SCALE - Structural CALculations Ensemble

The output from SCALE is a set of pages, neatly titled, dated and numbered, containing calculations and component details made to a standard suitable for submission to a checking authority.

Sample output calculations, detailing drawings, analysis runs and plots of results for nearly 4000 runs of SCALE, LUCID, SPADE and NL-STRESS, are included in this document. To jump directly to these calculations click on the bookmarks which should be displayed to the left of this document, or click on the highlighted proformas below to jump directly to sample output from that proforma.

Where a SCALE proforma has a # sign following its number, this indicates that that proforma pulls forward the moments, shears etc. from an NL-STRESS analysis into the proforma.

All proformas allow full workings to the Eurocodes and full workings to the British Standard, or are analytical and applicable to both. There are now over 600 proformas ready for the Eurocodes.

SCALE - Mathematics and Miscellaneous

sc020  Pages with heading only
sc021  Invoice for structural work
sc022  First page of calculations
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sc028  Solution of simultaneous equations
sc029  Solution of quadratic, cubic & quartic equations
sc030  Coordinate geometry
sc031  Differentiation of standard forms
sc032  Differentiation of a general function
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sc035  Mensuration of plane areas and solids
sc036  Statistics and quality control
sc037  Latent roots
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sc044  For testing only, automated running of all examples
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sc055  Cross referencing of numerical variables in proforma files
sc070  Simple test example
SCALE - Reinforced Concrete Design to Eurocode 2 and BS8110

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sc073  Rectangular beam - flexure, span/depth, curtailment, laps
sc074  Tee beam - flexure only
sc075  Tee beam - flexure, span/depth check
sc076  Design of walls
sc077  Wall to wall intersection
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sc111  Deep beams - Kong's method
sc112  Modular-ratio design of reinforcement for circular column
sc113  Modular-ratio calculation of stresses for circular column
sc117  Simply supported R.C. staircase
sc118  Concrete nibs
sc119  Fire resistance

SCALE - Reinforced & Prestressed Concrete Design: Eurocode 2/BS5400/DOT

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sc123  General reinforced concrete section - assessment
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email: ian@fitzroy.com

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sc340 Bolts
sc345 Column base
sc350 Steel guardrailing
Location: Triangle for main setting out of New Annexe

Solution of triangle given 2 sides and included angle

\[
\begin{align*}
\text{Length of side } b &= 10 \text{ m} \\
\text{Length of side } c &= 10 \text{ m} \\
\text{Included angle } A' &= 30^\circ \\
\text{From cosine formula} & \\
\text{a} &= \sqrt{(b^2 + c^2 - 2bc \cos A')} = 5.1764 \text{ m} \\
\text{B'} &= \text{DEG}(B) = 75^\circ \\
\text{C'} &= \text{DEG}(C) = 75^\circ
\end{align*}
\]
Simultaneous linear equations

\[
\begin{align*}
\text{a1} &= 2 \\
\text{b1} &= 1 \\
\text{c1} &= 1 \\
\text{d1} &= 1 \\
\text{e1} &= 11
\end{align*}
\]

with four unknowns

\[
\begin{align*}
\text{c2} &= 3 \\
\text{d2} &= 2 \\
\text{e2} &= 22 \\
\end{align*}
\]

The solution of four simultaneous equations:

\[
\begin{align*}
\text{a1}\cdot x + \text{b1}\cdot y + \text{c1}\cdot z + \text{d1}\cdot u &= \text{e1} \\
\text{a2}\cdot x + \text{b2}\cdot y + \text{c2}\cdot z + \text{d2}\cdot u &= \text{e2} \\
\text{a3}\cdot x + \text{b3}\cdot y + \text{c3}\cdot z + \text{d3}\cdot u &= \text{e3} \\
\text{a4}\cdot x + \text{b4}\cdot y + \text{c4}\cdot z + \text{d4}\cdot u &= \text{e4}
\end{align*}
\]

having coefficients:

\[
\begin{align*}
\text{a2} &= 1 \\
\text{b2} &= 2 \\
\text{a3} &= 3 \\
\text{b3} &= -1 \\
\text{a4} &= 7 \\
\text{b4} &= -6 \\
\text{c3} &= -1 \\
\text{d3} &= -1 \\
\text{c4} &= 1 \\
\text{d4} &= 5 \\
\text{e2} &= 22 \\
\text{e3} &= -6 \\
\text{e4} &= 18
\end{align*}
\]

Value of \(x\)

\[
x = 1
\]

Value of \(y\)

\[
y = 2
\]

Value of \(z\)

\[
z = 3
\]

Value of \(u\)

\[
u = 4
\]

Check by substitution in original equations

\[
\begin{align*}
\text{error1} &= \text{a1}\cdot x + \text{b1}\cdot y + \text{c1}\cdot z + \text{d1}\cdot u - \text{e1} = 0 \\
\text{error2} &= \text{a2}\cdot x + \text{b2}\cdot y + \text{c2}\cdot z + \text{d2}\cdot u - \text{e2} = 0 \\
\text{error3} &= \text{a3}\cdot x + \text{b3}\cdot y + \text{c3}\cdot z + \text{d3}\cdot u - \text{e3} = 0 \\
\text{error4} &= \text{a4}\cdot x + \text{b4}\cdot y + \text{c4}\cdot z + \text{d4}\cdot u - \text{e4} = 0
\end{align*}
\]
Location: Equation solution for 3 point problem

Equation to be solved is $x^3 - a.x^2 + b.x - c = 0$ where:

- Coefficient $a = 2$
- Coefficient $b = 3$
- Constant $c = 2.717$
Location: Equation solution for quadratic.

Equation to be solved is \( x^4 - a.x^3 + b.x^2 - c.x + d = 0 \) where:

Coefficient \( a = 3 \)
Coefficient \( b = -4 \)
Coefficient \( c = -2 \)
Constant \( d = 1 \)
Location: Setting out for entrance curve

Setting out circular arc

<table>
<thead>
<tr>
<th>Position</th>
<th>Chainage (m)</th>
<th>Offset (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>0.5</td>
<td>0.19631</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>0.36932</td>
</tr>
<tr>
<td>3</td>
<td>1.5</td>
<td>0.51998</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>0.64911</td>
</tr>
<tr>
<td>5</td>
<td>2.5</td>
<td>0.75735</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>0.84523</td>
</tr>
<tr>
<td>7</td>
<td>3.5</td>
<td>0.91317</td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>0.96148</td>
</tr>
<tr>
<td>9</td>
<td>4.5</td>
<td>0.99038</td>
</tr>
<tr>
<td>10</td>
<td>5</td>
<td>1</td>
</tr>
</tbody>
</table>

Length \( L = 10 \text{ m} \)
Offset at centre of arc \( H = 1 \text{ m} \)
Location: To find slope of function

Derivative of $x^n$ is $n \cdot x^{(n-1)}$.
Value of $x$ $x = 2$
Exponent $n = 3$
Derivative $dy/dx$ $y' = n \cdot x^{(n-1)} = 12$
Location: To find slope of water surface

Differentiation of a general function

Suppose that \( y_m2, y_m1, y_0, y_1 \) and \( y_2 \) are the values of \( f(x) \) at \( a-2h, a-h, a, a+h \) and \( a+2h \) respectively. Then by applying Taylor’s theorem it can be shown that, if \( h \) is small,

\[
f'(a) \approx \frac{8(y_1 - y_m1) - (y_2 - y_m2)}{12h}
\]

and

\[
f''(a) \approx \frac{16(y_1 + y_m1) - (y_2 + y_m2) - 30y_0}{12h^2}
\]

Increment taken as \( h = 1E-6 \)

Value of \( x \) at which derivative \( (dy/dx) \) required: \( a = 1 \)

Expression

\[ f(x) = 2x^2 \]

\[
\begin{align*}
 f(x) & \text{ at } a-2h \\
 y_{m2} & = 2x^2 = 1.999992 \\
 f(x) & \text{ at } a-h \\
 y_{m1} & = 2x^2 = 1.999996 \\
 f(x) & \text{ at } a \\
 y_0 & = 2x^2 = 2 \\
 f(x) & \text{ at } a+h \\
 y_1 & = 2x^2 = 2.000004 \\
 f(x) & \text{ at } a+2h \\
 y_2 & = 2x^2 = 2.000008
\end{align*}
\]

\[
\begin{align*}
 \frac{dy}{dx} & = \frac{8(y_1 - y_{m1}) - (y_2 - y_{m2})}{12h} = 4 \\
 \frac{d^2y}{dx^2} & = \frac{16(y_1 + y_{m1}) - (y_2 + y_{m2}) - 30y_0}{12h^2} = 3.999171364
\end{align*}
\]
Location: Area beneath pressure function

\[
\int_{a}^{b} c \cdot x^{n} \, dx = \left[ \frac{c \cdot x^{n+1}}{n+1} \right]_{a}^{b} \quad \text{for} \quad n \neq -1
\]

Constant \( c = 1 \)
Exponent \( n = 3 \)
Lower limit \( a = 1 \)
Upper limit \( b = 3 \)
Integral \( I = c \left( \frac{(b^{n+1})}{n+1} - \frac{(a^{n+1})}{n+1} \right) = 20 \)
Location: Revolve area under y=\(e^x\) about y axis from \(x=1\) to \(2\)

Integration of a general function using Simpson's rule

If \(y_1,y_2,y_3\) are the ordinates at \(a-h\), \(a\) and \(a+h\) respectively, Simpson's rule may be written:

\[
\int_{a-h}^{a+h} f(x) \, dx \approx \frac{h}{3} \left( f(a-h) + 4f(a) + f(a+h) \right)
\]

Usually Simpson's rule is applied by dividing the area between the ordinates \(y_1\) and \(y_{2n+1}\) into \(2n\) strips. This gives for 12 strips:

\[
\int_{a}^{b} f(x) \, dx \approx \frac{h}{3} \left( y_1 + y_{13} + 4(y_2 + y_4 + y_6 + y_8 + y_{10} + y_{12}) + 2(y_3 + y_5 + y_7 + y_9 + y_{11}) \right)
\]

where \(h = \frac{(b-a)}{12}\)

Lower limit \(a=1\)
Upper limit \(b=2\)
Width of strip \(h=\frac{(b-a)}{12}=0.08333333333\)
Function of \(x\) \(f(x)=2\pi x e^x\)
Writing \(k=4*(y(2)+y(4)+y(6)+y(8)+y(10)+y(12))=1111.327032\)
Integral \(I=\frac{h}{3}*(y(1)+y(13)+k+2*(y(3)+y(5)+y(7)+y(9)+y(11)))=46.42685176\)
### Location: Volume of concrete ball

#### Sphere & spherical cap

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of sphere</td>
<td>d = 0.25 m</td>
</tr>
<tr>
<td>Radius of sphere</td>
<td>r = d/2 = 0.125 m</td>
</tr>
<tr>
<td>Volume of sphere</td>
<td>V = ( \frac{4}{3} \pi r^3 ) = 0.0081812 m³</td>
</tr>
<tr>
<td>Curved surface area of sphere</td>
<td>C = 4πr² = 0.19635 m²</td>
</tr>
<tr>
<td>Height of altitude of cap</td>
<td>h = 0.05 m</td>
</tr>
<tr>
<td>Radius of base of cap</td>
<td>r₁ = ( \sqrt{2r \cdot h - h^2} ) = 0.1 m</td>
</tr>
<tr>
<td>Surface area of cap</td>
<td>S = 2πrh = 0.03927 m²</td>
</tr>
<tr>
<td>Volume of spherical cap</td>
<td>V = ( \frac{\pi}{6} (3r₁^2 + h^2) \cdot h ) = 0.85085E-3 m³</td>
</tr>
</tbody>
</table>
### Location: Cubes in 3rd floor slab

<table>
<thead>
<tr>
<th>Number of values in sample</th>
<th>N=12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured value</td>
<td>x(1)=30.5 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(2)=26.3 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(3)=32.6 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(4)=32.5 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(5)=28.9 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(6)=25 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(7)=32.1 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(8)=30.4 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(9)=31.4 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(10)=32.3 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(11)=33.5 N/mm²</td>
</tr>
<tr>
<td>Measured value</td>
<td>x(12)=33.4 N/mm²</td>
</tr>
<tr>
<td>No. in sample</td>
<td>12</td>
</tr>
<tr>
<td>Highest value</td>
<td>33.5 N/mm²</td>
</tr>
<tr>
<td>Lowest value</td>
<td>25 N/mm²</td>
</tr>
<tr>
<td>Range</td>
<td>8.5 N/mm²</td>
</tr>
<tr>
<td>Sum of values</td>
<td>Sum'x=335.5 N/mm²</td>
</tr>
<tr>
<td>Mean value</td>
<td>xbar=Sum'x/N=27.958 N/mm²</td>
</tr>
</tbody>
</table>

### Variance

\[ V = \frac{\sum d^2}{N-1} = 13.231 \]

### Standard deviation σ

\[ \sigma = \sqrt{V} = 3.6374 \text{ N/mm}² \]

### Coefficient of variation

\[ v = 100 \times \left( \frac{\sigma}{x_{\text{bar}}} \right) = 13.01 \% \]

### Percentage

\[ \text{per} = 5 \]

### Factor k as given by Himsworth

\[ k = \text{TABLE 1 for per}=5 \]

\[ k = 1.6795 \]

### Minimum desired strength

\[ x_0 = x_{\text{bar}} \times (1 - k \times v / 100) = 21.849 \text{ N/mm}² \]
Location: Latent roots for 2 x 2 matrix

Latent roots or characteristic values or eigenvalues

<table>
<thead>
<tr>
<th>Elements of K</th>
<th>k11=3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>k12=2</td>
</tr>
<tr>
<td></td>
<td>k21=1</td>
</tr>
<tr>
<td></td>
<td>k22=2</td>
</tr>
</tbody>
</table>

D=k1*k2-k21*k12=0
Location: Latent roots for 3 x 3 matrix

Latent roots or characteristic values or eigenvalues

Elements of K

k11=1250
k12=-500
k13=0
k21=-500
k22=875
k23=-375
k31=0
k32=-750
k33=750

Check | K - alpha.I | = 0  for first root:

k1=k11-alpha=-460.58
k2=k22-alpha=-835.58
k3=k33-alpha=-960.58

D=k1*(k2*k3-k23*k32)-k12*(k21*k3-k23*k31)+k13*(k21*k32-k2*k31)=-2.0336

Check | K - beta.I | = 0  for second root:

k1=k11-beta=250
k2=k22-beta=-125
k3=k33-beta=-250

D=k1*(k2*k3-k23*k32)-k12*(k21*k3-k23*k31)+k13*(k21*k32-k2*k31)=-0.64246

Check | K - gamma.I | = 0  for third root:

k1=k11-gamma=1085.6
k2=k22-gamma=710.58
k3=k33-gamma=585.58

D=k1*(k2*k3-k23*k32)-k12*(k21*k3-k23*k31)+k13*(k21*k32-k2*k31)=0
Location: Latent roots for 4 x 4 matrix

Latent roots or characteristic values or eigenvalues

Elements of $K$

$$\begin{align*}
  k_{11} &= -6 \\
  k_{12} &= 2 \\
  k_{13} &= 1 \\
  k_{14} &= -1 \\
  k_{21} &= 2 \\
  k_{22} &= -2 \\
  k_{23} &= 1 \\
  k_{24} &= 3 \\
  k_{31} &= 1 \\
  k_{32} &= 1 \\
  k_{33} &= -6 \\
  k_{34} &= 0 \\
  k_{41} &= -1 \\
  k_{42} &= 3 \\
  k_{43} &= 0 \\
  k_{44} &= -1
\end{align*}$$

Check $|K - \alpha I| = 0$ for first root:

$$\begin{align*}
  k_1 &= k_{11} - \alpha = 1.6701 \\
  k_2 &= k_{22} - \alpha = 5.6701 \\
  k_3 &= k_{33} - \alpha = 1.6701 \\
  k_4 &= k_{44} - \alpha = 6.6701
\end{align*}$$

$$\begin{align*}
  D_1 &= k_1(k_2(k_3k_4-k_{34}k_{43})-k_{23}((k_{32}k_4-k_{34}k_{42})+k_{24}((k_{32}k_4-k_{34}k_{43}))) \\
  &= 69.25
  \\
  D_2 &= -k_1(k_2(k_3k_4-k_{34}k_{43})-k_{23}((k_{32}k_4-k_{34}k_{42})+k_{24}((k_{32}k_4-k_{34}k_{43}))) \\
  &= -41.24
  \\
  D_3 &= k_1(k_2(k_3k_4-k_{34}k_{43})-k_{23}((k_{32}k_4-k_{34}k_{42})+k_{24}((k_{32}k_4-k_{34}k_{43}))) \\
  &= -12.48
  \\
  D_4 &= -k_1(k_2(k_3k_4-k_{34}k_{43})-k_{23}((k_{32}k_4-k_{34}k_{42})+k_{24}((k_{32}k_4-k_{34}k_{43}))) \\
  &= -15.491
  \\
  \text{Determinant} &= D_1 + D_2 + D_3 + D_4 = 0.038577
\end{align*}$$

Check $|K - \beta I| = 0$ for second root:

$$\begin{align*}
  k_1 &= k_{11} - \beta = 0.36953 \\
  k_2 &= k_{22} - \beta = 4.3695 \\
  k_3 &= k_{33} - \beta = 0.36953 \\
  k_4 &= k_{44} - \beta = 5.3695
\end{align*}$$

$$\begin{align*}
  D_1 &= k_1(k_2(k_3k_4-k_{34}k_{43})-k_{23}((k_{32}k_4-k_{34}k_{42})+k_{24}((k_{32}k_4-k_{34}k_{43}))) \\
  &= -0.0093137
  \\
  D_2 &= -k_1(k_2(k_3k_4-k_{34}k_{43})-k_{23}((k_{32}k_4-k_{34}k_{42})+k_{24}((k_{32}k_4-k_{34}k_{43}))) \\
  &= 0.58497
  \\
  D_3 &= k_1(k_2(k_3k_4-k_{34}k_{43})-k_{23}((k_{32}k_4-k_{34}k_{42})+k_{24}((k_{32}k_4-k_{34}k_{43}))) \\
  &= -0.72329
  \\
  D_4 &= -k_1(k_2(k_3k_4-k_{34}k_{43})-k_{23}((k_{32}k_4-k_{34}k_{42})+k_{24}((k_{32}k_4-k_{34}k_{43}))) \\
  &= 0.16811
  \\
  \text{Determinant} &= D_1 + D_2 + D_3 + D_4 = 0.020471
\end{align*}$$

Check $|K - \gamma I| = 0$ for third root:

$$\begin{align*}
  k_1 &= k_{11} - \gamma = -3.382 \\
  k_2 &= k_{22} - \gamma = 0.61797 \\
  k_3 &= k_{33} - \gamma = -3.382 \\
  k_4 &= k_{44} - \gamma = 1.618
\end{align*}$$

$$\begin{align*}
  D_1 &= k_1(k_2(k_3k_4-k_{34}k_{43})-k_{23}((k_{32}k_4-k_{34}k_{42})+k_{24}((k_{32}k_4-k_{34}k_{43}))) \\
  &= -86.034
\end{align*}$$
D2 = -k12*(k21*(k3*k4-k34*k43)-k23*(k31*k4-k34*k41)+k24*(k31*k43-k3 *k41)) = 45.416
D3 = k13*(k21*(k32*k4-k34*k42)-k2*(k31*k4-k34*k41)+k24*(k31*k42-k32 *k41)) = 14.236
D4 = -k14*(k21*(k32*k43-k3*k42)-k2*(k31*k43-k3*k41)+k23*(k31*k42-k32 *k41)) = 26.382

Determinant

D = D1 + D2 + D3 + D4 = 2.9468E-6

Check | k - delta.I | = 0 for fourth root:

k1 = k11 - delta = -7.6575
k2 = k22 - delta = -3.6575
k3 = k33 - delta = -7.6575
k4 = k44 - delta = -2.6575

D1 = k1*(k2*(k3*k4-k34*k43)-k23*(k32*k4-k34*k42)+k24*(k32*k43-k3 *k42)) = 21.868
D2 = -k12*(k21*(k3*k4-k34*k43)-k23*(k31*k4-k34*k41)+k24*(k31*k43-k33 *k41)) = -40.77
D3 = k13*(k21*(k32*k4-k34*k42)-k2*(k31*k4-k34*k41)+k24*(k31*k42-k32 *k41)) = -3.035
D4 = -k14*(k21*(k32*k43-k3*k42)-k2*(k31*k43-k3*k41)+k23*(k31*k42-k32 *k41)) = 21.938

Determinant

D = D1 + D2 + D3 + D4 = -7.3703E-6
Location: Largest latent root for 4 x 4 matrix

Latent roots or characteristic values or eigenvalues

No. of elements in square matrix \( \text{nel}=16 \)
Number of rows or columns \( a=\text{SQR} (\text{nel})=4 \)
Element 1 \( a(1)=5 \)
Element 2 \( a(2)=2 \)
Element 3 \( a(3)=1 \)
Element 4 \( a(4)=-1 \)
Element 5 \( a(5)=2 \)
Element 6 \( a(6)=9 \)
Element 7 \( a(7)=1 \)
Element 8 \( a(8)=3 \)
Element 9 \( a(9)=1 \)
Element 10 \( a(10)=1 \)
Element 11 \( a(11)=5 \)
Element 12 \( a(12)=0 \)
Element 13 \( a(13)=-1 \)
Element 14 \( a(14)=3 \)
Element 15 \( a(15)=0 \)
Element 16 \( a(16)=6 \)

Largest latent root \( \lambda=11.286 \)
**Location: Centre of gravity of component A-12**

<table>
<thead>
<tr>
<th>Weight</th>
<th>X-coordinate</th>
<th>Y-coordinate</th>
<th>Z-coordinate</th>
</tr>
</thead>
<tbody>
<tr>
<td>w(1) = 5000 kN</td>
<td>x(1) = 10 m</td>
<td>y(1) = 10 m</td>
<td>z(1) = 0 m</td>
</tr>
<tr>
<td>w(2) = 3000 kN</td>
<td>x(2) = 5 m</td>
<td>y(2) = 5 m</td>
<td>z(2) = 0 m</td>
</tr>
<tr>
<td>w(3) = 5000 kN</td>
<td>x(3) = 15 m</td>
<td>y(3) = 15 m</td>
<td>z(3) = 0 m</td>
</tr>
<tr>
<td>w(4) = 3000 kN</td>
<td>x(4) = 20 m</td>
<td>y(4) = 20 m</td>
<td>z(4) = 0 m</td>
</tr>
<tr>
<td>w(5) = 0 kN</td>
<td>x(5) = 0 m</td>
<td>y(5) = 0 m</td>
<td>z(5) = 0 m</td>
</tr>
</tbody>
</table>

Number of weights: 4
Total of all weights: 16000 kN

Total moment about x axis: \( \sigma_{wx} = 200000 \) kNm
Total moment about y axis: \( \sigma_{wy} = 200000 \) kNm
Total moment about z axis: \( \sigma_{wz} = 0 \) kNm

Centroid:
\[ x_{bar} = \frac{\sigma_{wx}}{\sigma_{w}} = 12.5 \text{ m} \]
\[ y_{bar} = \frac{\sigma_{wy}}{\sigma_{w}} = 12.5 \text{ m} \]
\[ z_{bar} = \frac{\sigma_{wz}}{\sigma_{w}} = 0 \text{ m} \]
Location: Example: symmetric matrix arithmetic

Total number of elements in matrix n=16

Elements of matrix A order

- left to right - top to bottom

\[
\begin{bmatrix}
1.36 & -0.48 & -1 & 0 \\
-0.48 & 1.64 & 0 & 0 \\
-1 & 0 & 1.36 & 0.48 \\
0 & 0 & 0.48 & 1.64
\end{bmatrix}
\]

MATRIX [A] AFTER INVERSION:

Elements of matrix A order

- left to right - top to bottom

\[
\begin{bmatrix}
2.5031 & 0.7326 & 2.0525 & -0.60073 \\
0.7326 & 0.82418 & 0.60073 & -0.17582 \\
2.0525 & 0.60073 & 2.5031 & -0.7326 \\
-0.60073 & -0.17582 & -0.7326 & 0.82418
\end{bmatrix}
\]
Greek alphabet

<table>
<thead>
<tr>
<th>Greek alphabet</th>
<th>Upper case</th>
<th>Lower case</th>
</tr>
</thead>
<tbody>
<tr>
<td>(with SCALE character references in parentheses)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alpha</td>
<td>A ( A )</td>
<td>α (224)</td>
</tr>
<tr>
<td>Beta</td>
<td>B ( B )</td>
<td>β (225)</td>
</tr>
<tr>
<td>Gamma</td>
<td>Γ (226)</td>
<td>γ (128)</td>
</tr>
<tr>
<td>Delta</td>
<td>Δ (129)</td>
<td>δ (235)</td>
</tr>
<tr>
<td>Epsilon</td>
<td>E ( E )</td>
<td>ε (238)</td>
</tr>
<tr>
<td>Zeta</td>
<td>Z ( Z )</td>
<td>–</td>
</tr>
<tr>
<td>Eta</td>
<td>H ( H )</td>
<td>η (131)</td>
</tr>
<tr>
<td>Theta</td>
<td>Θ (233)</td>
<td>θ (132)</td>
</tr>
<tr>
<td>Iota</td>
<td>I ( I )</td>
<td>–</td>
</tr>
<tr>
<td>Kappa</td>
<td>K ( K )</td>
<td>κ (134)</td>
</tr>
<tr>
<td>Lambda</td>
<td>–</td>
<td>λ (136)</td>
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<tr>
<td>Mu</td>
<td>M ( M )</td>
<td>μ (230)</td>
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<tr>
<td>Nu</td>
<td>N ( N )</td>
<td>–</td>
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<tr>
<td>Xi</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Omicron</td>
<td>O ( O )</td>
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<tr>
<td>Pi</td>
<td>Π (139)</td>
<td>π (227)</td>
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<tr>
<td>Rho</td>
<td>ρ (140)</td>
<td>–</td>
</tr>
<tr>
<td>Sigma</td>
<td>Σ (228)</td>
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<td>Omega</td>
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Mathematical Symbols

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<td>Τ ( T )</td>
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<tr>
<td>Phi</td>
<td>Φ (232)</td>
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<td>Ψ (144)</td>
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<tr>
<td>Omega</td>
<td>– (142)</td>
</tr>
</tbody>
</table>

Mathematical Symbols

(with SCALE character references in parentheses)

| Sterling      | £ ( £ )    |          |          |
| Dollar        | $ ( $ )    |          |          |
| Euro          | € (148)    |          |          |
| Diameter      | Ø ( 23)    |          |          |
| Infinity      | ∞ (236)    |          |          |
| Quarter       | ¼ (172)    |          |          |
| Half          | ½ (171)    |          |          |
| Squared       | ² (253)    |          |          |
| Cubed         | ³ (169)    |          |          |
| To the fourth | ⁴ (174)    |          |          |
| To the sixth  | ⁶ (175)    |          |          |
| Divide        | ÷ (246)    |          |          |
| Approx        | ≈ (247)    |          |          |
| Degrees       | ° (248)    |          |          |
| Identical     | ≡ (240)    |          |          |
| Plus-minus    | ± (241)    |          |          |
| Greater or equal | ≥ (242) |          |          |
| Less or equal | ≤ (243)    |          |          |
| Square root   | (251)      |          |          |
Imperial second moment of area $I = 0.0805941358024692 \ ft^4$

Metric second moment of area $= 0.695605958462723E-3 \ m^4$
$= 69560.5958462723 \ cm^4$
$= 695.605958462723E6 \ mm^4$
Sum lent (principal) \( s = 100 \)
Compound interest rate \( p = 5 \% \) /year
Interest paid every \( t = 1 \) months
Length of loan period \( n = 1 \) years
\[ T = n \times 12 = 12 \text{ months} \]
At the end of 12 months the principal amount is
\[ A = s \times (1 + p / (12/t \times 100))^{(T/t)} = 105.12 \]
Polynomial fit:

264.14 * x^0 ± 0.98294
0.75711 * x^1 ± 0.03386
-0.043598 * x^2 ± 0.0026634
0.32629E-3 * x^3 ± 30.279E-6
-1.0476E-6 * x^4 ± 0.16147E-6
1.5213E-9 * x^5 ± 0.40443E-9
-0.77857E-12 * x^6 ± 0.3838E-12
Location: Channel section used as a test example

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of section D</td>
<td>200 mm</td>
</tr>
<tr>
<td>Width of section B</td>
<td>200 mm</td>
</tr>
<tr>
<td>Thickness of web t</td>
<td>10 mm</td>
</tr>
<tr>
<td>Thickness of flange T</td>
<td>10 mm</td>
</tr>
</tbody>
</table>
Location: Small beam - high-yield steel - no compression steel

Bending in rectangular beams

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Moment before redistribution \( M_{bef} = 12 \text{ kNm} \)

Beam being analysed is considered as non-continuous.

Characteristic concrete strength \( f_{cu} = 35 \text{ N/mm}^2 \)

Max.aggregate size (for bar spc.) \( h_{agg} = 15 \text{ mm} \)

Char.strength of long'l bars \( f_y = 500 \text{ N/mm}^2 \)

Longitudinal reinforcement is high-yield steel.

Diameter of tension bars \( \text{dia} = 10 \text{ mm} \)

Diameter of link legs \( \text{dial} = 8 \text{ mm} \)

Designated exposure class is XC1

Specified fixing tolerance \( \text{tol} = 10 \text{ mm} \)

Nominal concrete cover \( \text{cover} = 25 \text{ mm} \)

Overall depth of section \( h = 275 \text{ mm} \)

Effective depth of section \( d = 237 \text{ mm} \)

Breadth of section \( b = 200 \text{ mm} \)

Longitudinal reinforcement

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength 500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 10 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars 2</td>
</tr>
<tr>
<td></td>
<td>arranged in a single layer</td>
</tr>
<tr>
<td></td>
<td>Cover to all steel 25 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required 122.58 mm²</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided 157.08 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided 0.2856 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided 1.2331 kg/m</td>
</tr>
<tr>
<td></td>
<td>Maximum clear spacing 180 mm</td>
</tr>
<tr>
<td></td>
<td>Link size assumed 8 mm</td>
</tr>
</tbody>
</table>
**Location:** Narrow deep beam - mild steel - no compression steel

**Bending in rectangular beams**

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Moment before redistribution \( M_{bef} = 1000 \text{ kNm} \)

Beam being analysed is considered as continuous.

Design moment (after redistrib.) \( M = 1000 \text{ kNm} \)

Characteristic concrete strength \( f_{cu} = 30 \text{ N/mm}^2 \)

Max. aggregate size (for bar spc.) \( h_{agg} = 15 \text{ mm} \)

Char. strength of long'1 bars \( f_y = 250 \text{ N/mm}^2 \)

Longitudinal reinforcement is mild steel.

Diameter of tension bars \( \text{dia}=25 \text{ mm} \)

Diameter of link legs \( \text{dial}=12 \text{ mm} \)

Designated exposure class is XC2

Specified fixing tolerance \( \text{tol}=10 \text{ mm} \)

Nominal concrete cover \( \text{cover}=35 \text{ mm} \)

Overall depth of section \( h=950 \text{ mm} \)

Effective depth of section \( d=865 \text{ mm} \)

Breadth of section \( b=300 \text{ mm} \)

**Longitudinal reinforcement**

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength 250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 25 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars 14</td>
</tr>
<tr>
<td></td>
<td>arranged in 4 layers</td>
</tr>
<tr>
<td></td>
<td>Cover to all steel 35 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required 6718.4 mm²</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided 6872.2 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided 2.4113 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided 53.947 kg/m</td>
</tr>
<tr>
<td></td>
<td>Maximum clear spacing 288 mm</td>
</tr>
<tr>
<td></td>
<td>Link size assumed 12 mm</td>
</tr>
</tbody>
</table>

**WARNING:**

As overall depth \( h \) exceeds 750 mm bars to control cracking must be provided on side faces of beam (Clause 3.12.5.4 and 3.12.11.2.6).
Location: Shallow beam - square twist - with compression steel

Bending in rectangular beams

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Moment before redistribution \( M_{bef} = 81.8 \text{ kNm} \)
Beam being analysed is considered as continuous.

Design moment (after redistrib.) \( M = 71 \text{ kNm} \)
Characteristic concrete strength \( f_{cu} = 40 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{agg} = 15 \text{ mm} \)
Char.strength of long'l bars \( f_y = 425 \text{ N/mm}^2 \)

Steel strength other than those given in Table 3.1 of BS8110.
Specified min.steel percentage \( \text{permin} = 0.16 \% \)
Diameter of tension bars \( \text{dia} = 12 \text{ mm} \)
Diameter of link legs \( \text{dial} = 10 \text{ mm} \)
Nominal concrete cover \( \text{cover} = 30 \text{ mm} \)
Overall depth of section \( h = 150 \text{ mm} \)
Effective depth of section \( d = 104 \text{ mm} \)
Breadth of section \( b = 1000 \text{ mm} \)

Longitudinal reinforcement

Diameter of compression bars \( \text{diac} = 12 \text{ mm} \)
Depth to compression bars \( d' = 46 \text{ mm} \)

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength ( 425 \text{ N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars ( 12 \text{ mm} )</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars ( 22 )</td>
</tr>
<tr>
<td></td>
<td>arranged in a single layer</td>
</tr>
<tr>
<td></td>
<td>Cover to all steel ( 30 \text{ mm} )</td>
</tr>
<tr>
<td></td>
<td>Area of steel required ( 2417.8 \text{ mm}^2 )</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided ( 2488.1 \text{ mm}^2 )</td>
</tr>
<tr>
<td></td>
<td>Percentage provided ( 1.6423 % )</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided ( 19.532 \text{ kg/m} )</td>
</tr>
<tr>
<td></td>
<td>Maximum clear spacing ( 148 \text{ mm} )</td>
</tr>
<tr>
<td></td>
<td>Link size assumed ( 10 \text{ mm} )</td>
</tr>
<tr>
<td>COMPRESSION</td>
<td>Characteristic strength 425 N/mm²</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>----------------------------------</td>
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<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 12 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars 28</td>
</tr>
<tr>
<td></td>
<td>arranged in a single layer</td>
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<tr>
<td></td>
<td>Cover to all steel 30 mm</td>
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<tr>
<td></td>
<td>Area of steel required 3158.9 mm²</td>
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<tr>
<td></td>
<td>Area of steel provided 3166.7 mm²</td>
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<td></td>
<td>Percentage provided 2.0902 %</td>
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<tr>
<td></td>
<td>Weight of steel provided 24.859 kg/m</td>
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<td></td>
<td>Minimum poss. link size 6 mm</td>
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<tr>
<td></td>
<td>Maximum spacing of links 144 mm</td>
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</table>
Bending in rectangular beams

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution \( M_{bef} = 5000 \text{ kNm} \)
Beam being analysed is considered as continuous.
Design moment (after redistrib.) \( M = 5000 \text{ kNm} \)
Characteristic concrete strength \( f_{cu} = 50 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{agg} = 15 \text{ mm} \)
Char.strength of long'1 bars \( f_y = 250 \text{ N/mm}^2 \)
Longitudinal reinforcement is mild steel.
Diameter of tension bars \( d_{ia} = 40 \text{ mm} \)
Diameter of link legs \( d_{ial} = 16 \text{ mm} \)
Designated exposure class is XC3
Specified fixing tolerance \( t_{ol} = 10 \text{ mm} \)
Nominal concrete cover \( c_{over} = 50 \text{ mm} \)
Overall depth of section \( h = 1000 \text{ mm} \)
Effective depth of section \( d = 914 \text{ mm} \)
Breadth of section \( b = 1000 \text{ mm} \)

Longitudinal reinforcement

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength 250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 40 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars 24</td>
</tr>
<tr>
<td></td>
<td>arranged in 3 layers</td>
</tr>
<tr>
<td></td>
<td>Cover to all steel 50 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required 29816 mm²</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided 30159 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided 2.9861 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided 236.75 kg/m</td>
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<tr>
<td></td>
<td>Maximum clear spacing 285 mm</td>
</tr>
<tr>
<td></td>
<td>Link size assumed 16 mm</td>
</tr>
</tbody>
</table>

WARNING:
As more than 1 layer of tension bars required, you should determine a revised effective depth corresponding to the position of the centroid of this reinforcement, and rerun this proforma.
Location: Ex1 - Beam with compression reinforcement

Bending in rectangular beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Moment before redistribution \( M_{bef} = 150 \text{kNm} \)
Beam being analysed is considered as non-continuous.
Char yield strength of reinforcement \( f_{yk} = 500 \text{N/mm}^2 \)
Max. aggregate size (for bar spc.) \( h_{agg} = 20 \text{mm} \)
Diameter of tension bars \( \text{dia} = 25 \text{mm} \)
Diameter of link legs \( \text{dial} = 8 \text{mm} \)
Effective depth of section \( d = 340 \text{mm} \)
Diameter of compression bars \( \text{diac} = 12 \text{mm} \)
Depth to compression steel \( d_2 = 45 \text{mm} \)

DESIGN

| Overall depth | 400 mm |
| Effective depth | 340 mm |
| Width of section | 250 mm |

SUMMARY

| Parameter K | 0.173 |
| Parameter K' | 0.1684 |
| Lever arm ratio \( z/d \) | 0.8185 |

FLEXURE

| Steel area provided | 1473 mm$^2$ |
| Diameter of bars | 25 mm |
| Number of bars | 3 |

TENSION REINFORCEMENT

| Steel area required | 1238 mm$^2$ |
| Steel percentage req. | 1.456% |
| Minimum area of steel | 128 mm$^2$ |
| Maximum area of steel | 4000 mm$^2$ |

COMPRESSION REINFORCEMENT

| Steel area provided | 226.2 mm$^2$ |
| Diameter of bars | 12 mm |
| Number of bars | 2 |

| Steel area required | 31.16 mm$^2$ |
| Minimum area required | 200 mm$^2$ |
Location: Ex2 - Redistribution example

Bending in rectangular beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Moment before redistribution \( M_{bef} = 460 \text{ kNm} \)
Beam being analysed is considered as continuous.
Section considered has a hogging moment.
Design moment (after redistrib.) \( M = 370 \text{ kNm} \)
Char yield strength of reinf’ment \( f_yk = 500 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( \text{dia} = 25 \text{ mm} \)
Diameter of link legs \( \text{dial} = 8 \text{ mm} \)
Effective depth of section \( d = 525 \text{ mm} \)
Diameter of compression bars \( \text{diac} = 20 \text{ mm} \)
Depth to compression steel \( d_2 = 100 \text{ mm} \)

| DESIGN | Overall depth | 600 mm |
| SUMMARY | Effective depth | 525 mm |
| Width of section | 300 mm |
| FLEXURE | Parameter K | 0.179 |
| Parameter K' | 0.1547 |
| Lever arm ratio z/d | 0.8369 |
| TENSION REINFORCEMENT | Steel area provided | 1963 mm$^2$ |
| Diameter of bars | 25 mm |
| Number of bars | 4 |
| Steel area required | 1946 mm$^2$ |
| Steel percentage req. | 1.235 % |
| Minimum area of steel | 210.1 mm$^2$ |
| Maximum area of steel | 7200 mm$^2$ |
| COMPRESSION REINFORCEMENT | Steel area provided | 628.3 mm$^2$ |
| Diameter of bars | 20 mm |
| Number of bars | 2 |
| Steel area required | 316.1 mm$^2$ |
| Minimum area required | 360 mm$^2$ |
Location: Ex3 - Shallow beam

Bending in rectangular beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Moment before redistribution \( M_{bef} = 425 \text{ kNm} \)

Beam being analysed is considered as continuous.

Char yield strength of reinf'ment \( f_y = 500 \text{ N/mm}^2 \)

Max. aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)

Diameter of tension bars \( d_{ia} = 25 \text{ mm} \)

Diameter of link legs \( d_{ial} = 8 \text{ mm} \)

Effective depth of section \( d = 340 \text{ mm} \)

Diameter of compression bars \( d_{iac} = 16 \text{ mm} \)

Depth to compression steel \( d_2 = 50 \text{ mm} \)

**DESIGN**

Overall depth \( 400 \text{ mm} \)

Effective depth \( 340 \text{ mm} \)

Width of section \( 600 \text{ mm} \)

**SUMMARY**

Effective depth \( 340 \text{ mm} \)

**FLEXURE**

Parameter K \( 0.1751 \)

Parameter K' \( 0.1684 \)

Lever arm ratio \( z/d \) \( 0.8185 \)

**TENSION REINFORCEMENT**

Steel area provided \( 3927 \text{ mm}^2 \)

Diameter of bars \( 25 \text{ mm} \)

Number of bars \( 8 \)

Steel area required \( 3507 \text{ mm}^2 \)

Steel percentage req. \( 1.719 \% \)

Minimum area of steel \( 340.5 \text{ mm}^2 \)

Maximum area of steel \( 9600 \text{ mm}^2 \)

**COMPRESSION REINFORCEMENT**

Steel area provided \( 603.2 \text{ mm}^2 \)

Diameter of bars \( 16 \text{ mm} \)

Number of bars \( 3 \)

Steel area required \( 128.4 \text{ mm}^2 \)

Minimum area required \( 480 \text{ mm}^2 \)
Location: Ex4 - Minimum arrangement

Bending in rectangular beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Moment before redistribution $M_{bef}=12$ kNm
Beam being analysed is considered as continuous.
Section considered has a sagging moment.
Char yield strength of reinforcement $f_{yk}=500$ N/mm$^2$
Max. aggregate size (for bar spc.) $h_{agg}=20$ mm
Diameter of tension bars $dia=12$ mm
Diameter of link legs $dial=8$ mm
Effective depth of section $d=345$ mm

DESIGN
- Overall depth $400$ mm
- Effective depth $345$ mm
- Width of section $200$ mm

SUMMARY
- Parameter K $0.0202$
- Parameter K' $0.1684$
- Lever arm ratio $z/d$ $0.95$

FLEXURE
- Steel area required $92.03$ mm$^2$
- Steel area provided $226.2$ mm$^2$
- Diameter of bars $12$ mm
- Number of bars $2$
- Steel percentage req. $0.1334$ %
- Minimum area of steel $92.03$ mm$^2$
- Maximum area of steel $3200$ mm$^2$
Location: Small beam - high-yield steel - no compression steel

Bending in rectangular beams with optional calculations for shear, lap lengths, bar curtailment & limiting span/effective depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Ultimate BM before redistribution Mbef=12 kNm
Beam being analysed is considered as non-continuous.
Characteristic concrete strength fcu=35 N/mm²
Max.aggregate size (for bar spc) hagg=15 mm
Char.strength of long'1 bars fy=500 N/mm²
Longitudinal reinforcement is high-yield steel.
Diameter of tension bars dia=10 mm
Diameter of link legs dial=8 mm
Char.strength of link steel fyv=500 N/mm²
High-yield steel shear reinforcement.
Designated exposure class is XC1
Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=25 mm
Overall depth of section h=275 mm
Effective depth of section d=237 mm
Breadth of section b=200 mm

Longitudinal reinforcement

| TENSION | Characteristic strength 500 N/mm² |
| REINFORCEMENT | Diameter of bars 10 mm |
| SUMMARY | Number of bars 2 |
|           | arranged in a single layer. |
|           | Cover to all steel 25 mm |
|           | Area of steel required 122.58 mm² |
|           | Area of steel provided 157.08 mm² |
|           | Percentage provided 0.2856 % |
|           | Weight of steel provided 1.2331 kg/m |
|           | Max.permissible spacing 180 mm |
|           | Link size assumed 8 mm |
Check on span/effective depth ratio

Basic ratio for simp.-sup.beam \( bs'd=20 \) see Table 3.9
As applied-moment factor \( M'bd2=M*1000*1000/(b*d^2)=1.0682 \) N/mm²
Mod.factor for tension steel from equation 7 (Table 3.10) \( \text{modf}_1=0.55+(477-fs)/(120*(.9+M'bd2))=1.4682 \)
Mod.factor for no comp.steel \( \text{modf}_2=1 \)
Maximum permissible \( \text{span/effective-depth ratio} \) \( ps'd=bs'd*\text{modf}_1*\text{modf}_2=29.364 \)
Span of beam (see Cls.3.4.1.2-4) \( \text{span}=5 \) m
Actual span/effective-depth ratio \( as'd=1000*\text{span}/d=21.097 \)
As this does not exceed 29.364, this is Acceptable.

Bending-moment capacities at steel curtailment points

As there are only two bars in tension, no curtailment is possible.

Anchorage and lap lengths – Clause 3.12.8 of BS8110: Part 1

Type 2 deformed bars: bond coefficient \( \beta=0.50 \) (Table 3.26)
Tension reinforcement:
Force in tension bar at yield \( Ft=fy/gammaS*PI*dia^2/4=34148 \) N
Ultimate anchorage bond stress \( fbu=\beta*SQR(fcu)=2.958 \) N/mm²
Anchorage and tension lap length \( \text{lap}=Ft/(PI*dia*fbu)=367.46 \) mm
\( =370 \) mm (rounded)
1.4 * tension lap length \( =520 \) mm (rounded)
2 * tension lap length \( =740 \) mm (rounded)

Shear reinforcement

Shear calculations are in accordance with Clauses 3.4.5 of Code
Support \( \text{av} \):
Design section

Location for shear calculation: 0.25 m from support
Effective breadth for shear \( bv=200 \) mm
Shear force due to ultimate load \( V=100 \) kN
Distance from support \( av=250 \) mm
No.of tension bars effective at section \( nbars=2 \)
Number of legs to be provided \( nlegs=2 \)
Chosen link spacing \( sv'=140 \) mm

Use 8 mm links (two legs), spaced at 140 mm centres along beam.

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>Number of legs</td>
<td>2</td>
</tr>
<tr>
<td>Spacing</td>
<td>140 mm</td>
</tr>
<tr>
<td>Approx.weight of links</td>
<td>2.1588 kg/m</td>
</tr>
</tbody>
</table>
Location: Narrow deep beam - mild steel - no compression steel

Bending in rectangular beams with optional calculations for shear, lap lengths, bar curtailment & limiting span/effective depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Ultimate BM before redistribution Mbef=1000 kNm
Beam being analysed is considered as continuous.
Design moment (after redistrib.) M=1000 kNm
Characteristic concrete strength fcu=30 N/mm²
Max.aggregate size (for bar spc) hagg=15 mm
Char.strength of long'l bars fy=250 N/mm²
Longitudinal reinforcement is mild steel.
Diameter of tension bars dia=25 mm
Diameter of link legs dial=12 mm
Char.strength of link steel fyv=250 N/mm²
Mild steel shear reinforcement.
Designated exposure class is XC2
Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=35 mm
Overall depth of section h=950 mm
Effective depth of section d=865 mm
Breadth of section b=300 mm

Longitudinal reinforcement

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength 250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 25 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars 14</td>
</tr>
<tr>
<td></td>
<td>arranged in 4 layers.</td>
</tr>
<tr>
<td></td>
<td>Cover to all steel 35 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required 6718.4 mm²</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided 6872.2 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided 2.4113 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided 53.947 kg/m</td>
</tr>
<tr>
<td></td>
<td>Max.permissible spacing 288 mm</td>
</tr>
<tr>
<td></td>
<td>Link size assumed 12 mm</td>
</tr>
</tbody>
</table>

As overall depth h exceeds 750 mm bars to control cracking must be provided on side faces of beam (Clause 3.12.5.4 & 3.12.11.2.6).
Check on span/effective depth ratio

Basic ratio for continuous beam  \( bs'd=26 \)  see Table 3.9
As applied-moment factor \( M'bd^2=M*1000*1000/(b*d^2)=4.455 \text{ N/mm}^2 \)
Mod.factor for tension steel from equation 7 (Table 3.10) \( \text{modf1}=0.55+(477-fs)/(120*(.9+M'bd^2))=1.0387 \)
Mod.factor for no comp.steel \( \text{modf2}=1 \)
Maximum permissible span/effective-depth ratio \( ps'd=bs'd*\text{modf1}*\text{modf2}=27.007 \)
Span of beam (see Cls.3.4.1.2-4) \( \text{span}=12 \text{ m} \)
As span exceeds 10 m and partitions/finishes are sensitive to deflection multiply basic ratio by 10/span (in m) \( \text{ps'}d=\text{ps}'d*10/\text{span}=22.506 \)
Actual span/effective-depth ratio as\( d'=1000*\text{span}/d=13.873 \)
As this does not exceed 22.506, this is Acceptable.

Bending-moment capacities at steel curtailment points

The number of tension bars is even, so reduce in pairs.

With 12 tension bars, effective steel area \( Asrd=5890.5 \text{ mm}^2 \)
Reduced design moment \( M_{\text{red}}=M*\text{nbtr}/n\text{bart}=857.14 \text{ kNm} \)

With 10 tension bars, effective steel area \( Asrd=4908.7 \text{ mm}^2 \)
Reduced design moment \( M_{\text{red}}=M*\text{nbtr}/n\text{bart}=714.29 \text{ kNm} \)

With 8 tension bars, effective steel area \( Asrd=3927 \text{ mm}^2 \)
Reduced design moment \( M_{\text{red}}=M*\text{nbtr}/n\text{bart}=571.43 \text{ kNm} \)

With 6 tension bars, effective steel area \( Asrd=2945.2 \text{ mm}^2 \)
Reduced design moment \( M_{\text{red}}=M*\text{nbtr}/n\text{bart}=428.57 \text{ kNm} \)

With 4 tension bars, effective steel area \( Asrd=1963.5 \text{ mm}^2 \)
Reduced design moment \( M_{\text{red}}=M*\text{nbtr}/n\text{bart}=285.71 \text{ kNm} \)

With 2 tension bars, effective steel area \( Asrd=981.75 \text{ mm}^2 \)
Reduced design moment \( M_{\text{red}}=M*\text{nbtr}/n\text{bart}=142.86 \text{ kNm} \)

No further curtailment is possible. However, bars must extend beyond the theoretical curtailment points as required below.

Min.extension to tension bars equal to effective depth \( d \) of 865 mm as this is not less than 12*tension bar diameter, i.e. 300 mm.
Anchorage and lap lengths - Clause 3.12.8 of BS8110: Part 1

Type 2 deformed bars: bond coefficient $\beta = 0.50$ (Table 3.26)

Tension reinforcement:
- Force in tension bar at yield $F_t = \frac{f_y}{\gamma_s} \pi d^2 / 4 = 106712$ N
- Ultimate anchorage bond stress $f_{bu} = \beta \sqrt{f_{cu}} = 2.7386$ N/mm²
- Anchorage and tension lap length $l_{ap} = \frac{F_t}{\pi d f_{bu}} = 496.13$ mm
  - $1.4 \times$ tension lap length $= 700$ mm (rounded)
  - $2 \times$ tension lap length $= 1000$ mm (rounded)

Shear reinforcement

Shear calculations are in accordance with Clauses 3.4.5 of Code

Support $\downarrow$ av $\uparrow$ Design section

Location for shear calculation: 0.5 m from support
- Effective breadth for shear $b_v = 300$ mm
- Shear force due to ultimate load $V = 500$ kN
- Distance from support $a_v = 500$ mm
- No. of tension bars effective at section $n_{bars} = 14$
- Number of legs to be provided $n_{legs} = 2$
- Chosen link spacing $s_v' = 160$ mm

Use 12 mm links (two legs), spaced at 160 mm centres along beam.

<table>
<thead>
<tr>
<th>SHEAR REINFORCEMENT SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
</tr>
<tr>
<td>Diameter of links</td>
</tr>
<tr>
<td>Number of legs</td>
</tr>
<tr>
<td>Spacing</td>
</tr>
<tr>
<td>Approx.weight of links</td>
</tr>
</tbody>
</table>
Location: Shallow beam - square twist - with compression steel

Bending in rectangular beams with optional calculations for shear,
lap lengths, bar curtailment & limiting span/effective depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Ultimate BM before redistribution Mbef=81.8 kNm
Beam being analysed is considered as continuous.
Design moment (after redistrib.) M=72 kNm
Characteristic concrete strength fcu=40 N/mm²
Max.aggregate size (for bar spc) hagg=15 mm
Char.strength of long'l bars fy=425 N/mm²
Steel strength other than those given in Table 3.1 of BS8110.
Specified min.steel percentage permin=0.16 %
Diameter of tension bars dia=12 mm
Diameter of link legs dial=8 mm
Char.strength of link steel fyv=250 N/mm²
Mild steel shear reinforcement.
Nominal concrete cover cover=30 mm
Overall depth of section h=150 mm
Effective depth of section d=104 mm
Breadth of section b=1010 mm

Longitudinal reinforcement

Depth to compression bars d'=44 mm

| TENSION REINFORCEMENT SUMMARY |
|------------------------------|-----------------|
| Characteristic strength     | 425 N/mm²       |
| Diameter of bars             | 12 mm           |
| Number of bars               | 22              |
| arranged in a single layer   |                 |
| Cover to all steel           | 30 mm           |
| Area of steel required       | 2453.5 mm²      |
| Area of steel provided       | 2488.1 mm²      |
| Percentage provided          | 1.6423 %        |
| Weight of steel provided     | 19.532 kg/m     |
| Max.permissible spacing      | 148 mm          |
| Link size assumed            | 8 mm            |
| COMPRESSION | Characteristic strength | 425 N/mm² |
| REINFORCEMENT | Diameter of bars | 12 mm |
| SUMMARY | Number of bars | 11 |
| | arranged in a single layer | |
| | Cover to all steel | 30 mm |
| | Area of steel required | 1234.7 mm² |
| | Area of steel provided | 1244.1 mm² |
| | Percentage provided | 0.82117 % |
| | Weight of steel provided | 9.766 kg/m |
| Minimum poss.link size | 6 mm |
| Maximum spacing of links | 144 mm |

**Check on span/effective depth ratio**

| Basic ratio for continuous beam | $bs'd=26$ see Table 3.9 |
| As applied-moment factor | $M'bd2=M*1000*1000/(b*d^2)=6.5909$ N/mm² |
| Mod.factor for tension steel from equation 7 (Table 3.10) | $modf1=0.55+(477-fs)/(120*(.9+M'bd2))=0.72752$ |
| Area of comp.steel provided | $As'pr=1244.1$ mm² |
| Percentage of compression steel | $per'n=100*As'pr/(b*d)=1.1844$ % |
| From equation 9 of BS8110, with percentage of comp.steel=1.1844 %, Mod.factor for compression steel | $modf2=1+per'n/(3+per'n)=1.283$ |
| Maximum permissible span/effective-depth ratio | $ps'd=bs'd*modf1*modf2=24.27$ |
| Span of beam (see Cls.3.4.1.2-4) | $span=2.4$ m |
| Actual span/effective-depth ratio as $d=1000*span/d=23.077$ | |
| As this does not exceed 24.27, this is Acceptable. | |

**Bending-moment capacities at steel curtailment points**

The number of tension bars is even, so reduce in pairs.

With 20 tension bars, effective steel area $Asrd=2261.9$ mm²
Reduced design moment $Mred=M*nbtr/nbart=65.455$ kNm
Reduced area of comp.steel reqd.
Provide 2 compression bars, giving an area of $As'rd=2261.9$ mm²
Provide 2 compression bars, giving an area of $As'rd=As'rd=2261.9$ mm²

With 18 tension bars, effective steel area $Asrd=2035.8$ mm²
Reduced design moment $Mred=M*nbtr/nbart=58.909$ kNm
No compression steel is now required.

With 16 tension bars, effective steel area $Asrd=1809.6$ mm²
Reduced design moment $Mred=M*nbtr/nbart=52.364$ kNm

With 14 tension bars, effective steel area $Asrd=1583.4$ mm²
Reduced design moment $Mred=M*nbtr/nbart=45.818$ kNm
With 12 tension bars, effective steel area $A_{sr}=1357.2$ mm$^2$
Reduced design moment $M_{red}=M*nbtr/nbart=39.273$ kNm

With 10 tension bars, effective steel area $A_{sr}=1131$ mm$^2$
Reduced design moment $M_{red}=M*nbtr/nbart=32.727$ kNm

With 8 tension bars, effective steel area $A_{sr}=904.78$ mm$^2$
Reduced design moment $M_{red}=M*nbtr/nbart=26.182$ kNm

With 6 tension bars, effective steel area $A_{sr}=678.58$ mm$^2$
Reduced design moment $M_{red}=M*nbtr/nbart=19.636$ kNm

With 4 tension bars, effective steel area $A_{sr}=452.39$ mm$^2$
Reduced design moment $M_{red}=M*nbtr/nbart=13.091$ kNm

With 2 tension bars, effective steel area $A_{sr}=226.19$ mm$^2$
Reduced design moment $M_{red}=M*nbtr/nbart=6.5455$ kNm

No further curtailment is possible. However, bars must extend beyond the theoretical curtailment points as required below.

Min. extension to tension bars equal to $12*bar$ diameter, i.e. 144 mm, as this is not less than the effective depth $d$ of 104 mm.

Min. extension to comp. bars equal to $12*bar$ diameter, i.e. 144 mm as this is not less than the effective depth $d$ of 104 mm.

**Anchorage and lap lengths - Clause 3.12.8 of BS8110: Part 1**

Plain bars: bond coefficient $\beta=0.28$ (Table 3.26)

Tension reinforcement:
Force in tension bar at yield $F_t=fy/\gamma_S*\pi*dia^2/4=41797$ N
Ultimate anchorage bond stress $f_{bu}=\beta*\sqrt{f_{cu}}=1.7709$ N/mm$^2$
Anchorage and tension lap length $lap=F_t/(\pi*dia*f_{bu})=626.07$ mm

$1.4* $ tension lap length = 880 mm (rounded)
$2* $ tension lap length = 1260 mm (rounded)

Compression reinforcement:

Plain bars: bond coefficient $\beta=0.35$ (Table 3.26)

Force in comp. bar at yield $F_{sc}=fy/\gamma_S*\pi*diac^2/4=41797$ N
Ultimate anchorage bond stress $f_{bu}=\beta*\sqrt{f_{cu}}=2.2136$ N/mm$^2$
Anchorage length in compression $lac=F_{sc}/(\pi*diac*f_{bu})=500.86$ mm

Compression lap length $(1.25*lac) = 630$ mm (rounded)

**Shear reinforcement**

Shear calculations are in accordance with Clauses 3.4.5 of Code

Location for shear calculation: 100 mm from support
Effective breadth for shear \( bv = 1010 \text{ mm} \)

Shear force due to ultimate load \( V = 50 \text{ kN} \)

Distance from support \( av = 100 \text{ mm} \)

No. of tension bars effective at section \( nbars = 22 \)

As \( bv \) exceeds 350 mm note the conditions in Clause 3.4.5.5:

i) that no longitudinal bar should be more than 150 mm from a vertical leg, and

ii) (because \( bv \) exceeds \( d \)), that the transverse spacing of the legs must not exceed the effective depth \( d \) (i.e. 104 mm).

Number of legs to be provided \( nlegs = 2 \)

Chosen link spacing \( sv' = 50 \text{ mm} \)

Use 8 mm links (two legs), spaced at 50 mm centres along beam.

When detailing steel, watch carefully the requirements of Cl.3.4.5.5.

<table>
<thead>
<tr>
<th>SHEAR</th>
<th>Characteristic strength</th>
<th>250 \text{ N/mm}²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of legs</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>50 mm</td>
</tr>
<tr>
<td></td>
<td>Approx. weight of links</td>
<td>16.54 \text{ kg/m}</td>
</tr>
</tbody>
</table>
Location: Large beam - mild steel - no compression steel

Bending in rectangular beams with optional calculations for shear, lap lengths, bar curtailment & limiting span/effective depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gamma_S=1.15
Ultimate BM before redistribution Mbef=5000 kNm
Beam being analysed is considered as continuous.
Design moment (after redistrib.) M=5000 kNm
Characteristic concrete strength fcu=50 N/mm²
Max.aggregate size (for bar spc) hagg=15 mm
Char.strength of long'1 bars fy=250 N/mm²
Longitudinal reinforcement is mild steel.
Diameter of tension bars dia=40 mm
Diameter of link legs dial=16 mm
Char.strength of link steel fyv=250 N/mm²
Mild steel shear reinforcement.
Designated exposure class is XC3
Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=50 mm
Overall depth of section h=1000 mm
Effective depth of section d=874 mm
Breadth of section b=1010 mm

Longitudinal reinforcement

| TENSION | Characteristic strength 250 N/mm² |
| REINFORCEMENT | Diameter of bars 40 mm |
| SUMMARY | Number of bars 26 |
|          | arranged in 3 layers. |
|          | Cover to all steel 50 mm |
|          | Area of steel required 31877 mm² |
|          | Area of steel provided 32673 mm² |
|          | Percentage provided 3.2349 % |
|          | Weight of steel provided 256.48 kg/m |
|          | Max.permissible spacing 289 mm |
|          | Link size assumed 16 mm |

As overall depth h exceeds 750 mm bars to control cracking must be provided on side faces of beam (Clause 3.12.5.4 & 3.12.11.2.6).
Check on span/effective depth ratio

Basic ratio for continuous beam  \( bs'd=26 \)  see Table 3.9
As applied-moment factor  \( M'bd2=M*1000*1000/(b*d^2)=6.4808 \) N/mm²
Mod.factor for tension steel from equation 7 (Table 3.10)  
\[
\text{modf1}=0.55+(477-fs)/(120*(.9+M'bd2)) =0.90497
\]
Mod.factor for no comp.steel  \( \text{modf2}=1 \)
Maximum permissible span/effective-depth ratio  \( ps'd=bs'd*\text{modf1}*\text{modf2}=23.529 \)
Span of beam (see C1s.3.4.1.2-4)  \( \text{span}=14 \) m
As span exceeds 10 m and partitions/finishes are sensitive to deflection multiply basic ratio by 10/span (in m)  \( ps'd=ps'd*10/\text{span} \)
\( =16.807 \)
Actual span/effective-depth ratio as  \( ps'd=1000*\text{span}/d=16.018 \)
As this does not exceed 16.807, this is Acceptable.

Bending-moment capacities at steel curtailment points

The number of tension bars is even, so reduce in pairs.

With 24 tension bars, effective steel area  \( Asrd=30159 \) mm²
Reduced design moment  \( Mred=M*nbtr/nbart=4615.4 \) kNm

With 22 tension bars, effective steel area  \( Asrd=27646 \) mm²
Reduced design moment  \( Mred=M*nbtr/nbart=4230.8 \) kNm

With 20 tension bars, effective steel area  \( Asrd=25133 \) mm²
Reduced design moment  \( Mred=M*nbtr/nbart=3846.2 \) kNm

With 18 tension bars, effective steel area  \( Asrd=22619 \) mm²
Reduced design moment  \( Mred=M*nbtr/nbart=3461.5 \) kNm

With 16 tension bars, effective steel area  \( Asrd=20106 \) mm²
Reduced design moment  \( Mred=M*nbtr/nbart=3076.9 \) kNm

With 14 tension bars, effective steel area  \( Asrd=17593 \) mm²
Reduced design moment  \( Mred=M*nbtr/nbart=2692.3 \) kNm

With 12 tension bars, effective steel area  \( Asrd=15080 \) mm²
Reduced design moment  \( Mred=M*nbtr/nbart=2307.7 \) kNm

With 10 tension bars, effective steel area  \( Asrd=12566 \) mm²
Reduced design moment  \( Mred=M*nbtr/nbart=1923.1 \) kNm

With 8 tension bars, effective steel area  \( Asrd=10053 \) mm²
Reduced design moment  \( Mred=M*nbtr/nbart=1538.5 \) kNm

With 6 tension bars, effective steel area  \( Asrd=7539.8 \) mm²
Reduced design moment  \( Mred=M*nbtr/nbart=1153.8 \) kNm
With 4 tension bars, effective steel area $A_{sr}=5026.5 \, \text{mm}^2$
Reduced design moment $M_{red}=M*nbtr/nbart=769.23 \, \text{kNm}$

With 2 tension bars, effective steel area $A_{sr}=2513.3 \, \text{mm}^2$
Reduced design moment $M_{red}=M*nbtr/nbart=384.62 \, \text{kNm}$

No further curtailment is possible. However, bars must extend beyond the theoretical curtailment points as required below.

Min. extension to tension bars equal to effective depth $d$ of 874 mm as this is not less than $12 \times$ tension bar diameter, i.e. 480 mm.

**Anchorage and lap lengths - Clause 3.12.8 of BS8110: Part 1**

Plain bars: bond coefficient $\beta=0.28$ (Table 3.26)

Tension reinforcement:

Force in tension bar at yield $F_t=f_y/gamma_S*PI*dia^2/4=273182 \, \text{N}$
Ultimate anchorage bond stress $f_{bu}=\beta*SQR(f_{cu})=1.9799 \, \text{N/mm}^2$
Anchorage and tension lap length $l_{ap}=F_t/(PI*dia*f_{bu})=1098 \, \text{mm}$
1.4 * tension lap length = 1500 mm (rounded)
2 * tension lap length = 2200 mm (rounded)

**Shear reinforcement**

Shear calculations are in accordance with Clauses 3.4.5 of Code

Location for shear calculation: 0.457 m from support
Effective breadth for shear $b_v=1010 \, \text{mm}$
Shear force due to ultimate load $V=1450 \, \text{kN}$
Distance from support $a_v=457 \, \text{mm}$
No. of tension bars effective at section $n_bars=26$
As $b_v$ exceeds 350 mm note the conditions in Clause 3.4.5.5:

i) that no longitudinal bar should be more than 150 mm from a vertical leg, and

ii) (because $b_v$ exceeds $d$), that the transverse spacing of the legs must not exceed the effective depth $d$ (i.e. 874 mm).

Number of legs to be provided $n_{legs}=2$
Chosen link spacing $s_v'=150 \, \text{mm}$

Use 16 mm links (two legs), spaced at 150 mm centres along beam.

When detailing steel, watch carefully the requirements of Cl.3.4.5.5.

**Summary**

- **Shear:** Characteristic strength 250 N/mm²
- **Reinforcement:** Diameter of links 16 mm
- **Summary:** Number of legs 2
- **Spacing:** 150 mm
- **Approx. weight of links:** 38.425 kg/m
Location: Ex1 - Beam with compression reinforcement

Bending in rectangular beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45\*d.

Design BM before redistribution \( M_{\text{bef}} = 150 \text{ kNm} \)
Beam being analysed is considered as non-continuous.
Char yield strength of reinforc'ment \( f_{\text{yk}} = 500 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{\text{agg}} = 20 \text{ mm} \)
Diameter of tension bars \( \text{dia} = 25 \text{ mm} \)
Diameter of link legs \( \text{dia}_l = 8 \text{ mm} \)

Section properties

| Effective depth of section | \( d = 340 \text{ mm} \) |
| Diameter of compression bars | \( \text{dia}_c = 12 \text{ mm} \) |
| Depth to compression steel | \( d_2 = 45 \text{ mm} \) |

DESIGN

| Overall depth | 400 mm |
| Effective depth | 340 mm |
| Width of section | 250 mm |
| Parameter K | 0.173 |
| Parameter K' | 0.1684 |
| Lever arm ratio z/d | 0.8185 |

TENSION REINFORCEMENT

| Steel area provided | 1473 \text{ mm}^2 |
| Diameter of bars | 25 mm |
| Number of bars | 3 |
| Steel area required | 1238 \text{ mm}^2 |
| Steel percentage req. | 1.456 \% |
| Minimum area of steel | 128 \text{ mm}^2 |
| Maximum area of steel | 4000 \text{ mm}^2 |

COMPRESSION REINFORCEMENT

| Steel area provided | 226.2 \text{ mm}^2 |
| Diameter of bars | 12 mm |
| Number of bars | 2 |
| Steel area required | 31.16 \text{ mm}^2 |
| Minimum area required | 200 \text{ mm}^2 |

Deflection check

| Effective span of beam | \( L = 5.4 \text{ m} \) |
| Actual span to depth ratio | \( l'd = L \times 1000/d = 15.88 \) |
| Allowable span/depth ratio | \( l'da = k \times N \times F1 \times F2 \times F3 = 17 \) |
DESIGN

Actual l/d ratio 15.88

SUMMARY

Basic l/d ratio 14.29

DEFLECTION

Span factor 1
Flange beam factor F1 1
Long spans factor F2 1
Steel stress factor F3 1.19
Allowable l/d ratio 17

Shear check

Location for shear calculation: At simple support
Shear force due to ultimate load $V_{Ed} = 110$ kN
Design shear stress $v_{Ed} = \frac{V_{Ed} \times 10^3}{0.9 \times bw \times d} = 1.438$ N/mm²
Concrete strut capacity $v_{Rdm} = 0.124 \times f_{ck} (1 - f_{ck}/250) = 3.274$ N/mm²
Angle of inclination of strut $\theta' = 22°$
Number of shear legs $n_{sl} = 2$
Chosen spacing $s = 250$ mm

SHEAR SUMMARY

Design shear force 110 kN
Design shear stress 1.438 N/mm²
Concrete strut capacity 3.274 N/mm²
Area/spacing ratio 0.3307
Diameter of links 8 mm
Area of links 100 mm²
Maximum spacing 255 mm
(based on 0.75d)
Actual spacing 250 mm
Location: Ex2 - Redistribution example

Bending in rectangular beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Design BM before redistribution $M_{\text{bef}}=460$ kNm
Beam being analysed is considered as continuous.
Section considered has a hogging moment.
Design moment (after redistrib.) $M=370$ kNm
Char yield strength of reinf’ment $f_{yk}=500$ N/mm²
Max.aggregate size (for bar spc.) $h_{agg}=20$ mm
Diameter of tension bars $\text{dia}=25$ mm
Diameter of link legs $d_{\text{ial}}=8$ mm

Section properties

Effective depth of section $d=525$ mm
Diameter of compression bars $d_{\text{iac}}=20$ mm
Depth to compression steel $d_{2}=100$ mm

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth 600 mm</th>
</tr>
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<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth 525 mm</td>
</tr>
<tr>
<td></td>
<td>Width of section 300 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K 0.179</td>
</tr>
<tr>
<td></td>
<td>Parameter K' 0.1547</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio $z/d$ 0.8369</td>
</tr>
<tr>
<td>TENSION REINFORCEMENT</td>
<td>Steel area provided 1963 mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars 25 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars 4</td>
</tr>
<tr>
<td></td>
<td>Steel area required 1946 mm²</td>
</tr>
<tr>
<td></td>
<td>Steel percentage req. 1.235 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel 210.1 mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel 7200 mm²</td>
</tr>
<tr>
<td>COMPRESSION REINFORCEMENT</td>
<td>Steel area provided 628.3 mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars 20 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars 2</td>
</tr>
<tr>
<td></td>
<td>Steel area required 316.1 mm²</td>
</tr>
<tr>
<td></td>
<td>Minimum area required 360 mm²</td>
</tr>
</tbody>
</table>

Shear check

Location for shear calculation: 500 mm from support
Shear force due to ultimate load $V_{\text{Ed}}=278$ kN
Design shear stress $\nu_{\text{Ed}}=V_{\text{Ed}}*10^3/(0.9*\text{bw}*d)=1.961$ N/mm²
Concrete strut capacity
\[ v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 2.79 \text{ N/mm}^2 \]

Angle of inclination of strut
\[ \theta' = 22^\circ \]

Number of shear legs
\[ n_{sl} = 2 \]

Chosen spacing
\[ s = 175 \text{ mm} \]

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force</td>
<td>278 kN</td>
</tr>
<tr>
<td>Design shear stress</td>
<td>1.961 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>2.79 N/mm²</td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.5413</td>
</tr>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>Area of links</td>
<td>100 mm²</td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>185 mm</td>
</tr>
<tr>
<td>Actual spacing</td>
<td>175 mm</td>
</tr>
</tbody>
</table>
Location: Ex3 - Shallow beam

Calculations in rectangular beams

Design BM before redistribution $M_{bef}=425$ kNm
Beam being analysed is considered as continuous.
Char yield strength of reinforcement $f_yk=500$ N/mm²
Max. aggregate size (for bar spc.) $h_{agg}=20$ mm
Diameter of tension bars $d_{ia}=25$ mm
Diameter of link legs $d_{ial}=8$ mm

Section properties

Effective depth of section $d=340$ mm
Diameter of compression bars $d_{iac}=16$ mm
Depth to compression steel $d2=50$ mm

| DESIGN | Overall depth 400 mm |
| SUMMARY | Effective depth 340 mm |
| Width of section 600 mm |
| FLEXURE | Parameter K 0.1751 |
| | Parameter K' 0.1684 |
| | Lever arm ratio $z/d$ 0.8185 |
| TENSION REINFORCEMENT | Steel area provided 3927 mm² |
| | Diameter of bars 25 mm |
| | Number of bars 8 |
| | Steel area required 3507 mm² |
| | Steel percentage req. 1.719 % |
| | Minimum area of steel 340.5 mm² |
| | Maximum area of steel 9600 mm² |
| COMPRESSION REINFORCEMENT | Steel area provided 603.2 mm² |
| | Diameter of bars 16 mm |
| | Number of bars 3 |
| | Steel area required 128.4 mm² |
| | Minimum area required 480 mm² |

Deflection check

Effective span of beam $L=5.2$ m
Actual span to depth ratio $l'd=L*1000/d=15.29$
Allowable span/depth ratio $l'da=k*N*F1*F2*F3=20.86$
### DESIGN
- Actual l/d ratio: 15.29

### SUMMARY
- Basic l/d ratio: 14.33

### DEFLECTION
- Span factor: 1.3
- Flange beam factor F1: 1
- Long spans factor F2: 1
- Steel stress factor F3: 1.12
- Allowable l/d ratio: 20.86

### Shear check
- Location for shear calculation: 500mm from support
- Shear force due to ultimate load: VEd=278 kN

\[
\text{Design shear stress} = \frac{V_{Ed} \times 10^3}{0.9 \times b \times d} = 1.514 \text{ N/mm}^2
\]

\[
\text{Concrete strut capacity} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 3.732 \text{ N/mm}^2
\]

\[
\text{Angle of inclination of strut} = \theta' = 22^\circ
\]

### SHEAR SUMMARY
- Number of shear legs: ns1=4
- Chosen spacing: s=225 mm

<table>
<thead>
<tr>
<th>Design shear force</th>
<th>278 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear stress</td>
<td>1.514 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.732 N/mm²</td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.8358</td>
</tr>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>Area of links</td>
<td>201 mm²</td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>240 mm</td>
</tr>
<tr>
<td>Actual spacing</td>
<td>225 mm</td>
</tr>
</tbody>
</table>
Location: Ex4 - Minimum arrangement

Bending in rectangular beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Design BM before redistribution Mbef=12 kNm
Beam being analysed is considered as continuous. Section considered has a sagging moment.
Char yield strength of reinforcement fyk=500 N/mm²
Max. aggregate size (for bar spc.) hagg=20 mm
Diameter of tension bars dia=12 mm
Diameter of link legs dial=8 mm

Section properties

Effective depth of section d=345 mm

| DESIGN | Overall depth | 400 mm |
| SUMMARY | Effective depth | 345 mm |
| Width of section | 200 mm |
| FLEXURE | Parameter K | 0.0202 |
| Parameter K' | 0.1684 |
| Lever arm ratio z/d | 0.95 |
| TENSION REINFORCEMENT | Steel area required | 92.03 mm² |
| Steel area provided | 226.2 mm² |
| Diameter of bars | 12 mm |
| Number of bars | 2 |
| Steel percentage req. | 0.1334 % |
| Minimum area of steel | 92.03 mm² |
| Maximum area of steel | 3200 mm² |

Deflection check

Effective span of beam L=3.4 m
Actual span to depth ratio l'd=L*1000/d=9.855
Allowable span/depth ratio l'da=k*N*F1*F2*F3=218.5
Absolute value of span/depth l'da=40*k=52

| DESIGN | Actual 1/d ratio | 9.855 |
| SUMMARY | Basic 1/d ratio | 112 |
| DEFLECTION | Span factor | 1.3 |
| | Flange beam factor F1 | 1 |
| | Long spans factor F2 | 1 |
| | Steel stress factor F3 | 1.5 |
| | Allowable 1/d ratio | 52 |
Shear check

Location for shear calculation: 300mm from support
Shear force due to ultimate load VEd=27 kN
Design shear stress vEd=VEd*10^3/(0.9*bw*d)=0.4348 N/mm²
Concrete strut capacity vRdm=0.124*fck*(1-fck/250)=2.79 N/mm²
Angle of inclination of strut theta'=22°
Number of shear legs nsl=2
Chosen spacing s=250 mm

SHEAR SUMMARY

Design shear force 27 kN
Design shear stress 0.4348 N/mm²
Concrete strut capacity 2.79 N/mm²
Area/spacing ratio 0.16
Diameter of links 8 mm
Area of links 100 mm²
Maximum spacing (based on 0.75d) 258.8 mm
Actual spacing 250 mm
Location: Small beam - high-yield steel - no compression steel

Design of flanged beams to resist bending

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

Moment before redistribution \( M_{bef} = 12 \, \text{kNm} \)

Beam being analysed is considered as non-continuous.

Characteristic concrete strength \( f_{cu} = 35 \, \text{N/mm}^2 \)

Max. aggregate size (for bar spc.) \( h_{agg} = 15 \, \text{mm} \)

Char. strength of long'1 bars \( f_y = 500 \, \text{N/mm}^2 \)

Longitudinal reinforcement is high-yield steel.

Diameter of tension bars \( \text{dia} = 12 \, \text{mm} \)

Diameter of link legs \( \text{dial} = 6 \, \text{mm} \)

Designated exposure class is XC1

Specified fixing tolerance \( \text{tol} = 10 \, \text{mm} \)

Nominal concrete cover \( \text{cover} = 25 \, \text{mm} \)

Overall depth of section \( h = 275 \, \text{mm} \)

Effective depth of section \( d = 238 \, \text{mm} \)

Breadth of flange \( b = 400 \, \text{mm} \)

Breadth of rib \( bw = 200 \, \text{mm} \)

Thickness of flange \( hf = 100 \, \text{mm} \)

### Longitudinal reinforcement

| TENSION | Characteristic strength 500 N/mm² |
| REINFORCEMENT | Diameter of bars 12 mm |
| SUMMARY | Number of bars 2 |
| | arranged in a single layer |
| | Cover to all steel 25 mm |
| | Area of steel required 122.07 mm² |
| | Area of steel provided 226.19 mm² |
| | Percentage provided: |
| | of gross section 0.30159 % |
| | of web area 0.41126 % |
| | Weight of steel provided 1.7756 kg/m |
| | Max. permissible spacing 261 mm |
| | Link size assumed 6 mm |
Location: Narrow deep beam - mild steel - no compression steel

Design of flanged beams to resist bending

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution $M_{bef}=1000$ kNm
Beam being analysed is considered as continuous.
Design moment (after redistrib.) $M=1000$ kNm
Characteristic concrete strength $f_{cu}=30$ N/mm²
Max.aggregate size (for bar spc.) $h_{agg}=15$ mm
Char.strength of long'1 bars $f_y=250$ N/mm²
Longitudinal reinforcement is mild steel.
Diameter of tension bars $d_{ia}=25$ mm
Diameter of link legs $d_{ial}=12$ mm
Designated exposure class is XC2
Specified fixing tolerance $tol=10$ mm
Nominal concrete cover $cover=35$ mm
Overall depth of section $h=950$ mm
Effective depth of section $d=800$ mm
Breadth of flange $b=1500$ mm
Breadth of rib $bw=300$ mm
Thickness of flange $hf=150$ mm

Longitudinal reinforcement

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength 250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 25 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars 13</td>
</tr>
<tr>
<td></td>
<td>arranged in 4 layers</td>
</tr>
<tr>
<td></td>
<td>Cover to all steel 35 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required 6052.6 mm²</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided 6381.4 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided:</td>
</tr>
<tr>
<td></td>
<td>of gross section 1.3723 %</td>
</tr>
<tr>
<td></td>
<td>of web area 2.2391 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided 50.094 kg/m</td>
</tr>
<tr>
<td></td>
<td>Max.permmissible spacing 297 mm</td>
</tr>
<tr>
<td></td>
<td>Link size assumed 12 mm</td>
</tr>
</tbody>
</table>

As overall depth $h$ exceeds 750 mm, bars to control cracking must be provided on side faces of beam (refer to Code Clause 3.12.5.4 and 3.12.11.2.9).
**Location: Shallow beam - square twist - with compression steel**

**Design of flanged beams to resist bending**

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution \( M_{bef} = 95 \text{ kNm} \)
Beam being analysed is considered as continuous.
Design moment (after redistrib.) \( M = 76 \text{ kNm} \)
Characteristic concrete strength \( f_{cu} = 35 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{agg} = 15 \text{ mm} \)
Char.strength of long'1 bars \( f_y = 425 \text{ N/mm}^2 \)
Steel strength other than those given in Table 3.1 of BS8110.
Min.percentage when \( bw/b \geq 0.4 \) \( \text{permn1} = 0.16 \% \)
Min.percentage when \( bw/b < 0.4 \) \( \text{permn2} = 0.22 \% \)
Diameter of tension bars \( \text{d}{\text{ia}} = 20 \text{ mm} \)
Diameter of link legs \( \text{d}{\text{ia}}l = 8 \text{ mm} \)
Nominal concrete cover \( \text{cover} = 30 \text{ mm} \)
Overall depth of section \( h = 175 \text{ mm} \)
Effective depth of section \( d = 127 \text{ mm} \)
Breadth of flange \( b = 1000 \text{ mm} \)
Breadth of rib \( bw = 300 \text{ mm} \)
Thickness of flange \( hf = 75 \text{ mm} \)

**Longitudinal reinforcement**

Depth to compression bars \( d' = 44 \text{ mm} \)

**TENSION**
**REINFORCEMENT**
**SUMMARY**

Characteristic strength \( 425 \text{ N/mm}^2 \)
Diameter of bars \( 20 \text{ mm} \)
Number of bars \( 7 \)
arranged in 2 layers
Cover to all steel \( 30 \text{ mm} \)
Area of steel required \( 1981.1 \text{ mm}^2 \)
Area of steel provided \( 2199.1 \text{ mm}^2 \)
Percentage provided:
of gross section \( 2.0944 \% \)
of web area \( 4.1888 \% \)
Weight of steel provided \( 17.263 \text{ kg/m} \)
Max.permissible spacing \( 147 \text{ mm} \)
Link size assumed \( 8 \text{ mm} \)
COMPRESSION

REINFORCEMENT

SUMMARY

Characteristic strength 425 N/mm²
Diameter of bars 12 mm
Number of bars 3
arranged in a single layer
Cover to all steel 30 mm
Area of steel required 300 mm²
Area of steel provided 339.29 mm²
Percentage provided:
  of gross section 0.32314 %
  of web area 0.64627 %
  of flange area 0.45239 %
Weight of steel provided 2.6634 kg/m
Minimum poss. link size 6 mm
Maximum spacing of links 144 mm

WARNING:
As more than 1 layer of tension bars required, you should
determine a revised effective depth corresponding to the
position of the centroid of this reinforcement, and rerun
this proforma.
**Location:** Large beam - high-yield steel - with compression steel

**Design of flanged beams to resist bending**

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Moment before redistribution $M_{bef}=9567$ kNm

Beam being analysed is considered as continuous.

Design moment (after redistrib.) $M=9345$ kNm

Characteristic concrete strength $f_{cu}=50$ N/mm$^2$

Max.aggregate size (for bar spc.) $h_{agg}=15$ mm

Char.strength of long'1 bars $f_y=500$ N/mm$^2$

Longitudinal reinforcement is high-yield steel.

Diameter of tension bars $d_{ia}=40$ mm

Diameter of link legs $d_{ial}=12$ mm

Designated exposure class is XC3

Specified fixing tolerance $tol=10$ mm

Nominal concrete cover $cover=50$ mm

Overall depth of section $h=1000$ mm

Effective depth of section $d=918$ mm

Breadth of flange $b=2000$ mm

Breadth of rib $bw=500$ mm

Thickness of flange $hf=175$ mm

**Longitudinal reinforcement**

Diameter of compression bars $d_{iac}=20$ mm

Depth to compression bars $d'=72$ mm

**TENSION**

**REINFORCEMENT**

**SUMMARY**

- Characteristic strength $500$ N/mm$^2$
- Diameter of bars $40$ mm
- Number of bars $22$
- arranged in 6 layers
- Cover to all steel $50$ mm
- Area of steel required $27319$ mm$^2$
- Area of steel provided $27646$ mm$^2$
- Percentage provided:
  - of gross section $3.6257\%$
  - of web area $5.5292\%$
- Weight of steel provided $217.02$ kg/m
- Max.permissible spacing $139$ mm
- Link size assumed $12$ mm
COMPRESSION

REINFORCEMENT

SUMMARY

Characteristic strength  500 N/mm²
Diameter of bars         20 mm
Number of bars           10
arranged in a single layer
Cover to all steel       50 mm
Area of steel required   3135.3 mm²
Area of steel provided   3141.6 mm²
Percentage provided:
  of gross section        0.41201 %
  of web area             0.62832 %
  of flange area          0.8976 %
Weight of steel provided 24.662 kg/m
Minimum poss.link size   6 mm
Maximum spacing of links 240 mm

WARNING:
As more than 1 layer of tension bars required, you should determine a revised effective depth corresponding to the position of the centroid of this reinforcement, and rerun this proforma.
Design of flanged beams to resist bending

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution Mbef=1148 kNm
Beam being analysed is considered as continuous.
Characteristic concrete strength fcu=30 N/mm²
Max.aggregate size (for bar spc.) hagg=15 mm
Char.strength of long'l bars fy=500 N/mm²
Longitudinal reinforcement is high-yield steel.

Diameter of tension bars dia=32 mm
Diameter of link legs dial=8 mm
Designated exposure class is XC1
Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=25 mm
Overall depth of section h=750 mm
Effective depth of section d=668 mm
Breadth of flange b=1780 mm
Breadth of rib bw=350 mm
Thickness of flange hf=300 mm

**Longitudinal reinforcement**

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength 500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 32 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars 6</td>
</tr>
<tr>
<td></td>
<td>arranged in 2 layers</td>
</tr>
<tr>
<td></td>
<td>Cover to all steel 25 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required 4190.5 mm²</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided 4825.5 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided:</td>
</tr>
<tr>
<td></td>
<td>of gross section 0.69783 %</td>
</tr>
<tr>
<td></td>
<td>of web area 1.8383 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided 37.88 kg/m</td>
</tr>
<tr>
<td></td>
<td>Max.permissible spacing 162 mm</td>
</tr>
<tr>
<td></td>
<td>Link size assumed 8 mm</td>
</tr>
</tbody>
</table>
**Location: Ex1 - Continuous T-beam**

**Bending in flanged beams**

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Distance between beam supports \( L = 8.2 \) m  
Moment before redistribution \( M_{bef} = 600 \) kNm  
Beam being analysed is considered as continuous.  
Section considered has a sagging moment.  
Overall depth of section \( h = 650 \) mm  
Average web width \( bw = 350 \) mm  
Flange thickness \( hf = 200 \) mm  
Char yield strength of reinft \( f_{yk} = 500 \) N/mm²  
Max.aggregate size (for bar spc.) \( h_{agg} = 20 \) mm  
Diameter of tension bars \( dia = 25 \) mm  
Diameter of link legs \( dial = 10 \) mm  
Effective depth of section \( d = 560 \) mm  
Section being considered is a T-beam.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth</th>
<th>650 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth</td>
<td>560 mm</td>
</tr>
<tr>
<td></td>
<td>Flange width</td>
<td>2194 mm</td>
</tr>
<tr>
<td></td>
<td>Web width</td>
<td>350 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K</td>
<td>0.0291</td>
</tr>
<tr>
<td></td>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio z/d</td>
<td>0.95</td>
</tr>
<tr>
<td>TENSION REINFORCEMENT</td>
<td>Steel area required</td>
<td>2594 mm²</td>
</tr>
<tr>
<td></td>
<td>Steel area provided</td>
<td>2945 mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Steel percentage req.</td>
<td>0.2111 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel</td>
<td>295.2 mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel</td>
<td>57044 mm²</td>
</tr>
</tbody>
</table>

**Deflection**

Actual span to depth ratio \( l'd = L *1000/d = 14.64 \)  
Allowable span/depth ratio \( l'da=k*N*F1*F2*F3 = 68.14 \)  
Absolute value of span/depth \( l'da = 40*k = 52 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Actual 1/d ratio</th>
<th>14.64</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Basic 1/d ratio</td>
<td>67.6</td>
</tr>
<tr>
<td>DEFLECTION</td>
<td>Span factor</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>Flange beam factor F1</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>Long spans factor F2</td>
<td>0.8537</td>
</tr>
<tr>
<td></td>
<td>Steel stress factor F3</td>
<td>1.135</td>
</tr>
<tr>
<td></td>
<td>Allowable 1/d ratio</td>
<td>52</td>
</tr>
</tbody>
</table>
Shear check

Location for shear calculation: 400mm from support
Shear force due to ultimate load $V_{Ed} = 267$ kN
Design shear stress $\nu_{Ed} = V_{Ed} \times 10^3 / (0.9 \times b \times d) = 1.514$ N/mm²
Concrete strut capacity $\nu_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck} / 250) = 3.274$ N/mm²
Angle of inclination of strut $\theta' = 22°$
Number of shear legs $n_{sl} = 4$
Chosen spacing $s = 250$ mm

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force $V_{Ed}$</td>
<td>267 kN</td>
</tr>
<tr>
<td>Design shear stress $\nu_{Ed}$</td>
<td>1.514 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity $\nu_{Rdm}$</td>
<td>3.274 N/mm²</td>
</tr>
<tr>
<td>Area/spacing ratio $\nu_{A/s}$</td>
<td>0.4874</td>
</tr>
<tr>
<td>Diameter of links $d$</td>
<td>10 mm</td>
</tr>
<tr>
<td>Area of links $A$</td>
<td>314 mm²</td>
</tr>
<tr>
<td>Maximum spacing $s$ (based on 0.75$d$)</td>
<td>420 mm</td>
</tr>
<tr>
<td>Actual spacing $s$</td>
<td>250 mm</td>
</tr>
</tbody>
</table>
Location: Ex2 - Support condition

Bending in flanged beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Distance between beam supports \( L = 8.2 \text{ m} \)
Moment before redistribution \( M_{\text{bef}} = 650 \text{ kNm} \)
Beam being analysed is considered as continuous.
Section considered has a hogging moment and will be designed as a rectangular section.

Design moment (after redistrib.) \( M = 570 \text{ kNm} \)
Overall depth of section \( h = 650 \text{ mm} \)
Average web width \( bw = 350 \text{ mm} \)
Flange thickness \( hf = 200 \text{ mm} \)
Char yield strength of reinft \( f_{yk} = 500 \text{ N/mm}^2 \)
Max. aggregate size (for bar spc.) \( h_{\text{agg}} = 20 \text{ mm} \)
 Diameter of tension bars \( \text{dia} = 25 \text{ mm} \)
 Diameter of link legs \( \text{dial} = 10 \text{ mm} \)
Effective depth of section \( d = 570 \text{ mm} \)

| DESIGN | Overall depth | 650 mm |
| SUMMARY | Effective depth | 570 mm |
| Width of section | 350 mm |
| FLEXURE | Parameter K | 0.1671 |
| Parameter K' | 0.1684 |
| Lever arm ratio z/d | 0.8203 |
| TENSION REINFORCEMENT | Steel area required | 2804 mm² |
| | Steel area provided | 2945 mm² |
| | Diameter of bars | 25 mm |
| | Number of bars | 6 |
| | Steel percentage req. | 1.405 % |
| | Minimum area of steel | 300.5 mm² |
| | Maximum area of steel | 9100 mm² |

Shear check

Location for shear calculation: 500 mm from support
Shear force due to ultimate load \( V_{\text{Ed}} = 367 \text{ kN} \)
Design shear stress \( v_{\text{Ed}} = V_{\text{Ed}} \times 10^{3} / (0.9 \times bw \times d) = 2.044 \text{ N/mm}^2 \)
Concrete strut capacity \( v_{\text{Rdm}} = 0.124 \times f_{ck} \times (1 - f_{ck} / 250) = 3.274 \text{ N/mm}^2 \)
Angle of inclination of strut \( \theta' = 22° \)
Number of shear legs \( n_{\text{sl}} = 2 \)
Chosen spacing \( s = 225 \text{ mm} \)
<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force</td>
<td>367 kN</td>
</tr>
<tr>
<td>Design shear stress</td>
<td>2.044 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.274 N/mm²</td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.6582</td>
</tr>
<tr>
<td>Diameter of links</td>
<td>10 mm</td>
</tr>
<tr>
<td>Area of links</td>
<td>157 mm²</td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>238 mm</td>
</tr>
<tr>
<td>Actual spacing</td>
<td>225 mm</td>
</tr>
</tbody>
</table>
**Location:** Ex3 - Continuous L-beam

**Bending in flanged beams**

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Distance between beam supports \( L = 9 \text{ m} \)

Moment before redistribution \( M_{bef} = 780 \text{ kNm} \)

Beam being analysed is considered as continuous.

Section considered has a sagging moment.

Overall depth of section \( h = 700 \text{ mm} \)

Average web width \( b_w = 350 \text{ mm} \)

Flange thickness \( h_f = 250 \text{ mm} \)

Char yield strength of reinft \( f_{yk} = 500 \text{ N/mm}^2 \)

Max.aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)

Diameter of tension bars \( d_{ia} = 32 \text{ mm} \)

Diameter of link legs \( d_{ia,l} = 10 \text{ mm} \)

Effective depth of section \( d = 600 \text{ mm} \)

Section being considered is a L-beam.

<table>
<thead>
<tr>
<th>Design</th>
<th>Overall depth</th>
<th>700 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Summary</td>
<td>Effective depth</td>
<td>600 mm</td>
</tr>
<tr>
<td></td>
<td>Flange width</td>
<td>1615 mm</td>
</tr>
<tr>
<td></td>
<td>Web width</td>
<td>350 mm</td>
</tr>
<tr>
<td>Flexure</td>
<td>Parameter K</td>
<td>0.0447</td>
</tr>
<tr>
<td></td>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio z/d</td>
<td>0.95</td>
</tr>
<tr>
<td>Tension Reinforcement</td>
<td>Steel area required</td>
<td>3147 mm²</td>
</tr>
<tr>
<td></td>
<td>Steel area provided</td>
<td>3217 mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>32 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Steel percentage req.</td>
<td>0.3248 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel</td>
<td>316.3 mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel</td>
<td>45220 mm²</td>
</tr>
</tbody>
</table>

**Deflection**

Actual span to depth ratio \( l'd = L \times 1000 / d = 15 \)

Allowable span/depth ratio \( l'da = k \times N \times F_1 \times F_2 \times F_3 = 28.79 \)

<table>
<thead>
<tr>
<th>Design</th>
<th>Actual 1/d ratio</th>
<th>15</th>
</tr>
</thead>
<tbody>
<tr>
<td>Summary</td>
<td>Basic 1/d ratio</td>
<td>34.82</td>
</tr>
<tr>
<td>Deflection</td>
<td>Span factor</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>Flange beam factor F1</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>Long spans factor F2</td>
<td>0.7778</td>
</tr>
<tr>
<td></td>
<td>Steel stress factor F3</td>
<td>1.022</td>
</tr>
<tr>
<td></td>
<td>Allowable 1/d ratio</td>
<td>28.79</td>
</tr>
</tbody>
</table>
**Location: Ex4 - Simply supported arrangement**

**Bending in flanged beams**

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Distance between beam supports \( L = 7.2 \) m
Moment before redistribution \( M_{bef} = 480 \) kNm
Beam being analysed is considered as non-continuous.
Overall depth of section \( h = 500 \) mm
Average web width \( b_w = 650 \) mm
Flange thickness \( h_f = 250 \) mm
Char yield strength of reinft \( f_y = 500 \) N/mm²
Max aggregate size (for bar spc.) \( h_{agg} = 20 \) mm
Diameter of tension bars \( d_{ia} = 25 \) mm
Diameter of link legs \( d_{ial} = 10 \) mm
Effective depth of section \( d = 425 \) mm
Section being considered is a T-beam.

**NOTE:** Clear distances vary.

**DESIGN**
Overall depth 500 mm
**SUMMARY**
Effective depth 425 mm
Flange width 2585 mm
Web width 650 mm

**FLEXURE**
Parameter K 0.0343
Parameter K' 0.1684
Lever arm ratio \( z/d \) 0.95

**TENSION REINFORCEMENT**
Steel area required 2734 mm²
Steel area provided 2945 mm²
Diameter of bars 25 mm
Number of bars 6
Steel percentage req. 0.2489 %
Minimum area of steel 416.1 mm²
Maximum area of steel 51700 mm²

**Deflection**
Actual span to depth ratio \( l'd = L \times 1000/d = 16.94 \)
Allowable span/depth ratio \( l'd = k \times N \times F_1 \times F_2 \times F_3 = 43.68 \)
Absolute value of span/depth \( l'd = 40 \times k = 40 \)

**DESIGN**
Actual 1/d ratio 16.94
**SUMMARY**
Basic 1/d ratio 52.14
**DEFLECTION**
Span factor 1
Flange beam factor \( F_1 \) 0.8
Long spans factor \( F_2 \) 0.9722
Steel stress factor \( F_3 \) 1.077
Allowable 1/d ratio 40
Shear check

Location for shear calculation: 400mm from support
Shear force due to ultimate load $V_{Ed}=267 \text{ kN}$
Design shear stress $v_{Ed}=V_{Ed}/(0.9 \times bw \times d)=1.074 \text{ N/mm}^2$
Concrete strut capacity $v_{Rdm}=0.124 \times f_{ck}(1-f_{ck}/250)=3.274 \text{ N/mm}^2$
Angle of inclination of strut $\theta'=22^\circ$
Number of shear legs $n_{sl}=2$

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Design shear force</th>
<th>$267 \text{ kN}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear stress</td>
<td>$1.074 \text{ N/mm}^2$</td>
<td></td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>$3.274 \text{ N/mm}^2$</td>
<td></td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.6422</td>
<td></td>
</tr>
<tr>
<td>Diameter of links</td>
<td>10 mm</td>
<td></td>
</tr>
<tr>
<td>Area of links</td>
<td>157 mm$^2$</td>
<td></td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>244 mm</td>
<td></td>
</tr>
<tr>
<td>Actual spacing</td>
<td>244 mm</td>
<td></td>
</tr>
</tbody>
</table>
Location: Small beam - high-yield steel - no compression steel

Bending in flanged beams with optional calculations for shear, lap lengths, bar curtailment and limiting span/effective-depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Ultimate BM before redistribution Mbef=12 kNm
Beam being analysed is considered as non-continuous.
Characteristic concrete strength fcu=35 N/mm²
Max.aggregate size (for bar spc) hagg=15 mm
Char.strength of long'1 bars fy=500 N/mm²
Longitudinal reinforcement is high-yield steel.
Diameter of tension bars dia=12 mm
Char.strength of link steel fyv=500 N/mm²
Diameter of link legs dial=6 mm
High-yield steel shear reinforcement.
Designated exposure class is XC1
Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=25 mm
Overall depth of section h=275 mm
Effective depth of section d=238 mm
Breadth of flange b=400 mm
Breadth of rib bw=200 mm
Thickness of flange hf=100 mm

Longitudinal reinforcement

| TENSION | Characteristic strength 500 N/mm² |
| REINFORCEMENT | Diameter of bars 12 mm |
| SUMMARY | Number of bars 2 |
| | arranged in a single layer |
| | Cover to all steel 25 mm |
| | Area of steel required 122.07 mm² |
| | Area of steel provided 226.19 mm² |
| | Percentage provided: |
| | of gross section 0.30159 % |
| | of web area 0.41126 % |
| | Weight of steel provided 1.7756 kg/m |
| | Max.permissible spacing 261 mm |
| | Link size assumed 6 mm |
Check on span/effective-depth ratio

Interpolating from Table 3.9 with $bw/b=0.5$

Basic ratio for simply-sup. span $bs'd=17.143$

As applied-moment factor $M'bd2=M*1000*1000/(b*d^2)$

$=0.52962 \text{ N/mm}^2$

Mod.factor for tension steel from
equation 7 (Table 3.10) $\text{modf1}=0.55+(477-fs)/(120*(0.9$ $+M'bd2))=2.2819$

but this cannot exceed 2, so $\text{modf1}=2$

Mod.factor for no comp.steel $\text{modf2}=1$

Maximum permissible
span/effective-depth ratio $ps'd=bs'd*\text{modf1}*\text{modf2}=34.286$

Span of beam (Clause 3.4.1.2-4) $\text{span}=7 \text{ m}$

Actual span/effective-depth ratio as $d'=1000*\text{span}/d=29.412$

As this does not exceed $34.286$, this is Acceptable.

Bending-moment capacities at steel curtailment points

As there are only two bars in tension, no curtailment is possible.

Anchorage and lap lengths (Clause 3.12.8 of BS8110: Part 1)

Type 2 deformed bars: bond coefficient $\beta=0.50$ (Table 3.26)

Tension reinforcement:
Force in tension bar at yield $F_t=f_y/\gamma_S*\pi*dia^2/4=49173 \text{ N}$

Ultimate anchorage bond stress $f_{bu}=\beta*SQR(f_{cu})=2.958 \text{ N/mm}^2$

Anchorage and tension lap length $lap=F_t/(\pi*dia*f_{bu})=440.95 \text{ mm}$ $=450 \text{ mm (rounded)}$

$1.4 \times \text{ tension lap length}$ $=620 \text{ mm (rounded)}$

$2 \times \text{ tension lap length}$ $=890 \text{ mm (rounded)}$

Shear reinforcement

Shear calculations are in accordance with Clauses 3.4.5 of Code.

Support $\uparrow$ Design section $\leftarrow av$

Location for shear calculation: $0.25 \text{ m from support}$

Effective breadth for shear $bv=200 \text{ mm}$

Shear force due to ultimate load $V=100 \text{ kN}$

Distance from support $av=250 \text{ mm}$

No. of tension bars effective at section $nbars=2$

Number of legs to be provided $n\text{legs}=2$

Chosen link spacing $sv'=80 \text{ mm}$

Use 6 mm links (two legs), spaced at 80 mm centres along beam.
<table>
<thead>
<tr>
<th>SHEAR</th>
<th>REINFORCEMENT</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
<td>Diameter of links</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Number of legs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Spacing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Approx.weight of links</td>
</tr>
</tbody>
</table>
Location: Narrow deep beam - mild steel - no compression steel

Bending in flanged beams with optional calculations for shear, lap lengths, bar curtailment and limiting span/effective-depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Ultimate BM before redistribution Mbef=1000 kNm
Beam being analysed is considered as continuous.
Design moment (after redistrib.) M=1000 kNm
Characteristic concrete strength fcu=30 N/mm²
Max.aggregate size (for bar spc) hagg=15 mm
Char.strength of long'l bars fy=250 N/mm²
Longitudinal reinforcement is mild steel.
Diameter of tension bars dia=25 mm
Char.strength of link steel fyv=250 N/mm²
Diameter of link legs dial=12 mm
Mild steel shear reinforcement.
Designated exposure class is XC2
Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=35 mm
Overall depth of section h=950 mm
Effective depth of section d=890.5 mm
Breadth of flange b=1500 mm
Breadth of rib bw=300 mm
Thickness of flange hf=150 mm

Longitudinal reinforcement

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength 250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 25 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars 12</td>
</tr>
<tr>
<td></td>
<td>arranged in a single layer</td>
</tr>
<tr>
<td></td>
<td>Cover to all steel 35 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required 5437.5 mm²</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided 5890.5 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided:</td>
</tr>
<tr>
<td></td>
<td>of gross section 1.2668 %</td>
</tr>
<tr>
<td></td>
<td>of web area 2.0668 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided 46.24 kg/m</td>
</tr>
<tr>
<td></td>
<td>Max.permmissible spacing 300 mm</td>
</tr>
<tr>
<td></td>
<td>Link size assumed 12 mm</td>
</tr>
</tbody>
</table>

As overall depth h exceeds 750 mm, bars to control cracking must be provided on side faces of beam ( Cl. 3.12.5.4 and 3.12.11.2.9 )
Check on span/effective-depth ratio

Interpolating from Table 3.9 with $bw/b=0.2$
Basic ratio for continuous span $bs'd=20.8$
As applied-moment factor $M'bd^2=M*1000*1000/(b*d^2)=0.8407 \text{ N/mm}^2$
Mod.factor for tension steel from equation 7 (Table 3.10) $\text{modf1}=0.55+(477-fs)/(120*(0.9+M'bd^2))=2.097$
but this cannot exceed 2, so $\text{modf1}=2$
Mod.factor for no comp.steel $\text{modf2}=1$
Maximum permissible span/effective-depth ratio $ps'd=bs'd*\text{modf1}*\text{modf2}=41.6$
Span of beam (Clause 3.4.1.2-4) span=17 m
As span exceeds 10 m and partitions/finishes are sensitive to deflection multiply basic ratio by 10/span (in m) $ps'd=ps'd*10/span=24.471$
Actual span/effective-depth ratio as $d=1000*\text{span}/d=19.09$
As this does not exceed 24.471, this is Acceptable.

Bending-moment capacities at steel curtailment points

The number of tension bars is even, so reduce in pairs.

With 10 tension bars, effective steel area $Asrd=4908.7 \text{ mm}^2$
Reduced design moment $Mred=M*nbtr/nbart=833.33 \text{ kNm}$

With 8 tension bars, effective steel area $Asrd=3927 \text{ mm}^2$
Reduced design moment $Mred=M*nbtr/nbart=666.67 \text{ kNm}$

With 6 tension bars, effective steel area $Asrd=2945.2 \text{ mm}^2$
Reduced design moment $Mred=M*nbtr/nbart=500 \text{ kNm}$

With 4 tension bars, effective steel area $Asrd=1963.5 \text{ mm}^2$
Reduced design moment $Mred=M*nbtr/nbart=333.33 \text{ kNm}$

With 2 tension bars, effective steel area $Asrd=981.75 \text{ mm}^2$
Reduced design moment $Mred=M*nbtr/nbart=166.67 \text{ kNm}$

No further curtailment is possible. However, bars must extend beyond the theoretical curtailment points as required below.

Min.extension to tension bars equal to eff.depth $d$ of 890.5 mm as this is not less than 12*tension bar diameter, i.e. 300 mm.
Anchorage and lap lengths (Clause 3.12.8 of BS8110: Part 1)

Plain bars: bond coefficient  \( \beta = 0.28 \) (Table 3.26)

Tension reinforcement:

- Force in tension bar at yield:  \( F_t = \frac{f_y}{\gamma_S} \pi d^2}{4} = 106712 \text{ N} \)
- Ultimate anchorage bond stress:  \( f_{bu} = \beta \sqrt{f_{cu}} = 1.5336 \text{ N/mm}^2 \)
- Anchorage and tension lap length:  \( \text{lap} = \frac{F_t}{\pi d f_{bu}} = 885.94 \text{ mm} \)
  
  = 890 mm (rounded)

- \( 1.4 \times \text{tension lap length} = 1250 \text{ mm (rounded)} \)
- \( 2 \times \text{tension lap length} = 1780 \text{ mm (rounded)} \)

Shear reinforcement

Shear calculations are in accordance with Clauses 3.4.5 of Code.

Location for shear calculation: 0.5 m from support

- Effective breadth for shear:  \( b_v = 300 \text{ mm} \)
- Shear force due to ultimate load:  \( V = 800 \text{ kN} \)
- Distance from support:  \( a_v = 500 \text{ mm} \)
- Diameter of tension bars:  \( d_{as} = 25 \text{ mm} \)
- No. of tension bars effective at section:  \( n_{bars} = 12 \)
- Number of legs to be provided:  \( n_{legs} = 2 \)
- Chosen link spacing:  \( s_v' = 75 \text{ mm} \)

Use 12 mm links (two legs), spaced at 75 mm centres along beam.

<table>
<thead>
<tr>
<th>SHEAR REINFORCEMENT</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>250 N/mm(^2)</td>
</tr>
<tr>
<td>Diameter of links</td>
<td>12 mm</td>
</tr>
<tr>
<td>Number of legs</td>
<td>2</td>
</tr>
<tr>
<td>Spacing</td>
<td>75 mm</td>
</tr>
<tr>
<td>Approx. weight of links</td>
<td>54.97 kg/m</td>
</tr>
</tbody>
</table>
Location: Shallow beam - square twist - with compression steel

Bending in flanged beams with optional calculations for shear, lap lengths, bar curtailment and limiting span/effective-depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Ultimate BM before redistribution Mbef=95 kNm
Beam being analysed is considered as continuous.
Design moment (after redistrib.) M=76 kNm
Characteristic concrete strength fcu=35 N/mm²
Max.aggregate size (for bar spc) hagg=15 mm
Char.strength of long'l bars fy=425 N/mm²
Steel strength other than those given in Table 3.1 of BS8110.
Min.percentage when bw/b>=0.4 permn1=0.16 %
Min.percentage when bw/b<0.4 permn2=0.22 %
Diameter of tension bars dia=20 mm
Char.strength of link steel fyv=250 N/mm²
Diameter of link legs dial=8 mm
Mild steel shear reinforcement.
Nominal concrete cover cover=30 mm
Overall depth of section h=175 mm
Effective depth of section d=127 mm
Breadth of flange b=1000 mm
Breadth of rib bw=300 mm
Thickness of flange hf=75 mm

Longitudinal reinforcement

Depth to compression bars d'=46 mm

| TENSION | Characteristic strength 425 N/mm² |
| REINFORCEMENT | Diameter of bars 20 mm |
| Number of bars | 7 |
| arranged in a single layer |
| Cover to all steel | 30 mm |
| Area of steel required | 1982.3 mm² |
| Area of steel provided | 2199.1 mm² |
| Percentage provided: |
| of gross section | 2.0944 % |
| of web area | 4.1888 % |
| Weight of steel provided | 17.263 kg/m |
| Max.permissible spacing | 147 mm |
| Link size assumed | 8 mm |
COMPRESSION  
Characteristic strength  425 N/mm²

REINFORCEMENT  
Diameter of bars  16 mm

SUMMARY  
Number of bars  2
arranged in a single layer
Cover to all steel  30 mm
Area of steel required  300.47 mm²
Area of steel provided  402.12 mm²
Percentage provided:
of gross section  0.38298 %
of web area  0.76595 %
of flange area  0.53617 %
Weight of steel provided 3.1567 kg/m
Minimum poss.link size  6 mm
Maximum spacing of links  192 mm

Check on span/effective-depth ratio

Interpolating from Table 3.9 with bw/b=0.3
Basic ratio for continuous span  bs'd=20.8
As applied-moment factor  M'bd2=M*1000*1000/(b*d²)=4.712 N/mm²
Mod.factor for tension steel from equation 7 (Table 3.10)  modf1=0.55+(477-fs)/(120*(0.9
+M'bd2))=0.78425
Area of comp.steel provided  As'pr=402.12 mm²
Percentage of compression steel  per'n=As'pr/(b*d)=0.31663 %
From Equation 9 of BS8110, with percentage of comp.steel=0.31663 %,
Mod.factor for compression steel  modf2=1+per'n/(3+per'n)=1.0955
Maximum permissible
span/effective-depth ratio  ps'd=bs'd*modf1*modf2=17.87
Span of beam (Clause 3.4.1.2-4)  span=2.3 m

WARNING:
Actual span/effective-depth ratio 18.11
Permissible span/effective-depth ratio 17.87
Hence unacceptable.
Location: Large beam - high-yield steel - with compression steel

Bending in flanged beams with optional calculations for shear, lap lengths, bar curtailment and limiting span/effective-depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Ultimate BM before redistribution Mbef=9567 kNm
Beam being analysed is considered as continuous.

Max.aggregate size (for bar spc) hagg=15 mm
Char.strength of long'l bars fy=500 N/mm²
Longitudinal reinforcement is high-yield steel.
Diameter of tension bars dia=40 mm
Char.strength of link steel fyv=500 N/mm²
Diameter of link legs dial=12 mm
High-yield steel shear reinforcement.
Designated exposure class is XC3

Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=50 mm
Overall depth of section h=1000 mm
Effective depth of section d=918 mm
Breadth of flange b=2000 mm
Breadth of rib bw=500 mm
Thickness of flange hf=175 mm

Longitudinal reinforcement

Depth to compression bars d'=72 mm

<table>
<thead>
<tr>
<th>TENSION</th>
<th>REINFORCEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>40 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>22</td>
</tr>
</tbody>
</table>

arranged in a single layer

<table>
<thead>
<tr>
<th>Cover to all steel</th>
<th>50 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of steel required</td>
<td>27319 mm²</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>27646 mm²</td>
</tr>
<tr>
<td>Percentage provided:</td>
<td></td>
</tr>
<tr>
<td>of gross section</td>
<td>3.6257 %</td>
</tr>
<tr>
<td>of web area</td>
<td>5.5292 %</td>
</tr>
</tbody>
</table>

Weight of steel provided 217.02 kg/m
Max.permissible spacing 139 mm
Link size assumed 12 mm
### COMPRESSION
- Characteristic strength: 500 N/mm²

### REINFORCEMENT
- Diameter of bars: 20 mm
- Number of bars: 10
- Arranged in a single layer

### SUMMARY
- Cover to all steel: 50 mm
- Area of steel required: 3135.3 mm²
- Area of steel provided: 3141.6 mm²
- Percentage provided:
  - of gross section: 0.41201%
  - of web area: 0.62832%
  - of flange area: 0.8976%
- Weight of steel provided: 24.662 kg/m
- Minimum poss. link size: 6 mm
- Maximum spacing of links: 240 mm

As overall depth h exceeds 750 mm, bars to control cracking must be provided on side faces of beam (Cl. 3.12.5.4 and 3.12.11.2.9)

Check on span/effective-depth ratio

Interpolating from Table 3.9 with bw/b=0.25
Basic ratio for continuous span: bs'd=20.8

**WARNING:**
As $f_s$ exceeds 333.3 N/mm² Table 3.10 is invalid (Cl. 3.4.6.5).
Either the steel yield stress is too high or the moment redistribution from the midspan section is excessive.
Location: Ex1 - Continuous T-beam

Bending in flanged beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.45*d.

Distance between beam supports \( L = 8.2 \text{ m} \)
Design BM before redistribution \( M_{bef} = 600 \text{ kNm} \)
Beam being analysed is considered as continuous.
Section considered has a sagging moment.
Overall depth of section \( h = 650 \text{ mm} \)
Average web width \( b_w = 350 \text{ mm} \)
Flange thickness \( h_f = 200 \text{ mm} \)
Char yield strength of reinft \( f_yk = 500 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( d_{ia} = 25 \text{ mm} \)
Diameter of link legs \( d_{ial} = 10 \text{ mm} \)
Effective depth of section \( d = 560 \text{ mm} \)

Section being considered is a T-beam.

**DESIGN**

Overall depth 650 mm

**SUMMARY**

Effective depth 560 mm
Flange width 2194 mm
Web width 350 mm

**FLEXURE**

Parameter K 0.0291
Parameter K' 0.1684
Lever arm ratio z/d 0.95

**TENSION REINFORCEMENT**

Steel area required 2594 mm²
Steel area provided 2945 mm²
Diameter of bars 25 mm
Number of bars 6
Steel percentage req. 0.2111%
Minimum area of steel 295.2 mm²
Maximum area of steel 57044 mm²

Deflection check

Actual span to depth ratio \( 1'd = L \times 1000/d = 14.64 \)
Allowable span/depth ratio \( 1'da = k \times N \times F_1 \times F_2 \times F_3 = 68.14 \)
Absolute value of span/depth \( 1'da = 40 \times k = 52 \)

**DESIGN**

Actual 1/d ratio 14.64

**SUMMARY**

Basic 1/d ratio 67.6

**DEFLECTION**

Span factor 1.3
Flange beam factor \( F_1 \) 0.8
Long spans factor \( F_2 \) 0.8537
Steel stress factor \( F_3 \) 1.135
Allowable 1/d ratio 52
Shear check

Location for shear calculation: 400mm from support
Shear force due to ultimate load $V_{Ed}=267$ kN
Design shear stress $\nu_{Ed}=V_{Ed}*10^3/(0.9*bw*d)=1.514$ N/mm²
Concrete strut capacity $\nu_{Rdm}=0.124*fck*(1-fck/250)=3.274$ N/mm²
Angle of inclination of strut $\theta'=22^\circ$
Number of shear legs $n_{sl}=4$
Chosen spacing $s=125$ mm

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Design shear force</th>
<th>267 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear stress</td>
<td>1.514 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.274 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.4874</td>
<td></td>
</tr>
<tr>
<td>Diameter of links</td>
<td>10 mm</td>
<td></td>
</tr>
<tr>
<td>Area of links</td>
<td>314 mm²</td>
<td></td>
</tr>
<tr>
<td>Maximum spacing (based on 0.75d)</td>
<td>420 mm</td>
<td></td>
</tr>
<tr>
<td>Actual spacing</td>
<td>125 mm</td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex2 - Support condition

Bending in flanged beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.45*d.

Distance between beam supports \( L = 8.2 \text{ m} \)
Design BM before redistribution \( M_{bef} = 650 \text{ kNm} \)
Beam being analysed is considered as continuous.
Section considered has a hogging moment and will be designed as a rectangular section.

Design moment (after redistrib.) \( M = 570 \text{ kNm} \)
Overall depth of section \( h = 650 \text{ mm} \)
Average web width \( b_w = 350 \text{ mm} \)
Flange thickness \( h_f = 200 \text{ mm} \)
Char yield strength of reinft \( f_y = 500 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( d_{ia} = 25 \text{ mm} \)
Diameter of link legs \( d_{ial} = 10 \text{ mm} \)
Effective depth of section \( d = 570 \text{ mm} \)

| DESIGN | Overall depth | 650 mm |
| SUMMARY | Effective depth | 570 mm |
| Width of section | 350 mm |
| FLEXURE | Parameter K | 0.1671 |
| | Parameter K' | 0.1684 |
| | Lever arm ratio z/d | 0.8203 |
| TENSION REINFORCEMENT | Steel area required | 2804 mm² |
| | Steel area provided | 2945 mm² |
| | Diameter of bars | 25 mm |
| | Number of bars | 6 |
| | Steel percentage req. | 1.405 % |
| | Minimum area of steel | 300.5 mm² |
| | Maximum area of steel | 9100 mm² |

Shear check

Location for shear calculation: 500 mm from support
Shear force due to ultimate load \( V_{Ed} = 367 \text{ kN} \)
Design shear stress \( v_{Ed} = V_{Ed} \times 10^3 / (0.9 \times b_w \times d) = 2.044 \text{ N/mm}^2 \)
Concrete strut capacity \( v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 3.274 \text{ N/mm}^2 \)
Angle of inclination of strut \( \theta' = 22^\circ \)
Number of shear legs \( n_{sl} = 2 \)
Chosen spacing \( s = 225 \text{ mm} \)
<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Design shear force</th>
<th>367 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear stress</td>
<td>2.044 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.274 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.6582</td>
<td></td>
</tr>
<tr>
<td>Diameter of links</td>
<td>10 mm</td>
<td></td>
</tr>
<tr>
<td>Area of links</td>
<td>157 mm²</td>
<td></td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>238 mm</td>
<td></td>
</tr>
<tr>
<td>Actual spacing</td>
<td>225 mm</td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex3 - Continuous L-beam

Bending in flanged beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.45*d.

Distance between beam supports \( L = 9 \text{ m} \)
Design BM before redistribution \( M_{\text{bef}} = 780 \text{ kNm} \)
Beam being analysed is considered as continuous.
Section considered has a sagging moment.

Overall depth of section \( h = 700 \text{ mm} \)
Average web width \( b_w = 350 \text{ mm} \)
Flange thickness \( h_f = 250 \text{ mm} \)
Char yield strength of reinft \( f_y = 500 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{\text{agg}} = 20 \text{ mm} \)
Diameter of tension bars \( \text{dia} = 32 \text{ mm} \)
Diameter of link legs \( \text{dial} = 10 \text{ mm} \)
Effective depth of section \( d = 600 \text{ mm} \)
Section being considered is a L-beam.

**DESIGN**
- Overall depth: 700 mm

**SUMMARY**
- Effective depth: 600 mm
- Flange width: 1615 mm
- Web width: 350 mm

**FLEXURE**
- Parameter \( K \): 0.0447
- Parameter \( K' \): 0.1684
- Lever arm ratio \( z/d \): 0.95

**TENSION REINFORCEMENT**
- Steel area required: 3147 mm²
- Steel area provided: 3217 mm²
- Diameter of bars: 32 mm
- Number of bars: 4
- Steel percentage req.: 0.3248 %
- Minimum area of steel: 316.3 mm²
- Maximum area of steel: 45220 mm²

**Deflection check**

Actual span to depth ratio \( 1'd = L*1000/d = 15 \)
Allowable span/depth ratio \( 1'da = k*N*F1*F2*F3 = 28.79 \)

**DESIGN**
- Actual 1/d ratio: 15

**SUMMARY**
- Basic 1/d ratio: 34.82

**DEFORMATION**
- Span factor: 1.3
- Flange beam factor \( F_1 \): 0.8
- Long spans factor \( F_2 \): 0.7778
- Steel stress factor \( F_3 \): 1.022
- Allowable 1/d ratio: 28.79
Location: Ex4 - Simply supported arrangement

Bending in flanged beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.45*d.

Distance between beam supports \( L = 7.2 \) m
Design BM before redistribution \( M_{\text{bef}} = 480 \) kNm
Beam being analysed is considered as non-continuous.
Overall depth of section \( h = 500 \) mm
Average web width \( b_w = 650 \) mm
Flange thickness \( h_f = 250 \) mm
Char yield strength of reinft \( f_y = 500 \) N/mm²
Max.aggregate size (for bar spc.) \( h_{\text{agg}} = 20 \) mm
Diameter of tension bars \( d_{\text{ia}} = 25 \) mm
Diameter of link legs \( d_{\text{ial}} = 10 \) mm
Effective depth of section \( d = 425 \) mm
Section being considered is a T-beam.
NOTE: Clear distances vary.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth</th>
<th>500 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth</td>
<td>425 mm</td>
</tr>
<tr>
<td></td>
<td>Flange width</td>
<td>2585 mm</td>
</tr>
<tr>
<td></td>
<td>Web width</td>
<td>650 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K</td>
<td>0.0343</td>
</tr>
<tr>
<td></td>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio ( z/d )</td>
<td>0.95</td>
</tr>
<tr>
<td>TENSION REINFORCEMENT</td>
<td>Steel area required</td>
<td>2734 mm²</td>
</tr>
<tr>
<td></td>
<td>Steel area provided</td>
<td>2945 mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Steel percentage req.</td>
<td>0.2489 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel</td>
<td>416.1 mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel</td>
<td>51700 mm²</td>
</tr>
</tbody>
</table>

Deflection check

Actual span to depth ratio \( 1'd = L \times 1000 / d = 16.94 \)
Allowable span/depth ratio \( 1'd = k \times N \times F_1 \times F_2 \times F_3 = 43.68 \)
Absolute value of span/depth \( 1'd = 40 \times k = 40 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Actual 1/d ratio</th>
<th>16.94</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Basic 1/d ratio</td>
<td>52.14</td>
</tr>
<tr>
<td>DEFLECTION</td>
<td>Span factor</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Flange beam factor ( F_1 )</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>Long spans factor ( F_2 )</td>
<td>0.9722</td>
</tr>
<tr>
<td></td>
<td>Steel stress factor ( F_3 )</td>
<td>1.077</td>
</tr>
<tr>
<td></td>
<td>Allowable 1/d ratio</td>
<td>40</td>
</tr>
</tbody>
</table>
Shear check

Location for shear calculation: 400mm from support
Shear force due to ultimate load \( V_{Ed} = 267 \text{ kN} \)
Design shear stress \( \nu_{Ed} = V_{Ed} \times 10^3 / (0.9 \times bw \times d) = 1.074 \text{ N/mm}^2 \)
Concrete strut capacity \( \nu_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 3.274 \text{ N/mm}^2 \)
Angle of inclination of strut \( \theta' = 22^\circ \)
Number of shear legs \( n_{sl} = 2 \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Design shear force</th>
<th>267 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design shear stress</td>
<td>1.074 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Concrete strut capacity</td>
<td>3.274 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Area/spacing ratio</td>
<td>0.6422</td>
</tr>
<tr>
<td></td>
<td>Diameter of links</td>
<td>10 mm</td>
</tr>
<tr>
<td></td>
<td>Area of links</td>
<td>157 mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum spacing</td>
<td>244 mm</td>
</tr>
<tr>
<td></td>
<td>Actual spacing</td>
<td>244 mm</td>
</tr>
</tbody>
</table>

Design of concrete walls containing reinforcement to BS8110

Design to BS8110 (1997)
Total length of wall section \( b = 3000 \text{ mm} \)
Thickness of wall section \( h = 300 \text{ mm} \)
Concrete grade \( f_{cu} = 30 \text{ N/mm}^2 \)
Steel strength \( f_y = 250 \text{ N/mm}^2 \)
Number of rows of bars (1 or 2) \( \text{rows} = 2 \)
Vertical bar size \( d_i = 20 \text{ mm} \)
Designated exposure class is XC2
Specified fixing tolerance \( \text{tol} = 10 \text{ mm} \)
Cover to main bars \( c = 35 \text{ mm} \)
Vertical bar spacing \( \text{space} = 125 \text{ mm} \)
Clear height of wall \( \text{lo} = 3.5 \text{ m} \)
The end condition at the top of the wall in accordance with Clause 3.8.1.6.2 of BS8110 is condition 2
The end condition at the bottom of the wall in accordance with Clause 3.8.1.6.2 of BS8110 is condition 1

BS8110 Classification: Stocky braced reinforced-concrete wall of normal-weight concrete

Maximum load supported

\[
nw = h \times (0.35 \times f_{cu} + 0.67 \times f_y \times \rho_1)\\
= 3991.9 \text{ kN per metre}
\]

Design of concrete walls containing reinforcement to BS8110

Design to BS8110 (1997)
Total length of wall section \( b = 4000 \text{ mm} \)
Thickness of wall section \( h = 750 \text{ mm} \)
Density of concrete \( \text{density} = 1200 \text{ kg/m}^3 \)
Concrete grade \( f_{cu} = 30 \text{ N/mm}^2 \)
Steel strength \( f_y = 500 \text{ N/mm}^2 \)
Number of rows of bars (1 or 2) \( \text{rows} = 2 \)
Vertical bar size \( d_i = 25 \text{ mm} \)
Designated exposure class is XC3
Specified fixing tolerance \( \text{tol} = 10 \text{ mm} \)
Cover to main bars \( c = 45 \text{ mm} \)
Vertical bar spacing \( \text{space} = 200 \text{ mm} \)
Clear height of wall \( h_0 = 3.5 \text{ m} \)

The end condition at the top of the wall in accordance with Clause 3.8.1.6.2 of BS8110 is condition 3
The end condition at the bottom of the wall in accordance with Clause 3.8.1.6.2 of BS8110 is condition 1

BS8110 Classification: Stocky unbraced reinforced-concrete wall

Since wall supports an approximately-symmetrical beam or slab system, requirements of Clause 3.9.3.3 of BS8110 apply.
Design section to resist axial thrust plus nominal moment corresponding to an eccentricity of 0.05\(h\) (i.e. 37.5 mm or 20 mm, whichever is greater.

Max. resistance \( n_w = \frac{h^2 (k_1 (d-k_2 h)+r_{h0} f_y (d-h/2))}{(d+e_x-h/2)} \)
\( = 11137 \text{ kN per metre} \)

Since the maximum load supported by a stocky braced wall is \( n_w = \frac{h^2 (0.35 f_{cu} + 0.7 f_y r_{h0})}{(d+e_x-h/2)} \)
\( = 9593.1 \text{ kN per metre} \),
it would be sensible to restrict the load-carrying capacity of a similar unbraced section to this value.

Design of concrete walls containing reinforcement to BS8110

\[ \text{Design to BS8110(1997)} \]
Total length of wall section \( b = 3000 \text{ mm} \)
Thickness of wall section \( h = 100 \text{ mm} \)
Concrete grade \( f_{cu} = 30 \text{ N/mm}^2 \)
Steel strength \( f_y = 500 \text{ N/mm}^2 \)
Number of rows of bars (1 or 2) \( \text{rows=} 2 \)
Vertical bar size \( d_i = 12 \text{ mm} \)
Designated exposure class is XC1
Specified fixing tolerance \( \text{tol=} 10 \text{ mm} \)
Cover to main bars \( c = 25 \text{ mm} \)
Vertical bar spacing \( \text{space=} 100 \text{ mm} \)
Clear height of wall \( l_o = 3.5 \text{ m} \)
The end condition at the top of the wall in accordance with Clause 3.8.1.6.2 of BS8110 is condition 1
The end condition at the bottom of the wall in accordance with Clause 3.8.1.6.2 of BS8110 is condition 1

BS8110 Classification: Slender braced reinforced-concrete wall of normal-weight concrete

In this case the requirements of Clause 3.9.3.7.1 of BS8110 apply. First determine in-plane forces on section, due to in-plane axial loads plus moments resulting from horizontal forces, by an elastic analysis assuming that no tension occurs in the concrete. Then calculate effects of transverse moments separately, and combine with axial force acting at point concerned, obtained from previous calculation, using rigorous limit-state analysis. Total transverse moment must sum an additional moment due to slenderness with that resulting from applied loads, by considering wall as slender column bending about minor axis (i.e. use Proforma 91). If wall is reinforced by single central layer such additional moments must be doubled (see BS8110, Cl.3.9.3.7.3).
Location: Medium thick wall using normal-weight concrete

Design of concrete walls containing reinforcement to EC2

Char yield strength of reinforcement $f_{yk}=500 \text{ N/mm}^2$
Number of rows of bars (1 or 2) $\text{rows}=2$
Vertical bar diameter $\text{dia}=16 \text{ mm}$
Horizontal bar diameter $\text{dia}=12 \text{ mm}$

Section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall thickness</td>
<td>$h=300 \text{ mm}$</td>
</tr>
<tr>
<td>Length of wall section</td>
<td>$b=1000 \text{ mm}$</td>
</tr>
<tr>
<td>Depth of comp.reinforcement</td>
<td>$d_2=55 \text{ mm}$</td>
</tr>
</tbody>
</table>

Loading details

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear height between restraints</td>
<td>$l=3.5 \text{ m}$</td>
</tr>
<tr>
<td>Design ultimate axial load</td>
<td>$N=2000 \text{ kN/m}$</td>
</tr>
<tr>
<td>Larger initial end mt. (positive)</td>
<td>$M_2=114.5 \text{ kNm/m}$</td>
</tr>
<tr>
<td>Smaller initial end moment</td>
<td>$M_1=0 \text{ kNm/m}$</td>
</tr>
</tbody>
</table>

Slenderness (EC2 Clause 5.8.3.2)

About Y-Y axis:
Wall is considered to be braced about the stronger axis.
Fixity condition at top (1-3) $cty=2$
Fixity condition at bottom (1-3) $cby=2$

About Z-Z axis:
Wall is considered to be braced about the weaker axis.
Fixity condition at top (1-3) $ctz=2$
Fixity condition at bottom (1-3) $cbz=2$
For Y-Y stronger axis:
Slenderness $l_{amy}=l_{oy}/iy=10.306$
Design moments (Clause 5.8.8.2)

Design moment

Design moment
M Ed=Mo2=129.38E6 Nmm

Axial load and moment considerations

Effective/overall-depth ratio

d2h=d2/h=0.18333

Axial load ratio

Nrat=NEd/(b*h*fck)=0.26667

Moment ratio

Mrat=MEd/(b*h^2*fck)=0.0575

Longitudinal reinforcement

Chart data for rectangular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'.

Area of steel required

As=val*fck/fyk*b*h=0 mm²/m

Minimum permitted area of steel

Asmin=0.002*b*h=600 mm²/m

Provide minimum steel

As=Asmin=600 mm²/m

Number of bars provided

nbars=8 /m

DESIGN SUMMARY

Wall thickness
300 mm

Clear wall height
3.5 m

Design axial load
2000 kN/m

Design BM (Z-Z axis)
129.38 kNm/m

Vert.reinf.diameter
16 mm

Area of steel required
600 mm²/m

Area of steel provided
1608.5 mm²/m

Horiz.reinf.diameter
12 mm

Horiz.reinf.spacing
400 mm
Location: EX2 - Thick wall using normal-weight concrete

Design of concrete walls containing reinforcement to EC2

\[ h = \text{wall thickness} \]
\[ \text{horizontal bars to lie outside vertical bars} \]
\[ \text{vertical reinforcement} \]

Char yield strength of reinforcement \( f_{yk} = 500 \text{ N/mm}^2 \)
Number of rows of bars (1 or 2) \( \text{rows} = 2 \)
Vertical bar diameter \( \text{dia} = 25 \text{ mm} \)
Horizontal bar diameter \( \text{dial} = 10 \text{ mm} \)

**Section properties**

Wall thickness \( h = 750 \text{ mm} \)
Length of wall section \( b = 1000 \text{ mm} \)
Depth of comp. reinforcement \( d_2 = 68 \text{ mm} \)

**Loading details**

Clear height between restraints \( l = 3.5 \text{ m} \)
Design ultimate axial load \( N = 3000 \text{ kN/m} \)
Larger initial end mt. (positive) \( M_2 = 100 \text{ kNm/m} \)
Smaller initial end moment \( M_1 = 0 \text{ kNm/m} \)

**Slenderness (EC2 Clause 5.8.3.2)**

About Y-Y axis:
Wall is considered to be unbraced about the stronger axis.
Fixity condition at top (1-3) \( c_{ty} = 1 \)
Fixity condition at bottom (1-3) \( c_{by} = 1 \)
About Z-Z axis:
Wall is considered to be braced about the weaker axis.
Fixity condition at top (1-3) \( c_{tz} = 1 \)
Fixity condition at bottom (1-3) \( c_{bz} = 1 \)
For Y-Y stronger axis:
Slenderness \( l_{amy} = l_{oy}/l_{iy} = 14.549 \)
Design moments (Clause 5.8.8.2)

Design moment

\[ M_{Ed} = M_{o2} = 119.69 \times 10^6 \text{ Nmm} \]

Axial load and moment considerations

Effective/overall-depth ratio

\[ \frac{d_2}{h} = \frac{d_2}{h} = 0.090667 \]

Axial load ratio

\[ N_{rat} = \frac{N_{Ed}}{b \times h \times f_{ck}} = 0.16 \]

Moment ratio

\[ M_{rat} = \frac{M_{Ed}}{b \times h^2 \times f_{ck}} = 0.0085111 \]

Longitudinal reinforcement

Chart data for rectangular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'.

Area of steel required

\[ A_s = \frac{A_{val} \times f_{ck}}{f_{yk} \times b \times h} = 0 \text{ mm}^2/\text{m} \]

Minimum permitted area of steel

\[ A_{smin} = 0.002 \times b \times h = 1500 \text{ mm}^2/\text{m} \]

Provide minimum steel

\[ A_s = A_{smin} = 1500 \text{ mm}^2/\text{m} \]

Number of bars provided

\[ n_{bars} = 8 /\text{m} \]

DESIGN SUMMARY

| Wall thickness | 750 mm |
| Clear wall height | 3.5 m |
| Design axial load | 3000 kN/m |
| Design BM (Z-Z axis) | 119.69 kNm/m |
| Vert. reinft. diameter | 25 mm |
| Area of steel required | 1500 mm²/m |
| Area of steel provided | 3927 mm²/m |
| Horiz. reinft. diameter | 10 mm |
| Horiz. reinft. spacing | 400 mm |
Location: EX3 - Plain concrete thin wall

Design of concrete walls containing reinforcement to EC2

Char yield strength of reinf'ment \( f_{yk} = 500 \, \text{N/mm}^2 \)
Number of rows of bars (1 or 2) \( \text{rows} = 1 \)
Vertical bar diameter \( \text{dia} = 12 \, \text{mm} \)
Horizontal bar diameter \( \text{dia} = 10 \, \text{mm} \)

Section properties

Wall thickness \( h = 120 \, \text{mm} \)
Length of wall section \( b = 1000 \, \text{mm} \)
Depth of comp.reinforcement \( d_2 = 45 \, \text{mm} \)

Loading details

Clear height between restraints \( l = 3.5 \, \text{m} \)
Design ultimate axial load \( N = 300 \, \text{kN/m} \)
Larger initial end mt. (positive) \( M_2 = 12 \, \text{kNm/m} \)
Smaller initial end moment \( M_1 = 0 \, \text{kNm/m} \)

Slenderness (EC2 Clause 5.8.3.2)

About Y-Y axis:
Wall is considered to be braced about the stronger axis.
Fixity condition at top (1-3) \( c_{ty} = 1 \)
Fixity condition at bottom (1-3) \( c_{by} = 1 \)

About Z-Z axis:
Wall is considered to be braced about the weaker axis.
Fixity condition at top (1-2) \( c_{tz} = 1 \)
Fixity condition at bottom (1-2) \( c_{bz} = 1 \)

For Y-Y stronger axis:
Slenderness \( l_{amy} = \frac{\text{loy}}{\text{iy}} = 9.0933 \)
Axial resistance \( N_{rd} = 33.425 \, \text{kN per metre} \).
**Location: EX4 - Plain concrete wall**

**Design of concrete walls containing reinforcement to EC2**

![Diagram of wall section with reinforcement](Image)

- **Char yield strength of reinforcement (fyk):** 500 N/mm²
- **Number of rows of bars (1 or 2):** rows = 2
- **Vertical bar diameter:** dia = 10 mm
- **Horizontal bar diameter:** dia = 10 mm

**Section properties**

- **Wall thickness:** h = 300 mm
- **Length of wall section:** b = 1000 mm
- **Depth of comp. reinforcement:** d2 = 50 mm

**Loading details**

- **Clear height between restraints:** l = 3.5 m
- **Design ultimate axial load:** N = 2000 kN/m
- **Larger initial end moment (positive):** M2 = 10 kNm/m
- **Smaller initial end moment:** M1 = 0 kNm/m

**Slenderness (EC2 Clause 5.8.3.2)**

**About Y-Y axis:**
- Wall is considered to be braced about the stronger axis.
- Fixity condition at top (1-3): cty = 2
- Fixity condition at bottom (1-3): cby = 2

**About Z-Z axis:**
- Wall is considered to be braced about the weaker axis.
- Fixity condition at top (1-2): ctz = 2
- Fixity condition at bottom (1-2): cbz = 2

**For Y-Y stronger axis:**
- Slenderness: lamy = loy/iy = 10.306
- Axial resistance: Nrd = 1950.5 kN per metre.
**Location: Ex5 - RC Wall**

**Design of concrete walls containing reinforcement to EC2**

---

**h = wall thickness**

- **horizontal bars to lie outside vertical bars**

- **vertical reinforcement**

---

Char yield strength of reinforcement $f_{yk}=500 \text{ N/mm}^2$

- Number of rows of bars (1 or 2) $\text{rows}=2$
- Vertical bar diameter $\text{dia}=25 \text{ mm}$
- Horizontal bar diameter $\text{dial}=10 \text{ mm}$

---

**Section properties**

- Wall thickness $h=250 \text{ mm}$
- Length of wall section $b=1000 \text{ mm}$
- Depth of comp. reinforcement $d_2=58 \text{ mm}$

---

**Loading details**

- Clear height between restraints $l=3.5 \text{ m}$
- Design ultimate axial load $N=3000 \text{ kN/m}$
- Larger initial end mt. (positive) $M_2=100 \text{ kNm/m}$
- Smaller initial end moment $M_1=0 \text{ kNm/m}$

---

**Slenderness (EC2 Clause 5.8.3.2)**

**About Y-Y axis:**

- Wall is considered to be braced about the stronger axis.
- Fixity condition at top (1-3) $cty=3$
- Fixity condition at bottom (1-3) $cby=3$

**About Z-Z axis:**

- Wall is considered to be braced about the weaker axis.
- Fixity condition at top (1-3) $ctz=3$
- Fixity condition at bottom (1-3) $cbz=3$

**For Y-Y stronger axis:**

- Slenderness $l_\text{amy}=loy/iy=12.124$
Design moments (Clause 5.8.8.2)

The design moment is taken as the maximum of:

\[
\begin{align*}
Mo_2 &= 126.25 \times 10^6 \text{ Nmm} \\
M_{Ed1} &= M_{oe} + M_2 = 156.98 \times 10^6 \text{ Nmm} \\
M_{Ed2} &= M_{o1} + 0.5 \times M_2 = 61.613 \times 10^6 \text{ Nmm}
\end{align*}
\]

Design Moment

\[
M_{Ed} = 156.98 \times 10^6 \text{ Nmm}
\]

Axial load and moment considerations

Effective/overall-depth ratio \(d_2h = d_2/h = 0.232\)
Axial load ratio \(N_{rat} = N_{Ed}/(b \times h \times f_{ck}) = 0.48\)
Moment ratio \(M_{rat} = M_{Ed}/(b \times h^2 \times f_{ck}) = 0.10047\)

Longitudinal reinforcement

Chart data for rectangular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'.

Area of steel required \(A_s = \text{val} \times f_{ck}/f_{yk} \times b \times h = 3891.8 \text{ mm}^2/\text{m}\)
Number of bars provided \(n_{bars} = 8 /\text{m}\)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall thickness</td>
</tr>
<tr>
<td>Clear wall height</td>
</tr>
<tr>
<td>Design axial load</td>
</tr>
<tr>
<td>Design BM (Z-Z axis)</td>
</tr>
<tr>
<td>Vert.reinf.diameter</td>
</tr>
<tr>
<td>Area of steel required</td>
</tr>
<tr>
<td>Area of steel provided</td>
</tr>
<tr>
<td>Horiz.reinf.diameter</td>
</tr>
<tr>
<td>Horiz.reinf.spacing</td>
</tr>
</tbody>
</table>
**Location:** Ex6 - Example p175 from TCC Worked Examples - EC2:Volume 1

**Design of concrete walls containing reinforcement to EC2**

Char yield strength of reinforcement \( f_{yk} = 500 \text{ N/mm}^2 \)

- Number of rows of bars (1 or 2): \( \text{rows} = 2 \)
- Vertical bar diameter: \( \text{dia} = 16 \text{ mm} \)
- Horizontal bar diameter: \( \text{dia} = 10 \text{ mm} \)

**Section properties**

- Wall thickness: \( h = 200 \text{ mm} \)
- Length of wall section: \( b = 1000 \text{ mm} \)
- Depth of comp.reinforcement: \( d_2 = 43 \text{ mm} \)

**Loading details**

- Clear height between restraints: \( l = 3.1 \text{ m} \)
- Design ultimate axial load: \( N = 1149.4 \text{ kN/m} \)
- Larger initial end mt. (positive): \( M_2 = 31.6 \text{ kNm/m} \)
- Smaller initial end moment: \( M_1 = 0 \text{ kNm/m} \)

**Slenderness (EC2 Clause 5.8.3.2)**

About Y-Y axis:
- Wall is considered to be braced about the stronger axis.
- Fixity condition at top (1-3): \( c_{ty} = 1 \)
- Fixity condition at bottom (1-3): \( c_{by} = 1 \)

About Z-Z axis:
- Wall is considered to be braced about the weaker axis.
- Fixity condition at top (1-3): \( c_{tz} = 1 \)
- Fixity condition at bottom (1-3): \( c_{bz} = 1 \)

For Y-Y stronger axis:
- Slenderness: \( \lambda_{y} = \frac{h}{\text{iy}} = 8.054 \)
Design moments (Clause 5.8.8.2)

Design moment

\[ M_{Ed} = M_0 = 38.281 \times 10^6 \text{ Nm} \]

Axial load and moment considerations

Effective/overall-depth ratio

\[ d_2 = d_2/h = 0.215 \]

Axial load ratio

\[ N_{rat} = N_{Ed}/(b \times h \times f_{ck}) = 0.19157 \]

Moment ratio

\[ M_{rat} = M_{Ed}/(b \times h^2 \times f_{ck}) = 0.031901 \]

Longitudinal reinforcement

Chart data for rectangular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'.

Area of steel required

\[ A_s = v_a \times f_{ck}/f_{yk} \times b \times h = 0 \text{ mm}^2/\text{m} \]

Minimum permitted area of steel

\[ A_{s, min} = 0.002 \times b \times h = 400 \text{ mm}^2/\text{m} \]

Provide minimum steel

\[ A_s = A_{s, min} = 400 \text{ mm}^2/\text{m} \]

Number of bars provided

\[ n_{bars} = 6/\text{m} \]

Wall thickness

200 mm

DESIGN SUMMARY

Clear wall height

3.1 m

Design axial load

1149.4 kN/m

Design BM (Z-Z axis)

38.281 kNm/m

Vert. reinft. diameter

16 mm

Area of steel required

400 mm$^2$/m

Area of steel provided

1206.4 mm$^2$/m

Horiz. reinft. diameter

10 mm

Horiz. reinft. spacing

400 mm
Location: Ex1 - Thin wall, HY steel, high moment, no concrete splay

Calculation of reinforcement for wall-to-wall intersections

Calculations are in accordance with BS8110-1:1997.

Concrete grade \( f_{cu} = 30 \text{ N/mm}^2 \)
Main steel strength \( f_y = 500 \text{ N/mm}^2 \)
Ultimate moment \( M = 125 \text{ kNm/m} \)
Overall wall thickness \( h = 150 \text{ mm} \)
Depth of splay (if provided) \( h_s = 0 \text{ mm} \)
Bar diameter \( d_{ia} = 25 \text{ mm} \)
Partial safety factor for steel \( \gamma_S = 1.15 \)

Cover to bars

Designated exposure class is XC2
Specified fixing tolerance \( t_{ol} = 10 \text{ mm} \)
Cover to main bars \( c = 35 \text{ mm} \)
Distance between bar centres \( z = h - 2c - d_{ia} = 55 \text{ mm} \)
Effective depth \( d = h - c - d_{ia}/2 = 102.5 \text{ mm} \)
Extra eff. depth due to splay \( a = h_s + c + d_{ia}/2 = 47.5 \text{ mm} \)

Method based on data given in 'Reinforcement detailing of frame corner joints' by P.S. Balint & H.P.J. Taylor (C&CA pub. 42.462. 1972).
Main steel

Steel area required
Limiting bar spacing (c.to c.)
Chosen bar spacing
Area of main steel provided

Splay reinforcement

Although a splay is not provided within the corner, diagonal bars will still be included within the corner itself. 

Calculated area of splay reinforcement:

Area of splay bars provided

Link reinforcement

Link steel strength
Minimum permissible link steel
Chosen size of link
Provide 5 No/20 mm size links at corner.

Minimum radius of bend for main U-bars according to C&CA Report:

DESIGN SUMMARY

Area of main steel required
Area of main steel provided
Percentage of main steel provided
Area of splay steel required
Area of splay steel provided
Area of link steel required
Area of link steel provided
Main steel: 25 mm bars at 90 mm centres (5454.2 mm²/m)
Splay steel: 16 mm bars at 90 mm centres (2234 mm²/m)
Link steel: 5 No/20 mm links/metre (3141.6 mm²/m)
Ratio of splay steel/main steel required
Ratio of splay steel/main steel provided
Minimum radius of bend for main U-bars 300 mm
Location: Ex2 - Thick wall, mild steel, low moment w. concrete splay

Calculation of reinforcement for wall-to-wall intersections

Calculations are in accordance with BS8110-1:1997.

Concrete grade \( f_{cu} = 30 \text{ N/mm}^2 \)
Main steel strength \( f_y = 250 \text{ N/mm}^2 \)
Ultimate moment \( M = 30 \text{ kNm/m} \)
Overall wall thickness \( h = 1000 \text{ mm} \)
Depth of splay (if provided) \( h_s = 250 \text{ mm} \)
Bar diameter \( d = 25 \text{ mm} \)
Partial safety factor for steel \( \gamma_S = 1.15 \)

Cover to bars

Designated exposure class is XC2
Specified fixing tolerance \( t_{ol} = 10 \text{ mm} \)
Cover to main bars \( c = 40 \text{ mm} \)
Distance between bar centres \( z = h - 2c - d = 895 \text{ mm} \)
Effective depth \( d = h - c - d/2 = 947.5 \text{ mm} \)
Extra eff. depth due to splay \( a = h_s + c + d/2 = 302.5 \text{ mm} \)

Noor formula 1

This design method is based on an article by Dr F.A. Noor, which appeared in 'Concrete' in July 1977.
Main steel

Chosen ratio of splay/main steel  ratio=0.5
Area of main steel required:
compute  \( c_1 = \frac{10200000 \cdot M - 1000 \cdot d^2 \cdot \text{SQR}(f_{cu})}{1000} \)
compute  \( c_2 = \frac{d \cdot \left( 350 \cdot \text{SQR}(f_{cu}) + 3.11 \cdot \text{ratio} \cdot f_y \right)}{2.1847E6} \)
Area of main steel required  \( A_{st} = \frac{c_1}{c_2} = -2110.7 \text{ mm}^2/\text{m} \)
Limiting bar spacing (c.to c.)  \( S_{\text{Max}} = 5 \cdot \text{INT} \left( \frac{\text{dia} + S_{\text{Max}}}{5} \right) = 625 \text{ mm} \)
Chosen bar spacing  \( \text{space}_2 = 120 \text{ mm} \)
Area of main steel provided  \( A_{st\text{pro}} = \frac{\pi}{4} \cdot \text{dia}^2 \cdot 1000 / \text{space}_2 = 4090.6 \text{ mm}^2/\text{m} \)

Splay reinforcement

Area of splay steel required  \( A_{ss} = \text{ratio} \cdot A_{st} = 2000 \text{ mm}^2/\text{m} \)
Chosen size of splay bar  \( d_{S2} = 20 \text{ mm} \)
Area of splay bars provided  \( A_{ss\text{pro}} = \frac{\pi}{4} \cdot d_{S2}^2 \cdot 1000 / \text{space}_2 = 2618 \text{ mm}^2/\text{m} \)

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of main steel required</td>
<td>4000 mm² per metre</td>
</tr>
<tr>
<td>Area of main steel provided</td>
<td>4090.6 mm² per metre</td>
</tr>
<tr>
<td>Percentage of main steel provided</td>
<td>0.40906 %</td>
</tr>
<tr>
<td>Area of splay steel required</td>
<td>2000 mm² per metre</td>
</tr>
<tr>
<td>Area of splay steel provided</td>
<td>2618 mm² per metre</td>
</tr>
<tr>
<td>Main steel: 25 mm bars at 120 mm centres</td>
<td>(4090.6 mm²/m)</td>
</tr>
<tr>
<td>Splay steel: 20 mm bars at 120 mm centres</td>
<td>(2618 mm²/m)</td>
</tr>
<tr>
<td>Ratio of splay steel/main steel required</td>
<td>0.5</td>
</tr>
<tr>
<td>Ratio of splay steel/main steel provided</td>
<td>0.64</td>
</tr>
<tr>
<td>Minimum radius of bend for main U-bars</td>
<td>145 mm</td>
</tr>
</tbody>
</table>
Location: Ex3 - Medium wall, HY steel, with concrete splay

Calculation of reinforcement for wall-to-wall intersections

Calculations are in accordance with BS8110-1:1997.
Concrete grade \( f_{cu} = 35 \text{ N/mm}^2 \)  
Main steel strength \( f_y = 500 \text{ N/mm}^2 \)  
Ultimate moment \( M = 200 \text{ kNm/m} \)  
Overall wall thickness \( h = 500 \text{ mm} \)  
Depth of splay (if provided) \( h_s = 250 \text{ mm} \)  
Bar diameter \( d = 20 \text{ mm} \)  
Partial safety factor for steel \( \gamma_S = 1.15 \)

Cover to bars

Designated exposure class is XC3  
Specified fixing tolerance \( \text{tol} = 10 \text{ mm} \)  
Cover to main bars \( c = 40 \text{ mm} \)  
Distance between bar centres \( z = h - 2c - d = 400 \text{ mm} \)  
Effective depth \( d = h - c - d = 450 \text{ mm} \)  
Extra eff. depth due to splay \( a = h_s + c + d = 300 \text{ mm} \)  
This design method is based on an article by Dr F.A. Noor, which appeared in 'Concrete' in July 1977.

Main steel

Chosen ratio of splay/main steel \( \text{ratio} = 0.5 \)  
Area of main steel required \( = 822.85 \text{ mm}^2/\text{m} \)  
Chosen bar spacing \( \text{space}_3 = 155 \text{ mm} \)  
Area of main steel provided \( \text{Astpro} = \pi/4 \times d^2 \times 1000 / \text{space}_3 = 2026.8 \text{ mm}^2/\text{m} \)

Splay reinforcement

Area of splay steel required \( \text{Ass} = \text{ratio} \times \text{Ast} = 1000 \text{ mm}^2/\text{m} \)  
Size of splay bar \( d_{S3} = 16 \text{ mm} \)  
Area of splay bars provided \( \text{Asspro} = \pi/4 \times d_{S3}^2 \times 2 \times 1000 / \text{space}_3 = 1297.2 \text{ mm}^2/\text{m} \)
## DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of main steel required</td>
<td>2000 mm$^2$ per metre</td>
</tr>
<tr>
<td>Area of main steel provided</td>
<td>2026.8 mm$^2$ per metre</td>
</tr>
<tr>
<td>Percentage of main steel provided</td>
<td>0.40537 %</td>
</tr>
<tr>
<td>Area of splay steel required</td>
<td>1000 mm$^2$ per metre</td>
</tr>
<tr>
<td>Area of splay steel provided</td>
<td>1297.2 mm$^2$ per metre</td>
</tr>
<tr>
<td>Main steel: 20 mm bars at 155 mm centres</td>
<td>(2026.8 mm$^2$/m)</td>
</tr>
<tr>
<td>Splay steel: 16 mm bars at 155 mm centres</td>
<td>(1297.2 mm$^2$/m)</td>
</tr>
<tr>
<td>Ratio of splay steel/main steel required</td>
<td>0.5</td>
</tr>
<tr>
<td>Ratio of splay steel/main steel provided</td>
<td>0.64</td>
</tr>
<tr>
<td>Minimum radius of bend for main U-bars</td>
<td>180 mm</td>
</tr>
</tbody>
</table>
Location: Ex1 - Thin wall, HY steel, high moment, no concrete splay

Calculation of reinforcement for wall-to-wall intersections

The detail below entails intersecting pairs of vertical U-bars together with diagonal splay bars and diagonal links binding splay reinforcement and trimming bars round outside of corner (if required). Calculations are in accordance with BS EN 1992-1-1.

Design data

Char yield strength of r'ment \( f_yk = 500 \text{ N/mm}^2 \)
Design moment \( \text{MED} = 125 \text{ kNm/m} \)
Overall wall thickness \( h = 150 \text{ mm} \)
Depth of splay (if provided) \( hs = 0 \text{ mm} \)
Bar diameter (vertical U-bars) \( dia = 25 \text{ mm} \)
Partial safety factor for steel \( \gamma_{S} = 1.15 \)

Cover to main reinforcement

Distance between bar centres \( z = h - 2c - dia = 55 \text{ mm} \)
Effective depth \( d = h - c - dia/2 = 102.5 \text{ mm} \)
Extra eff. depth due to splay \( a = hs + c + dia/2 = 47.5 \text{ mm} \)

Design method by C&CA (basic detail)

Method based on data given in 'Reinforcement detailing of frame corner joints' by P.S. Balint & H.P.J. Taylor (C&CA pub. 42.462. 1972).

SCALE 5.48 Office 1007 Proforma 77
Main steel

Absolute maximum permitted clear bar spacing is 400 mm.
Limiting bar spacing (c.to c.) \( \text{SMax}=5\times\text{INT}((\text{dia}+3\times\text{d})/5)=330 \text{ mm} \)
Maximum allowable bar spacing to accommodate steel required=90 mm
Chosen bar spacing \( \text{space1}=90 \text{ mm} \)
Area of main steel provided \( \text{Astpro}=\pi/4\times\text{dia}^2\times1000/\text{space1} =5454.2 \text{ mm}^2/\text{m} \)
Corresponding steel percentage \( \text{perpro}=100\times\text{Astpro}/(1000\times\text{h})=3.6361 \% \)

Splay reinforcement

Although a splay is not provided within the corner, diagonal bars will still need to be included within the corner itself.
Calculated area of splay reinforcement:
\[
\text{Ass}=\sqrt{2}\times(\text{Med}\times1E6-0.6\times\text{Ast}\times\text{fyk}/\text{gamS}\times\text{z})/(\text{fyk}/\text{gamS}\times(a+z))=1586.7 \text{ mm}^2
\]
Chosen size of splay bar \( \text{diS1}=16 \text{ mm} \)
Area of splay bars provided \( \text{Asspro}=\pi/4\times\text{diS1}^2\times1000/\text{space1} =2234 \text{ mm}^2/\text{m} \)

Link reinforcement

Link steel strength \( \text{fyv}=500 \text{ N/mm}^2 \)
Minimum permissible link steel \( \text{Asv}=0.6\times\text{fyk}\times\text{Ast}/\text{fyv}=3136.4 \text{ mm}^2/\text{m} \)
Number of links required per metre 5 No.
Chosen size of link \( \text{Ldia}=20 \text{ mm} \)
Provide 5 No/20 mm size links at corner.
Area of link steel provided \( \text{Asvpro}=\text{Ldia}\times\text{Ldia}\times\pi/2\times\text{NumLink}/2 =3141.6 \text{ mm}^2/\text{m} \)
Minimum radius of bend in U-bars according to C&CA Report:
\[
\text{MinRad}=0.35\times\text{dia}\times\text{fyk}/\text{fcu}\times(5\times\text{dia}+2\times\text{c})/(\text{dia}+2\times\text{c})=299.34 \text{ mm}
\]
Min.radius of bend for main U-bars=300 mm.

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of main steel required</td>
<td>5227.3 mm² per metre</td>
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<tr>
<td>Area of main steel provided</td>
<td>5454.2 mm² per metre</td>
</tr>
<tr>
<td>Percentage of main steel provided</td>
<td>3.6361 %</td>
</tr>
<tr>
<td>Area of splay steel required</td>
<td>1586.7 mm² per metre</td>
</tr>
<tr>
<td>Area of splay steel provided</td>
<td>2234 mm² per metre</td>
</tr>
<tr>
<td>Area of link steel required</td>
<td>3136.4 mm² per metre</td>
</tr>
<tr>
<td>Area of link steel provided</td>
<td>3141.6 mm² per metre</td>
</tr>
<tr>
<td>Main steel : 25 mm bars at 90 mm centres</td>
<td>(5454.2 mm²/m)</td>
</tr>
<tr>
<td>Splay steel: 16 mm bars at 90 mm centres</td>
<td>(2234 mm²/m)</td>
</tr>
<tr>
<td>Link steel : 5 No/20 mm links/metre</td>
<td>(3141.6 mm²/m)</td>
</tr>
<tr>
<td>Ratio of splay steel/main steel required</td>
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<td>Minimum radius of bend for main U-bars</td>
<td>300 mm</td>
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</table>
**Location:** Ex2 - Thick wall, mild steel, low moment w. concrete splay

**Calculation of reinforcement for wall-to-wall intersections**

The detail below entails intersecting pairs of vertical U-bars together with diagonal splay bars and diagonal links binding splay reinforcement and trimming bars round outside of corner (if required). Calculations are in accordance with BS EN 1992-1-1.

**Design data**

- **Char yield strength of r'ment** \( f_yk = 500 \text{ N/mm}^2 \)
- **Design moment** \( M_{Ed} = 30 \text{ kNm/m} \)
- **Overall wall thickness** \( h = 1000 \text{ mm} \)
- **Depth of splay (if provided)** \( h_{s} = 250 \text{ mm} \)
- **Bar diameter (vertical U-bars)** \( d_{ia} = 25 \text{ mm} \)
- **Partial safety factor for steel** \( \gamma_S = 1.15 \)

**Cover to main reinforcement**

- **Cover to main bars** \( c = 40 \text{ mm} \)
- **Distance between bar centres** \( z = h - 2c - d_{ia} = 895 \text{ mm} \)
- **Effective depth** \( d = h - c - d_{ia}/2 = 947.5 \text{ mm} \)
- **Extra eff.depth due to splay** \( a = h - c - d_{ia}/2 = 302.5 \text{ mm} \)

**Design method by C&CA (basic detail)**

Method based on data given in 'Reinforcement detailing of frame corner joints' by P.S. Balint & H.P.J. Taylor (C&CA pub. 42.462. 1972).
Main steel

Absolute maximum permitted clear bar spacing is 400 mm.
Maximum allowable bar spacing to accommodate steel required = 245 mm
Chosen bar spacing = 245 mm
Area of main steel provided = \( \frac{\pi/4 \times \text{dia} \times \text{dia} \times 1000}{\text{space1}} \) = 2003.6 mm²/m
Corresponding steel percentage = \( \frac{100 \times \text{Astpro}}{1000 \times \text{h}} \) = 0.20036 %

Splay reinforcement

Calculated area of splay reinforcement:
\[ \text{Ass} = \sqrt{2} \times \left( \text{MED} \times 1E6 - 0.6 \times \text{Ast} \times \frac{\text{fyk}}{\gamma_S} \times \text{z} \right) / \left( \frac{\text{fyk}}{\gamma_S} \times (a + z) \right) \] = -1186.9 mm²

Link reinforcement

Minimum radius of bend in U-bars according to C&CA Report:
\[ \text{MinRad} = 0.35 \times \text{dia} \times \frac{\text{fyk}}{\text{fcu}} \times (5 \times \text{dia} + 2 \times c) / (\text{dia} + 2 \times c) \] = 284.72 mm
Min. radius of bend for main U-bars = 285 mm.

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of main steel required</td>
<td>2000 mm² per metre</td>
</tr>
<tr>
<td>Area of main steel provided</td>
<td>2003.6 mm² per metre</td>
</tr>
<tr>
<td>Percentage of main steel provided</td>
<td>0.20036 %</td>
</tr>
<tr>
<td>Area of splay steel required</td>
<td>See note below</td>
</tr>
<tr>
<td>Main steel : 25 mm bars at 245 mm centres</td>
<td>( 2003.6 mm²/m )</td>
</tr>
<tr>
<td>Ratio of splay steel/main steel required</td>
<td>0</td>
</tr>
<tr>
<td>Minimum radius of bend for main U-bars</td>
<td>285 mm</td>
</tr>
</tbody>
</table>
Location: Ex3 - Medium wall, HY steel, with concrete splay

Calculation of reinforcement for wall-to-wall intersections

The detail below entails intersecting pairs of vertical U-bars together with diagonal splay bars and diagonal links binding splay reinforcement and trimming bars round outside of corner (if required). Calculations are in accordance with BS EN 1992-1-1.

**Design data**

- Char yield strength of reinforcement: \( f_{yk} = 500 \text{ N/mm}^2 \)
- Design moment: \( M_{Ed} = 300 \text{ kNm/m} \)
- Overall wall thickness: \( h = 500 \text{ mm} \)
- Depth of splay (if provided): \( h_s = 250 \text{ mm} \)
- Bar diameter (vertical U-bars): \( \text{dia} = 25 \text{ mm} \)
- Partial safety factor for steel: \( \gamma_{S} = 1.15 \)
- Cover to main bars: \( c = 30 \text{ mm} \)
- Distance between bar centres: \( z = h - 2c - \text{dia} = 415 \text{ mm} \)
- Effective depth: \( d = h - c - \text{dia}/2 = 457.5 \text{ mm} \)
- Extra eff. depth due to splay: \( a = h_s + c + \text{dia}/2 = 292.5 \text{ mm} \)

**Design method by C&CA (basic detail)**

Method based on data given in 'Reinforcement detailing of frame corner joints' by P.S. Balint & H.P.J. Taylor (C&CA pub. 42.462.1972).
Main steel

Absolute maximum permitted clear bar spacing is 400 mm.
Limiting bar spacing (c.to c.) SMax=5*INT((dia+SMax)/5)=400 mm
Maximum allowable bar spacing to accommodate steel required=295 mm
Chosen bar spacing spacel=295 mm
Area of main steel provided Astpro=PI/4*dia*dia*1000/spacel =1664 mm²/m
Corresponding steel percentage perpro=100*Astpro/(1000*h)=0.3328%

Splay reinforcement

Calculated area of splay reinforcement:
Ass=SQR(2)*(MEd*1E6-0.6*Ast*fyk/gamS*z)/(fyk/gamS*(a+z))=551.69 mm²
Chosen size of splay bar diS1=16 mm
Area of splay bars provided Asspro=PI/4*diS1^2*1000/spacel =681.57 mm²/m

Link reinforcement

Minimum radius of bend in U-bars according to C&CA Report:
MinRad=0.35*dia*fyk/fcu*(5*dia+2*c)/(dia+2*c) =272.06 mm
Min. radius of bend for main U-bars=275 mm.

DESIGN SUMMARY

<table>
<thead>
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<th>Description</th>
<th>Value</th>
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<tbody>
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<td>Area of main steel required</td>
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<tr>
<td>Area of main steel provided</td>
<td>1664 mm² per metre</td>
</tr>
<tr>
<td>Percentage of main steel provided</td>
<td>0.3328 %</td>
</tr>
<tr>
<td>Area of splay steel required</td>
<td>551.69 mm² per metre</td>
</tr>
<tr>
<td>Area of splay steel provided</td>
<td>681.57 mm² per metre</td>
</tr>
<tr>
<td>Main steel : 25 mm bars at 295 mm centres</td>
<td>(1664 mm²/m)</td>
</tr>
<tr>
<td>Splay steel: 16 mm bars at 295 mm centres</td>
<td>(681.57 mm²/m)</td>
</tr>
<tr>
<td>Ratio of splay steel/main steel required</td>
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</tr>
<tr>
<td>Ratio of splay steel/main steel provided</td>
<td>0.4096</td>
</tr>
<tr>
<td>Minimum radius of bend for main U-bars</td>
<td>275 mm</td>
</tr>
</tbody>
</table>
Location: Ex4 - Using Noor formula 1

Calculation of reinforcement for wall-to-wall intersections

The detail below entails intersecting pairs of vertical U-bars together with diagonal splay bars and diagonal links binding splay reinforcement and trimming bars round outside of corner (if required). Calculations are in accordance with BS EN 1992-1-1.

![Diagram of reinforcement layout]

Design data

- Char yield strength of reinforcement: $f_{yK}=500$ N/mm$^2$
- Design moment: $M_{Ed}=30$ kNm/m
- Overall wall thickness: $h=1000$ mm
- Depth of splay (if provided): $h_s=250$ mm
- Bar diameter (vertical U-bars): $diameter=25$ mm
- Partial safety factor for steel: $\gamma_{S}=1.15$

Cover to main reinforcement

- Cover to main bars: $c=40$ mm
- Distance between bar centres: $z=h-2c-diameter=895$ mm
- Effective depth: $d=h-c-diameter/2=947.5$ mm
- Extra eff. depth due to splay: $a=h-s+diame/2=302.5$ mm

Design method using Noor formula 1

This calculation deals with the design of the reinforcement to resist bending at wall-to-wall intersections where the design moment tends to 'open' the corner. Tests have shown that many details fail at well below their calculated strength, and the concrete details described in this proforma have been developed to overcome this problem. The design method adopted here is based on an article by Dr F.A. Noor, which appeared in a publication entitled 'Concrete' in July 1977.
Main steel

Chosen ratio of splay/main steel ratio=0.5
Area of main steel required:
compute \[ c1=(10200000*\text{MEd}-1000*d^2*\text{SQR}(\text{fcu})) \]
compute \[ c2=(d*(350*\text{SQR}(\text{fcu})+3.11*\text{ratio*fyk})) \]
Area of main steel required \[ Ast=c1/c2=-1806.1 \text{ mm}^2/\text{m} \]
Maximum allowable bar spacing=245 mm
Chosen bar spacing \[ \text{space2}=245 \text{ mm} \]
Area of main steel provided \[ Astpro=\text{PI}/4*\text{dia}^2*1000/\text{space2} =2003.6 \text{ mm}^2/\text{m} \]

Splay reinforcement

Area of splay steel required \[ Ass=\text{ratio}*Ast=1000 \text{ mm}^2/\text{m} \]
Chosen size of splay bar \[ \text{diS2}=20 \text{ mm} \]
Area of splay bars provided \[ Asspro=\text{PI}/4*\text{diS2}^2*1000/\text{space2} =1282.3 \text{ mm}^2/\text{m} \]

Link reinforcement

Minimum radius of bend in U-bars according to C&CA Report:
\[ \text{MinRad}=0.35*\text{dia*fyk}/\text{fcu}*(5*\text{dia}+2*c)/(\text{dia}+2*c) \]
\[ =284.72 \text{ mm} \]
Min. radius of bend for main U-bars=285 mm.

DESIGN SUMMARY

| Area of main steel required | 2000 \text{ mm}^2 \text{ per metre} |
| Area of main steel provided | 2003.6 \text{ mm}^2 \text{ per metre} |
| Percentage of main steel provided | 0.20036 \% |
| Area of splay steel required | 1000 \text{ mm}^2 \text{ per metre} |
| Area of splay steel provided | 1282.3 \text{ mm}^2 \text{ per metre} |
| Main steel : 25 mm bars at 245 mm centres ( 2003.6 \text{ mm}^2/\text{m} ) |
| Splay steel: 20 mm bars at 245 mm centres ( 1282.3 \text{ mm}^2/\text{m} ) |
| Ratio of splay steel/main steel required | 0.5 |
| Ratio of splay steel/main steel provided | 0.64 |
| Minimum radius of bend for main U-bars | 285 mm |
Location: Ex5 - Using Noor formula 2

Calculation of reinforcement for wall-to-wall intersections

The detail entails intersecting pairs of horizontal U-bars together with diagonal splay bars. This design method is based on an article by Dr F.A. Noor, published in 'Concrete' in July 1977. Calculations are in accordance with BS EN 1992-1-1.

Design data

Char yield strength of r'ment $f_{yk}=500$ N/mm²
Design moment $M_{Ed}=300$ kNm/m
Overall wall thickness $h=600$ mm
Depth of splay (if provided) $h_s=250$ mm
Bar diameter (horizontal U-bars) $dia=20$ mm
Partial safety factor for steel $\gamma_S=1.15$

Cover to main reinforcement

Cover to main bars $c=30$ mm
Distance between bar centres $z=h-2c-dia=520$ mm
Effective depth $d=h-c-dia/2=560$ mm
Extra eff. depth due to splay $a=h_s+c+dia/2=290$ mm

This design method is based on an article by Dr F.A. Noor, which appeared in 'Concrete' in July 1977.

Main steel

Chosen ratio of splay/main steel ratio=0.5
Area of main steel required=$973.14$ mm²/m.
Maximum allowable bar spacing 260 mm
Chosen bar spacing $space_3=260$ mm
Area of main steel provided $Astpro=\pi/4*dia*dia*1000/space_3$
  $=1208.3$ mm²/m
Splay reinforcement

Area of splay steel required: $\text{Ass} = \text{ratio} \times \text{Ast} = 600 \text{ mm}^2/\text{m}$
Size of splay bar: $d_{\text{S3}} = 16 \text{ mm}$
Area of splay bars provided: $\text{Ass}_{\text{pro}} = \frac{\pi}{4} \times d_{\text{S3}}^2 \times 1000 / \text{space}_3 = 773.32 \text{ mm}^2/\text{m}$

**DESIGN SUMMARY**

<table>
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<tr>
<th>Area of main steel required</th>
<th>1200 mm$^2$ per metre</th>
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</thead>
<tbody>
<tr>
<td>Area of main steel provided</td>
<td>1208.3 mm$^2$ per metre</td>
</tr>
<tr>
<td>Percentage of main steel provided</td>
<td>0.20138 %</td>
</tr>
<tr>
<td>Area of splay steel required</td>
<td>600 mm$^2$ per metre</td>
</tr>
<tr>
<td>Area of splay steel provided</td>
<td>773.32 mm$^2$ per metre</td>
</tr>
<tr>
<td>Main steel: 20 mm bars at 260 mm centres (1208.3 mm$^2$/m)</td>
<td></td>
</tr>
<tr>
<td>Splay steel: 16 mm bars at 260 mm centres (773.32 mm$^2$/m)</td>
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</tr>
<tr>
<td>Ratio of splay steel/main steel required</td>
<td>0.5</td>
</tr>
<tr>
<td>Ratio of splay steel/main steel provided</td>
<td>0.64</td>
</tr>
<tr>
<td>Minimum radius of bend for main U-bars</td>
<td>200 mm</td>
</tr>
</tbody>
</table>
**Location:** Medium-sized section. Low shear force.

**Shear in rectangular and flanged beams, with choice of av enhancement**

The shear calculations are in accordance with the requirements of Clause 3.4.5

Design to BS8110 (1997) with partial safety factor for steel gammaS=1.15

Shear force due to ultimate loads $V=400 \text{ kN}$

Overall depth of section $h=550 \text{ mm}$

Effective depth of section $d=500 \text{ mm}$

Effective breadth for shear $b_v=200 \text{ mm}$

Characteristic concrete strength $f_{cu}=40 \text{ N/mm}^2$

Distance to support $a_v=500 \text{ mm}$

Diameter of compression bars $d_{iac}=0 \text{ mm}$

Diameter of tension bars $d_{ia}=25 \text{ mm}$

Diameter of link legs $d_{ial}=8 \text{ mm}$

Number of tension bars at section $n_{bars}=2$

Char. strength of link steel $f_{yv}=500 \text{ N/mm}^2$

Number of legs to be provided $n_{legs}=4$

Use 8 mm links (4 legs), spaced at 170 mm centres along beam.

Approx. total weight of links 5.8 kg per metre
Location: Large section. High shear force. High-yield steel.

Shear in rectangular and flanged beams, with choice of

av enhancement

The shear calculations are in accordance with the requirements of Clause 3.4.5

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Shear force due to ultimate loads V=7000 kN

Overall depth of section h=1500 mm
Effective depth of section d=1450 mm
Effective breadth for shear bv=1000 mm
Characteristic concrete strength fcu=40 N/mm²
Distance to support av=3000 mm
Diameter of compression bars diac=25 mm
Diameter of tension bars dia=40 mm
Diameter of link legs dial=10 mm
Number of tension bars at section nbars=12
Char.strength of link steel fyv=500 N/mm²

Since bv exceeds 350 mm, note the condition in Clause 3.4.5.5 that no longitudinal bar should be more than 150 mm from a vertical leg.

Number of legs to be provided nlegs=10

Use 10 mm links (10 legs), spaced at 80 mm centres along beam.
Approx.total weight of links 159.5 kg per metre
Location: Small section. High shear enhancement.

Shear in rectangular and flanged beams, with choice of
av enhancement

The shear calculations are in accordance with the requirements of Clause 3.4.5

Design to BS8110 (1997) with partial safety factor for steel gammaS=1.15
It is assumed that nominal shear reinforcement at least will be provided
Shear force due to ultimate loads V=25 kN
Overall depth of section h=100 mm
Effective depth of section d=75 mm
Effective breadth for shear bv=75 mm
Characteristic concrete strength fcu=35 N/mm²
Distance to support av=25 mm
Diameter of compression bars diac=0 mm
Diameter of tension bars dia=12 mm
Diameter of link legs dial=8 mm
Number of tension bars at section nbars=2
Char.strength of link steel fyv=500 N/mm²
Number of legs to be provided nlegs=2
Proposed spacing sv greater than 0.75*d= 0.75*75 =56.25 mm
Reset link spacing to limiting value of 50 mm (rounded).

Use 8 mm links (two legs), spaced at 50 mm centres along beam.
Approx.total weight of links 1.8 kg per metre
Location: Ex1 - Default example

Shear in rectangular and flanged beams

Design shear force $V_{Ed}=345$ kN
Overall depth of section $h=600$ mm
Effective depth of section $d=550$ mm
Effective breadth for shear $b_w=350$ mm
Char yield strength (reinf'ment) $f_yk=500$ N/mm²
Diameter of links $d_{al}=10$ N/mm²

Shear reinforcement

Design shear stress $v_{Ed}=\frac{V_{Ed}*10^3}{(0.9*b_w*d)}=1.9913$ N/mm²
Concrete strut strength $v_{Rdm}=0.124*f_{ck}*(1-f_{ck}/250)$
$=3.2736$ N/mm²
Angle of inclination of strut $\theta'=22^\circ$
Number of shear legs $n_{sl}=2$
Area of shear legs $A_{sw}=n_{sl}*\pi/4*d_{al}^2=157.08$ mm²
Spacing of links $s_{max}=\text{INT}(A_{sw}/A_{sw}'s)=244$ mm
Chosen spacing $s=225$ mm

Check crushing strength

Angle of inclination of strut $\theta'=22^\circ$
compute $\text{comp}=\cot(\theta)=1/\tan(\theta)=2.4751$
Maximum shear resistance:
$V_{Rdmax}=0.36*b_w*d*(1-f_{ck}/250)*f_{ck}/((\text{comp}+1/\text{comp})*1000)=635.45$ kN
As $V_{Ed} \leq V_{Rdmax}$ (345 kN ≤ 635.45 kN), design shear force OK.

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>345 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear strength</td>
<td>1.9913 N/mm²</td>
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</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.2736 N/mm²</td>
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<tr>
<td>Area-spacing ratio</td>
<td>0.64121</td>
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</tr>
<tr>
<td>Area of links</td>
<td>157 mm²</td>
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<tr>
<td>Maximum spacing</td>
<td>244 mm</td>
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</tr>
<tr>
<td>Actual spacing</td>
<td>225 mm</td>
<td></td>
</tr>
</tbody>
</table>
Location: Typical continuous slab reinforced with high-yield steel

Design of solid r.c. slabs reinforced in tension only.

Calculations for singly-reinforced slabs using formulae in Cl.3.4.4.4, i.e. assuming rectangular stress block and restricting NA depth to 0.4*d at supports (except supports to cantilevers) and 0.5*d near midspan (redistribution, if any, limited to 10%).

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Section location: Near midspan of span with both ends continuous.

Ultimate moment (after redstrb.) M=350 kNm/m
Characteristic concrete strength fcu=40 N/mm²
Characteristic strength of steel fy=500 N/mm²
Diameter of tension bars dia=20 mm
Designated exposure class is XC2
Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=35 mm
Overall thickness of slab h=400 mm
Effective depth of section d=355 mm
Chosen spacing of tension bars pch=125 mm
Diameter of distribution bars diamn=12 mm
Spacing of distribution bars pchDA=200 mm

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing of bars</td>
<td>125 mm</td>
</tr>
<tr>
<td></td>
<td>Effective depth</td>
<td>355 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required</td>
<td>2476.2 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided</td>
<td>2513 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Percentage provided</td>
<td>0.62825 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided</td>
<td>19.73 kg/m²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DISTRIBUTION</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing of bars</td>
<td>200 mm</td>
</tr>
<tr>
<td></td>
<td>Depth to bar centres</td>
<td>339 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required</td>
<td>520 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided</td>
<td>565 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Percentage provided</td>
<td>0.14125 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided</td>
<td>4.44 kg/m²</td>
</tr>
</tbody>
</table>
Check on span/effective-depth ratio

Basic ratio for continuous slab \( bs'd=26 \) (see Table 3.9)
Area of tension steel provided \( Aspr=2513 \text{ mm}^2 \)
Service stress in this steel \( fs=2*fy*As/(3*Aspr)=328.45 \text{ N/mm}^2 \)
Mod.factor for tension steel \( modf1=0.88664 \)
Mod.factor for no comp.steel \( modf2=1.0 \)
Maximum permissible span/effective-depth ratio \( ps'd=bs'd*modf1*modf2=23.053 \)
Effective span of slab \( span=8 \text{ m} \)
True span/effective-depth ratio \( as'd=1000*span/d=22.535 \)
As this does not exceed \( 23.053 \), this is Acceptable.
Location: Thick, heavily-loaded continuous slab, mild steel reinfmt.

Design of solid r.c. slabs reinforced in tension only.

Calculations for singly-reinforced slabs using formulae in Cl.3.4.4.4, i.e. assuming rectangular stress block and restricting NA depth to 0.4*d at supports (except supports to cantilevers) and 0.5*d near midspan (redistribution, if any, limited to 10%).

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

Section location: Near midspan of span with both ends continuous.

Ultimate moment (after redistribution) \( M = 2500 \text{ kNm/m} \)

Characteristic concrete strength \( f_{cu} = 35 \text{ N/mm}^2 \)

Characteristic strength of steel \( f_y = 250 \text{ N/mm}^2 \)

Diameter of tension bars \( \text{dia} = 40 \text{ mm} \)

Designated exposure class is XC2

Specified fixing tolerance \( \text{tol} = 10 \text{ mm} \)

Nominal concrete cover \( \text{cover} = 45 \text{ mm} \)

Overall thickness of slab \( h = 750 \text{ mm} \)

Effective depth of section \( d = 685 \text{ mm} \)

Chosen spacing of tension bars \( p_{ch} = 50 \text{ mm} \)

Diameter of distribution bars \( \text{diamn} = 20 \text{ mm} \)

Spacing of distribution bars \( p_{chDA} = 170 \text{ mm} \)

### TENSION REINFORCEMENT SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>40 mm</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>50 mm</td>
</tr>
<tr>
<td>Effective depth</td>
<td>685 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>21404 mm²/m</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>25132 mm²/m</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>3.3509 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>197.29 kg/m²</td>
</tr>
</tbody>
</table>

### DISTRIBUTION REINFORCEMENT SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>170 mm</td>
</tr>
<tr>
<td>Depth to bar centres</td>
<td>655 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>1800 mm²/m</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>1847 mm²/m</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.24627 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>14.5 kg/m²</td>
</tr>
</tbody>
</table>
Check on span/effective-depth ratio

Basic ratio for continuous slab \(bs'd=26\) (see Table 3.9)
Area of tension steel provided \(Aspr=25132\) mm²
Service stress in this steel \(fs=2*fy*As/(3*Aspr)=141.94\) N/mm²
Mod.factor for tension steel \(modf1=0.99833\)
Mod.factor for no comp.steel \(modf2=1.0\)
Maximum permissible
span/effective-depth ratio \(ps'd=bs'd*modf1*modf2=25.956\)
Effective span of slab \(span=12\) m
Although span exceeds 10 m, partitions/finishes are not considered sensitive to deflection, so modification to basic ratio is unnecessary.
True span/effective-depth ratio \(as'd=1000*span/d=17.518\)
As this does not exceed 25.956, this is Acceptable.
Design of solid r.c. slabs reinforced in tension only.

Calculations for singly-reinforced slabs using formulae in Cl.3.4.4.4, i.e. assuming rectangular stress block and restricting NA depth to 0.4*d at supports (except supports to cantilevers) and 0.5*d near midspan (redistribution, if any, limited to 10%).

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Section location: Near midspan of span with both ends continuous.

Ultimate moment (after redstrb.) $M=15 \text{ kNm/m}$

Characteristic concrete strength $f_{cu}=60 \text{ N/mm}^2$

Characteristic strength of steel $f_y=425 \text{ N/mm}^2$

$f_y$ has been set to less than the characteristic strength in order to achieve a design which satisfies span/effective-depth requirements.

Diameter of tension bars $\text{dia}=16 \text{ mm}$

Designated exposure class is XC4

Specified fixing tolerance $\text{tol}=10 \text{ mm}$

Nominal concrete cover $\text{cover}=30 \text{ mm}$

Overall thickness of slab $h=100 \text{ mm}$

Effective depth of section $d=62 \text{ mm}$

Chosen spacing of tension bars $\text{pch}=200 \text{ mm}$

Diameter of distribution bars $\text{diamn}=8 \text{ mm}$

Spacing of distribution bars $\text{pchDA}=190 \text{ mm}$

**TENSION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>425 N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>200 mm</td>
</tr>
<tr>
<td>Effective depth</td>
<td>62 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>710.35 mm$^2$/m</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>1005 mm$^2$/m</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>1.005 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>7.89 kg/m$^2$</td>
</tr>
</tbody>
</table>

**DISTRIBUTION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>425 N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>8 mm</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>190 mm</td>
</tr>
<tr>
<td>Depth to bar centres</td>
<td>50 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>130 mm$^2$/m</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>264 mm$^2$/m</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.264 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>2.07 kg/m$^2$</td>
</tr>
</tbody>
</table>
Check on span/effective-depth ratio

Basic ratio for continuous slab \( bs'd = 26 \) (see Table 3.9)
Area of tension steel provided \( Aspr = 1005 \text{ mm}^2 \)
Service stress in this steel \( fs = 2*fy*As/(3*Aspr) = 200.26 \text{ N/mm}^2 \)
Mod.factor for tension steel \( modf1 = 1.0302 \)
Mod.factor for no comp.steel \( modf2 = 1.0 \)
Maximum permissible
span/effective-depth ratio \( ps'd = bs'd*modf1*modf2 = 26.786 \)
Effective span of slab \( span = 8 \text{ m} \)
True span/effective-depth ratio \( as'd = 1000*span/d = 129.03 \)
Perm. span/effective-depth ratio 26.786
Hence unacceptable.

WARNING:
Either provide a thicker slab, or reduce the service stress.
This can be achieved by repeating the calculations and employing a lower value for the characteristic steel strength.
Location: Ex1 - Continuous slab

Bending in solid slabs (with comp.steel if reqd.),
designed per metre width, with checks on minimum steel

and span/effective-depth ratio

Calculations to EN 1992-1-1:2004

Eurocode 2 : Design of concrete structures & assume the use of a simplified rectangular concrete stress-block, and that the depth to the NA is restricted to 0.45d

Design BM before redistribution Mbef=625 kNm
Section being analysed is considered as continuous.
Section considered has a hogging moment.
Design BM (after redistribution) M=600 kNm

Char yield strength of reinft fyk=500 N/mm²
Max.aggregate size (for bar spc.) hagg=20 mm
Diameter of tension bars dia=25 mm
Diameter of distribution bars diad=12 mm
Effective depth of section d=350 mm

DESIGN

Overall depth 400 mm
Effective depth 350 mm

SUMMARY

Parameter K 0.1633
Parameter K' 0.1684
Lever arm ratio z/d 0.8255
Steel area (tension) 4776 mm²/m
Steel percentage req. 1.365 %
Minimum area of steel 527.2 mm²/m
Maximum area of steel 16000 mm²/m
Distribution steel 955.2 mm²/m

Chosen spacing of tension bars pch=100 mm
Diameter of distribution bars diad=12 mm
Chosen spacing of distn. bars pchd=110 mm

TENSION REINFORCEMENT

Diameter of bars 25 mm
Spacing of bars 100 mm
Area of steel required 4776 mm²/m
Area of steel provided 4908 mm²/m

DISTRIBUTION REINFORCEMENT

Diameter of bars 12 mm
Spacing of bars 110 mm
Area of steel required 955.2 mm²/m
Area of steel provided 1028 mm²/m
Shear

Location for shear calculation: 350mm from internal support
Shear force due to ultimate load $V_{Ed}=250$ kN
Term for shear resistance $vR_{dc}=(\rho_1*f_{ck})^{1/3}=3.478$
Shear resistance $V_{Rdc}=C_{Rdc}*ks*vR_{dc}*bw*d/1000=256.5$ kN
Minimum stress $v_{min}=0.035*ks^{1.5}*f_{ck}^0.5=0.4461$ N/mm²
Minimum shear resistance $V_{Rdm}=v_{min}*bw*d/1000=156.1$ kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Design shear force</th>
<th>250 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear resistance</td>
<td>256.5 kN</td>
</tr>
</tbody>
</table>

Crack control check

Unfactored permanent load on slab $G_k=100$ kN
Unfactored variable load on slab $Q_k=40$ kN
Permanent load factor $\gamma_G=1.35$
Variable load factor $\gamma_Q=1.5$
Factor $\psi_2$ for variable load $\psi_2=0.3$
SLS stress in reinforcement $\sigma_{SLS}=500/1.15*(A_s/A_{spr})*ratio*1/delta=253.1$ N/mm²
Design crack width $w_k=0.3$ mm
Max allowable SLS stress $\sigma_{SLS2}=TABLE 7.3$ for $p_{ch}=100$

$=320$ N/mm²

The bar spacing 100 mm used complies with Table 7.3N of EC2 Part 1:2004 hence satisfactory.
Location: Ex2 - Simply supported slab

Bending in solid slabs (with comp. steel if reqd.),
designed per metre width, with checks on minimum steel
and span/effective-depth ratio

Calculations to EN 1992-1-1:2004
Eurocode 2 : Design of concrete structures & assume the use of a simplified rectangular concrete stress-block, and that the depth to the NA is restricted to 0.45d

Design BM before redistribution Mbef=150 kNm
Section being analysed is considered as non-continuous.
Char. yield strength of reinft fyk=500 N/mm²
Max. aggregate size (for bar spc.) hagg=20 mm
Diameter of tension bars dia=16 mm
Diameter of distribution bars diad=8 mm
Effective depth of section d=245 mm

DESIGN
Overall depth 300 mm
SUMMARY
Effective depth 245 mm
FLEXURE
Parameter K 0.0714
Parameter K' 0.1684
Lever arm ratio z/d 0.9324
Steel area (tension) 1510 mm²/m
Steel percentage req. 0.6164 %
Minimum area of steel 408.9 mm²/m
Maximum area of steel 12000 mm²/m
Distribution steel 408.9 mm²/m

Chosen spacing of tension bars pch=130 mm
Diameter of distribution bars diad=8 mm
Chosen spacing of distrn. bars pchd=120 mm

TENSION REINFORCEMENT
Diameter of bars 16 mm
Spacing of bars 130 mm
Area of steel required 1510 mm²/m
Area of steel provided 1546 mm²/m

DISTRIBUTION REINFORCEMENT
Diameter of bars 8 mm
Spacing of bars 120 mm
Area of steel required 408.9 mm²/m
Area of steel provided 418 mm²/m

Deflection check
Effective span of beam L=4.3 m
Actual span to depth ratio l'd=L*1000/d=17.55
Allowable span/depth ratio l'da=k*N*F1*F2*F3=19.98
DESIGN
Actual l/d ratio  17.55
SUMMARY
Basic l/d ratio  19.52
DEFLECTION
Span factor  1
Flange beam factor F1  1
Long spans factor F2  1
Steel stress factor F3  1.024
Allowable l/d ratio  19.98

Note: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for highly stressed concrete (C30/37 & ρ=1.5%) as follows:

Simply supported spans  14
End spans  18
Interior spans  20

Shear

Location for shear calculation:  250mm from support
Shear force due to ultimate load  VEd=125 kN
Term for shear resistance  vRdc=(rho1*fck)^(1/3)=2.227
Shear resistance  VRdc=CRdc*ks*vRdc*bw*d/1000=124.6 kN
Minimum stress  vmin=0.035*ks^1.5*fck^0.5=0.5438 N/mm²
Minimum shear resistance  VRdm=vmin*bw*d/1000=133.2 kN
Modified shear resistance  VRdc=VRdm=133.2 kN

SHEAR SUMMARY
Design shear force  125 kN
Design shear resistance  133.2 kN

Crack control check

Unfactored permanent load on slab Gk=50 kN
Unfactored variable load on slab Qk=20 kN
Permanent load factor  gamG=1.35
Variable load factor  gamQ=1.5
Factor ψ2 for variable load  psi2=0.3
SLS stress in reinforcement σs  SLSs=500/1.15*(As/Aspr)*ratio*1/delta=243.9 N/mm²
Design crack width  wk=0.3 mm
Max allowable SLS stress σs  SLSs2=TABLE 7.3 for pch=130 =296 N/mm²

The bar spacing 130 mm used complies with Table 7.3N of EC2 Part 1:2004 hence satisfactory.
Location: Ex3 - Lightly loaded section

Bending in solid slabs (with comp.steel if reqd.),
designed per metre width, with checks on minimum steel
and span/effective-depth ratio

Calculations to EN 1992-1-1:2004
Eurocode 2 : Design of concrete structures & assume the use of a simplified rectangular concrete stress-block, and that the depth to the NA is restricted to 0.45d

Design BM before redistribution Mbef=15 kNm
Section being analysed is considered as non-continuous.
Char yield strength of reinft fyk=500 N/mm²
Max.aggregate size (for bar spc.) hagg=20 mm
Diameter of tension bars dia=10 mm
Diameter of distribution bars diad=10 mm
Effective depth of section d=160 mm

DESIGN
Overall depth 200 mm

SUMMARY
Effective depth 160 mm

FLEXURE
Parameter K 0.0195
Parameter K' 0.1684
Lever arm ratio z/d 0.95
Steel area (tension) 241 mm²/m
Steel percentage req. 0.1506 %
Minimum area of steel 241 mm²/m
Maximum area of steel 8000 mm²/m
Distribution steel 241 mm²/m

Chosen spacing of tension bars pch=325 mm

TENSION (AND DISTRIBUTION)
Diameter of bars 10 mm
Spacing of bars 325 mm
Area of steel required 241 mm²/m
Area of steel provided 241 mm²/m

Deflection check

Effective span of beam L=4.3 m
Actual span to depth ratio 1'd=L*1000/d=26.88
Allowable span/depth ratio 1'da=k*N*F1*F2*F3=115.9
Absolute value of span/depth 1'da=40*k=40

DESIGN
Actual 1/d ratio 26.88

SUMMARY
Basic 1/d ratio 115.9

DEFLECTION
Span factor 1
Flange beam factor F1 1
Long spans factor F2 1
Steel stress factor F3 1
Allowable 1/d ratio 40
NOTE: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for lightly stressed concrete (C30/37 & ρ=0.5%) as follows:

<table>
<thead>
<tr>
<th>Span Type</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simply supported spans</td>
<td>20</td>
</tr>
<tr>
<td>End spans</td>
<td>26</td>
</tr>
<tr>
<td>Interior spans</td>
<td>30</td>
</tr>
</tbody>
</table>

Shear

Location for shear calculation: 150 mm from support
Shear force due to ultimate load \( V_{Ed} = 14 \text{ kN} \)
Term for shear resistance \( V_{Rdc} = (\rho_{01} \cdot f_{ck})^{(1/3)} = 1.312 \)
Shear resistance \( V_{Rdc} = C_{Rdc} \cdot v_{Rdc} \cdot b \cdot w \cdot d / 1000 = 50.39 \text{ kN} \)
Minimum stress \( v_{min} = 0.035 \cdot k_{S} \cdot 1.5 \cdot f_{ck} \cdot 0.5 = 0.5422 \text{ N/mm}^2 \)
Minimum shear resistance \( V_{Rdm} = v_{min} \cdot b \cdot w \cdot d / 1000 = 86.75 \text{ kN} \)
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 86.75 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Design shear force</th>
<th>14 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear resistance</td>
<td>86.75 kN</td>
</tr>
</tbody>
</table>

**Crack control check**
Location: Ex4 - End span

Bending in solid slabs (with comp.steel if reqd.),
designed per metre width, with checks on minimum steel
and span/effective-depth ratio

Calculations to EN 1992-1-1:2004

![Diagram of bending in solid slabs]

Design BM before redistribution \( M_{bef} = 85 \text{ kNm} \)
Section being analysed is considered as continuous.
Section considered has a sagging moment.
Char yield strength of reinft \( f_{yk} = 500 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( d_{ia} = 16 \text{ mm} \)
Diameter of distribution bars \( d_{iad} = 10 \text{ mm} \)
Effective depth of section \( d = 185 \text{ mm} \)

| DESIGN  | Overall depth | 225 mm |
| SUMMARY | Effective depth | 185 mm |
| FLEXURE | Parameter K | 0.0828 |
|         | Parameter K' | 0.1684 |
|         | Lever arm ratio \( z/d \) | 0.9207 |
|         | Steel area (tension) | 1148 mm²/m |
|         | Steel percentage req. | 0.6204 % |
|         | Minimum area of steel | 278.6 mm²/m |
|         | Maximum area of steel | 9000 mm²/m |
|         | Distribution steel | 278.6 mm²/m |

Chosen spacing of tension bars \( p_{ch} = 175 \text{ mm} \)
Diameter of distribution bars \( d_{iad} = 10 \text{ mm} \)
Chosen spacing of distn. bars \( p_{chd} = 280 \text{ mm} \)

| TENSION REINFORCEMENT | Diameter of bars | 16 mm |
|                       | Spacing of bars | 175 mm |
|                       | Area of steel required | 1148 mm²/m |
|                       | Area of steel provided | 1148 mm²/m |
| DISTRIBUTION REINFORCEMENT | Diameter of bars | 10 mm |
|                          | Spacing of bars | 280 mm |
|                          | Area of steel required | 278.6 mm²/m |
|                          | Area of steel provided | 280 mm²/m |

Deflection check

Effective span of beam \( L = 4.3 \text{ m} \)
Actual span to depth ratio \( l'd = L \times 1000 / d = 23.24 \)
Allowable span/depth ratio \( l'da = k \times N \times F1 \times F2 \times F3 = 23.73 \)
DESIGN

Actual l/d ratio 23.24

SUMMARY

Basic l/d ratio 18.25

DEFLECTION

Span factor 1.3

Flange beam factor F1 1

Long spans factor F2 1

Steel stress factor F3 1

Allowable l/d ratio 23.73

Note: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for highly stressed concrete (C30/37 & ρ=1.5%) as follows:

Simply supported spans 14
End spans 18
Interior spans 20

Shear

Location for shear calculation: 200 mm from support
Shear force due to ultimate load $V_{Ed}=68$ kN
Term for shear resistance $V_{Rdc}=(\rho_{1}\cdot f_{ck})^{(1/3)}=2.104$
Shear resistance $V_{Rdc}=C_{Rdc}\cdot k_{s}\cdot V_{Rdc}\cdot b_{w}\cdot d/1000=93.4$ kN
Minimum stress $v_{min}=0.035\cdot k_{s}\cdot 1.5\cdot f_{ck}\cdot 0.5=0.5422$ N/mm²
Minimum shear resistance $V_{Rdm}=v_{min}\cdot b_{w}\cdot d/1000=100.3$ kN
Modified shear resistance $V_{Rdc}=V_{Rdm}=100.3$ kN

SHEAR SUMMARY

Design shear force 68 kN
Design shear resistance 100.3 kN

Crack control check

Unfactored permanent load on slab $G_{k}=40$ kN
Unfactored variable load on slab $Q_{k}=20$ kN
Permanent load factor $\gamma_{G}=1.35$
Variable load factor $\gamma_{Q}=1.5$
Factor $\psi_{2}$ for variable load $\psi_{2}=0.3$
SLS stress in reinforcement $\sigma_{s}=500/1.15\cdot(As/Aspr)\cdot ratio\cdot 1/\delta=238.1$ N/mm²
Design crack width $w_{k}=0.3$ mm
Max allowable SLS stress $\sigma_{s1}=TABLE$ 7.2 for dia=16

=240 N/mm²

The bar size 16 mm used complies with Table 7.2N of EC2 Part 1:2004 hence satisfactory.
Location: Ex5 - Simply supported slab with accurate F3 evaluation

Bending in solid slabs (with comp.steel if reqd.),
designed per metre width, with checks on minimum steel

and span/effective-depth ratio

Calculations to EN 1992-1-1:2004
Eurocode 2 : Design of concrete
structures & assume the use of a
simplified rectangular concrete
stress-block, and that the depth
to the NA is restricted to 0.45d

Design BM before redistribution \( M_{bef} = 150 \text{ kNm} \)
Section being analysed is considered as non-continuous.
Char yield strength of reinft \( f_{yk} = 500 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( \text{dia}=16 \text{ mm} \)
Diameter of distribution bars \( \text{diad}=8 \text{ mm} \)
Effective depth of section \( d=245 \text{ mm} \)

### DESIGN
- Overall depth \( 300 \text{ mm} \)
- Effective depth \( 245 \text{ mm} \)

### SUMMARY
- Parameter K \( 0.0714 \)
- Parameter K' \( 0.1684 \)
- Lever arm ratio \( z/d = 0.9324 \)
- Steel area (tension) \( 1510 \text{ mm}^2/\text{m} \)
- Steel percentage req. \( 0.6164 \% \)
- Minimum area of steel \( 408.9 \text{ mm}^2/\text{m} \)
- Maximum area of steel \( 12000 \text{ mm}^2/\text{m} \)
- Distribution steel \( 408.9 \text{ mm}^2/\text{m} \)

Chosen spacing of tension bars \( \text{pch}=130 \text{ mm} \)
Diameter of distribution bars \( \text{diad}=8 \text{ mm} \)
Chosen spacing of distn. bars \( \text{pchd}=120 \text{ mm} \)

### TENSION REINFORCEMENT
- Diameter of bars \( 16 \text{ mm} \)
- Spacing of bars \( 130 \text{ mm} \)
- Area of steel required \( 1510 \text{ mm}^2/\text{m} \)
- Area of steel provided \( 1546 \text{ mm}^2/\text{m} \)

### DISTRIBUTION REINFORCEMENT
- Diameter of bars \( 8 \text{ mm} \)
- Spacing of bars \( 120 \text{ mm} \)
- Area of steel required \( 408.9 \text{ mm}^2/\text{m} \)
- Area of steel provided \( 418 \text{ mm}^2/\text{m} \)

Deflection check

Effective span of beam \( L=4.3 \text{ m} \)
Unfactored permanent load on slab \( G_k=50 \text{ kN} \)
Unfactored variable load on slab \( Q_k=20 \text{ kN} \)
Permanent load factor \( \gamma_{G}=1.35 \)
Variable load factor \( \gamma_{Q} = 1.5 \)
Actual span to depth ratio \( l'/d = 1.1 \times 1000/d = 17.55 \)
Factor \( \psi_2 \) for variable load \( \psi_2 = 0.3 \)
SLS stress in reinforcement \( S_{LS} = 500/(1.15 \times (A_s/A_{spr}) \times \text{ratio} \times \delta) = 243.9 \text{ N/mm}^2 \)
Allowable span/depth ratio \( l'/d_a = k \times N \times F_1 \times F_2 \times F_3 = 24.8 \)

**DESIGN**

- Actual 1/d ratio: 17.55
- Basic 1/d ratio: 19.52

**SUMMARY**

- Span factor: 1
- Flange beam factor \( F_1 \): 1
- Long spans factor \( F_2 \): 1
- Steel stress factor \( F_3 \): 1.271
- Allowable 1/d ratio: 24.8

Note: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for highly stressed concrete (C30/37 & \( \rho = 1.5\% \)) as follows:

- Simply supported spans: 14
- End spans: 18
- Interior spans: 20

**Shear**

Location for shear calculation: 250mm from support
Shear force due to ultimate load \( V_{Ed} = 125 \text{ kN} \)
Term for shear resistance \( v_{Rdc} = (\rho_1 \times f_{ck})^{(1/3)} = 2.227 \)
Shear resistance \( V_{Rdc} = C R_{dc} \times k_s \times V_{Rdc} \times b \times d/1000 = 124.6 \text{ kN} \)
Minimum stress \( v_{min} = 0.035 \times k_s \times 1.5 \times f_{ck} \times 0.5 = 0.5438 \text{ N/mm}^2 \)
Minimum shear resistance \( V_{Rdm} = v_{min} \times b \times d/1000 = 133.2 \text{ kN} \)
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 133.2 \text{ kN} \)

**SHEAR SUMMARY**

- Design shear force: 125 kN
- Design shear resistance: 133.2 kN
Location: Typical continuous slab reinforced with high-yield steel

Bending in solid slabs (with comp.steel if reqd.),
designed per metre width, with checks on minimum steel
and span/effective-depth ratio

Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to d/2.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Moment before redistribution \(M_{bef}=625\,\text{kNm/m}\)
Slab containing section being analysed is considered as continuous.
Design moment (after redistrib.) \(M=600\,\text{kNm/m}\)
Characteristic concrete strength \(f_{cu}=25\,\text{N/mm}^2\)
Characteristic steel strength \(f_y=500\,\text{N/mm}^2\)
Longitudinal reinforcement is high-yield steel.

\[ \begin{align*}
\text{Diameter of tension bars} & \quad \text{dia}=20\,\text{mm} \\
\text{Designated exposure class} & \quad \text{is XC1} \\
\text{Specified fixing tolerance} & \quad \text{tol}=10\,\text{mm} \\
\text{Nominal concrete cover} & \quad \text{cover}=25\,\text{mm} \\
\text{Overall thickness of slab} & \quad h=400\,\text{mm} \\
\text{Effective depth of section} & \quad d=365\,\text{mm} \\
\text{Diameter of compression bars} & \quad \text{diac}=16\,\text{mm} \\
\text{Depth to compression steel} & \quad d'=33\,\text{mm} \\
\text{Chosen comp.bar spacing (c.to c.)} & \quad pchCA=250\,\text{mm} \\
\text{Chosen spacing of tension bars} & \quad pch=60\,\text{mm} \\
\text{Diameter of distribution bars} & \quad \text{diamn}=12\,\text{mm} \\
\text{Spacing of distribution bars} & \quad pchDA=200\,\text{mm} \\
\end{align*} \]

**TENSION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 (\text{N/mm}^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 (\text{mm})</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>60 (\text{mm})</td>
</tr>
<tr>
<td>Effective depth</td>
<td>365 (\text{mm})</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>4771.5 (\text{mm}^2/\text{m})</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>5235 (\text{mm}^2/\text{m})</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>1.3088 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>41.09 (\text{kg/m}^2)</td>
</tr>
</tbody>
</table>

**COMPRESSION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 (\text{N/mm}^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 (\text{mm})</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 (\text{mm})</td>
</tr>
<tr>
<td>Depth to bar centres</td>
<td>33 (\text{mm})</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>800 (\text{mm}^2/\text{m})</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>804 (\text{mm}^2/\text{m})</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.201 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>6.31 (\text{kg/m}^2)</td>
</tr>
<tr>
<td>DISTRIBUTION</td>
<td>Characteristic strength</td>
</tr>
<tr>
<td>-------------</td>
<td>-------------------------</td>
</tr>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing of bars</td>
</tr>
<tr>
<td></td>
<td>Depth to bar centres</td>
</tr>
<tr>
<td></td>
<td>Area of steel required</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided</td>
</tr>
<tr>
<td></td>
<td>Percentage provided</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided</td>
</tr>
</tbody>
</table>

Check on span/effective-depth ratio

Basic ratio for slab
continuous at one edge  
bs'd=23

Mod.factor for tension steel  
modf1=0.79755

Area of compression steel prov.  
As'pr=804 mm²

Percentage of compression steel  
per'=0.201 %

From Equation 9 of BS8110, with percentage of comp.steel=0.201 %,
Mod.factor for compression steel  
modf2=1+per'/(3+per')=1.0628

Maximum permissible
span/effective-depth ratio  
ps'd=bs'd*modf1*modf2=19.496

Effective span of slab  
span=5 m

True span/effective-depth ratio  
as'd=1000*span/d=13.699

As this does not exceed  
19.496, this is Acceptable.
Location: Thick, heavily-loaded continuous slab, mild steel reinfmt.

Bending in solid slabs (with comp.steel if reqd.),
designed per metre width, with checks on minimum steel
and span/effective-depth ratio

Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to d/2.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Moment before redistribution \( M_{\text{bef}} = 3000 \text{ kNm/m} \)

Slab containing section being analysed is considered as continuous.

Design moment (after redistrib.) \( M = 2500 \text{ kNm/m} \)

Characteristic concrete strength \( f_{\text{cu}} = 35 \text{ N/mm}^2 \)

Characteristic steel strength \( f_y = 250 \text{ N/mm}^2 \)

Longitudinal reinforcement is mild steel.

Diameter of tension bars \( \text{dia} = 40 \text{ mm} \)

Designated exposure class is XC2

Specified fixing tolerance \( \text{tol} = 10 \text{ mm} \)

Nominal concrete cover \( \text{cover} = 45 \text{ mm} \)

Overall thickness of slab \( h = 750 \text{ mm} \)

Effective depth of section \( d = 685 \text{ mm} \)

Diameter of compression bars \( \text{diac} = 25 \text{ mm} \)

Depth to compression steel \( d' = 57.5 \text{ mm} \)

Chosen comp.bar spacing (c.to c.) \( pchCA = 325 \text{ mm} \)

Chosen spacing of tension bars \( pch = 60 \text{ mm} \)

Diameter of distribution bars \( \text{diamn} = 20 \text{ mm} \)

Spacing of distribution bars \( \text{pchDA} = 170 \text{ mm} \)

**TENSION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>40 mm</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>60 mm</td>
</tr>
<tr>
<td>Effective depth</td>
<td>685 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>20621 mm²/m</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>20943 mm²/m</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>2.7924 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>164.4 kg/m²</td>
</tr>
</tbody>
</table>

**COMPRESSION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>325 mm</td>
</tr>
<tr>
<td>Depth to bar centres</td>
<td>57.5 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>1500 mm²/m</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>1510 mm²/m</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.20133 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>11.85 kg/m²</td>
</tr>
</tbody>
</table>
DISTRIBUTION

Characteristic strength 250 N/mm²

REINFORCEMENT

Diameter of bars 20 mm
Spacing of bars 170 mm

SUMMARY

Depth to bar centres 655 mm
Area of steel required 1800 mm²/m
Area of steel provided 1847 mm²/m
Percentage provided 0.24627 %
Weight of steel provided 14.5 kg/m²

Check on span/effective-depth ratio

Basic ratio for continuous slab bs’d=26 (see Table 3.9)
Mod.factor for tension steel modf₁=0.92476
Area of compression steel prov. As’pr=1510 mm²
Percentage of compression steel per’=0.20133 %
From Equation 9 of BS8110, with percentage of comp.steel=0.20133 %,
Mod.factor for compression steel modf₂=1+per’/(3+per’)=1.0629
Maximum permissible
span/effective-depth ratio ps’d=bs’d*modf₁*modf₂=25.556
Effective span of slab span=12 m
Although span exceeds 10 m, partitions/finishes are not considered
sensitive to deflection, so modification to basic ratio is unnecessary.
True span/effective-depth ratio as’d=1000*span/d=17.518
As this does not exceed 25.556, this is Acceptable.
Location: Lightly-loaded thin slab with non-standard reinforcement

Bending in solid slabs (with comp.steel if reqd.),
designed per metre width, with checks on minimum steel
and span/effective-depth ratio

Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to d/2.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Moment before redistribution Mbef=15 kNm/m
Slab containing section being analysed is considered as non-continuous.
Characteristic concrete strength $f_{cu}=60 \text{ N/mm}^2$
Characteristic steel strength $f_y=425 \text{ N/mm}^2$
Steel strength other than those given in Table 3.1 of BS8110.
Specified min.steel percentage $\permin=0.16\%$
Specified min.slab thickness $\mindep=215 \text{ mm}$
Diameter of tension bars $\dia=16 \text{ mm}$
Designated exposure class is XC4
Specified fixing tolerance $\tol=10 \text{ mm}$
Nominal concrete cover $\cover=30 \text{ mm}$
Overall thickness of slab $h=100 \text{ mm}$
Effective depth of section $d=62 \text{ mm}$
Chosen spacing of tension bars $p_{ch}=200 \text{ mm}$
Diameter of distribution bars $\diamn=8 \text{ mm}$
Spacing of distribution bars $p_{chDA}=190 \text{ mm}$

**TENSION REINFORCEMENT**

- **SUMMARY**
  - Characteristic strength 425 N/mm²
  - Diameter of bars 16 mm
  - Spacing of bars 200 mm
  - Effective depth 62 mm
  - Area of steel required $710.35 \text{ mm}^2$/m
  - Area of steel provided 1005 mm²/m
  - Percentage provided 1.005 %
  - Weight of steel provided 7.89 kg/m²

**DISTRIBUTION REINFORCEMENT**

- **SUMMARY**
  - Characteristic strength 425 N/mm²
  - Diameter of bars 8 mm
  - Spacing of bars 190 mm
  - Depth to bar centres 50 mm
  - Area of steel required 160 mm²/m
  - Area of steel provided 264 mm²/m
  - Percentage provided 0.264 %
  - Weight of steel provided 2.07 kg/m²
### Check on span/effective-depth ratio

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic ratio for simp.-sup.slab</td>
<td>bs'd=20 (see Table 3.9)</td>
</tr>
<tr>
<td>Mod.factor for tension steel</td>
<td>modf1=1.0302</td>
</tr>
<tr>
<td>Mod.factor for no comp.steel</td>
<td>modf2=1.0</td>
</tr>
<tr>
<td>Maximum permissible span/effective-depth ratio</td>
<td>ps'd=bs'd<em>modf1</em>modf2=20.605</td>
</tr>
<tr>
<td>Effective span of slab</td>
<td>span=2.5 m</td>
</tr>
</tbody>
</table>

**WARNING:**

- **True span/effective-depth ratio**: 40.323
- **Perm.span/effective-depth ratio**: 20.605

Hence unacceptable.
Location: Ex1 - Continuous slab

Bending in solid slabs (with comp.steel if reqd.),
designed per metre width, with checks on minimum steel
and span/effective-depth ratio

Calculations to EN 1992-1-1:2004 Eurocode 2 : Design of concrete structures & assume the use of a simplified rectangular concrete stress-block, and that the depth to the NA is restricted to 0.45d

Design BM before redistribution \( M_{bef} = 625 \text{ kNm} \)
Section being analysed considered as continuous.
Section considered has a hogging moment.
Design BM (after redistribution) \( M = 600 \text{ kNm} \)

Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( \text{dia} = 25 \text{ mm} \)
Diameter of distribution bars \( \text{diad} = 12 \text{ mm} \)
Effective depth of section \( d = 350 \text{ mm} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth 400 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth 350 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K 0.1633</td>
</tr>
<tr>
<td></td>
<td>Parameter K' 0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio z/d 0.8255</td>
</tr>
<tr>
<td></td>
<td>Steel area (tension) 4776 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Steel percentage req. 1.365 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel 527.2 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel 16000 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Distribution steel 955.2 mm²/m</td>
</tr>
</tbody>
</table>

Chosen spacing of tension bars \( p_{ch} = 100 \text{ mm} \)

Spacing of distribution steel

| Diameter of distribution bars \( \text{dia} = 12 \text{ mm} \) |
| Chosen spacing of distn. bars \( p_{chd} = 110 \text{ mm} \) |

| TENSION REINFORCEMENT | \( \text{Diameter of bars} = 25 \text{ mm} \) |
|                       | \( \text{Spacing of bars} = 100 \text{ mm} \) |
|                       | \( \text{Area of steel required} = 4776 \text{ mm}^2/\text{m} \) |
|                       | \( \text{Area of steel provided} = 4908 \text{ mm}^2/\text{m} \) |

| DISTRIBUTION REINFORCEMENT | \( \text{Diameter of bars} = 12 \text{ mm} \) |
|                           | \( \text{Spacing of bars} = 110 \text{ mm} \) |
|                           | \( \text{Area of steel required} = 955.2 \text{ mm}^2/\text{m} \) |
|                           | \( \text{Area of steel provided} = 1028 \text{ mm}^2/\text{m} \) |
Shear check

Location for shear calculation: 350mm from internal support
Shear force due to ultimate load $V_{Ed} = 250$ kN
Term for shear resistance $v_{Rdc} = (\rho_1^1 * f_{ck})^{1/(1/3)} = 3.478$
Shear resistance $VR_{dc} = CR_{dc} * k_s * bw * d / 1000 = 256.5$ kN
Minimum stress $v_{min} = 0.035^1 * ks^1.5 * f_{ck}^0.5 = 0.4461$ N/mm²
Minimum shear resistance $VR_{dm} = v_{min} * bw * d / 1000 = 156.1$ kN

SHEAR SUMMARY
Design shear force 250 kN
Design shear resistance 256.5 kN

Crack control check

Unfactored permanent load on slab $G_k = 100$ kN
Unfactored variable load on slab $Q_k = 40$ kN
Permanent load factor $\gamma_G = 1.35$
Variable load factor $\gamma_Q = 1.5$
Factor $\psi_2$ for variable load $\psi_2 = 0.3$
SLS stress in reinforcement $\sigma_s = 500/1.15 * (A_s/A_{spr}) * \text{ratio} * 1/\delta = 253.1$ N/mm²
Design crack width $w_k = 0.3$ mm
Max allowable SLS stress $\sigma_s = \text{TABLE 7.3 for } p_{ch} = 100 = 320$ N/mm²

The bar spacing 100 mm used complies with Table 7.3N of EC2 Part 1:2004 hence satisfactory.
Location: Ex2 - Simply supported slab

Bending in solid slabs (with comp. steel if reqd.),
designed per metre width, with checks on minimum steel
and span/effective-depth ratio

Calculations to EN 1992-1-1:2004
Eurocode 2: Design of concrete structures & assume the use of a
simplified rectangular concrete stress-block, and that the depth
to the NA is restricted to 0.45d

Design BM before redistribution Mbef=150 kNm
Section being analysed is considered as non-continuous.
Char yield strength of reinft. f_yk=500 N/mm²
Max. aggregate size (for bar spc.) h_{agg}=20 mm
Diameter of tension bars dia=16 mm
Diameter of distribution bars diad=8 mm
Effective depth of section d=245 mm

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth</th>
<th>300 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth</td>
<td>245 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K</td>
<td>0.0714</td>
</tr>
<tr>
<td></td>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio z/d</td>
<td>0.9324</td>
</tr>
<tr>
<td></td>
<td>Steel area (tension)</td>
<td>1510 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Steel percentage req.</td>
<td>0.6164 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel</td>
<td>408.9 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel</td>
<td>12000 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Distribution steel</td>
<td>408.9 mm²/m</td>
</tr>
</tbody>
</table>

Chosen spacing of tension bars pch=130 mm

Spacing of distribution steel

| Diameter of distribution bars | diad=8 mm |
| Chosen spacing of distn. bars | pchd=120 mm |

| TENSION REINFORCEMENT | Diameter of bars | 16 mm |
| | Spacing of bars | 130 mm |
| | Area of steel required | 1510 mm²/m |
| | Area of steel provided | 1546 mm²/m |

| DISTRIBUTION REINFORCEMENT | Diameter of bars | 8 mm |
| | Spacing of bars | 120 mm |
| | Area of steel required | 408.9 mm²/m |
| | Area of steel provided | 418 mm²/m |
Deflection check

Effective span of beam L=4.3 m
Actual span to depth ratio l'd=L*1000/d=17.55
Allowable span/depth ratio l'da=k*N*F1*F2*F3=19.98

DESIGN
Actual 1/d ratio 17.55
SUMMARY
Basic 1/d ratio 19.52
DEFLECTION
Span factor 1
Flange beam factor F1 1
Long spans factor F2 1
Steel stress factor F3 1.024
Allowable 1/d ratio 19.98

Note: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for highly stressed concrete (C30/37 & ρ=1.5%) as follows:
Simply supported spans 14
End spans 18
Interior spans 20

Shear check

Location for shear calculation: 250mm from support
Shear force due to ultimate load VEd=125 kN
Term for shear resistance vRdc=(rho1*fck)^(1/3)=2.227
Shear resistance VRdc=CRdc*ks*vRdc*bw*d/1000=124.6 kN
Minimum stress vmin=0.035*ks^1.5*fck^0.5=0.5438 N/mm²
Minimum shear resistance VRdm=vmin*bw*d/1000=133.2 kN
Modified shear resistance VRdc=VRdm=133.2 kN

SHEAR SUMMARY
Design shear force 125 kN
Design shear resistance 133.2 kN

Crack control check

Unfactored permanent load on slab Gk=50 kN
Unfactored variable load on slab Qk=20 kN
Permanent load factor gamG=1.35
Variable load factor gamQ=1.5
Factor w2 for variable load psi2=0.3
SLS stress in reinforcement çs SLSs=500/1.15*(As/Aspr)*ratio*1/delta=243.9 N/mm²
Design crack width wk=0.3 mm
Max allowable SLS stress çs SLSs2=TABLE 7.3 for pch=130 =296 N/mm²

The bar spacing 130 mm used complies with Table 7.3N of EC2 Part 1:2004 hence satisfactory.
Location: Ex3 - Lightly loaded section

Bending in solid slabs (with comp.steel if reqd.),
designed per metre width, with checks on minimum steel

and span/effective-depth ratio

Calculations to EN 1992-1-1:2004
Eurocode 2 : Design of concrete structures & assume the use of a simplified rectangular concrete stress-block, and that the depth to the NA is restricted to 0.45d

Design BM before redistribution  $\text{M}_{\text{bef}}=15 \text{ kNm}$
Section being analysed is considered as non-continuous.
Char yield strength of reinft. $f_y=500 \text{ N/mm}^2$
Max.aggregate size (for bar spc.) $h_{\text{agg}}=20 \text{ mm}$
Diameter of tension bars $d_{\text{ia}}=10 \text{ mm}$
Diameter of distribution bars $d_{\text{iad}}=10 \text{ mm}$
Effective depth of section $d=165 \text{ mm}$

DESIGN
Overall depth 200 mm
SUMMARY
Effective depth 165 mm
FLEXURE
Parameter $K$ 0.0184
Parameter $K'$ 0.1684
Lever arm ratio $z/d$ 0.95
Steel area (tension) $248.5 \text{ mm}^2/\text{m}$
Steel percentage req. $0.1506 \%$
Minimum area of steel $248.5 \text{ mm}^2/\text{m}$
Maximum area of steel $8000 \text{ mm}^2/\text{m}$
Distribution steel $248.5 \text{ mm}^2/\text{m}$

Chosen spacing of tension bars $p_{\text{ch}}=300 \text{ mm}$

TENSION (AND DISTRIBUTION)
REINFORCEMENT
Diameter of bars 10 mm
Spacing of bars 300 mm
Area of steel required $248.5 \text{ mm}^2/\text{m}$
Area of steel provided $261 \text{ mm}^2/\text{m}$

Deflection check
Effective span of beam $L=4.3 \text{ m}$
Actual span to depth ratio $l'/d=L\times1000/d=26.06$
Allowable span/depth ratio $l'/da=k\times N\times F_1 \times F_2 \times F_3=121.7$
Absolute value of span/depth $l'/da=40 \times k=40$

DESIGN
Actual $l'/d$ ratio 26.06
SUMMARY
Basic $l'/d$ ratio 115.9
DEFLECTION
Span factor 1
Flange beam factor $F_1$ 1
Long spans factor $F_2$ 1
Steel stress factor $F_3$ 1.05
Allowable $l'/d$ ratio 40
NOTE: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for lightly stressed concrete (C30/37 & p=0.5%) as follows:

<table>
<thead>
<tr>
<th>Type of Span</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simply supported spans</td>
<td>20</td>
</tr>
<tr>
<td>End spans</td>
<td>26</td>
</tr>
<tr>
<td>Interior spans</td>
<td>30</td>
</tr>
</tbody>
</table>

**Shear check**

Location for shear calculation: 150 mm from support

Shear force due to ultimate load: \( V_{Ed} = 14 \text{ kN} \)

Term for shear resistance: \( V_{Rdc} = (\rho_1 \times f_{ck})^{(1/3)} = 1.334 \)

Shear resistance: \( V_{Rdc} = C_{Rdc} \times k_s \times V_{Rdc} \times b_w \times d / 1000 = 52.82 \text{ kN} \)

Minimum stress: \( v_{min} = 0.035 \times k_s \times 1.5 \times f_{ck} \times 0.5 = 0.5422 \text{ N/mm}^2 \)

Minimum shear resistance: \( V_{Rdm} = v_{min} \times b_w \times d / 1000 = 89.47 \text{ kN} \)

Modified shear resistance: \( V_{Rdc} = V_{Rdm} = 89.47 \text{ kN} \)

**SHEAR SUMMARY**

Design shear force: 14 kN

Design shear resistance: 89.47 kN

**Crack control check**
Location: Ex4 - End span

Bending in solid slabs (with comp.steel if reqd.),
designed per metre width, with checks on minimum steel
and span/effective-depth ratio

Calculations to EN 1992-1-1:2004
Eurocode 2 : Design of concrete structures & assume the use of a simplified rectangular concrete stress-block, and that the depth to the NA is restricted to 0.45d

Design BM before redistribution Mbef=85 kNm
Section being analysed is considered as continuous.
Section considered has a sagging moment.
Char yield strength of reinft. fyk=500 N/mm²
Max.aggregate size (for bar spc.) hagg=20 mm
Diameter of tension bars dia=16 mm
Diameter of distribution bars diad=10 mm
Effective depth of section d=185 mm

DESIGN
Overall depth 225 mm
Effective depth 185 mm

SUMMARY
Effective depth          185 mm

FLEXURE
Parameter K  0.0828
Parameter K'  0.1684
Lever arm ratio z/d  0.9207
Steel area (tension) 1148 mm²/m
Steel percentage req. 0.6204 %
Minimum area of steel 278.6 mm²/m
Maximum area of steel 9000 mm²/m
Distribution steel 278.6 mm²/m

Chosen spacing of tension bars pch=175 mm

Spacing of distribution steel

Diameter of distribution bars diad=10 mm
Chosen spacing of distn. bars pchd=280 mm

TENSION REINFORCEMENT
Diameter of bars 16 mm
Spacing of bars 175 mm
Area of steel required 1148 mm²/m
Area of steel provided 1148 mm²/m

DISTRIBUTION REINFORCEMENT
Diameter of bars 10 mm
Spacing of bars 280 mm
Area of steel required 278.6 mm²/m
Area of steel provided 280 mm²/m
Deflection check

Effective span of beam                  L=4.3 m
Actual span to depth ratio            l'd=L*1000/d=23.24
Allowable span(depth ratio           l'da=k*N*F1*F2*F3=23.73

DESIGN                                 Actual l/d ratio         23.24
SUMMARY                                Basic l/d ratio          18.25
DEFLECTION                             Span factor              1.3
Flange beam factor F1                 1
Long spans factor F2                   1
Steel stress factor F3                 1
Allowable l/d ratio                    23.73

Note: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for highly stressed concrete (C30/37 & ρ=1.5%) as follows:

Simply supported spans     14
End spans                                      18
Interior spans                 20

Shear check

Location for shear calculation: 200 mm from support
Shear force due to ultimate load       VEd=68 kN
Term for shear resistance            vRdc=(rho1*fck)^(1/3)=2.104
Shear resistance                   VRdc=CRdc*ks*vRdc*bw*d/1000=93.4 kN
Minimum stress                     vmin=0.035*ks^1.5*fck^0.5=0.5422 N/mm²
Minimum shear resistance            VRdm=vmin*bw*d/1000=100.3 kN
Modified shear resistance           VRdc=VRdm=100.3 kN

SHEAR SUMMARY                      Design shear force       68 kN
                                      Design shear resistance 100.3 kN

Crack control check

Unfactored permanent load on slab Gk=40 kN
Unfactored variable load on slab    Qk=20 kN
Permanent load factor              gamG=1.35
Variable load factor               gamQ=1.5
Factor ψ2 for variable load        psi2=0.3
SLS stress in reinforcement σs     SLSs=500/1.15*(As/Aspr)*ratio*1/delta=238.1 N/mm²
Design crack width                 wk=0.3 mm
Max allowable SLS stress σs         SLSs1=TABLE 7.2 for dia=16
                                      =240 N/mm²

The bar size 16 mm used complies with Table 7.2N of EC2 Part 1:2004 hence satisfactory.
Location: Ex5 - Simply supported slab

Bending in solid slabs (with comp. steel if reqd.),
designed per metre width, with checks on minimum steel

and span/effective-depth ratio

Calculations to EN 1992-1-1:2004

Eurocode 2 : Design of concrete structures & assume the use of a simplified rectangular concrete stress-block, and that the depth to the NA is restricted to 0.45d

Design BM before redistribution $M_{bef}=150$ kNm

Section being analysed is considered as non-continuous.

Char yield strength of reinfor. $f_{yk}=500$ N/mm²

Max. aggregate size (for bar spc.) $h_{agg}=20$ mm

Diameter of tension bars $d_{ia}=16$ mm

Diameter of distribution bars $d_{iad}=8$ mm

Effective depth of section $d=245$ mm

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth 300 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth 245 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K 0.0714</td>
</tr>
<tr>
<td></td>
<td>Parameter K' 0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio $z/d$ 0.9324</td>
</tr>
<tr>
<td></td>
<td>Steel area (tension) $1510$ mm²/m</td>
</tr>
<tr>
<td></td>
<td>Steel percentage req. 0.6164 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel $408.9$ mm²/m</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel $12000$ mm²/m</td>
</tr>
<tr>
<td></td>
<td>Distribution steel $408.9$ mm²/m</td>
</tr>
</tbody>
</table>

Chosen spacing of tension bars $p_{ch}=130$ mm

Spacing of distribution steel

| Diameter of distribution bars $d_{iad}=8$ mm |
| Chosen spacing of distn. bars $p_{chd}=120$ mm |

| TENSION REINFORCEMENT | Diameter of bars 16 mm |
|                       | Spacing of bars 130 mm |
|                       | Area of steel required $1510$ mm²/m |
|                       | Area of steel provided $1546$ mm²/m |

| DISTRIBUTION REINFORCEMENT | Diameter of bars 8 mm |
|                           | Spacing of bars 120 mm |
|                           | Area of steel required $408.9$ mm²/m |
|                           | Area of steel provided 418 mm²/m |
### Deflection check

<table>
<thead>
<tr>
<th>Effective span of beam</th>
<th>L=4.3 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored permanent load on slab</td>
<td>Gk=50 kN</td>
</tr>
<tr>
<td>Unfactored variable load on slab</td>
<td>Qk=20 kN</td>
</tr>
<tr>
<td>Permanent load factor</td>
<td>gamG=1.35</td>
</tr>
<tr>
<td>Variable load factor</td>
<td>gamQ=1.5</td>
</tr>
<tr>
<td>Actual span to depth ratio</td>
<td>l'd=L*1000/d=17.55</td>
</tr>
<tr>
<td>Factor 2 for variable load</td>
<td>psi2=0.3</td>
</tr>
<tr>
<td>SLS stress in reinforcement σs</td>
<td>SLSs=500/1.15*(As/Aspr)<em>ratio</em>1/delta=243.9 N/mm²</td>
</tr>
<tr>
<td>Allowable span/depth ratio</td>
<td>l'da=k<em>N</em>F1<em>F2</em>F3=24.8</td>
</tr>
</tbody>
</table>

#### DESIGN
- Actual 1/d ratio: 17.55

#### SUMMARY
- Basic 1/d ratio: 19.52

#### DEFLECTION
- Span factor: 1
- Flange beam factor F1: 1
- Long spans factor F2: 1
- Steel stress factor F3: 1.271
- Allowable 1/d ratio: 24.8

Note: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for highly stressed concrete (C30/37 & ρ=1.5%) as follows:
- Simply supported spans: 14
- End spans: 18
- Interior spans: 20

### Shear check

<table>
<thead>
<tr>
<th>Location for shear calculation: 250mm from support</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear force due to ultimate load</td>
</tr>
<tr>
<td>Term for shear resistance</td>
</tr>
<tr>
<td>Shear resistance</td>
</tr>
<tr>
<td>Minimum stress</td>
</tr>
<tr>
<td>Minimum shear resistance</td>
</tr>
<tr>
<td>Modified shear resistance</td>
</tr>
</tbody>
</table>

#### SHEAR SUMMARY
- Design shear force: 125 kN
- Design shear resistance: 133.2 kN
Location: Typical continuous slab reinforced with high-yield steel

Bending in solid slabs (with comp.steel if reqd.),
designed for a given width, with checks on minimum steel
and span/effective-depth ratio

Calculations are in accordance with
Clause 3.4.4.4 of BS8110: Part 1 and
thus assume the use of a simplified
rectangular concrete stress-block
and that the depth to the neutral
axis is restricted to d/2.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution Mbef=1875 kNm
Slab containing section being analysed is considered as continuous.
Design moment (after redistrib.) M=1800 kNm
Characteristic concrete strength fcu=30 N/mm²
Characteristic steel strength fy=500 N/mm²
Longitudinal reinforcement is high-yield steel.
Diameter of tension bars dia=20 mm
Designated exposure class is XCl
Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=25 mm
Overall thickness of slab h=400 mm
Effective depth of section d=365 mm
Breadth of slab b=3000 mm
Chosen spacing of tension bars pch=60 mm
Diameter of distribution bars diamn=12 mm
Spacing of distribution bars pchDA=200 mm

TENSION
REINFORCEMENT
SUMMARY
Characteristic strength 500 N/mm²
Diameter of bars 20 mm
Number of bars 49
Spacing of bars 60 mm
Effective depth 365 mm
Area of steel required 14386 mm²
Area of steel provided 15393 mm²
Percentage provided 1.2828 %
Weight of steel provided 120.84 kg/m

DISTRIBUTION
REINFORCEMENT
SUMMARY
Characteristic strength 500 N/mm²
Diameter of bars 12 mm
Depth to bar centres 349 mm
Area of steel required 520 mm²/m
Area of steel provided 565 mm²/m
Percentage provided 0.14125 %
Weight of steel provided 4.44 kg/m²
Check on span/effective-depth ratio

Basic ratio for slab
continuous at one edge  bs'd=23
Mod.factor for tension steel  modf1=0.78517
Mod.factor for no comp.steel  modf2=1.0
Maximum permissible
span/effective-depth ratio  ps'd=bs'd*modf1*modf2=18.059
Effective span of slab  span=5 m
True span/effective-depth ratio  as'd=1000*span/d=13.699
As this does not exceed  18.059, this is Acceptable.
Location: Thick, heavily-loaded continuous slab, mild steel reinfmt.

Bending in solid slabs (with comp.steel if reqd.),
designed for a given width, with checks on minimum steel
and span/effective-depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to d/2.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution Mbef=6000 kNm
Slab containing section being analysed is considered as continuous.
Design moment (after redistrib.) M=5000 kNm
Characteristic concrete strength fcu=35 N/mm²
Characteristic steel strength fy=250 N/mm²
Longitudinal reinforcement is mild steel.
Diameter of tension bars dia=40 mm
Designated exposure class is XC2
Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=45 mm
Overall thickness of slab h=750 mm
Effective depth of section d=685 mm
Breadth of slab b=2000 mm
Diameter of compression bars diac=25 mm
Depth to compression steel d'=57.5 mm
Chosen comp.bar spacing (c.to c.) pchCA=260 mm
Chosen spacing of tension bars pch=50 mm
Diameter of distribution bars diamn=20 mm
Spacing of distribution bars pchDA=170 mm

**TENSION**  
Characteristic strength 250 N/mm²

**REINFORCEMENT**  
Diameter of bars 40 mm
Number of bars 38
Spacing of bars 50 mm
Effective depth 685 mm
Area of steel required 41241 mm²
Area of steel provided 47752 mm²
Percentage provided 3.1835 %
Weight of steel provided 374.85 kg/m
Reinforced concrete design to BS8110 & Eurocode 2

Solid slab - flexure, tens & comp steel, span/d

Date: 02/12/19

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COMPRESSION
Characteristic strength 250 N/mm²

REINFORCEMENT
Diameter of bars 25 mm
Number of bars 8
Spacing of bars 260 mm
Depth to bar centres 57.5 mm
Area of steel required 3000 mm²
Area of steel provided 3927 mm²
Percentage provided 0.2618%
Weight of steel provided 30.83 kg/m

SUMMARY
Number of bars 8
Spacing of bars 260 mm
Depth to bar centres 57.5 mm
Area of steel required 3000 mm²
Area of steel provided 3927 mm²
Percentage provided 0.2618%
Weight of steel provided 30.83 kg/m

DISTRIBUTION
Characteristic strength 250 N/mm²

REINFORCEMENT
Diameter of bars 20 mm
Spacing of bars 170 mm
Depth to bar centres 655 mm
Area of steel required 1800 mm²/m
Area of steel provided 1847 mm²/m
Percentage provided 0.24627%
Weight of steel provided 14.5 kg/m²

SUMMARY
Spacing of bars 170 mm
Depth to bar centres 655 mm
Area of steel required 1800 mm²/m
Area of steel provided 1847 mm²/m
Percentage provided 0.24627%
Weight of steel provided 14.5 kg/m²

Check on span/effective-depth ratio

Basic ratio for continuous slab bs'd=26 (see Table 3.9)
Mod.factor for tension steel modf1=0.95713
Area of compression steel prov. As'pr=3927 mm²
Percentage of comp.steel per'=0.2618%

From Equation 9 of BS8110, with percentage of comp.steel=0.2618 %,
Mod.factor for compression steel modf2=1+per'/(3+per')=1.0803
Maximum permissible
span/effective-depth ratio ps'd=bs'd*modf1*modf2=26.883
Effective span of slab span=12 m

Although span exceeds 10 m, partitions/finishes are not considered sensitive to deflection, so modification to basic ratio is unnecessary.

True span/effective-depth ratio as'd=1000*span/d=17.518

As this does not exceed 26.883, this is Acceptable.
Location: Lightly-loaded thin slab with non-standard reinforcement

Bending in solid slabs (with comp.steel if reqd.),
designed for a given width, with checks on minimum steel
and span/effective-depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to d/2.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Moment before redistribution Mbef=60 kNm
Slab containing section being analysed is considered as non-continuous.
Characteristic concrete strength fcu=60 N/mm²
Characteristic steel strength fy=425 N/mm²
Steel strength other than those given in Table 3.1 of BS8110.
Specified min.steel percentage permin=0.16 %
Specified min.slab thickness mindep=215 mm
Diameter of tension bars dia=16 mm
Designated exposure class is XC4
Specified fixing tolerance tol=10 mm
Nominal concrete cover cover=30 mm
Overall thickness of slab h=100 mm
Effective depth of section d=62 mm
Breadth of slab b=4000 mm
Chosen spacing of tension bars pch=200 mm
Diameter of distribution bars diamn=8 mm
Spacing of distribution bars pchDA=190 mm

**TENSION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>425 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>20</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>200 mm</td>
</tr>
<tr>
<td>Effective depth</td>
<td>62 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>2841.4 mm²</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>4021 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>1.0053 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>31.56 kg/m</td>
</tr>
</tbody>
</table>

**DISTRIBUTION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>425 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>8 mm</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>190 mm</td>
</tr>
<tr>
<td>Depth to bar centres</td>
<td>50 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>160 mm²/m</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>264 mm²/m</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.264 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>2.07 kg/m²</td>
</tr>
</tbody>
</table>
Check on span/effective-depth ratio

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic ratio for simp.-sup.slab</td>
<td>bs'd=20 (see Table 3.9)</td>
</tr>
<tr>
<td>Mod.factor for tension steel</td>
<td>modf1=1.0303</td>
</tr>
<tr>
<td>Mod.factor for no comp.steel</td>
<td>modf2=1.0</td>
</tr>
<tr>
<td>Maximum permissible span/effective-depth ratio</td>
<td>ps'd=bs'd<em>modf1</em>modf2=20.606</td>
</tr>
<tr>
<td>Effective span of slab</td>
<td>span=2.5 m</td>
</tr>
</tbody>
</table>

WARNING:

True span/effective-depth ratio 40.323
Perm.span/effective-depth ratio 20.606
Hence unacceptable.
Location: Ex1 - Continuous slab

Bending in solid slabs (with comp.steel if reqd.),
designed for a given width, with checks on minimum steel
and span/effective-depth ratio

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the NA is restricted to 0.45d.

Design BM before redistribution $M_{bef}=625$ kNm
Section being analysed is considered as continuous.
Section considered has a hogging moment
Design BM (after redistribution) $M=600$ kNm

Char yield strength of r'ment $f_yk=500$ N/mm²
Max.aggregate size (for bar spc.) $h_{agg}=20$ mm
Diameter of tension bars $dia=25$ mm
Diameter of distribution bars $diad=12$ mm
Effective depth of section $d=350$ mm
Breadth of section $b=1000$ mm

Main reinforcement

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
<th>FLEXURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall depth</td>
<td>400 mm</td>
<td>Effective depth</td>
</tr>
<tr>
<td>Parameter K</td>
<td>0.1633</td>
<td>Parameter K'</td>
</tr>
<tr>
<td>Lever arm ratio $z/d$</td>
<td>0.8255</td>
<td>Steel area (tension)</td>
</tr>
<tr>
<td>Steel percentage req.</td>
<td>1.365 %</td>
<td>Minimum area of steel</td>
</tr>
<tr>
<td>Maximum area of steel</td>
<td>16000 mm²</td>
<td>Distribution steel</td>
</tr>
</tbody>
</table>

Spacing of bars - tension reinforcement

Maximum pitch of bars (<3h) $p_{chmx}=400$ mm
Calculated pitch of bars $pitch=b*PI*dia^2/(4*As)=102.8$ mm
Chosen spacing of tension bars $p_{ch}=100$ mm
Spacing of distribution steel

Diameter of distribution bars \( d_{iad} = 12 \text{ mm} \)
Chosen spacing of distn. bars \( p_{chd} = 110 \text{ mm} \)

**TENSION REINFORCEMENT**

- Diameter of bars \( 25 \text{ mm} \)
- Spacing of bars \( 100 \text{ mm} \)
- Area of steel required \( 4776 \text{ mm}^2 \)
- Area of steel provided \( 4908 \text{ mm}^2 \)

**DISTRIBUTION REINFORCEMENT**

- Diameter of bars \( 12 \text{ mm} \)
- Spacing of bars \( 110 \text{ mm} \)
- Area of steel required \( 955.2 \text{ mm}^2/\text{m} \)
- Area of steel provided \( 1028 \text{ mm}^2/\text{m} \)

Shear check

Location for shear calculation: 350mm from internal support
Shear force due to ultimate load \( V_{Ed} = 250 \text{ kN} \)
Term for shear resistance \( v_{Rdc} = (\rho_{1} f_{ck})^{(1/3)} = 3.478 \)
Shear resistance \( V_{Rdc} = C_{Rdc} \cdot k_{s} \cdot v_{Rdc} \cdot b_{w} \cdot d/1000 = 256.5 \text{ kN} \)
Minimum stress \( v_{min} = 0.035 \cdot k_{s}^{1.5} \cdot f_{ck}^{0.5} = 0.4461 \text{ N/mm}^2 \)
Minimum shear resistance \( V_{Rdm} = v_{min} \cdot b_{w} \cdot d/1000 = 156.1 \text{ kN} \)

**SHEAR SUMMARY**

- Design shear force \( 250 \text{ kN} \)
- Design shear resistance \( 256.5 \text{ kN} \)

Crack control check

Unfactored permanent load on slab \( G_k = 100 \text{ kN} \)
Unfactored variable load on slab \( Q_k = 40 \text{ kN} \)
Permanent load factor \( \gamma_{G} = 1.35 \)
Variable load factor \( \gamma_{Q} = 1.5 \)
Factor \( \psi_2 \) for variable load \( \psi_2 = 0.3 \)
SLS stress in reinforcement \( \sigma_{SLS} = 500/1.15 \cdot (A_s/Aspr) \cdot \text{ratio} \cdot 1 \)
\( /\delta = 253.1 \text{ N/mm}^2 \)
Design crack width \( w_{k} = 0.3 \text{ mm} \)
Max allowable SLS stress \( \sigma_{SLS} = 320 \text{ N/mm}^2 \)

The bar spacing 100 mm used complies with Table 7.3N of EC2 Part 1:2004 hence satisfactory.
Location: Ex2 - Simply supported slab

Bending in solid slabs (with comp. steel if reqd.),

designed for a given width, with checks on minimum steel

and span/effective-depth ratio

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the NA is restricted to 0.45d.

Design BM before redistribution $M_{bef}=150 \text{ kNm}$
Section being analysed is considered as non-continuous.
Char yield strength of r'ment $f_{yk}=500 \text{ N/mm}^2$
Max. aggregate size (for bar spc.) $h_{agg}=20 \text{ mm}$
Diameter of tension bars $d_{ia}=16 \text{ mm}$
Diameter of distribution bars $d_{iad}=8 \text{ mm}$
Effective depth of section $d=245 \text{ mm}$
Breadth of section $b=1000 \text{ mm}$

Main reinforcement

| DESIGN | Overall depth | 300 mm |
| SUMMARY | Effective depth | 245 mm |
| FLEXURE | Parameter K | 0.0714 |
| | Parameter K' | 0.1684 |
| | Lever arm ratio z/d | 0.9324 |
| | Steel area (tension) | 1510 mm² |
| | Steel percentage req. | 0.6164 % |
| | Minimum area of steel | 408.9 mm² |
| | Maximum area of steel | 12000 mm² |
| | Distribution steel | 408.9 mm²/m |

Spacing of bars - tension reinforcement

| Maximum pitch of bars (<3h) | $p_{chmx}=400 \text{ mm}$ |
| Calculated pitch of bars | $p_{itch}=b\times\pi\times d_{ia}^2/(4\times A_{s})=133.1 \text{ mm}$ |
| Chosen spacing of tension bars | $p_{ch}=130 \text{ mm}$ |
Spacing of distribution steel

| Diameter of distribution bars | diad=8 mm |
| Chosen spacing of distribution bars | pchd=120 mm |

TENSION REINFORCEMENT

| Diameter of bars | 16 mm |
| Spacing of bars | 130 mm |
| Area of steel required | 1510 mm² |
| Area of steel provided | 1546 mm² |

DISTRIBUTION REINFORCEMENT

| Diameter of bars | 8 mm |
| Spacing of bars | 120 mm |
| Area of steel required | 408.9 mm²/m |
| Area of steel provided | 418 mm²/m |

Deflection check

Effective span of beam | L=4.3 m |
Actual span to depth ratio | l'd=L*1000/d=17.55 |
Allowable span/depth ratio | l'da=k*N*F1*F2*F3=19.98 |

DESIGN

| Actual 1/d ratio | 17.55 |
| Basic 1/d ratio | 19.52 |

DEFLECTION

| Span factor | 1 |
| Flange beam factor F1 | 1 |
| Long spans factor F2 | 1 |
| Steel stress factor F3 | 1.024 |
| Allowable 1/d ratio | 19.98 |

Note: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for highly stressed concrete (C30/37 & p=1.5%) as follows:

| Simply supported spans | 14 |
| End spans | 18 |
| Interior spans | 20 |

Shear check

| Location for shear calculation: | 250mm from support |
| Shear force due to ultimate load | VEd=125 kN |
| Term for shear resistance | vRd=(rho1*fck)^1/3=2.227 |
| Shear resistance | VRdc=CRdc*ks*vRdc*bw*d/1000=124.6 kN |
| Minimum stress | vmin=0.035*ks^1.5*fck^0.5=0.5438 N/mm² |
| Minimum shear resistance | VRdm=vmin*bw*d/1000=133.2 kN |
| Modified shear resistance | VRdc=VRdm=133.2 kN |

SHEAR SUMMARY

| Design shear force | 125 kN |
| Design shear resistance | 133.2 kN |

Crack control check

| Unfactored permanent load on slab | Gk=50 kN |
| Unfactored variable load on slab | Qk=20 kN |
| Permanent load factor | gamG=1.35 |
| Variable load factor | gamQ=1.5 |
| Factor ψ2 for variable load | psi2=0.3 |
SLS stress in reinforcement $\sigma_s = \frac{500/1.15 \times (A_s/A_{spr}) \times \text{ratio} \times 1}{\Delta} = 243.9 \text{ N/mm}^2$

Design crack width $w_k = 0.3 \text{ mm}$

Max allowable SLS stress $\sigma_s = $ TABLE 7.3 for $p_{ch}=130$

$= 296 \text{ N/mm}^2$

The bar spacing $130 \text{ mm}$ used complies with Table 7.3N of EC2 Part 1:2004 hence satisfactory.
Location: Ex3 - Lightly loaded section

Bending in solid slabs (with comp. steel if reqd.),
designed for a given width, with checks on minimum steel
and span/effective-depth ratio

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the NA is restricted to 0.45d.

Design BM before redistribution \( M_{bef} = 15 \text{ kNm} \)
Section being analysed is considered as non-continuous.
Char yield strength of reinforcement \( f_{yk} = 500 \text{ N/mm}^2 \)
Max. aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( d_{ia} = 10 \text{ mm} \)
Diameter of distribution bars \( d_{iad} = 10 \text{ mm} \)
Effective depth of section \( d = 165 \text{ mm} \)
Breadth of section \( b = 1000 \text{ mm} \)

Main reinforcement

| DESIGN | Overall depth | 200 mm |
| SUMMARY | Effective depth | 165 mm |
| FLEXURE | Parameter K | 0.0184 |
| | Parameter K' | 0.1684 |
| | Lever arm ratio \( z/d \) | 0.95 |
| | Steel area (tension) | 248.5 mm\(^2\) |
| | Steel percentage req. | 0.1506% |
| | Minimum area of steel | 248.5 mm\(^2\) |
| | Maximum area of steel | 8000 mm\(^2\) |
| | Distribution steel | 248.5 mm\(^2\)/m |

Spacing of bars - tension reinforcement

| TENSION (AND DISTRIBUTION) | Diameter of bars | 10 mm |
| REINFORCEMENT | Spacing of bars | 300 mm |
| | Area of steel required | 248.5 mm\(^2\) |
| | Area of steel provided | 261 mm\(^2\) |
Deflection check

Effective span of beam \( L = 4.3 \text{ m} \)
Actual span to depth ratio \( l'/d = L*1000/d = 26.06 \)
Allowable span/depth ratio \( l'/da = k*N*F1*F2*F3 = 121.7 \)
Absolute value of span/depth \( l'/da = 40*k = 40 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Actual l/d ratio</th>
<th>26.06</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Basic l/d ratio</td>
<td>115.9</td>
</tr>
<tr>
<td>DEFLECTION</td>
<td>Span factor</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Flange beam factor F1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Long spans factor F2</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Steel stress factor F3</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td>Allowable l/d ratio</td>
<td>40</td>
</tr>
</tbody>
</table>

NOTE: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for lightly stressed concrete (C30/37 & \( \rho = 0.5\% \)) as follows:
- Simply supported spans 20
- End spans 26
- Interior spans 30

Shear check

Location for shear calculation: 150 mm from support
Shear force due to ultimate load \( V_{Ed} = 14 \text{ kN} \)
Term for shear resistance \( v_{Rdc} = (\rho_1*f_{ck})^{(1/3)} = 1.334 \)
Shear resistance \( VR_{dc} = CR_{dc}*ks*v_{Rdc}*bw*d/1000 = 52.82 \text{ kN} \)
Minimum stress \( v_{min} = 0.035*ks^{1.5}*f_{ck}^{0.5} = 0.5422 \text{ N/mm}^2 \)
Minimum shear resistance \( VR_{dm} = v_{min}*bw*d/1000 = 89.47 \text{ kN} \)
Modified shear resistance \( VR_{dc} = VR_{dm} = 89.47 \text{ kN} \)

SHEAR SUMMARY
- Design shear force 14 kN
- Design shear resistance 89.47 kN

Crack control check
Location: Ex4 - End span

Bending in solid slabs (with comp.steel if reqd.), designed for a given width, with checks on minimum steel and span/effective-depth ratio

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the NA is restricted to 0.45d.

Design BM before redistribution $M_{bef}=85 \text{kNm}$
Section being analysed is considered as continuous.
Section considered has a sagging moment
Char yield strength of r'ment $f_{yk}=500 \text{N/mm}^2$
Max.aggregate size (for bar spc.) $\text{hagg}=20 \text{mm}$
Diameter of tension bars $\text{dia}=16 \text{mm}$
Diameter of distribution bars $\text{diad}=10 \text{mm}$
Effective depth of section $d=185 \text{mm}$
Breadth of section $b=1000 \text{mm}$

Main reinforcement

| DESIGN          | Overall depth | 225 mm |
| SUMMY           | Effective depth | 185 mm |
| FLEXURE         | Parameter K | 0.0828 |
|                 | Parameter K' | 0.1684 |
|                 | Lever arm ratio z/d | 0.9207 |
|                 | Steel area (tension) | 1148 mm$^2$ |
|                 | Steel percentage req. | 0.6204 % |
|                 | Minimum area of steel | 278.6 mm$^2$ |
|                 | Maximum area of steel | 9000 mm$^2$ |
|                 | Distribution steel | 278.6 mm$^2$/m |

Spacing of bars - tension reinforcement

Maximum pitch of bars ($<3h$) $p_{chnx}=400 \text{ mm}$
Calculated pitch of bars $\text{pitch}=b*\pi*\text{dia}^2/(4*\text{As})=175.2 \text{ mm}$
Chosen spacing of tension bars $p_{ch}=175 \text{ mm}$
Spacing of distribution steel

Diameter of distribution bars \( d_{id} = 10 \text{ mm} \)
Chosen spacing of distn. bars \( p_{chd} = 280 \text{ mm} \)

**TENSION REINFORCEMENT**
- Diameter of bars \( d = 16 \text{ mm} \)
- Spacing of bars \( s = 175 \text{ mm} \)
- Area of steel required \( A_{sr} = 1148 \text{ mm}^2 \)
- Area of steel provided \( A_{sp} = 1148 \text{ mm}^2 \)

**DISTRIBUTION REINFORCEMENT**
- Diameter of bars \( d = 10 \text{ mm} \)
- Spacing of bars \( s = 280 \text{ mm} \)
- Area of steel required \( A_{sr} = 278.6 \text{ mm}^2/m \)
- Area of steel provided \( A_{sp} = 280 \text{ mm}^2/m \)

**Deflection check**

Effective span of beam \( L = 4.3 \text{ m} \)
Actual span to depth ratio \( l'/d = L \times 1000/d = 23.24 \)
Allowable span/depth ratio \( l'/da = k \times N \times F1 \times F2 \times F3 = 23.73 \)

**DESIGN**
- Actual 1/d ratio \( = 23.24 \)
**SUMMARY**
- Basic 1/d ratio \( = 18.25 \)
**DEFLECTION**
- Span factor \( = 1.3 \)
- Flange beam factor \( F1 = 1 \)
- Long spans factor \( F2 = 1 \)
- Steel stress factor \( F3 = 1 \)
- Allowable 1/d ratio \( = 23.73 \)

Note: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for highly stressed concrete \((C30/37 & \rho=1.5\%)\) as follows:

<table>
<thead>
<tr>
<th>Type of Span</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simply supported spans</td>
<td>14</td>
</tr>
<tr>
<td>End spans</td>
<td>18</td>
</tr>
<tr>
<td>Interior spans</td>
<td>20</td>
</tr>
</tbody>
</table>

**Shear check**

Location for shear calculation: \( 200 \text{ mm from support} \)
Shear force due to ultimate load \( V_{Ed} = 68 \text{ kN} \)
Term for shear resistance \( v_{Rdc} = (\rho_{01} \times f_{ck})^{(1/3)} = 2.104 \)
Shear resistance \( V_{Rdc} = C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d / 1000 = 93.4 \text{ kN} \)
Minimum stress \( v_{min} = 0.035 \times k_s \times 1.5 \times f_{ck} \times 0.5 = 0.5422 \text{ N/mm}^2 \)
Minimum shear resistance \( V_{Rdm} = v_{min} \times b_w \times d / 1000 = 100.3 \text{ kN} \)
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 100.3 \text{ kN} \)

**SHEAR SUMMARY**
- Design shear force \( = 68 \text{ kN} \)
- Design shear resistance \( = 100.3 \text{ kN} \)

**Crack control check**

Unfactored permanent load on slab \( G_k = 40 \text{ kN} \)
Unfactored variable load on slab \( Q_k = 20 \text{ kN} \)
Permanent load factor \( \gamma_{G} = 1.35 \)
Variable load factor \( \gamma_{Q} = 1.5 \)
Factor \( \psi_2 \) for variable load \( \psi_2 = 0.3 \)
SLS stress in reinforcement $s$  
\[ SLS_s = \frac{500}{1.15} \times \frac{(As/Aspr) \times \text{ratio} \times 1}{\delta} = 238.1 \text{ N/mm}^2 \]

Design crack width $wk = 0.3 \text{ mm}$

Max allowable SLS stress $s$  
\[ SLS_{s1} = \text{TABLE 7.2 for dia=16} = 240 \text{ N/mm}^2 \]

The bar size 16 mm used complies with Table 7.2N of EC2 Part 1:2004 hence satisfactory.
Bending in solid slabs (with comp. steel if reqd.),
designed for a given width, with checks on minimum steel
and span/effective-depth ratio

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the NA is restricted to 0.45d.

Design BM before redistribution \( M_{bef} = 375 \text{ kNm} \)
Section being analysed is considered as non-continuous.
Char yield strength of reinforcement \( f_{yk} = 500 \text{ N/mm}^2 \)
Max. aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( \text{dia} = 20 \text{ mm} \)
Diameter of distribution bars \( \text{diad} = 12 \text{ mm} \)
Effective depth of section \( d = 235 \text{ mm} \)
Breadth of section \( b = 1000 \text{ mm} \)

Main reinforcement

Diameter of compression bars \( \text{diac} = 20 \text{ mm} \)
Depth to compression steel \( d_2 = 45 \text{ mm} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth</th>
<th>300 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth</td>
<td>235 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K</td>
<td>0.1698</td>
</tr>
<tr>
<td></td>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio z/d</td>
<td>0.8185</td>
</tr>
<tr>
<td></td>
<td>Steel area (tension)</td>
<td>4485 mm²</td>
</tr>
<tr>
<td></td>
<td>Steel area (compress.)</td>
<td>600 mm²</td>
</tr>
<tr>
<td></td>
<td>Steel percentage req.</td>
<td>1.908 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel</td>
<td>428.8 mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel</td>
<td>12000 mm²</td>
</tr>
<tr>
<td></td>
<td>Distribution steel</td>
<td>896.9 mm²/m</td>
</tr>
</tbody>
</table>

Spacing of bars - tension reinforcement

Maximum pitch of bars (<3h) \( p_{chmx} = 400 \text{ mm} \)
Calculated pitch of bars \( \text{pitch} = b \cdot \pi \cdot \text{dia}^2 / (4 \cdot \text{As}) = 70.05 \text{ mm} \)
Chosen spacing of tension bars \( \text{pch} = 70 \text{ mm} \)
Spacing of distribution steel

Diameter of distribution bars diad=12 mm
Chosen spacing of distn. bars pchd=125 mm
Chosen spacing of compress bars pchc=250 mm

TENSION REINFORCEMENT

Diameter of bars 20 mm
Spacing of bars 70 mm
Area of steel required 4485 mm²
Area of steel provided 4487 mm²

DISTRIBUTION REINFORCEMENT

Diameter of bars 12 mm
Spacing of bars 125 mm
Area of steel required 896.9 mm²/m
Area of steel provided 904 mm²/m

COMPRESSION

AND DISTRIBUTION

REINFORCEMENT

Area of steel required 600 mm²
Area of steel provided 1256 mm²

Deflection check

Effective span of beam L=4.3 m
Unfactored permanent load on slab Gk=460 kN
Unfactored variable load on slab Qk=230 kN
Permanent load factor gamG=1.35
Variable load factor gamQ=1.5
Actual span to depth ratio l'd=L*1000/d=18.3
Factor Z2 for variable load psi2=0.3
SLS stress in reinforcement ås SLSs=500/1.15*(As/Aspr)*ratio*1/delta=238 N/mm²
Allowable span/depth ratio l'da=k*N*F1*F2*F3=18.57

DESIGN

Actual l/d ratio 18.3

SUMMARY

Basic l/d ratio 14.26

DEFLECTION

Span factor 1
Flange beam factor F1 1
Long spans factor F2 1
Steel stress factor F3 1.303
Allowable l/d ratio 18.57

Note: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for highly stressed concrete (C30/37 & ρ=1.5%) as follows:

Simply supported spans 14
End spans 18
Interior spans 20

Shear check

Location for shear calculation: 250mm from support
Shear force due to ultimate load VEd=125 kN
Term for shear resistance vRdc=(rho1*fck)^(1/3)=3.367
Shear resistance VRdc=CRdc*ks*vRdc*bw*d/1000=182.6 kN
Minimum stress vmin=0.035*ks^1.5*fck^0.5=0.5901 N/mm²
Minimum shear resistance VRdm=vmin*bw*d/1000=138.7 kN
<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Design shear force</th>
<th>125 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design shear resistance</td>
<td>182.6 kN</td>
</tr>
</tbody>
</table>
Location: High shear on moderately thick slab (High-yield steel)

Shear in longitudinal zone of solid slab of given width

Since distance from support varies throughout zone considered, enhancement of shear resistance due to proximity to support is not applicable.

The calculations for shear in the zone of solid slab are in accordance with Clauses 3.5.5 of BS8110 throughout. Support $d$ zone $d$ = effective depth.

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$

Effective depth $d = 400$ mm

Breadth of slab zone $b = 1000$ mm

Characteristic concrete strength $f_{cu} = 30$ N/mm²

Diameter of tension bars $dia = 25$ mm

Number of tension bars $nbars = 4$

Shear force due to ult.loads $V = 900$ kN

Design shear stress of $2.25$ N/mm² exceeds critical stress of $0.52979$ N/mm², so links to resist balance of shear forces are necessary.

Characteristic strength of links $f_{yv} = 500$ N/mm²

Number of transverse legs $nlegs = 4$

Diameter of link legs $dial = 12$ mm

Use 4 No. 12 mm dia. legs transversely ($f_{yv} = 500$ N/mm²) and spaced at 110 mm centres in direction of span. Transverse spacing must not exceed effective depth of 400 mm
Location: High shear on thin wide slab (High-yield steel)

Shear in longitudinal zone of solid slab of given width

Since distance from support varies throughout zone considered, enhancement of shear resistance due to proximity to support is not applicable.

The calculations for solid slab in accordance with Clauses 3.5.5 of BS8110 throughout. In the case of a uniform load, the design section to be considered need not be less than a minimum distance from support equal to effective depth.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Effective depth d=200 mm
Breadth of slab zone b=5000 mm
Characteristic concrete strength fcu=35 N/mm²
Diameter of tension bars dia=16 mm
Number of tension bars nbars=50
Shear force due to ult.loads V=2200 kN

Design shear stress of 2.2 N/mm² exceeds critical stress of 0.84227 N/mm², so links to resist balance of shear forces are necessary.

Characteristic strength of links fyv=500 N/mm²
Number of transverse legs nlegs=26
Diameter of link legs dial=8 mm

Use 26 No. 8 mm dia. legs transversely (fyv=500 N/mm²) and spaced at 80 mm centres in direction of span. Transverse spacing must not exceed effective depth of 200 mm.
**Location: Low shear on deep narrow slab (High-yield steel)**

Shear in longitudinal zone of solid slab of given width

Since distance from support varies throughout zone considered, enhancement of shear resistance due to proximity to support is not applicable.

The calculations for shear in the zone of solid slab are in accordance with Clauses 3.5.5 of BS8110 throughout.

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Effective depth $d=1000$ mm

Breadth of slab zone $b=200$ mm

Characteristic concrete strength $f_{cu}=30$ N/mm$^2$

Diameter of tension bars $d_{ia}=20$ mm

Number of tension bars $n_{bars}=2$

Shear force due to ult.loads $V=50$ kN

**WARNING:**

As $v_c=0.36309$ N/mm$^2$ exceeds $v=0.25$ N/mm$^2$ no shear links are needed.
Location: Low shear slab

Design data

Design shear force $\text{VE}_d = 51 \text{ kN}$
Overall depth of section $h = 250 \text{ mm}$
Effective depth of section $d = 204 \text{ mm}$
Effective breadth for shear $b_w = 1000 \text{ mm}$
Char yield strength (reinf'ment) $f_{yk} = 500 \text{ N/mm}^2$
Diameter of tension bars $\text{dia} = 10 \text{ mm}$
Number of tension bars $n_{bars} = 10$
Term for shear resistance $v_{Rdc} = (\rho_1 \cdot f_{ck})^{(1/3)} = 2.3096$
Shear resistance $V_{Rdc} = C_{Rdc} \cdot v_{Rdc} \cdot b_w \cdot d / 1000 = 112.52 \text{ kN}$
Minimum stress $v_{min} = 0.035 \cdot ks^{1.5} \cdot f_{ck}^{0.5}$
= 0.55587 $\text{N/mm}^2$
Minimum shear resistance $V_{Rdm} = v_{min} \cdot b_w \cdot d / 1000 = 113.4 \text{ kN}$
Modified shear resistance $V_{Rdc} = V_{Rdm} = 113.4 \text{ kN}$

SHEAR SUMMARY

Design shear force $51 \text{ kN}$
Design shear resistance $113.4 \text{ kN}$

Shear in rib of ribbed slab (no enhancement)

As distance from support varies throughout zone considered, enhancement of shear resistance due to proximity to support is not applicable.

Calculations for ribbed slab in accordance with Clauses 3.6.4 and 3.6.1.3 of BS8110.

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

With uniform load, design section considered need not be nearer to support than effective depth

Effective depth $d=300$ mm
Average rib width $b_v=400$ mm
Characteristic concrete strength $f_{cu}=30$ N/mm$^2$
Diameter of tension bars $d_{ia}=20$ mm
Number of tension bars in rib $n_{bars}=3$
Ultimate shear force in rib $V=320$ kN

Design shear stress of 2.6667 N/mm$^2$ exceeds critical stress of 0.66585 N/mm$^2$, so links to resist balance of shear forces are essential.

Characteristic strength of links $f_{yv}=500$ N/mm$^2$
Number of transverse legs $n_{legs}=4$
Diameter of link legs $d_{ial}=10$ mm

**LINK SUMMARY**

- Reinforcement strength 500 N/mm$^2$
- Bar diameter 10 mm
- Number of legs 4
- Area 314 mm$^2$
- Centres 170 mm
Location: Large rib. Low shear. High-yield steel.

Shear in rib of ribbed slab (no enhancement)

As distance from support varies throughout zone considered, enhancement of shear resistance due to proximity to support is not applicable.

Calculations for ribbed slab in accordance with Clauses 3.6.4 and 3.6.1.3 of BS8110.

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

With uniform load, design section considered need not be nearer to support than effective depth

Effective depth $d=600$ mm
Average rib width $b_v=300$ mm
Characteristic concrete strength $f_{cu}=25$ N/mm$^2$
Diameter of tension bars $d_{ia}=20$ mm
Number of tension bars in rib $n_{bars}=3$
Ultimate shear force in rib $V=50$ kN

Since design shear stress of $0.27778$ N/mm$^2$ is less than critical shear stress of $0.46029$ N/mm$^2$, no special shear reinforcement is required.

However as there are 3 bars in rib, links to ensure cover are recommended at 1 to 1.5m spacing (See Clause 3.6.6.3).
**Location:** Small rib. High shear. High-yield steel.

Shear in rib of ribbed slab (no enhancement)

As distance from support varies throughout zone considered, enhancement of shear resistance due to proximity to support is not applicable.

Calculations for ribbed slab in accordance with Clauses 3.6.4 and 3.6.1.3 of BS8110.

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

With uniform load, design section considered need not be nearer to support than effective depth.

Effective depth $d=300 \text{ mm}$
Average rib width $b_v=150 \text{ mm}$
Characteristic concrete strength $f_{cu}=35 \text{ N/mm}^2$
Diameter of tension bars $d_{ia}=20 \text{ mm}$
Number of tension bars in rib $n_{bars}=2$
Ultimate shear force in rib $V_{ult}=210 \text{ kN}$

Design shear stress of $4.6667 \text{ N/mm}^2$ exceeds critical stress of $0.84915 \text{ N/mm}^2$, so links to resist balance of shear forces are essential.

Characteristic strength of links $f_{yv}=500 \text{ N/mm}^2$
Number of transverse legs $n_{legs}=2$
Diameter of link legs $d_{ial}=10 \text{ mm}$

**LINK SUMMARY**

<table>
<thead>
<tr>
<th>Reinforcement strength</th>
<th>$500 \text{ N/mm}^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bar diameter</td>
<td>$10 \text{ mm}$</td>
</tr>
<tr>
<td>Number of legs</td>
<td>$2$</td>
</tr>
<tr>
<td>Area</td>
<td>$157 \text{ mm}^2$</td>
</tr>
<tr>
<td>Centres</td>
<td>$110 \text{ mm}$</td>
</tr>
</tbody>
</table>
Location: Ex1 - Shallow/wide rib. High shear.

Shear in ribbed slab


d=300 mm
bw=400 mm
fck=30 N/mm²
fyk=500 N/mm²
dia=20 mm
nbars=3
VEd=320 kN

Partial safety factor for r'ment gams=1.15
Partial safety factor for conc. gamc=1.5
fyd=fyk/gams=434.78 N/mm²
Aswm=0.08*bw*fck^0.5/fyk=0.35054
nsl=2
Asw=nsl*PI/4*dial^2=157.08 mm²
smax=INT(Asw/Asw's)=144 mm
s=140 mm

As VEd ≤ VRdmax ( 320 kN ≤ 396.12 kN ), design shear force OK.

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>320 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear strength</td>
<td>2.963 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.2736 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>1.0904</td>
<td></td>
</tr>
<tr>
<td>Area of links</td>
<td>157 mm²</td>
<td></td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>144 mm</td>
<td></td>
</tr>
<tr>
<td>Actual spacing</td>
<td>140 mm</td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex2 - Large rib. Low shear. High-yield steel.

Shear in ribbed slab

Effective depth \( d = 600 \text{ mm} \)
Average rib width \( bw = 400 \text{ mm} \)
Char cylinder compress strength \( f_{ck} = 25 \text{ N/mm}^2 \)
Char yield strength of links \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of tension bars \( \text{dia} = 20 \text{ mm} \)
Number of tension bars in rib \( \text{nbars} = 3 \)
Ultimate shear force in rib \( V_{Ed} = 320 \text{ kN} \)

Shear reinforcement

Partial safety factor for r'ment \( \gamma_{ams} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Design yield strength of links \( f_{yd} = \frac{f_{yk}}{\gamma_{ams}} = 434.78 \text{ N/mm}^2 \)
Minimum ratio area to spacing \( A_{sw} = 0.08 \times bw \times f_{ck}^{0.5} / f_{yk} = 0.32 \)
Number of shear legs \( n_{sl} = 2 \)
Diameter of link legs \( \text{dial} = 8 \text{ mm} \)
Area of shear legs \( A_{sw} = \text{nsl} \times \pi / 4 \times \text{dial}^2 = 100.53 \text{ mm}^2 \)
Spacing of links \( s_{max} = \text{INT} (A_{sw} / A_{sw}'s) = 184 \text{ mm} \)
Chosen spacing \( s = 175 \text{ mm} \)

Check crushing strength

Angle of inclination of strut \( \theta' = 22^\circ \)
compute \( \text{comp} = \cot(\theta) = 1 / \tan(\theta) = 2.4751 \)
Maximum shear resistance:

\[
VR_{dmax} = 0.36 \times bw \times d \times (1 - f_{ck} / 250) \times f_{ck} / ((\text{comp} + 1 / \text{comp}) \times 1000) = 675.21 \text{ kN}
\]
As \( V_{Ed} \leq VR_{dmax} \) (320 kN \( \leq 675.21 \text{ kN} \)), design shear force OK.

**Shear Summary**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>320 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear strength</td>
<td>1.4815 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>2.79 N/mm²</td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.54519</td>
</tr>
<tr>
<td>Area of links</td>
<td>100 mm²</td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>184 mm</td>
</tr>
<tr>
<td>Actual spacing</td>
<td>175 mm</td>
</tr>
</tbody>
</table>
Location: Ex3 - Small rib. High shear. High-yield steel.

Shear in ribbed slab


Effective depth  d=300 mm
Average rib width  bw=400 mm
Char cylinder compress strength  fck=35 N/mm²
Char yield strength of links  fyk=500 N/mm²
Diameter of tension bars  dia=20 mm
Number of tension bars in rib  nbars=2
Ultimate shear force in rib  VEd=480 kN

Shear reinforcement

Partial safety factor for r'ment  gams=1.15
Partial safety factor for conc.  gamc=1.5
Design yield strength of links  fyd=fyk/gams=434.78 N/mm²
Minimum ratio area to spacing  Aswm=0.08*bw*fck^0.5/fyk=0.37863
Number of shear legs  nsl=2
Diameter of link legs  dial=12 mm
Area of shear legs  Asw=nsl*PI/4*dial^2=226.19 mm²
Spacing of links  smax=INT(Asw/Asw's)=125 mm

Check crushing strength

Angle of inclination of strut  theta'=23.792°
compute  comp=cot(θ)=1/tan(θ)=2.2681
Maximum shear resistance:
VRdmax=0.36*bw*d*(1-fck/250)*fck/((comp+1/comp)*1000)=480 kN
As VEd ≤ VRdmax ( 480 kN ≤ 480 kN ), design shear force OK.

SHEAR SUMMARY

Shear force  480 kN
Design shear strength  4.4444 N/mm²
Concrete strut capacity  3.7324 N/mm²
Area/spacing ratio  1.8028
Area of links  226 mm²
Maximum spacing  125 mm
Actual spacing  125 mm
Location: Ex4 - Low shear (no shear links needed).

Shear in ribbed slab


Effective depth $d=600$ mm
Average rib width $bw=300$ mm
Char cylinder compress strength $f_{ck}=20$ N/mm$^2$
Char yield strength of links $f_{yk}=500$ N/mm$^2$
Diameter of tension bars $dia=20$ mm
Number of tension bars in rib $nbars=3$
Ultimate shear force in rib $V_{Ed}=50$ kN

As $V_{Ed} < V_{Rdc}$ (50 kN < 74.535 kN), shear r'ment is not required. However, consider providing nominal links to allow for fabrication.
Location: Large square slab with all edges free

Design moments in uniformly loaded two-way slabs to BS8110

Calculation of design moments according to Clauses 3.5.3 of BS8110.

\[
\text{dead load} = g_k \text{ kN/m}^2 \\
\text{live load} = q_k \text{ kN/m}^2
\]

Length of shorter side \( l_x = 20 \text{ m} \)
Length of longer side \( l_y = 20 \text{ m} \)
Characteristic dead load \( g_k = 4 \text{ kN/m}^2 \)
Characteristic imposed load \( q_k = 5 \text{ kN/m}^2 \)

All four edges simply supported

\[
\begin{array}{c}
g_k \text{ kN/m}^2 \\
q_k \text{ kN/m}^2
\end{array}
\]

Design moment for short span \( l_x \)
\[
msx = alpx \times n \times 1y \times l_x^2 = 6800 \text{ kNm}
\]

Design moment for long span \( l_y \)
\[
msy = alpsy \times n \times l_x^3 = 6800 \text{ kNm}
\]

Reinforcement must be provided over full width.
No torsion steel is required.
Location: Long rectangular slab with all edges fixed

Design moments in uniformly loaded two-way slabs to BS8110

Calculation of design moments according to Clauses 3.5.3 of BS8110.

dead load = gk kN/m²
live load = qk kN/m²

Length of shorter side           lx=10 m
Length of longer side             ly=20 m
Characteristic dead load          gk=4 kN/m²
Characteristic imposed load       qk=5 kN/m²

All four edges continuous

Coefficients from Table 3.14.
Design moment for short span \( lx \) (over width of 15 m)

<table>
<thead>
<tr>
<th>Condition</th>
<th>Equation</th>
<th>Moment (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>At continuous edges</td>
<td>( msxe = \text{betsxn} \times n \times \text{mstrwy} \times lx^2 )</td>
<td>1285.2</td>
</tr>
<tr>
<td>At midspan</td>
<td>( msxm = \text{betsxp} \times n \times \text{mstrwy} \times lx^2 )</td>
<td>979.2</td>
</tr>
</tbody>
</table>

Design moment for long span \( ly \) (over width of 7.5 m)

<table>
<thead>
<tr>
<th>Condition</th>
<th>Equation</th>
<th>Moment (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>At continuous edges</td>
<td>( msye = \text{betsyn} \times n \times \text{mstrwx} \times ly^2 )</td>
<td>326.4</td>
</tr>
<tr>
<td>At midspan</td>
<td>( msym = \text{betsyp} \times n \times \text{mstrwx} \times ly^2 )</td>
<td>244.8</td>
</tr>
</tbody>
</table>
Location: Slab with two adjacent edges fixed

Design moments in uniformly loaded two-way slabs to BS8110

Calculation of design moments according to Clauses 3.5.3 of BS8110.

\[
\begin{align*}
\text{dead load} &= g_k \text{ kN/m}^2 \\
\text{live load} &= q_k \text{ kN/m}^2
\end{align*}
\]

Length of shorter side \( l_x = 6 \text{ m} \)
Length of longer side \( l_y = 9 \text{ m} \)
Characteristic dead load \( g_k = 4 \text{ kN/m}^2 \)
Characteristic imposed load \( q_k = 5 \text{ kN/m}^2 \)

Two adjacent edges discontinuous

Coefficients from Table 3.14.
**Design moment for short span lx (over width of 6.75 m)**

- At continuous edge: \( msxe=betsxn*n*mstrwy*lx^2=257.77 \text{ kNm} \)
- At midspan: \( msxm=betsxp*n*mstrwy*lx^2=194.98 \text{ kNm} \)

**Design moment for long span ly (over width of 4.5 m)**

- At continuous edge: \( msye=betsyn*n*mstrwx*lx^2=99.144 \text{ kNm} \)
- At midspan: \( msym=betsyp*n*mstrwx*lx^2=74.909 \text{ kNm} \)
Location: Large square slab with all edges free

Design moments in uniformly loaded two-way slabs to EC2

Bending moment coefficients from Table 5.3 of the ISE "Manual for the design of concrete building structures to Eurocode 2" will be used in the following calculations.

\[
g_k \text{ kN/m}^2 \\
q_k \text{ kN/m}^2
\]

Length of shorter side \( l_x = 20 \text{ m} \)
Length of longer side \( l_y = 20 \text{ m} \)
Characteristic permanent load \( g_k = 4 \text{ kN/m}^2 \)
Characteristic variable load \( q_k = 5 \text{ kN/m}^2 \)

**All four edges simply supported**

\[
g_k \text{ kN/m}^2 \\
q_k \text{ kN/m}^2
\]

Design moment for short span \( l_x \) \( m_{sx} = a l_p s_x n l_y l_x^2 = 6450 \text{ kNm} \)
Design moment for long span \( l_y \) \( m_{sy} = a l_p s_y n l_x^3 = 6450 \text{ kNm} \)

Reinforcement must be provided over full width.
No torsion steel is required.
Location: Long rectangular slab with all edges fixed

Design moments in uniformly loaded two-way slabs to EC2

Bending moment coefficients from Table 5.3 of the ISE "Manual for the design of concrete building structures to Eurocode 2" will be used in the following calculations.

\[
gk = \text{permanent load} \\
qk = \text{variable load}
\]

Length of shorter side \( lx = 10 \) m
Length of longer side \( ly = 20 \) m
Characteristic permanent load \( gk = 4 \) kN/m²
Characteristic variable load \( qk = 5 \) kN/m²

**All four edges continuous**

Coefficients from Table 5.3
Design moment for short span 1x - over width of 15 m

At continuous edges
msxe = betsxn * n * mstrwy * lx^2 = 1219.1 kNm
At midspan
msxm = betsxp * n * mstrwy * lx^2 = 928.8 kNm

Design moment for long span 1y - over width of 7.5 m

At continuous edges
msye = betsyn * mstrwx * lx^2 = 309.6 kNm
At midspan
msym = betsyp * mstrwx * lx^2 = 232.2 kNm
Location: Slab with two adjacent edges fixed

Design moments in uniformly loaded two-way slabs to EC2

Bending moment coefficients from Table 5.3 of the ISE "Manual for the design of concrete building structures to Eurocode 2" will be used in the following calculations.

Length of shorter side \( l_x = 6 \text{ m} \)
Length of longer side \( l_y = 9 \text{ m} \)
Characteristic permanent load \( g_k = 4 \text{ kN/m}^2 \)
Characteristic variable load \( q_k = 5 \text{ kN/m}^2 \)

Two adjacent edges discontinuous

Coefficients from Table 5.3
Design moment for short span lx - over width of 6.75 m

At continuous edge  \( msxe = \text{betsxn} \times n \times \text{mstrwy} \times lx^2 = 244.51 \text{ kNm} \)

At midspan  \( msxm = \text{betsxp} \times n \times \text{mstrwy} \times lx^2 = 184.95 \text{ kNm} \)

Design moment for long span ly - over width of 4.5 m

At continuous edge  \( msye = \text{betsyn} \times n \times \text{mstrwx} \times lx^2 = 94.041 \text{ kNm} \)

At midspan  \( msym = \text{betsyp} \times n \times \text{mstrwx} \times lx^2 = 71.053 \text{ kNm} \)
Location: Typical two-way slab with all edges fixed

Design of two-way slabs rectangular slabs supporting uniform loads

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

Calculation of design moments in accordance with Clauses 3.5.3 of BS8110 throughout.

\[
\text{dead load} = g_k \, \text{kN/m}^2 \\
\text{live load} = q_k \, \text{kN/m}^2
\]

Length of shorter side \( l_x = 6 \, \text{m} \)
Length of longer side \( l_y = 8 \, \text{m} \)
Characteristic dead load \( g_k = 4 \, \text{kN/m}^2 \)
Characteristic imposed load \( q_k = 5 \, \text{kN/m}^2 \)
Overall depth of slab \( h = 200 \, \text{mm} \)
Characteristic concrete strength \( f_{cu} = 30 \, \text{N/mm}^2 \)
Characteristic strength of steel \( f_y = 500 \, \text{N/mm}^2 \)
Bond type (0, 1 or 2) Type = 2
Size of upper bars in x-direction \( d_{aux} = 20 \, \text{mm} \)
Size of lower bars in x-direction \( d_{ax} = 20 \, \text{mm} \)
Size of upper bars in y-direction \( d_{auy} = 16 \, \text{mm} \)
Size of lower bars in y-direction \( d_{ay} = 16 \, \text{mm} \)
Designated exposure class is XC2
Specified fixing tolerance \( t_{ol} = 10 \, \text{mm} \)
Designated exposure class is XC2
Specified fixing tolerance \( t_{ol} = 10 \, \text{mm} \)
Nominal cover at top of slab \( t_{cover} = 35 \, \text{mm} \)
Nominal cover at bottom of slab \( b_{cover} = 35 \, \text{mm} \)
Effective depths:
Of top steel:
in x-direction \( d_{ux} = 155 \, \text{mm} \)
in y-direction \( d_{uy} = d_{ux} - (d_{aux} + d_{auy})/2 = 137 \, \text{mm} \)
Of bottom steel:
in x-direction \( d_{dx} = 155 \, \text{mm} \)
in y-direction \( d_{dy} = d_{dx} - (d_{ax} + d_{ay})/2 = 137 \, \text{mm} \)

Coefficients from Table 3.14

All four edges continuous
Chosen pitch for tension steel \( \text{pitchC}=480 \text{ mm} \)
Chosen pitch for tension steel \( \text{pitchC}=420 \text{ mm} \)
Chosen pitch for tension steel \( \text{pitchC}=480 \text{ mm} \)
Chosen pitch for tension steel \( \text{pitchC}=420 \text{ mm} \)

Mod.factor for tension steel:
From equation 7 (Table 3.10) \( \text{modf1}=0.55+(477-fs)/(120*(0.9+M'bd2))=2.3092 \)
but this cannot exceed 2, so \( \text{modf1}=2 \)
Mod.factor for no comp.steel \( \text{modf2}=1 \)
Thus maximum permissible
span/effective-depth ratio \( \text{ps}'d=bs'd*\text{modf1*modf2}=52 \)
Actual span/effective-depth ratio \( \text{as}'d=1000*\text{span}/dx=38.71 \)
This does not exceed permr.ratio of 52, and is thus acceptable.

SUMMARY OF REINFORCEMENT REQUIRED ( \( f_y=500 \text{ N/mm}^2 \) )

At edges in x-direction:
\[
\begin{array}{ll}
\text{Characteristic strength} & 500 \text{ N/mm}^2 \\
\text{Area of steel required} As_x & 2171.9 \text{ mm}^2 \\
\text{Area of steel provided} As_{px} & 4084.1 \text{ mm}^2 \\
\text{Percentage of steel provided} & 0.34034 \% \\
\text{Size of bars} & 20 \text{ mm} \\
\text{Number of bars provided} & 13 \\
\text{Spacing of bars} & 480 \text{ mm centres} \\
\text{Minimum bond length required} & 425 \text{ mm} \\
\text{Effective depth adopted} & 155 \text{ mm} \\
\text{Weight of steel provided} & 32.06 \text{ kg/m run} \\
\end{array}
\]

At midspan in x-direction:
\[
\begin{array}{ll}
\text{Characteristic strength} & 500 \text{ N/mm}^2 \\
\text{Area of steel required} As_x & 1636.5 \text{ mm}^2 \\
\text{Area of steel provided} As_{px} & 4084.1 \text{ mm}^2 \\
\text{Percentage of steel provided} & 0.34034 \% \\
\text{Size of bars} & 20 \text{ mm} \\
\text{Number of bars provided} & 13 \\
\text{Spacing of bars} & 480 \text{ mm centres} \\
\text{Minimum bond length required} & 320 \text{ mm} \\
\text{Effective depth adopted} & 155 \text{ mm} \\
\text{Weight of steel provided} & 32.06 \text{ kg/m run} \\
\end{array}
\]

At edges in y-direction:
\[
\begin{array}{ll}
\text{Characteristic strength} & 500 \text{ N/mm}^2 \\
\text{Area of steel required} As_y & 1245.9 \text{ mm}^2 \\
\text{Area of steel provided} As_{py} & 2211.7 \text{ mm}^2 \\
\text{Percentage of steel provided} & 0.24574 \% \\
\text{Size of bars} & 16 \text{ mm} \\
\text{Number of bars provided} & 11 \\
\text{Spacing of bars} & 420 \text{ mm centres} \\
\text{Minimum bond length required} & 360 \text{ mm} \\
\text{Effective depth adopted} & 137 \text{ mm} \\
\text{Weight of steel provided} & 17.362 \text{ kg/m run} \\
\end{array}
\]
At midspan in y-direction:

- Characteristic strength: 500 N/mm²
- Area of steel required Asym: 1170 mm²
- Area of steel provided Asym: 2211.7 mm²
- Percentage of steel provided: 0.24574 %
- Size of bars: 16 mm
- Number of bars provided: 11
- Spacing of bars: 420 mm centres
- Minimum bond length required: 270 mm
- Effective depth adopted: 137 mm
- Weight of steel provided: 17.362 kg/m run

Minimum tension reinforcement in edge strips

- Min. tension area (Table 3.25) = \( \frac{\text{Asmin}}{100} \times 1000 \times h \)
  \( = 260 \) mm² per metre
- Diameter of distribution bars: diamin = 12 mm
- Maximum pitch (c.to c.of bars) = \( 3 \times d_{uy} + \text{diamin} = 423 \) mm

As this is less than 750 mm, this is satisfactory.

Chosen pitch for tension steel = pitchC = 420 mm

Minimum reinforcement in edge strips:

- Characteristic strength: 500 N/mm²
- Minimum area required Asmin: 260 mm² per m
- Steel area provided Asmpro: 269 mm² per m
- Percentage of steel provided: 0.1345 %
- Size of bars: 12 mm
- Spacing of bars: 420 mm centres
- Minimum lap length required: 300 mm
- Weight of steel provided: 2.1117 kg/m²

No torsion reinforcement is required (see Rule 7 in Cl. 3.5.3.5).
Location: Large thick two-way slab reinforced with mild steel

Design of two-way slabs rectangular slabs supporting uniform loads

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

Calculation of design moments in accordance with Clauses 3.5.3 of BS8110 throughout.

\[
\begin{align*}
\text{dead load} &= g_k \text{kN/m}^2 \\
\text{live load} &= q_k \text{kN/m}^2 \\
\end{align*}
\]

Length of shorter side \( lx = 10 \) m
Length of longer side \( ly = 20 \) m
Characteristic dead load \( g_k = 3 \) kN/m²
Characteristic imposed load \( q_k = 4 \) kN/m²

Overall depth of slab \( h = 500 \) mm
Characteristic concrete strength \( f_{cu} = 25 \) N/mm²
Characteristic strength of steel \( f_y = 250 \) N/mm²
Specified min. steel percentage \( \text{permin} = 0.24 \% \)
Specified min. slab thickness \( \text{mindep} = 250 \) mm
Bond type (0, 1 or 2) \( \text{Type} = 0 \)
Size of upper bars in x-direction \( \text{dia}_x = 25 \) mm
Size of lower bars in x-direction \( \text{dia}_x = 25 \) mm
Size of upper bars in y-direction \( \text{dia}_y = 20 \) mm
Size of lower bars in y-direction \( \text{dia}_y = 20 \) mm
Designated exposure class is XC1
Specified fixing tolerance \( \text{tol} = 10 \) mm

Effective depths:
Of top steel: in x-direction \( \text{dux} = 462.5 \) mm
in y-direction \( \text{duy} = \text{dux} - (\text{dia}_x + \text{dia}_y)/2 = 440 \) mm
Of bottom steel: in x-direction \( \text{dx} = 462.5 \) mm
in y-direction \( \text{dy} = \text{dx} - (\text{dia}_x + \text{dia}_y)/2 = 440 \) mm

Two adjacent edges discontinuous

Coefficients from Table 3.14
Chosen pitch for tension steel pitchC=400 mm
Chosen pitch for tension steel pitchC=260 mm
Chosen pitch for tension steel pitchC=400 mm
Chosen pitch for tension steel pitchC=260 mm

Mod. factor for tension steel:
From equation 7 (Table 3.10) modf1=0.55+(477-fs)/(120*(0.9 +M'bd2))=2.6631
but this cannot exceed 2, so modf1=2
Mod. factor for no comp. steel modf2=1
Thus maximum permissible
span/effective-depth ratio ps'd=bs'd*modf1*modf2=46
Actual span/effective-depth ratio as'd=1000*span/dx=21.622
This does not exceed permis.ratio of 46, and is thus acceptable.

**SUMMARY OF REINFORCEMENT REQUIRED** ( fy=250 N/mm²  )

<table>
<thead>
<tr>
<th>Location</th>
<th>Characteristic strength</th>
<th>Area of steel required Asx</th>
<th>Area of steel provided Aspx</th>
<th>Percentage of steel provided</th>
<th>Size of bars</th>
<th>Number of bars provided</th>
<th>Spacing of bars</th>
<th>Minimum bond length required</th>
<th>Effective depth adopted</th>
<th>Weight of steel provided</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>At edges in x-direction:</strong></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>250 N/mm²</td>
<td>18000 mm²</td>
<td>18653 mm²</td>
<td>0.24871 %</td>
<td>25 mm</td>
<td>38</td>
<td>400 mm centres</td>
<td>810 mm</td>
<td>462.5 mm</td>
<td>146.43 kg/m run</td>
</tr>
<tr>
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<tr>
<td><strong>At midspan in x-direction:</strong></td>
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</tr>
<tr>
<td></td>
<td>250 N/mm²</td>
<td>18000 mm²</td>
<td>18653 mm²</td>
<td>0.24871 %</td>
<td>25 mm</td>
<td>38</td>
<td>400 mm centres</td>
<td>610 mm</td>
<td>462.5 mm</td>
<td>146.43 kg/m run</td>
</tr>
<tr>
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<td></td>
</tr>
<tr>
<td><strong>At edges in y-direction:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>250 N/mm²</td>
<td>9000 mm²</td>
<td>9110.6 mm²</td>
<td>0.24295 %</td>
<td>20 mm</td>
<td>29</td>
<td>260 mm centres</td>
<td>445 mm</td>
<td>440 mm</td>
<td>71.518 kg/m run</td>
</tr>
</tbody>
</table>

Area calculated is the minimum requirement given in Table 3.25.
At midspan in y-direction:

- Characteristic strength: 250 N/mm²
- Area of steel required (Asym): 9000 mm²
- Area of steel provided (Asym): 9110.6 mm²
- Percentage of steel provided: 0.24295%
- Size of bars: 20 mm
- Number of bars provided: 29
- Spacing of bars: 260 mm centres
- Minimum bond length required: 445 mm
- Effective depth adopted: 440 mm
- Weight of steel provided: 71.518 kg/m run

Area calculated is the minimum requirement given in Table 3.25.

Use minimum steel as detailed above for edge strips.

See Rules 5 and 6 in Cl.3.5.3.5 for details of torsion r'ment.
Location: Thin heavily-loaded square slab, one freely supported edge

Design of two-way slabs rectangular slabs supporting uniform loads

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Calculation of design moments in accordance with Clauses 3.5.3 of BS8110 throughout.

Length of shorter side \( lx = 5 \text{ m} \)
Length of longer side \( ly = 5 \text{ m} \)
Characteristic dead load \( gk = 4 \text{ kN/m}^2 \)
Characteristic imposed load \( qk = 5 \text{ kN/m}^2 \)
Overall depth of slab \( h = 150 \text{ mm} \)
Characteristic concrete strength \( f_{cu} = 60 \text{ N/mm}^2 \)
Characteristic strength of steel \( f_y = 500 \text{ N/mm}^2 \)
Bond type (0,1 or 2) \( \text{Type}=2 \)
Size of upper bars in x-direction \( dia_x = 16 \text{ mm} \)
Size of lower bars in x-direction \( dia_x = 16 \text{ mm} \)
Size of upper bars in y-direction \( dia_y = 16 \text{ mm} \)
Size of lower bars in y-direction \( dia_y = 16 \text{ mm} \)
Designated exposure class is XC4
Specified fixing tolerance \( tol = 10 \text{ mm} \)
Specified fixing tolerance \( tol = 10 \text{ mm} \)
Nominal cover at top of slab \( t_{cover} = 30 \text{ mm} \)
Nominal cover at bottom of slab \( b_{cover} = 30 \text{ mm} \)
Effective depths:
Of top steel: in y-direction \( du_y = 112 \text{ mm} \)
in x-direction \( du_x = du_y - (dia_x + dia_y)/2 = 96 \text{ mm} \)
Of bottom steel: in y-direction \( dy = 112 \text{ mm} \)
in x-direction \( dx = dy - (dia_x + dia_y)/2 = 96 \text{ mm} \)

One long edge discontinuous

Coefficients from Table 3.14
Chosen pitch for tension steel    pitchC=350 mm
Chosen pitch for tension steel    pitchC=300 mm
Chosen pitch for tension steel    pitchC=350 mm
Chosen pitch for tension steel    pitchC=300 mm

Mod.factor for tension steel:
From equation 7 (Table 3.10)      modf1=0.55+(477-fs)/(120*(0.9
+M'bd2))=2.0199
but this cannot exceed 2, so      modf1=2
Mod.factor for no comp.steel      modf2=1
Thus maximum permissible
span/effective-depth ratio         ps'd=bs'd*modf1*modf2=46
Actual span/effective-depth ratio as'd=1000*span/dx=52.083
This exceeds permissible ratio of 46, and is thus not acceptable.

SUMMARY OF REINFORCEMENT REQUIRED ( fy=500 N/mm² )

At edges in x-direction:
Characteristic strength        500 N/mm²
Area of steel required Asxe    1254 mm²
Area of steel provided Aspxe   2613.8 mm²
Percentage of steel provided   0.46468 %
Size of bars                   16 mm
Number of bars provided        13
Spacing of bars                300 mm centres
Minimum bond length required   220 mm
Effective depth adopted        96 mm
Weight of steel provided       20.518 kg/m run

At midspan in x-direction:
Characteristic strength        500 N/mm²
Area of steel required Asxm    964.64 mm²
Area of steel provided Aspxm   2613.8 mm²
Percentage of steel provided   0.46468 %
Size of bars                   16 mm
Number of bars provided        13
Spacing of bars                300 mm centres
Minimum bond length required   195 mm
Effective depth adopted        96 mm
Weight of steel provided       20.518 kg/m run

At edges in y-direction:
Characteristic strength        500 N/mm²
Area of steel required Asye    1019.8 mm²
Area of steel provided Aspye   2211.7 mm²
Percentage of steel provided   0.39319 %
Size of bars                   16 mm
Number of bars provided        11
Spacing of bars                350 mm centres
Minimum bond length required   210 mm
Effective depth adopted        112 mm
Weight of steel provided       17.362 kg/m run

SCALE 5.48                Office 1007                Proforma 86
At midspan in y-direction:

- **Characteristic strength**: 500 N/mm²
- **Area of steel required Asym**: 771.71 mm²
- **Area of steel provided Asym**: 2211.7 mm²
- **Percentage of steel provided**: 0.39319 %
- **Size of bars**: 16 mm
- **Number of bars provided**: 11
- **Spacing of bars**: 350 mm centres
- **Minimum bond length required**: 195 mm
- **Effective depth adopted**: 112 mm
- **Weight of steel provided**: 17.362 kg/m run

Minimum tension reinforcement in edge strips

- **Min. tension area (Table 3.25)**: \( A_{\text{min}} = \frac{\text{permin}}{100} \times 100 \times h \)
  \( = 195 \text{ mm}^2 \) per metre
- **Diameter of distribution bars**: \( d_{\text{amin}} = 12 \text{ mm} \)
- **Maximum pitch (c.to c.of bars)**: \( p_{\text{chm}} = 3 \times d_{\text{x}} + d_{\text{amin}} = 300 \text{ mm}, \)
- As this is less than 750 mm, this is satisfactory.
- **Chosen pitch for tension steel**: \( \text{pitchC} = 300 \text{ mm} \)

Minimum reinforcement in edge strips:

- **Characteristic strength**: 500 N/mm²
- **Minimum area required Asmin**: 195 mm² per m
- **Steel area provided Asmpro**: 376 mm² per m
- **Percentage of steel provided**: 0.25067 %
- **Size of bars**: 12 mm
- **Spacing of bars**: 300 mm centres
- **Minimum lap length required**: 300 mm
- **Weight of steel provided**: 2.9516 kg/m²

See Rule 6 in Clause 3.5.3.5 for details of torsion reinforcement.
**Location:** Ex1 - Typical two-way slab with all edges fixed

**Design of two-way slabs rectangular slabs supporting uniform loads**

The calculations are in accordance with EC2 Part 1-1 and the IStructE "Manual for the design of concrete building structures to EC2".

Partial safety factor for steel \( g_{ms} = 1.15 \)
Partial safety factor for conc. \( g_{mc} = 1.5 \)
Partial safety factor for bond \( g_{mb} = 1.5 \)

![Diagram of a two-way slab](image)

- Length of shorter side \( lx = 6 \) m
- Length of longer side \( ly = 8 \) m
- Characteristic permanent load \( g_k = 4 \) kN/m²
- Characteristic variable load \( q_k = 5 \) kN/m²
- Permanent load factor \( g_{mG} = 1.35 \)
- Variable load factor \( g_{mQ} = 1.5 \)
- Overall depth of slab \( h = 200 \) mm
- Size of upper bars in x-direction \( d_{aux} = 20 \) mm
- Size of lower bars in x-direction \( d_{ax} = 20 \) mm
- Size of upper bars in y-direction \( d_{auy} = 16 \) mm
- Size of lower bars in y-direction \( d_{ay} = 16 \) mm
- Nominal cover at top of slab \( t_{cover} = 35 \) mm
- Nominal cover at bottom of slab \( b_{cover} = 35 \) mm
- Effective depths:
  - Of top steel: in x-direction \( d_{ux} = 155 \) mm
  - In y-direction \( d_{uy} = d_{ux} - (d_{aux} + d_{auy})/2 = 137 \) mm
  - Of bottom steel: in x-direction \( d_{dx} = 155 \) mm
  - In y-direction \( d_{dy} = d_{dx} - (d_{ax} + d_{ay})/2 = 137 \) mm

The coefficients for the edges of the slab are given as follows:

- **All four edges continuous**
  - Coefficients from the ISE Manual to EC2, Table 5.3.
Chosen spacing of tension bars  pchxe=400 mm
Chosen spacing of tension bars  pchye=400 mm
Chosen spacing of tension bars  pchxm=400 mm
Chosen spacing of tension bars  pchym=400 mm

Check on span/effective-depth ratio (in lx direction)

Actual span to depth ratio  l'd=L*1000/d=43.8
Allowable span/depth ratio  l'da=k*N*F1*F2*F3=145.6
Absolute value of span/depth  l'da=40*k=60

DESIGN
SUMMARY
DEFLECTION

NOTE: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for lightly stressed concrete (C30/37 & f=0.5%) as follows:
Simply supported spans  20
End spans  26
Interior spans  30

SUMMARY OF REINFORCEMENT REQUIRED  ( f=500 N/mm² )

At edges in x-direction:
Characteristic strength  500 N/mm²
Area of steel required Asxe  2060 mm²
Area of steel provided Aspxe  5027 mm²
Percentage of steel provided  0.4189 %
Size of bars  20 mm
Number of bars provided  16
Spacing of bars  400 mm centres
Minimum bond length required  330 mm
Effective depth adopted  155 mm

At midspan in x-direction:
Characteristic strength  500 N/mm²
Area of steel required Asxm  1552 mm²
Area of steel provided Aspxm  5027 mm²
Percentage of steel provided  0.4189 %
Size of bars  20 mm
Number of bars provided  16
Spacing of bars  400 mm centres
Minimum bond length required  249 mm
Effective depth adopted  155 mm
At edges in y-direction:

- Characteristic strength: $500 \text{ N/mm}^2$
- Area of steel required $A_{sy}$: $1182 \text{ mm}^2$
- Area of steel provided $A_{psy}$: $2413 \text{ mm}^2$
- Percentage of steel provided: $0.2681\%$
- Size of bars: $16 \text{ mm}$
- Number of bars provided: $12$
- Spacing of bars: $400 \text{ mm centres}$
- Minimum bond length required: $316 \text{ mm}$
- Effective depth adopted: $137 \text{ mm}$

At midspan in y-direction:

- Characteristic strength: $500 \text{ N/mm}^2$
- Area of steel required $A_{sm}$: $886.3 \text{ mm}^2$
- Area of steel provided $A_{spym}$: $2413 \text{ mm}^2$
- Percentage of steel provided: $0.2681\%$
- Size of bars: $16 \text{ mm}$
- Number of bars provided: $12$
- Spacing of bars: $400 \text{ mm centres}$
- Minimum bond length required: $237 \text{ mm}$
- Effective depth adopted: $137 \text{ mm}$

Minimum tension reinforcement in edge strips

- Diameter of distribution bars: $d_{\text{amin}}=12 \text{ mm}$
- Chosen spacing of tension bars: $p_{\text{chmin}}=400 \text{ mm}$

Minimum reinforcement in edge strips:

- Characteristic strength: $500 \text{ N/mm}^2$
- Minimum area required $A_{\text{min}}$: $206 \text{ mm}^2$ per m
- Steel area provided $A_{\text{smpro}}$: $282 \text{ mm}^2$ per m
- Size of bars: $12 \text{ mm}$
- Spacing of bars: $400 \text{ mm centres}$

Shear check

Case 1

- Area of steel required: $A_{s}=2060 \text{ mm}^2$
- Area of tension steel provided: $A_{\text{spr}}=5027 \text{ mm}^2$
- Effective depth of section: $d=155 \text{ mm}$
- Location for shear calculation: Shear on long side of slab
- Shear force due to ultimate load: $V_{Eд}=235 \text{ kN}$
- Effective breadth for shear: $b_w=6000 \text{ mm}$
- Term for shear resistance: $v_{\text{Rdc}}=(\rho_{\text{1}}\cdot f_{\text{ck}})^{(1/3)}=2.382$
- Shear resistance: $V_{\text{Rdc}}=C_{\text{Rdc}}\cdot k_s\cdot v_{\text{Rdc}}\cdot b_w\cdot d/1000=531.6 \text{ kN}$
- Minimum stress: $v_{\text{min}}=0.035\cdot k_s^{1.5}\cdot f_{\text{ck}}^{0.5}=0.495 \text{ N/mm}^2$
- Minimum shear resistance: $V_{\text{Rdm}}=v_{\text{min}}\cdot b_w\cdot d/1000=460.3 \text{ kN}$
SHEAR SUMMARY

- Design shear force: 235 kN
- Design shear resistance: 531.6 kN

Crack control check

Measures to control cracking are not necessary where the overall depth does not exceed 200 mm as per Clause 7.3.3(1).
Location: Ex2 - Large thick two-way slab reinforced with mild steel

Design of two-way slabs rectangular slabs supporting uniform loads

The calculations are in accordance with EC2 Part 1-1 and the IStructE "Manual for the design of concrete building structures to EC2".

Partial safety factor for steel $\gamma_m = 1.15$
Partial safety factor for conc. $\gamma_c = 1.5$
Partial safety factor for bond $\gamma_b = 1.5$

$$
\begin{align*}
g_k &\text{ kN/m}^2 \\
q_k &\text{ kN/m}^2 \\
\gamma_m &\text{ permanent load} \\
\gamma_c &\text{ variable load}
\end{align*}
$$

Length of shorter side $l_x = 10$ m
Length of longer side $l_y = 20$ m
Characteristic permanent load $g_k = 3$ kN/m$^2$
Characteristic variable load $q_k = 4$ kN/m$^2$
Permanent load factor $\gamma_m G = 1.35$
Variable load factor $\gamma_c Q = 1.5$
Overall depth of slab $h = 500$ mm
Size of upper bars in x-direction $d_{aux} = 25$ mm
Size of lower bars in x-direction $d_{ax} = 25$ mm
Size of upper bars in y-direction $d_{auy} = 20$ mm
Size of lower bars in y-direction $d_{ay} = 20$ mm
Nominal cover at top of slab $t_{cover} = 30$ mm
Nominal cover at bottom of slab $b_{cover} = 30$ mm
Effective depths:
\begin{align*}
\text{Of top steel: in x-direction } &d_{ux} = 457.5 \text{ mm} \\
&d_{uy} = d_{ux} - (d_{aux} + d_{auy}) / 2 = 435 \text{ mm} \\
\text{Of bottom steel: in x-direction } &d_{x} = 462.5 \text{ mm}
\end{align*}
Location: Ex3 - Heavily-loaded slab with one freely supported edge

Design of two-way slabs rectangular slabs supporting uniform loads

The calculations are in accordance with EC2 Part 1-1 and the IStructE "Manual for the design of concrete building structures to EC2".

Partial safety factor for steel \( g_{as} = 1.15 \)
Partial safety factor for conc. \( g_{mc} = 1.5 \)
Partial safety factor for bond \( g_{mb} = 1.5 \)

\[
g_k \text{ kN/m}^2 \quad g_k = \text{permanent load}
\]
\[
q_k \text{ kN/m}^2 \quad q_k = \text{variable load}
\]

Length of shorter side \( l_x = 5 \text{ m} \)
Length of longer side \( l_y = 5 \text{ m} \)
Characteristic permanent load \( g_k = 4 \text{ kN/m}^2 \)
Characteristic variable load \( q_k = 5 \text{ kN/m}^2 \)
Permanent load factor \( g_{mG} = 1.35 \)
Variable load factor \( g_{mQ} = 1.5 \)
Overall depth of slab \( h = 175 \text{ mm} \)
Size of upper bars in x-direction \( d_{aux} = 16 \text{ mm} \)
Size of lower bars in x-direction \( d_{ax} = 16 \text{ mm} \)
Size of upper bars in y-direction \( d_{auy} = 16 \text{ mm} \)
Size of lower bars in y-direction \( d_{ay} = 16 \text{ mm} \)
Nominal cover at top of slab \( t_{cover} = 35 \text{ mm} \)
Nominal cover at bottom of slab \( b_{cover} = 35 \text{ mm} \)
Effective depths:
  Of top steel: in y-direction \( d_{uy} = 132 \text{ mm} \)
    in x-direction \( d_{ux} = d_{uy} - (d_{aux} + d_{auy})/2 = 116 \text{ mm} \)
  Of bottom steel: in y-direction \( d_y = 132 \text{ mm} \)
    in x-direction \( d_x = d_y - (d_{ax} + d_{ay})/2 = 116 \text{ mm} \)

Coefficient from the ISE Manual to EC2, Table 5.3.
Chosen spacing of tension bars  pchye=400 mm
Chosen spacing of tension bars  pchxe=400 mm
Chosen spacing of tension bars  pchym=400 mm
Chosen spacing of tension bars  pchxm=400 mm

Check on span/effective-depth ratio (in lx direction)

Actual span to depth ratio  1'd=L*1000/d=43.1
Allowable span/depth ratio  1'da=k*N*F1*F2*F3=248.4
Absolute value of span/depth  l'da=40*k=52

DESIGN

Summary of reinforcement required ( fy=500 N/mm² )

At edges in x-direction:

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of steel required Asxe</td>
<td>984.4 mm²</td>
</tr>
<tr>
<td>Area of steel provided Aspxe</td>
<td>2011 mm²</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>0.3064 %</td>
</tr>
<tr>
<td>Size of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of bars provided</td>
<td>10</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>400 mm centres</td>
</tr>
<tr>
<td>Minimum bond length required</td>
<td>199 mm</td>
</tr>
<tr>
<td>Effective depth adopted</td>
<td>116 mm</td>
</tr>
</tbody>
</table>

At midspan in x-direction:

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of steel required Asxm</td>
<td>921 mm²</td>
</tr>
<tr>
<td>Area of steel provided Aspxm</td>
<td>2011 mm²</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>0.3064 %</td>
</tr>
<tr>
<td>Size of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of bars provided</td>
<td>10</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>400 mm centres</td>
</tr>
<tr>
<td>Minimum bond length required</td>
<td>186 mm</td>
</tr>
<tr>
<td>Effective depth adopted</td>
<td>116 mm</td>
</tr>
</tbody>
</table>
At edges in y-direction:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Area of steel required Asye</td>
<td>1048 mm²</td>
</tr>
<tr>
<td>Area of steel provided Aspye</td>
<td>2011 mm²</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>0.3064 %</td>
</tr>
<tr>
<td>Size of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of bars provided</td>
<td>10</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>400 mm centres</td>
</tr>
<tr>
<td>Minimum bond length required</td>
<td>212 mm</td>
</tr>
<tr>
<td>Effective depth adopted</td>
<td>132 mm</td>
</tr>
</tbody>
</table>

At midspan in y-direction:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Area of steel required Asym</td>
<td>1048 mm²</td>
</tr>
<tr>
<td>Area of steel provided Asym</td>
<td>2011 mm²</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>0.3064 %</td>
</tr>
<tr>
<td>Size of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of bars provided</td>
<td>10</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>400 mm centres</td>
</tr>
<tr>
<td>Minimum bond length required</td>
<td>212 mm</td>
</tr>
<tr>
<td>Effective depth adopted</td>
<td>132 mm</td>
</tr>
</tbody>
</table>

Minimum tension reinforcement in edge strips

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of distribution bars</td>
<td>diamin=12 mm</td>
</tr>
<tr>
<td>Chosen spacing of tension bars</td>
<td>pchmin=400 mm</td>
</tr>
</tbody>
</table>

Minimum reinforcement in edge strips:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Minimum area required Asmin</td>
<td>279 mm² per m</td>
</tr>
<tr>
<td>Steel area provided Aspro</td>
<td>282 mm² per m</td>
</tr>
<tr>
<td>Size of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>400 mm centres</td>
</tr>
</tbody>
</table>
Location: Ex4 - Two-way slab reinforced with accurate F3 evaluation

Design of two-way slabs rectangular slabs supporting uniform loads

The calculations are in accordance with EC2 Part 1-1 and the IStructE "Manual for the design of concrete building structures to EC2".

Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Partial safety factor for bond \( \gamma_{mb} = 1.5 \)

\[
g_k \text{ kN/m}^2 \\
q_k \text{ kN/m}^2
\]

Length of shorter side \( l_x = 10 \text{ m} \)
Length of longer side \( l_y = 20 \text{ m} \)
Characteristic permanent load \( g_k = 3 \text{ kN/m}^2 \)
Characteristic variable load \( q_k = 4 \text{ kN/m}^2 \)
Permanent load factor \( \gamma_{mg} = 1.35 \)
Variable load factor \( \gamma_{Q} = 1.5 \)
Overall depth of slab \( h = 500 \text{ mm} \)
Size of upper bars in x-direction \( d_{iax} = 25 \text{ mm} \)
Size of lower bars in x-direction \( d_{x} = 25 \text{ mm} \)
Size of upper bars in y-direction \( d_{iauy} = 20 \text{ mm} \)
Size of lower bars in y-direction \( d_{ay} = 20 \text{ mm} \)
Nominal cover at top of slab \( t_{cover} = 30 \text{ mm} \)
Nominal cover at bottom of slab \( b_{cover} = 30 \text{ mm} \)
Effective depths:
Of top steel: in x-direction \( d_{ux} = 457.5 \text{ mm} \)
\( d_{uy} = d_{ux} - (d_{iax} + d_{iauy})/2 = 435 \text{ mm} \)
Of bottom steel: in x-direction \( d_{x} = 462.5 \text{ mm} \)
**Location:** Ex5 - Typical two-way slab with all edges discontinuous

**Design of two-way slabs rectangular slabs supporting uniform loads**

The calculations are in accordance with EC2 Part 1-1 and the IStructE "Manual for the design of concrete building structures to EC2".

- Partial safety factor for steel $g_{ms} = 1.15$
- Partial safety factor for conc. $g_{mc} = 1.5$
- Partial safety factor for bond $g_{mb} = 1.5$

Length of shorter side $l_x = 1.7$ m
Length of longer side $l_y = 1.7$ m
Characteristic permanent load $g_k = 112$ kN/m²
Characteristic variable load $q_k = 0$ kN/m²
Permanent load factor $g_{mG} = 1.35$
Variable load factor $g_{mQ} = 1.5$
Overall depth of slab $h = 400$ mm
Size of upper bars in x-direction $d_{aux} = 16$ mm
Size of lower bars in x-direction $d_{ax} = 16$ mm
Size of upper bars in y-direction $d_{auy} = 16$ mm
Size of lower bars in y-direction $d_{ay} = 16$ mm
Nominal cover at top of slab $t_{cover} = 20$ mm
Nominal cover at bottom of slab $b_{cover} = 20$ mm
Effective depths:
  - Of top steel: in x-direction $d_{ux} = 372$ mm
    in y-direction $d_{uy} = d_{ux} - (d_{aux} + d_{auy})/2 = 356$ mm
  - Of bottom steel: in x-direction $d_{dx} = 322$ mm
    in y-direction $d_{dy} = d_{dx} - (d_{ax} + d_{ay})/2 = 306$ mm

All four edges discontinuous

Coefficients from the ISE Manual to EC2, Table 5.3.
Chosen spacing of tension bars  pchxm=340 mm
Chosen spacing of tension bars  pchym=360 mm

SUMMARY OF REINFORCEMENT REQUIRED ( fy=500 N/mm² )

At midspan in x-direction:

Characteristic strength  500 N/mm²
Area of steel required  Asxm  749.1 mm²
Area of steel provided  Aspxm  804.2 mm²
Percentage of steel provided  0.1577 %
Size of bars  16 mm
Number of bars provided  4
Spacing of bars  340 mm centres
Minimum bond length required  439 mm
Effective depth adopted  322 mm

At midspan in y-direction:

Characteristic strength  500 N/mm²
Area of steel required  Asym  711.9 mm²
Area of steel provided  Aspym  804.2 mm²
Percentage of steel provided  0.1577 %
Size of bars  16 mm
Number of bars provided  4
Spacing of bars  360 mm centres
Minimum bond length required  417 mm
Effective depth adopted  306 mm

Minimum tension reinforcement in edge strips

Diameter of distribution bars  diamin=16 mm
Chosen spacing of tension bars  pchmin=280 mm

Minimum reinforcement in edge strips:

Characteristic strength  500 N/mm²
Minimum area required  Asmin  678 mm² per m
Steel area provided  Aspro  718 mm² per m
Size of bars  16 mm
Spacing of bars  280 mm centres

Shear check

Case 1

Area of steel required  As=675 mm²
Area of tension steel provided  Aspr=1810 mm²
Effective depth of section  d=372 mm
Location for shear calculation:
Shear force due to ultimate load  VEd=65 kN
Effective breadth for shear  bw=1000 mm
Term for shear resistance  vRdc=(rho1*fck)^(1/3)=2.135
Shear resistance  VRdc=CRdc*ks*vRdc*bw*d/1000=165.2 kN
Minimum stress  vmin=0.035*ks^1.5*fck^0.5=0.5051 N/mm²
Minimum shear resistance  VRdm=vmin*bw*d/1000=187.9 kN
Modified shear resistance  VRdc=VRdm=187.9 kN
<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force</td>
<td>65 kN</td>
</tr>
<tr>
<td>Design shear resistance</td>
<td>187.9 kN</td>
</tr>
</tbody>
</table>
Location: Large square internal panel

Bending moments to BS8110 (AMD7583) for flat slabs without drop panels

Calculations for simplified method described in Clauses 3.7.2 of Code.

Shorter span of slab panel \( l_x = 20 \text{ m} \)
Longer span of slab panel \( l_y = 20 \text{ m} \)
Plan dim. of columns in x-dir. \( C_x = 400 \text{ mm} \)
Plan dim. of columns in y-dir. \( C_y = 400 \text{ mm} \)
Eff. slab depth for moment transfer to columns \( d = 350 \text{ mm} \)
Characteristic dead load \( g_k = 4 \text{ kN/m}^2 \)
Characteristic imposed load \( q_k = 2 \text{ kN/m}^2 \)
Characteristic concrete strength \( f_{cu} = 35 \text{ N/mm}^2 \)

Support conditions: - Interior support -

Interior support - Interior support - Interior support
### Summary of design moments (moments in kNm/metre width)

<p>| | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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<td>mtlhₓᵣ=314.77</td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

All moments are in kNm per metre width.
**Location:** Long rectangular panel along free edge of slab system

**Bending moments to BS8110 (AMD7583) for flat slabs without drop panels**

Calculations for simplified method described in Clauses 3.7.2 of Code.

![Diagram of bending moments](image)

Shorter span of slab panel \( l_x = 10 \text{ m} \)
Longer span of slab panel \( l_y = 12 \text{ m} \)
Plan dim.of columns in x-dir. \( C_x = 250 \text{ mm} \)
Plan dim.of columns in y-dir. \( C_y = 300 \text{ mm} \)
Eff.slab depth for moment transfer to columns \( d = 200 \text{ mm} \)
Characteristic dead load \( g_k = 3.2 \text{ kN/m}^2 \)
Characteristic imposed load \( q_k = 2.5 \text{ kN/m}^2 \)
Characteristic concrete strength \( f_{cu} = 30 \text{ N/mm}^2 \)

Support conditions: Free edge

Interior support

Interior support

Penultimate support
Summary of design moments (moments in kNm/metre width)

\[
\begin{align*}
\text{mtrhxl} &= 21.6 & \text{mtrhxm} &= 0 & \text{mtrhxr} &= 21.6 \\
\text{mlhyl} &= 108.32 & \text{mspbyl} &= 84.624 & \text{mtlhyr} &= 108.32 \\
\text{mlhym} &= 36.107 & \text{mspbym} &= 69.238 & \text{mtlhyr} &= 108.32 \\
\text{mlhxl} &= 124.2 & \text{mthxm} &= 29.57 & \text{mlxh} &= 124.2
\end{align*}
\]

All moments are in kNm per metre width.

mtrhxl, mtrhxm, mtrhxr, mtlhyl, mtlhym, mtlhxl, mspbyl, mspbym, mtrhyr, and mtlhyr are in kNm.

mspbxl, mspbxm, mspbxr, mlhyl, mlhym, mlhxl are in kNm per metre width.
Location: Rectangular penultimate panel

Bending moments to BS8110 (AMD7583) for flat slabs without drop panels

Calculations for simplified method described in Clauses 3.7.2 of Code.

Shorter span of slab panel $l_x = 11$ m
Longer span of slab panel $l_y = 12$ m
Plan dim. of columns in x-dir. $C_x = 200$ mm
Plan dim. of columns in y-dir. $C_y = 250$ mm
Eff. slab depth for moment transfer to columns $d = 250$ mm
Characteristic dead load $g_k = 2$ kN/m$^2$
Characteristic imposed load $q_k = 2$ kN/m$^2$
Characteristic concrete strength $f_{cu} = 30$ N/mm$^2$

Support conditions:
- Penultimate support
- Interior support

REFERENCE: SCALE Proforma 087. (ans=3)
**Summary of design moments (moments in kNm/metre width)**

<table>
<thead>
<tr>
<th>mtrhxl = 98.081</th>
<th>mtrhxm = 27.664</th>
<th>mtrhxr = 98.081</th>
<th>All moments are in kNm per metre width.</th>
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</thead>
<tbody>
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<td>mtlhyr = 107.37</td>
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<td>mtlhxm = 19.957</td>
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</tr>
</tbody>
</table>
Location: Large square internal panel

Bending moments to EC2 for flat slabs without drop panels

This simplified method assumes that the flat slab has at least three spans or bays in each direction and the ratio of the longest span to the shortest does not exceed 1.2. The max values of the bending moment and shear forces in each direction will be obtained from Table 5.4 in the IStructE Manual for the design of concrete building structures to Eurocode 2. This assumes 20% redistribution of bending moments.

Shorter span of slab panel \( l_x = 20 \text{ m} \)
Longer span of slab panel \( l_y = 20 \text{ m} \)
Plan dim. of columns in x-dir. \( C_x = 400 \text{ mm} \)
Plan dim. of columns in y-dir. \( C_y = 400 \text{ mm} \)
Moment transfer to columns \( d = 350 \text{ mm} \)
Characteristic permanent load \( g_k = 4 \text{ kN/m}^2 \)
Characteristic variable load \( q_k = 2 \text{ kN/m}^2 \)

Support conditions:

Effective diameter of column \( h_c = \sqrt{\frac{4 \times C_x \times C_y}{\pi}} = 451.35 \text{ mm} \)
Design ult load per unit area \( n = 1.35 \times g_k + 1.5 \times q_k = 8.4 \text{ kN/m}^2 \)
Total design load on panel \( F = n \times l_x \times l_y = 3360 \text{ kN} \)
Summary of design moments (moments in kNm/metre width)

All moments are in kNm per metre width.

\[
\begin{align*}
\text{mtrhxl} &= 300.46 & \text{mtrhxm} &= 100.15 & \text{mtrhxr} &= 300.46 \\
\text{mtlhyl} &= 300.46 & \text{mspbyl} &= 232.85 & \text{mtrhyl} &= 300.46 \\
\text{mtlhyl} &= 300.46 & \text{mspbyl} &= 232.85 & \text{mtrhyl} &= 300.46 \\
\text{mtlhyl} &= 300.46 & \text{mspbyl} &= 232.85 & \text{mtrhyl} &= 300.46 \\
\text{mtlhyl} &= 300.46 & \text{mspbyl} &= 232.85 & \text{mtrhyl} &= 300.46
\end{align*}
\]
Location: Long rectangular panel along free edge of slab system

Bending moments to EC2 for flat slabs without drop panels

This simplified method assumes that the flat slab has at least three spans or bays in each direction and the ratio of the longest span to the shortest does not exceed 1.2. The max values of the bending moment and shear forces in each direction will be obtained from Table 5.4 in the IStructE Manual for the design of concrete building structures to Eurocode 2. This assumes 20% redistribution of bending moments.

![Diagram of slab with dimensions and support conditions]

Shorter span of slab panel \( lx = 10 \text{ m} \)
Longer span of slab panel \( ly = 12 \text{ m} \)
Plan dim.of columns in x-dir. \( Cx = 250 \text{ mm} \)
Plan dim.of columns in y-dir. \( Cy = 300 \text{ mm} \)
Moment transfer to columns \( d = 200 \text{ mm} \)
Characteristic permanent load \( gk = 3.2 \text{ kN/m}^2 \)
Characteristic variable load \( qk = 2.5 \text{ kN/m}^2 \)

Support conditions:

- Free edge
- Interior support
- Penultimate support

Effective diameter of column \( h_c = \sqrt{4 \times Cx \times Cy / \pi} = 309.02 \text{ mm} \)
Design ult load per unit area \( n = 1.35 \times gk + 1.5 \times qk = 8.07 \text{ kN/m}^2 \)
Total design load on panel \( F = n \times lx \times ly = 968.4 \text{ kN} \)
Summary of design moments (moments in kNm/metre width)

<table>
<thead>
<tr>
<th>Moment Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>mtrhx1</td>
<td>20.4 kNm</td>
</tr>
<tr>
<td>mtrhxm</td>
<td>0 kNm</td>
</tr>
<tr>
<td>mtrhxr</td>
<td>20.4 kNm</td>
</tr>
</tbody>
</table>

All moments are in kNm per metre width.

<table>
<thead>
<tr>
<th>Moment Type</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>mtlhyl</td>
<td>103.08 kNm</td>
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<tr>
<td>mspbyl</td>
<td>80.532</td>
</tr>
<tr>
<td>mtrhyl</td>
<td>103.08 kNm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Moment Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>mtlhym</td>
<td>34.361 kNm</td>
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<tr>
<td>mspbym</td>
<td>65.89</td>
</tr>
<tr>
<td>mtrhym</td>
<td>34.361 kNm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Moment Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>mtlhx1</td>
<td>118.19 kNm</td>
</tr>
<tr>
<td>mtlhxm</td>
<td>28.141</td>
</tr>
<tr>
<td>mtlhxr</td>
<td>118.19 kNm</td>
</tr>
</tbody>
</table>

All moments are in kNm per metre width.
Location: Rectangular penultimate panel

Bending moments to EC2 for flat slabs without drop panels

This simplified method assumes that the flat slab has at least three spans or bays in each direction and the ratio of the longest span to the shortest does not exit 1.2. The max values of the bending moment and shear forces in each direction will be obtained from Table 5.4 in the IStructE Manual for the design of concrete building structures to Eurocode 2. This assumes 20% redistribution of bending moments.

Shorter span of slab panel \( l_x = 11 \text{ m} \)
Longer span of slab panel \( l_y = 12 \text{ m} \)
Plan dim. of columns in x-dir. \( C_x = 200 \text{ mm} \)
Plan dim. of columns in y-dir. \( C_y = 250 \text{ mm} \)
Moment transfer to columns \( d = 250 \text{ mm} \)
Characteristic permanent load \( g_k = 2 \text{ kN/m}^2 \)
Characteristic variable load \( q_k = 2 \text{ kN/m}^2 \)

Support conditions:

Effective diameter of column \( h_c = \sqrt{4 \times C_x \times C_y / \pi} = 252.31 \text{ mm} \)
Design ult load per unit area \( n = 1.35 \times g_k + 1.5 \times q_k = 5.7 \text{ kN/m}^2 \)
Total design load on panel \( F = n \times l_x \times l_y = 752.4 \text{ kN} \)
### Summary of design moments (moments in kNm/metre width)

<table>
<thead>
<tr>
<th>X</th>
<th>Y</th>
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<tr>
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<td>mtlhxr=67.219</td>
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</tr>
</tbody>
</table>
**Location:** High shear force on small area of moderately-thick slab

**Punching shear round concentrated load in accordance with BS8110**

Calculations are in accordance with Clauses 3.7.7 of Part 1 of BS8110. First steel perimeter located at d/2 from load face (see Cl.3.7.7.6). Design to BS8110:1997, partial safety factor for steel gams=1.15.

Average flexural steel percentage $p=1.2\%$

Characteristic concrete strength $f_{cu}=40\,N/mm^2$

Design ult. load per unit area $n=10\,kN/m^2$

Char. strength of shear reinf. $f_{yv}=500\,N/mm^2$

Initial dia. for shear reinf. $d_{ial}=12\,mm$

According to Clause 3.4.5.8 of BS8110, absolute maximum shear stress must not exceed $5\,N/mm^2$ and thus $v_{lim}=5\,N/mm^2$.

Maximum design shear stress $v_{max}=1000\cdot V_{eff}/(u_0\cdot d)=4.9342\,N/mm^2$

**Calculations for Zone 1 i.e. for Steel Perimeters 0 and 1**

Distance from load face to shear perimeter being checked $l_p=480\,mm$

Number of legs to be provided along inner steel perimeter 0 $n_i=23$

Location of Steel Perimeter 0 : 160 mm from face of loaded area.
Along Steel Perimeter 0 : provide 12 mm dia legs at 130 mm spacing.
Number of legs on perimeter 0 $n_i=\text{INT}(\text{usi}/\text{spi})+1=25$

Location of Steel Perimeter 1 : 400 mm from face of loaded area.
Along Steel Perimeter 1 : provide 12 mm dia legs at 150 mm spacing.
Number of legs on perimeter 1 $n_o=\text{INT}(\text{uso}/\text{sp})+1=35$
Calculations for Zone 2 i.e. for Steel Perimeter 2

Distance from load face to shear perimeter being checked = 720 mm

Location of Steel Perimeter 2: 640 mm from face of loaded area.
Along Steel Perimeter 2: provide 12 mm dia legs at 480 mm spacing.
Number of legs on perimeter 2 = INT(uso/sp)+1 = 15

Calculations for Zone 3 i.e. for Steel Perimeter 3

Distance from load face to shear perimeter being checked = 960 mm

Location of Steel Perimeter 3: 880 mm from face of loaded area.
Along Steel Perimeter 3: provide 12 mm dia legs at 480 mm spacing.
Number of legs on perimeter 3 = INT(uso/sp)+1 = 19

Calculations for Zone 4 i.e. for Steel Perimeter 4

Distance from load face to shear perimeter being checked = 1200 mm

As v does not exceed vc, no shear reinforcement is required on steel perimeter 4 and thus the calculations are complete.

Total number of 12 mm links required is 94

Note: All shear reinforcement must be anchored round at least one layer of tension steel (Clause 3.7.7.6)
**Location:** Moderate shear force on thin slab using mild steel

**Punching shear round concentrated load in accordance with BS8110**

Calculations are in accordance with Clauses 3.7.7 of Part 1 of BS8110. First steel perimeter located at d/2 from load face (see Cl.3.7.7.6). Design to BS8110:1997, partial safety factor for steel gains=1.15.

Average flexural steel percentage \( p = 0.4 \% \)

Characteristic concrete strength \( f_{cu} = 30 \, \text{N/mm}^2 \)

Design ult.load per unit area \( n = 10 \, \text{kN/m}^2 \)

Char. strength of shear reinf. \( f_{y} = 250 \, \text{N/mm}^2 \)

Initial dia. for shear reinf. \( d_{ial} = 6 \, \text{mm} \)

According to Clause 3.4.5.8 of BS8110, absolute maximum shear stress must not exceed 0.8 \( f_{cu} \) and thus \( v_{lim} = 0.8 \times \text{SQR}(f_{cu}) = 4.3818 \, \text{N/mm}^2 \)

Maximum design shear stress \( v_{max} = 1000 \times \text{Ve}ff/(uo \times d) = 2.0833 \, \text{N/mm}^2 \)

**Calculations for Zone 1 i.e. for Steel Perimeters 0 and 1**

Distance from load face to shear perimeter being checked \( l_p = 180 \, \text{mm} \)

Number of legs to be provided along inner steel perimeter 0 \( n_i = 23 \)

Location of Steel Perimeter 0 : 60 mm from face of loaded area.

Along Steel Perimeter 0 : provide 6 mm dia legs at 100 mm spacing.

Number of legs on perimeter 0 \( n_i = \text{INT}(us_i/sp_i) + 1 = 25 \)

Location of Steel Perimeter 1 : 150 mm from face of loaded area.

Along Steel Perimeter 1 : provide 6 mm dia legs at 100 mm spacing.

Number of legs on perimeter 1 \( n_o = \text{INT}(us_o/sp) + 1 = 33 \)
Calculations for Zone 2 i.e. for Steel Perimeter 2

Distance from load face to shear perimeter being checked \( l_p = 270 \) mm

Location of Steel Perimeter 2 : 240 mm from face of loaded area.
Along Steel Perimeter 2 : provide 6 mm dia legs at 180 mm spacing.
Number of legs on perimeter 2 \( \text{no} = \text{INT}(uso/sp) + 1 = 22 \)

Calculations for Zone 3 i.e. for Steel Perimeter 3

Distance from load face to shear perimeter being checked \( l_p = 360 \) mm

Location of Steel Perimeter 3 : 330 mm from face of loaded area.
Along Steel Perimeter 3 : provide 6 mm dia legs at 180 mm spacing.
Number of legs on perimeter 3 \( \text{no} = \text{INT}(uso/sp) + 1 = 26 \)

Calculations for Zone 4 i.e. for Steel Perimeter 4

Distance from load face to shear perimeter being checked \( l_p = 450 \) mm

Location of Steel Perimeter 4 : 420 mm from face of loaded area.
Along Steel Perimeter 4 : provide 6 mm dia legs at 180 mm spacing.
Number of legs on perimeter 4 \( \text{no} = \text{INT}(uso/sp) + 1 = 30 \)

Calculations for Zone 5 i.e. for Steel Perimeter 5

Distance from load face to shear perimeter being checked \( l_p = 540 \) mm

As \( v \) does not exceed \( v_c \), no shear reinforcement is required on steel perimeter 5 and thus the calculations are complete.

Total number of 6 mm links required is 136

Note: All shear reinforcement must be anchored round at least one layer of tension steel (Clause 3.7.7.6)
Location: High shear force on thick slab using high-yield steel

Punching shear round concentrated load in accordance with BS8110

Calculations are in accordance with Clauses 3.7.7 of Part I of BS8110. First steel perimeter located at d/2 from load face (see Cl.3.7.7.6). Design to BS8110:1997, partial safety factor for steel gams=1.15.

Note:
As the first plane that must be considered is at 1.5*d from face of concentrated load (see Figure 3.17 in Code), no enhancement of vc (Clause 3.7.7.4) is applicable.

Average flexural steel percentage p=0.15 %
Characteristic concrete strength \( f_{cu} = 30 \text{ N/mm}^2 \)
Design ult.load per unit area \( n = 14 \text{ kN/m}^2 \)
Char. strength of shear reinf. \( f_{yv} = 500 \text{ N/mm}^2 \)
Initial dia. for shear reinf. \( \text{dial} = 8 \text{ mm} \)
According to Clause 3.4.5.8 of BS8110, absolute maximum shear stress must not exceed 0.8 \( f_{cu} \) and thus \( v_{lim} = 0.8 \times \text{SQR}(f_{cu}) = 4.3818 \text{ N/mm}^2 \)
Maximum design shear stress \( v_{max} = 1000 \times \text{Ve}/(uo \times d) = 2.8571 \text{ N/mm}^2 \)

Calculations for Zone 1 i.e. for Steel Perimeters 0 and 1

Distance from load face to shear perimeter being checked \( l_p = 1500 \text{ mm} \)
Number of legs to be provided along inner steel perimeter 0 \( n_i = 104 \)
Location of Steel Perimeter 0 : 500 mm from face of loaded area.
Along Steel Perimeter 0 : provide 8 mm dia legs at 50 mm spacing.
Number of legs on perimeter 0 \( n_i = \text{INT}(usi/spi)+1=123 \)
Location of Steel Perimeter 1 : 1250 mm from face of loaded area.
Along Steel Perimeter 1 : provide 8 mm dia legs at 80 mm spacing.
Number of legs on perimeter 1 \( n_o = \text{INT}(uso/sp)+1=152 \)
Calculations for Zone 2 i.e. for Steel Perimeter 2

Distance from load face to shear perimeter being checked \( l_p = 2250 \text{ mm} \)

As \( v \) does not exceed \( v_c \), no shear reinforcement is required on steel perimeter 2 and thus the calculations are complete.

Total number of 8 mm links required is 275

Note: All shear reinforcement must be anchored round at least one layer of tension steel (Clause 3.7.7.6)
Location: Ex1 - High shear force on moderately-thick slab

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel)  \( g_{\text{ams}} = 1.15 \)
Partial safety factor (concrete)  \( g_{\text{amc}} = 1.5 \)

Definitions:
- loaded area refers to the cross-sectional area of the column
- load perimeter refers to the column perimeter
- load face refers to the column face
- face of the load refers to the face of the column
- loaded area dimensions refers to column area dimensions

Design applied shear force  \( V_{\text{Ed}} = 2300 \text{ kN} \)
Average effective depth  \( d = 320 \text{ mm} \)
Dimension of loaded area  \( c_x = 400 \text{ mm} \)
Dimension of loaded area  \( c_y = 550 \text{ mm} \)

Check shear stress at the load perimeter

Punching shear force  \( V_{\text{Ed}} = 2300 \text{ kN} \)
Shear stress at load perimeter  \( v_{\text{Ed}} = \beta V_{\text{Ed}} / (u_0 d) = 4.3503 \text{ N/mm}^2 \)
Maximum punching shear resistance  \( v_{\text{Rdmax}} = 0.5 v f_{\text{cd}} = 5.5808 \text{ N/mm}^2 \)
As \( v_{\text{Ed}} \leq v_{\text{Rdmax}} \), shear stress at load perimeter is within the maximum punching shear resistance. Hence satisfactory.

Check shear stress at control perimeter \( u_1 \) at 2d from load face

Reinforcement ratio in x-dir  \( p_{\text{lx}} = 0.012 \)
Reinforcement ratio in y-dir  \( p_{\text{ly}} = 0.012 \)
As \( v_{\text{Ed}} > v_{\text{Rdc}} \), punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required
Outer control perimeter $U_{out} = \beta \times V_{Ed} \times 1000 / (d \times v_{Rdc}) = 11416 \text{ mm}$
Radius to $U_{out}$ $r_{out} = (U_{out} - l_{cf}) / (2 \times \pi) = 1514.5 \text{ mm}$
The outer perimeter of shear reinforcement should be at a distance not greater than $1.5d$ from the outer control perimeter $U_{out}$. Radius to outer perimeter of shear reinforcement $Out_{per} = r_{out} - 1.5 \times d = 1034.5 \text{ mm from load face}$

**Shear links on basic control perimeter $u_1$**

Spac. inside 2d control perimeter $St = 450 \text{ mm}$
Spac. outside control perimeter $u_1 St' = 600 \text{ mm}$
Char. yield strength of reinft. $f_{yk} = 500 \text{ N/mm}^2$
Diameter of shear reinforcement $dial = 12 \text{ mm}$
Spacing of links to be used $spac = 270 \text{ mm}$
Use minimum H 12 (113.1 mm²) legs of links @ 270 mm c/c around perimeter $u_1$ (i.e. in the tangential direction).

**Summary of punching shear links**

In the following calculation the first perimeter will be taken at $0.5d$ from the face of the load and subsequent perimeters will be spaced at $0.75d$ apart.

Links on perimeter 1:
Use minimum H 12 (113.1 mm²) legs of links @ 130 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 12 (113.1 mm²) legs of links @ 200 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 12 (113.1 mm²) legs of links @ 270 mm c/c in the tangential direction and 240 mm c/c in the radial direction.
Links on perimeters beyond basic control perimeter u1

Links on perimeter 4:
Use minimum H 12 (113.1 mm²) legs of links @ 340 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 5:
Use minimum H 12 (113.1 mm²) legs of links @ 410 mm c/c in the tangential direction and 240 mm c/c in the radial direction.
Location: Ex2 - Moderate shear force on thin slab

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel) \( g_{ams} = 1.15 \)
Partial safety factor (concrete) \( g_{amc} = 1.5 \)

Definitions:
- loaded area refers to the cross-sectional area of the column
- load perimeter refers to the column perimeter
- load face refers to the column face
- face of the load refers to the face of the column
- loaded area dimensions refers to column area dimensions

\[
\begin{align*}
\text{First link perimeter} & \leq 0.5d \text{ from face of loaded area.} \\
\text{Link perimeters} & \leq 0.75d \text{ apart.} \\
\text{\( \leq 1.5d \quad \leq 0.5d \)}
\end{align*}
\]

Design applied shear force \( V_{Ed} = 400 \text{ kN} \)
Average effective depth \( d = 120 \text{ mm} \)
Dimension of loaded area \( cx = 400 \text{ mm} \)
Dimension of loaded area \( cy = 600 \text{ mm} \)

Check shear stress at the load perimeter

Punching shear force \( V_{Ed} = 400 \text{ kN} \)
Shear stress at load perimeter \( v_{Ed} = \beta_{Ed} V_{Ed} \frac{1000}{(u_{o}d)} = 1.9167 \text{ N/mm}^2 \)
Maximum punching shear resistance \( V_{Rd}\text{max} = 0.5v_{fcd} = 5.5808 \text{ N/mm}^2 \)
As \( v_{Ed} \leq V_{Rd}\text{max} \), shear stress at load perimeter is within the maximum punching shear resistance. Hence satisfactory.

Check shear stress at control perimeter \( u_1 \) at 2d from load face

Reinforcement ratio in x-dir \( p_{lx} = 0.004 \)
Reinforcement ratio in y-dir \( p_{ly} = 0.004 \)
As \( V_{Ed} > V_{Rdc} \) punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required
Outer control perimeter: \( U_{out} = \beta \cdot \frac{V_{Ed} \cdot 1000}{d \cdot v_{Rdc}} \) = 6833.8 mm

Radius to \( U_{out} \): \( r_{out} = \frac{(U_{out} - lcf)}{(2 \cdot \pi)} \) = 769.33 mm

The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter \( U_{out} \). Radius to outer perimeter of shear reinforcement: \( \text{Outper} = r_{out} - 1.5 \cdot d \) = 589.33 mm from load face

**Shear links on basic control perimeter \( u_1 \)**

Spac. inside 2d control perimeter: \( S_t = 180 \text{ mm} \)

Spac. outside control perimeter \( u_1 \): \( S'_t = 240 \text{ mm} \)

Char. yield strength of reinft.: \( f_{yk} = 500 \text{ N/mm}^2 \)

Diameter of shear reinforcement: \( d_{al} = 6 \text{ mm} \)

Spacing of links to be used: \( S_{pc} = 180 \text{ mm} \)

Use minimum H 6 (28.274 mm²) legs of links @ 180 mm c/c around perimeter \( u_1 \) (i.e. in the tangential direction).

**Summary of punching shear links**

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.

![Diagram of punching shear links](image)

Links on perimeter 1:
Use minimum H 6 (28.274 mm²) legs of links @ 130 mm c/c in the tangential direction and 90 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 6 (28.274 mm²) legs of links @ 160 mm c/c in the tangential direction and 90 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 6 (28.274 mm²) legs of links @ 180 mm c/c in the tangential direction and 90 mm c/c in the radial direction.
Links on perimeters beyond basic control perimeter u1

Links on perimeter 4:
Use minimum H 6 (28.274 mm²) legs of links @ 225 mm c/c in the tangential direction and 90 mm c/c in the radial direction.

Links on perimeter 5:
Use minimum H 6 (28.274 mm²) legs of links @ 240 mm c/c in the tangential direction and 90 mm c/c in the radial direction.

Links on perimeter 6:
Use minimum H 6 (28.274 mm²) legs of links @ 240 mm c/c in the tangential direction and 90 mm c/c in the radial direction.

Links on perimeter 7:
Use minimum H 6 (28.274 mm²) legs of links @ 240 mm c/c in the tangential direction and 90 mm c/c in the radial direction.
Location: Ex3 - High shear force on thick slab using high-yield steel

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel) \( \gamma_{ms} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.5 \)

Definitions:
- loaded area refers to the cross-sectional area of the column
- load perimeter refers to the column perimeter
- load face refers to the column face
- face of the load refers to the face of the column
- loaded area dimensions refers to column area dimensions

First link perimeter at \( \leq 0.5d \) from face of loaded area.
Link perimeters at \( \leq 0.75d \) apart.

Basic control perimeter is at \( 2d \) from the face of the loaded area.
Sr = spacing of links in the radial direction.

Design applied shear force \( V_{Ed} = 6000 \) kN
Average effective depth \( d = 1000 \) mm
Dimension of loaded area \( cx = 350 \) mm
Dimension of loaded area \( cy = 700 \) mm

Check shear stress at the load perimeter

Punching shear force \( V_{Ed} = 6000 \) kN
Shear stress at load perimeter \( \tau_{Ed} = \beta_{b} \times V_{Ed} \times 1000 / (\mu \times d) = 3.2857 \) N/mm²
Maximum punching shear resistance \( V_{Rdmax} = 0.5 \times \tau_{fcd} = 4.5 \) N/mm²
As \( V_{Ed} \leq V_{Rdmax} \), shear stress at load perimeter is within the maximum punching shear resistance. Hence satisfactory.

Check shear stress at control perimeter \( u_1 \) at \( 2d \) from load face

Reinforcement ratio in x-dir \( p_{1x} = 0.0015 \)
Reinforcement ratio in y-dir \( p_{1y} = 0.0015 \)
As \( V_{Ed} > V_{Rdc} \) punching shear reinforcement needs to be provided.
Outer control perimeter where shear links are no longer required

Outer control perimeter

\[ U_{\text{out}} = \beta V_{Ed} 1000 / (d \cdot v_{Rdc}) = 22647 \text{ mm} \]

Radius to \( U_{\text{out}} \)

\[ r_{\text{out}} = (U_{\text{out}} - l_{cf}) / (2 \cdot \pi) = 3270.2 \text{ mm} \]

The outer perimeter of shear reinforcement should be at a distance not greater than 1.5\( d \) from the outer control perimeter \( U_{\text{out}} \). Radius to outer perimeter of shear reinforcement \( r_{\text{outer}} = r_{\text{out}} - 1.5 \cdot d \)

= 1770.2 mm from load face

Shear links on basic control perimeter \( u_1 \)

Spacing inside 2\( d \) control perimeter  \( S_t = 1500 \text{ mm} \)

Spacing outside control perimeter \( u_1 \)  \( S'_t = 2000 \text{ mm} \)

Char yield strength of reinft.  \( f_yk = 500 \text{ N/mm}^2 \)

Diameter of shear reinforcement  \( d_{\text{ial}} = 10 \text{ mm} \)

Spacing of links to be used  \( S_{\text{pac}} = 280 \text{ mm} \)

Use minimum H 10 (78.54 mm\(^2\)) legs of links @ 280 mm c/c around perimeter \( u_1 \) (i.e. in the tangential direction).

Summary of punching shear links

In the following calculation the first perimeter will be taken at 0.5\( d \) from the face of the load and subsequent perimeters will be spaced at 0.75\( d \) apart.

\[ \begin{array}{ccc}
\leq 2d & \leq 0.5d & \leq 0.75d \\
3 & 2 & 1 \\
1 & 2 & 3 \\
\end{array} \]

radial direction

Links on perimeter 1:

Use minimum H 10 (78.54 mm\(^2\)) legs of links @ 100 mm c/c in the tangential direction and 750 mm c/c in the radial direction.

Links on perimeter 2:

Use minimum H 10 (78.54 mm\(^2\)) legs of links @ 190 mm c/c in the tangential direction and 750 mm c/c in the radial direction.

Links on perimeter 3:

Use minimum H 10 (78.54 mm\(^2\)) legs of links @ 280 mm c/c in the tangential direction and 750 mm c/c in the radial direction.
Links on perimeters beyond basic control perimeter u1
Location: Ex4 - High shear force internal load case

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel)  \( g_{ams} = 1.15 \)
Partial safety factor (concrete)  \( g_{amc} = 1.5 \)

Definitions:
- loaded area refers to the cross-sectional area of the column
- load perimeter refers to the column perimeter
- load face refers to the column face
- face of the load refers to the face of the column
- loaded area dimensions refers to column area dimensions

First link perimeter at \( \leq 0.5d \) from face of loaded area.
Link perimeters at \( \leq 0.75d \) apart.

\( cx \) and \( cy \) are the loaded area dimensions.

Basic control perimeter is at \( 2d \) from the face of the loaded area.
\( Sr = \) spacing of links in the radial direction.
\( \leq 1.5d \) \( \leq 0.5d \)

Design applied shear force \( V_{Ed} = 1204.8 \) kN
Average effective depth \( d = 250 \) mm
Dimension of loaded area \( cx = 400 \) mm
Dimension of loaded area \( cy = 400 \) mm

Check shear stress at the load perimeter

Punching shear force \( V_{Ed} = 1204.8 \) kN
Shear stress at load perimeter \( v_{Ed} = \beta_*V_{Ed}*(u_0*d)^1000/(u_0*d) = 3.4638 \) N/mm²
Maximum punching shear resistance \( v_{R,dmax} = 0.5*v_fcd = 5.28 \) N/mm²
As \( v_{Ed} \leq v_{R,dmax} \), shear stress at load perimeter is within the maximum punching shear resistance. Hence satisfactory.

Check shear stress at control perimeter \( u_1 \) at \( 2d \) from load face

Reinforcement ratio in x-dir \( p_{1x} = 0.0085 \)
Reinforcement ratio in y-dir \( p_{1y} = 0.0048 \)
As \( v_{Ed} > v_{R,dc} \) punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required
Outer control perimeter \( U_{out} = \beta \cdot V_E \cdot 1000 / (d \cdot v_{Rdc}) = 9119.2 \, \text{mm} \)

Radius to \( U_{out} \) \( r_{out} = (U_{out} - l_{cf}) / (2 \cdot \pi) = 1196.7 \, \text{mm} \)

The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter \( U_{out} \). Radius to outer perimeter of shear reinforcement \( Outper = r_{out} - 1.5 \cdot d = 821.72 \, \text{mm} \) from load face

**Shear links on basic control perimeter \( u_1 \)**

- Spac.inside 2d control perimeter \( St = 350 \, \text{mm} \)
- Spac.outside control perimeter \( u_1 \) \( St' = 500 \, \text{mm} \)
- Char yield strength of reinft. \( f_{yk} = 500 \, \text{N/mm}^2 \)
- Diameter of shear reinforcement \( d_{al} = 10 \, \text{mm} \)
- Spacing of links to be used \( spac = 275 \, \text{mm} \)

Use minimum H 10 ( 78.54 mm² ) legs of links @ 275 mm c/c around perimeter \( u_1 \) (i.e. in the tangential direction).

**Summary of punching shear links**

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.

Links on perimeter 1:
Use minimum H 10 ( 78.54 mm² ) legs of links @ 140 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 10 ( 78.54 mm² ) legs of links @ 205 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 10 ( 78.54 mm² ) legs of links @ 275 mm c/c in the tangential direction and 185 mm c/c in the radial direction.
Links on perimeters beyond basic control perimeter u1

Links on perimeter 4:
Use minimum H 10 (78.54 mm²) legs of links @ 345 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 5:
Use minimum H 10 (78.54 mm²) legs of links @ 415 mm c/c in the tangential direction and 185 mm c/c in the radial direction.
Location: Ex5 - Moderate shear force edge load

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel) $\gamma_{ams}=1.15$
Partial safety factor (concrete) $\gamma_{amc}=1.5$

Definitions:
- loaded area refers to the cross-sectional area of the column
- load perimeter refers to the column perimeter
- load face refers to the column face
- face of the load refers to the face of the column
- loaded area dimensions refers to column area dimensions

First link perimeter at $\leq 0.5d$ from face of loaded area.
Link perimeters at $\leq 0.75d$ apart.

Design applied shear force $V_{Ed}=609.58$ kN
Average effective depth $d=250$ mm
Dimension of loaded area $cx=400$ mm
Dimension of loaded area $cy=400$ mm

Check shear stress at the load perimeter

Punching shear force $V_{Ed}=609.58$ kN
Load dimension $c_1=400$ mm
Load dimension $c_2=400$ mm
Shear stress at load perimeter $v_{Ed}=\beta_{Ed}*V_{Ed}*1000/(u_0*d)=2.9684$ N/mm²
Maximum punching shear resistance $v_{Rdmax}=0.5*v*f_{cd}=5.28$ N/mm²
As $v_{Ed} \leq v_{Rdmax}$, shear stress at load perimeter is within the maximum punching shear resistance. Hence satisfactory.

Check shear stress at control perimeter $u_1$ at 2d from load face

Reinforcement ratio in x-dir $p_{ lx}=0.0089$
Reinforcement ratio in y-dir $p_{ ly}=0.0069$
As $v_{Ed} > v_{Rd}$ punching shear reinforcement needs to be provided.
Outer control perimeter where shear links are no longer required

Outer control perimeter

Outer control perimeter where shear links are no longer required

Radius to Uout

The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter Uout. Radius to outer perimeter of shear reinforcement

\[ \text{Outer} = \text{out} = 1.5 \times d \]

= 929.21 mm from load face

Shear links on basic control perimeter u1

Spac.inside 2d control perimeter

Spac.outside control perimeter u1

Char yield strength of reinft.

Diameter of shear reinforcement

Spacing of links to be used

Use minimum H 10 (78.54 mm²) legs of links @ 255 mm c/c around perimeter u1 (i.e. in the tangential direction).

Summary of punching shear links

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.

Links on perimeter 1:
Use minimum H 10 (78.54 mm²) legs of links @ 145 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 10 (78.54 mm²) legs of links @ 200 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 10 (78.54 mm²) legs of links @ 255 mm c/c in the tangential direction and 185 mm c/c in the radial direction.
Links on perimeters beyond basic control perimeter ul

Links on perimeter 4:
Use minimum H 10 (78.54 mm²) legs of links @ 315 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 5:
Use minimum H 10 (78.54 mm²) legs of links @ 370 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 6:
Use minimum H 10 (78.54 mm²) legs of links @ 425 mm c/c in the tangential direction and 185 mm c/c in the radial direction.
Location: Ex6 - Moderate shear force corner load

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel) $g_{ams}=1.15$
Partial safety factor (concrete) $g_{amc}=1.5$

Definitions:
- loaded area refers to the cross-sectional area of the column
- load perimeter refers to the column perimeter
- load face refers to the column face
- face of the load refers to the face of the column
- loaded area dimensions refers to column area dimensions

First link perimeter at $\leq 0.5d$ from face of loaded area.
Link perimeters at $\leq 0.75d$ apart.

Design applied shear force $V_{Ed}=300$ kN
Average effective depth $d=250$ mm
Dimension of loaded area $cx=400$ mm, $cy=400$ mm

Check shear stress at the load perimeter

Punching shear force $V_{Ed}=300$ kN
Load dimension $c_1=400$ mm
Load dimension $c_2=400$ mm
Shear stress at load perimeter $v_{Ed}=\beta \cdot V_{Ed} \cdot 1000/(u_0 \cdot d) = 2.4$ N/mm²
Maximum punching shear resistance $v_{Rdmax}=0.5 \cdot v \cdot f_{cd}=5.28$ N/mm²
As $v_{Ed} \leq v_{Rdmax}$, shear stress at load perimeter is within the maximum punching shear resistance. Hence satisfactory.

Check shear stress at control perimeter $u_1$ at $2d$ from load face

Reinforcement ratio in x-dir $p_{lx}=0.0089$
Reinforcement ratio in y-dir $p_{ly}=0.0069$
As $v_{Ed} > v_{Rdc}$ punching shear reinforcement needs to be provided.
Outer control perimeter where shear links are no longer required

Outer control perimeter

\[ \text{Uout} = \beta m V_E d / (d^2 r_d c) = 2766.9 \text{ mm} \]

Radius to \( \text{Uout} \)

\[ \text{rout} = (\text{Uout} - l_{cf}) / (\pi / 2) = 1284 \text{ mm} \]

The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter \( \text{Uout} \). Radius to outer perimeter of shear reinforcement \( \text{Outper} = \text{rout} - 1.5 \times d \)

\[ = 908.98 \text{ mm from load face} \]

Shear links on basic control perimeter \( \text{u1} \)

Spac.inside 2d control perimeter \( \text{St}=350 \text{ mm} \)

Spac.outside control perimeter \( \text{u1} \) \( \text{St}'=500 \text{ mm} \)

Char yield strength of reinft. \( \text{f}_{yk}=500 \text{ N/mm}^2 \)

Diameter of shear reinforcement \( \text{dial}=10 \text{ mm} \)

Spacing of links to be used \( \text{spac}=255 \text{ mm} \)

Use minimum H 10 ( 78.54 mm² ) legs of links @ 255 mm c/c around perimeter \( \text{u1} \) (i.e. in the tangential direction).

Summary of punching shear links

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.

\[
\begin{array}{|c|c|c|}
\hline
& \text{d} & \\
\hline
1 & \leq 0.75d & \leq 0.75d \\
2 & \leq 0.75d & \leq 0.75d \\
3 & \leq 2d & \text{basic control perimeter \( \text{u1} \)} \\
\hline
\end{array}
\]

\[ 1, 2, 3 \text{ indicate link perimeters} \]

Links on perimeter 1:
Use minimum H 10 (78.54 mm²) legs of links @ 175 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 10 (78.54 mm²) legs of links @ 230 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 10 (78.54 mm²) legs of links @ 255 mm c/c in the tangential direction and 185 mm c/c in the radial direction.
Links on perimeters beyond basic control perimeter u1

Links on perimeter 4:
Use minimum H 10 (78.54 mm²) legs of links @ 345 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 5:
Use minimum H 10 (78.54 mm²) legs of links @ 400 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 6:
Use minimum H 10 (78.54 mm²) legs of links @ 455 mm c/c in the tangential direction and 185 mm c/c in the radial direction.
Location: Ex7 - Internal rectangular load head

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel) \( g_{ams} = 1.15 \)
Partial safety factor (concrete) \( g_{amc} = 1.5 \)

Definitions:
- loaded area refers to the cross-sectional area of the column head
- load perimeter refers to the column head perimeter
- load head face refers to the column head face
- face of the load head refers to the face of the column head
- loaded area dimensions refers to column head area dimensions

For circular columns/piles
\( cx = cy = \text{diameter of loaded area} \)

Basic control perimeter is at \( 2d \) from the vertical face of the load head.
\( Sr = \text{spacing of links in the radial direction}. \)

Design applied shear force \( V_{Ed} = 2300 \, \text{kN} \)
Average effective depth \( d = 320 \, \text{mm} \)
Projection of head from load \( 1H_1 = 500 \, \text{mm} \)
Load head height below slab \( h_H = 300 \, \text{mm} \)
Load dimension \( c_1 = 400 \, \text{mm} \)
Projection of head from load \( 1H_2 = 500 \, \text{mm} \)
Load head height below slab \( h_H = 300 \, \text{mm} \)
Load dimension \( c_2 = 550 \, \text{mm} \)

Check shear stress at the load head perimeter

Punching shear force \( V_{Ed} = 2300 \, \text{kN} \)
Shear stress at load head \( \nu_{Ed} = \beta \times V_{Ed} \times 1000 / (u_0 \times d) = 1.401 \, \text{N/mm}^2 \)
Maximum punching shear resistance \( \nu_{Rd_{max}} = 0.5 \times \nu \times f_{cd} = 5.5808 \, \text{N/mm}^2 \)
As \( \nu_{Ed} \leq \nu_{Rd_{max}} \), shear stress at load head perimeter is within the maximum punching shear resistance. Hence satisfactory.
Check shear stress at control perimeter u1 at 2d

from load head vertical face

Reinforcement ratio in x-dir \( p1x = 0.012 \)
Reinforcement ratio in y-dir \( p1y = 0.012 \)
As \( vEd > vRdc \) punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required

Outer control perimeter \( Uout = \beta VEd \times 1000 / (d \times vRdc) = 11416 \text{ mm} \)
Radius to \( Uout \) \( r_{out} = (Uout - lcf) / (2 \times PI) = 877.93 \text{ mm} \)
The outer perimeter of shear reinforcement required should be placed at a distance not greater than 1.5d from the outer control perimeter \( Uout \).
Radius to outer perimeter of shear reinforcement from the load head vertical face is as follows:
\[ \text{Outper} = r_{out} - 1.5 \times d = 397.93 \text{ mm} \]

Shear links on basic control perimeter u1

Spac.inside 2d control perimeter \( S_t = 450 \text{ mm} \)
Spac.outside control perimeter u1 \( S'_t = 600 \text{ mm} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of shear reinforcement \( d_{ail} = 12 \text{ mm} \)
Spacing of links to be used \( s_{pac} = 450 \text{ mm} \)
Use minimum H 12 (113.1 mm²) legs of links @ 450 mm c/c around perimeter u1 (i.e. in the tangential direction).

NOTE:
Punching shear links on perimeter 3 (i.e. at 2d from the load head) shown below are optional and could be omitted. This is because the radius \( \text{Outper} < 1.25 \times d \) (397.93 mm < 400 mm).

Summary of punching shear links

\[ \begin{array}{cccc}
3 & 2 & 1 & 1, 2, 3 \text{ indicate link perimeters} \\
\end{array} \]

basic control perimeter u1.
radial direction

\[ \begin{array}{cccc}
\leq 2d & \leq 0.5d & \leq 0.75d & \text{load head} \\
\end{array} \]
Links on perimeter 1:
Use minimum H 12 (113.1 mm$^2$) legs of links @ 450 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 12 (113.1 mm$^2$) legs of links @ 450 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 12 (113.1 mm$^2$) legs of links @ 450 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

**Links on perimeters beyond basic control perimeter u1**

**Load head dimensions - rectangular**

Internal load:

![Diagram of load head dimensions (X-direction)](image)

SECTION THROUGH LOAD HEAD (X-direction)

Internal load:

![Diagram of load head dimensions (Y-direction)](image)

SECTION THROUGH LOAD HEAD (Y-direction)
**Location:** Ex8 - Internal circular load head

**Punching shear round concentrated load in accordance with EC2**

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.

Partial safety factor (steel)\( g_{ams}=1.15 \)
Partial safety factor (concrete)\( g_{amc}=1.5 \)

Definitions:
- loaded area refers to the cross-sectional area of the column head
- load perimeter refers to the column head perimeter
- load head face refers to the column head face
- face of the load head refers to the face of the column head
- loaded area dimensions refers to column head area dimensions

For circular columns/piles
\( c_x=c_y=\text{diameter of loaded area} \)

Basic control perimeter is at 2\( d \) from the vertical face of the load head.

\( S_r = \text{spacing of links in the radial direction} \)

Design applied shear force\( V_{Ed}=2300 \text{ kN} \)
Average effective depth\( d=320 \text{ mm} \)
Projection of head from load\( l_H=500 \text{ mm} \)
Load head height below slab\( h_H=300 \text{ mm} \)
Load dimension (i.e. diameter)\( c=400 \text{ mm} \)

**Check shear stress at the load head perimeter**

Punching shear force\( V_{Ed}=2300 \text{ kN} \)
Shear stress at load head\( v_{Ed}=\beta V_{Ed}*1000/(u_0*d)=1.8793 \text{ N/mm}^2 \)
Maximum punching shear resistance\( v_{Rd,max}=0.5*v*f_{cd}=5.5808 \text{ N/mm}^2 \)
As\( v_{Ed} \leq v_{Rd,max} \), shear stress at load head perimeter is within the maximum punching shear resistance. Hence satisfactory.

**Check shear stress at control perimeter \( u_1 \) at 2\( d \)**

from load head vertical face

Reinforcement ratio in x-dir\( p_{1x}=0.012 \)
Reinforcement ratio in y-dir\( p_{1y}=0.012 \)
As\( v_{Ed} > v_{Rd,c} \), punching shear reinforcement needs to be provided.
Outer control perimeter where shear links are no longer required

Outer control perimeter  

Uout = \beta \cdot VEd \cdot 1000 \div (d \cdot vRdc) = 11416 \text{ mm}

Radius to Uout  

rout = (Uout - lcf) \div (2 \cdot PI) = 1116.9 \text{ mm}

The outer perimeter of shear reinforcement required should be placed at a distance not greater than 1.5d from the outer control perimeter Uout. Radius to outer perimeter of shear reinforcement from the load head vertical face is as follows:

Outper = rout - 1.5 \cdot d = 636.94 \text{ mm}

Shear links on basic control perimeter u1

Spac. inside 2d control perimeter  
St = 450 \text{ mm}

Spac. outside control perimeter u1  
St' = 600 \text{ mm}

Char yield strength of reinft.  
fyk = 500 \text{ N/mm}^2

Diameter of shear reinforcement  
dial = 12 \text{ mm}

Spacing of links to be used  
spac = 450 \text{ mm}

Use minimum H 12 (113.1 \text{ mm}^2) legs of links @ 450 mm c/c around perimeter u1 (i.e. in the tangential direction).

Summary of punching shear links

Links on perimeter 1:
Use minimum H 12 (113.1 \text{ mm}^2) legs of links @ 340 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 12 (113.1 \text{ mm}^2) legs of links @ 435 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 12 (113.1 \text{ mm}^2) legs of links @ 450 mm c/c in the tangential direction and 240 mm c/c in the radial direction.
Links on perimeters beyond basic control perimeter u1

Load head dimensions - circular

Internal load:

SECTION THROUGH LOAD HEAD
**Location:** Ex9 - Edge rectangular load with load head

**Punching shear round concentrated load in accordance with EC2**

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.

Partial safety factor (steel) $g_{ams}=1.15$

Partial safety factor (concrete) $g_{mac}=1.5$

Definitions:
- loaded area refers to the cross-sectional area of the column head
- load perimeter refers to the column head perimeter
- load head face refers to the column head face
- face of the load head refers to the face of the column head
- loaded area dimensions refers to column head area dimensions

For circular columns/piles $cx=cy=diameter$ of loaded area

Basic control perimeter is at 2d from the vertical face of the load head.

$Sr$ = spacing of links in the radial direction.

First link perimeter at $\leq 0.5d$ from face of load head.

Link perimeters at $\leq 0.75d$ apart.

$\leq 1.5d$ $\leq 0.5d$

Design applied shear force $V_{Ed}=609.58$ kN

Average effective depth $d=250$ mm

Projection of head from load $l_{H1}=500$ mm

Load head height below slab $h_{H1}=300$ mm

Load dimension $c_{1}=400$ mm

Projection of head from load $l_{H2}=500$ mm

Load head height below slab $h_{H2}=300$ mm

Load dimension $c_{2}=400$ mm

**Check shear stress at the load head perimeter**

Punching shear force $V_{Ed}=609.58$ kN

Shear stress at load head $v_{Ed}=\beta*V_{Ed}*1000/(uo*d)=1.5877$ N/mm²

Maximum punching shear resistance $v_{Rdmax}=0.5*v_{fcd}=5.28$ N/mm²

As $v_{Ed} \leq v_{Rdmax}$, shear stress at load head perimeter is within the maximum punching shear resistance. Hence satisfactory.
Check shear stress at control perimeter u1 at 2d

from load head vertical face

Reinforcement ratio in x-dir \( p_{lx} = 0.0089 \)
Reinforcement ratio in y-dir \( p_{ly} = 0.0069 \)
As \( v_{Ed} > v_{Rdc} \) punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required

Outer control perimeter \( U_{out} = \beta V_{Ed} \times 1000 / (d \times v_{Rdc}) = 5247.3 \text{ mm} \)
Radius to \( U_{out} \) \( r_{out} = (U_{out} - lcf) / (\pi) = 985.9 \text{ mm} \)
The outer perimeter of shear reinforcement required should be placed at a distance not greater than 1.5d from the outer control perimeter \( U_{out} \).
Radius to outer perimeter of shear reinforcement from the load head vertical face is as follows:
\[ \text{Outper} = r_{out} - 1.5 \times d = 610.9 \text{ mm} \]

Shear links on basic control perimeter u1

Spac.inside 2d control perimeter \( S_t = 350 \text{ mm} \)
Spac.outside control perimeter u1 \( S_t' = 500 \text{ mm} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of shear reinforcement \( d_{al} = 8 \text{ mm} \)
Spacing of links to be used \( S_{ac} = 295 \text{ mm} \)
Use minimum H 8 (50.265 mm²) legs of links @ 295 mm c/c around perimeter u1 (i.e. in the tangential direction).

Summary of punching shear links

Links on perimeter 1:
Use minimum H 8 (50.265 mm²) legs of links @ 200 mm c/c in the tangential direction and 185 mm c/c in the radial direction.
Links on perimeter 2:
Use minimum H 8 (50.265 mm²) legs of links @ 245 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 8 (50.265 mm²) legs of links @ 295 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

**Links on perimeters beyond basic control perimeter u1**

Links on perimeter 4:
Use minimum H 8 (50.265 mm²) legs of links @ 340 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

**Load head dimensions - rectangular**

Edge load:

```
+----------------+-----------------+-----------------
|                |                 |                 |
|                |                 |                 |
|                |                 |                 |
|                |                 |                 |
+----------------+-----------------+-----------------
```

**SECTION THROUGH LOAD HEAD (X-direction)**

Edge load:

```
+----------------+----------------+-----------------+-----------------
|                |                 |                 |                 |
|                |                 |                 |                 |
|                |                 |                 |                 |
|                |                 |                 |                 |
+----------------+----------------+-----------------+-----------------
```

**SECTION THROUGH LOAD HEAD (Y-direction)**
**Location:** Ex10 - Corner rectangular load with load head

**Punching shear round concentrated load in accordance with EC2**

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel)  \( g_{ams} = 1.15 \)
Partial safety factor (concrete)  \( g_{amc} = 1.5 \)

Definitions:
- loaded area refers to the cross-sectional area of the column head
- load perimeter refers to the column head perimeter
- load head face refers to the column head face
- face of the load head refers to the face of the column head
- loaded area dimensions refers to column head area dimensions

For circular columns/piles
\[ cx = cy = \text{diameter of loaded area} \]

Basic control perimeter is at 2\( d \) from the vertical face of the load head.
\[ Sr = \text{spacing of links in the radial direction.} \]

\[ \leq 1.5d \quad \leq 0.5d \]

Design applied shear force  \( V_{Ed} = 300 \text{ kN} \)
Average effective depth  \( d = 250 \text{ mm} \)
Projection of head from load  \( l_{H1} = 500 \text{ mm} \)
Load head height below slab  \( h_{H} = 300 \text{ mm} \)
Load dimension  \( c_1 = 400 \text{ mm} \)
Projection of head from load  \( l_{H2} = 500 \text{ mm} \)
Load head height below slab  \( h_{H} = 300 \text{ mm} \)
Load dimension  \( c_2 = 400 \text{ mm} \)

**Check shear stress at the load head perimeter**

Punching shear force  \( V_{Ed} = 300 \text{ kN} \)
Shear stress at load head  \[ v_{Ed} = \frac{\beta \cdot V_{Ed} \cdot 1000}{(u_0 \cdot d)} = 2.4 \text{ N/mm}^2 \]
Maximum punching shear resistance  \( v_{Rd_{max}} = 0.5 \cdot v \cdot f_{cd} = 5.28 \text{ N/mm}^2 \)
As  \( v_{Ed} \leq v_{Rd_{max}} \), shear stress at load head perimeter is within the maximum punching shear resistance. Hence satisfactory.
Check shear stress at control perimeter \( u_1 \) at 2d

from load head vertical face

Reinforcement ratio in x-dir \( p_{1x} = 0.0089 \)
Reinforcement ratio in y-dir \( p_{1y} = 0.0069 \)
As \( v_{Ed} > v_{Rdc} \) punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required

Outer control perimeter \( U_{out} = \beta \cdot V_{Ed} \cdot 1000 / (d \cdot v_{Rdc}) = 2766.9 \text{ mm} \)
Radius to \( U_{out} \) \( r_{out} = (U_{out} - lcf) / (\pi / 2) = 1284 \text{ mm} \)
The outer perimeter of shear reinforcement required should be placed at a distance not greater than 1.5\( d \) from the outer control perimeter \( U_{out} \).
Radius to outer perimeter of shear reinforcement from the load head vertical face is as follows:
\[ \text{Outper} = r_{out} - 1.5 \cdot d = 908.98 \text{ mm} \]

Shear links on basic control perimeter \( u_1 \)

Spac.inside 2d control perimeter \( S_t = 350 \text{ mm} \)
Spac.outside control perimeter \( u_1 \) \( S'_t = 500 \text{ mm} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
 Diameter of shear reinforcement \( d_{al} = 8 \text{ mm} \)
Spacing of links to be used \( s_{pc} = 185 \text{ mm} \)
Use minimum H 8 (50.265 mm\(^2\)) legs of links @ 185 mm c/c around perimeter \( u_1 \) (i.e. in the tangential direction).

Summary of punching shear links

Links on perimeter 1:
Use minimum H 8 (50.265 mm\(^2\)) legs of links @ 110 mm c/c in the tangential direction and 185 mm c/c in the radial direction.
Links on perimeter 2:
Use minimum H 8 (50.265 mm²) legs of links @ 150 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 8 (50.265 mm²) legs of links @ 185 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

**Links on perimeters beyond basic control perimeter ul**

Links on perimeter 4:
Use minimum H 8 (50.265 mm²) legs of links @ 220 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 5:
Use minimum H 8 (50.265 mm²) legs of links @ 255 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

Links on perimeter 6:
Use minimum H 8 (50.265 mm²) legs of links @ 290 mm c/c in the tangential direction and 185 mm c/c in the radial direction.

**Load head dimensions - rectangular**

Corner load:

![SECTION THROUGH LOAD HEAD (X-direction)](image)

Corner load:

![SECTION THROUGH LOAD HEAD (Y-direction)](image)
Location: Ex11 - As Ex1 but using circular column 400mm diameter

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel) $\gamma_{ms} = 1.15$
Partial safety factor (concrete) $\gamma_{mc} = 1.5$

Definitions:
- loaded area refers to the cross-sectional area of the column
- load perimeter refers to the column perimeter
- load face refers to the column face
- face of the load refers to the face of the column
- loaded area dimensions refers to column area dimensions

First link perimeter at $\leq 0.5d$ from face of loaded area.
Link perimeters at $\leq 0.75d$ apart.

$\leq 1.5d$ $\leq 0.5d$

Design applied shear force $V_{Ed} = 1400$ kN
Average effective depth $d = 320$ mm
Dimension of loaded area $cx = 300$ mm
Dimension of loaded area $cy = 300$ mm

Check shear stress at the load perimeter

Punching shear force $V_{Ed} = 1400$ kN
Shear stress at load perimeter $v_{Ed} = \beta_{v} V_{Ed} \times 1000/(u_{o}d) = 4.1927$ N/mm²
Maximum punching shear resistance $V_{RD_{max}} = 0.5v_{fcd} = 5.5808$ N/mm²
As $v_{Ed} \leq v_{RD_{max}}$, shear stress at load perimeter is within the maximum punching shear resistance. Hence satisfactory.

Check shear stress at control perimeter $u_1$ at 2d from load face

Reinforcement ratio in x-dir $p_{lx} = 0.012$
Reinforcement ratio in y-dir $p_{ly} = 0.012$
As $v_{Ed} > v_{Rdc}$ punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required
Outer control perimeter

\[ U_{out} = \beta \cdot V_{Ed} \cdot 1000 / (d \cdot v_{Rdc}) = 6949 \text{ mm} \]

Radius to \( U_{out} \)

\[ r_{out} = (U_{out} - l_{cf}) / (2 \cdot \pi) = 914.98 \text{ mm} \]

The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter \( U_{out} \). Radius to outer perimeter of shear reinforcement \( Out_{per} = r_{out} - 1.5 \cdot d \)

= 434.98 mm from load face

**Shear links on basic control perimeter u1**

- Spac.inside 2d control perimeter \( St = 450 \text{ mm} \)
- Spac.outside control perimeter u1 \( St' = 600 \text{ mm} \)
- Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
- Diameter of shear reinforcement \( d_{al} = 12 \text{ mm} \)
- Spacing of links to be used \( s_{pc} = 450 \text{ mm} \)

Use minimum H 12 (113.1 mm²) legs of links @ 450 mm c/c around perimeter u1 (i.e. in the tangential direction).

**Summary of punching shear links**

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.

![Diagram of shear links](attachment: shear_links_diagram.png)

Links on perimeter 1:
Use minimum H 12 (113.1 mm²) legs of links @ 230 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 12 (113.1 mm²) legs of links @ 390 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 12 (113.1 mm²) legs of links @ 450 mm c/c in the tangential direction and 240 mm c/c in the radial direction.
Links on perimeters beyond basic control perimeter u1
Location: Ex12 - Assume zero load head dimensions (lH=hH=0 mm)

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel) \( g_{ams} = 1.15 \)
Partial safety factor (concrete) \( g_{mc} = 1.5 \)

Definitions:
- loaded area refers to the cross-sectional area of the column head
- load perimeter refers to the column head perimeter
- load head face refers to the column head face
- face of the load head refers to the face of the column head
- loaded area dimensions refers to column head area dimensions

For circular columns/piles
\( cx = cy = \text{diameter of loaded area} \)

Basic control perimeter is at 2d from the vertical face of the load head.
\( Sr = \text{spacing of links in the radial direction.} \)

\( \leq 1.5d \)
\( \leq 0.5d \)

Design applied shear force \( V_{Ed} = 1400 \text{ kN} \)
Average effective depth \( d = 320 \text{ mm} \)
Projection of head from load \( l_H = 0 \text{ mm} \)
Load head height below slab \( h_H = 0 \text{ mm} \)
Load dimension (i.e. diameter) \( c = 400 \text{ mm} \)

Check shear stress at the load head perimeter

Punching shear force \( V_{Ed} = 1400 \text{ kN} \)
Shear stress at load head \( v_{Ed} = \beta \frac{V_{Ed} \cdot 1000}{(u_0 \cdot d)} = 4.0037 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5 \cdot v \cdot f_{cd} = 5.5808 \text{ N/mm}^2 \)
As \( v_{Ed} \leq v_{Rdmax} \), shear stress at load head perimeter is within the maximum punching shear resistance. Hence satisfactory.

Check shear stress at control perimeter \( u_1 \) at 2d from load head vertical face

Reinforcement ratio in x-dir \( p_{lx} = 0.012 \)
Reinforcement ratio in y-dir \( p_{ly} = 0.012 \)
As \( v_{Ed} > v_{Rd} \) punching shear reinforcement needs to be provided.
Outer control perimeter where shear links are no longer required

Outer control perimeter \( \text{Uout}=\beta VEd*1000/(d*vRdc) = 6949 \text{ mm} \)
Radius to \( \text{Uout} \) \( \text{rout}=(\text{Uout}-lcf)/(2*\text{PI}) = 905.96 \text{ mm} \)
The outer perimeter of shear reinforcement required should be placed at a distance not greater than 1.5d from the outer control perimeter \( \text{Uout} \).
Radius to outer perimeter of shear reinforcement from the load head vertical face is as follows:
\( \text{Outper} = \text{rout}-1.5*d = 425.96 \text{ mm} \)

Shear links on basic control perimeter \( \text{u1} \)

Spac.inside 2d control perimeter \( \text{St}=450 \text{ mm} \)
Spac.outside control perimeter \( \text{u1} \) \( \text{St}'=600 \text{ mm} \)
Char yield strength of reinft. \( \text{fyk}=500 \text{ N/mm}^2 \)
Diameter of shear reinforcement \( \text{dial}=12 \text{ mm} \)
Spacing of links to be used \( \text{spac}=450 \text{ mm} \)
Use minimum H 12 (113.1 mm²) legs of links @ 450 mm c/c around perimeter \( \text{u1} \) (i.e. in the tangential direction).

Summary of punching shear links

Links on perimeter 1:
Use minimum H 12 (113.1 mm²) legs of links @ 240 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 12 (113.1 mm²) legs of links @ 405 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 12 (113.1 mm²) legs of links @ 450 mm c/c in the tangential direction and 240 mm c/c in the radial direction.
Links on perimeters beyond basic control perimeter ul
Location: Ex13 - As Ex1 but with opening near loaded area

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel) gams=1.15
Partial safety factor (concrete) gamc=1.5

Definitions:
- loaded area refers to the cross-sectional area of the column
- load perimeter refers to the column perimeter
- load face refers to the column face
- face of the load refers to the face of the column
- loaded area dimensions refers to column area dimensions

\[ cx \text{ and } cy \text{ are the loaded area dimensions.} \]
\[ \text{Basic control perimeter is at 2d from the face of the loaded area.} \]
\[ Sr = \text{spacing of links in the radial direction.} \]

First link perimeter at \( \leq 0.5d \) from face of loaded area.
Link perimeters at \( \leq 0.75d \) apart.

Design applied shear force \( VEd=2300 \text{ kN} \)
Average effective depth \( d=320 \text{ mm} \)
Dimension of loaded area \( cx=400 \text{ mm} \)
Dimension of loaded area \( cy=550 \text{ mm} \)

Check shear stress at the load perimeter

Punching shear force \( VEd=2300 \text{ kN} \)
Shear stress at load perimeter \( vEd=\beta*VEd*1000/(\muo*d)=4.3503 \text{ N/mm}^2 \)
Maximum punching shear resistance \( VRdmax=0.5*v*fcd=5.5808 \text{ N/mm}^2 \)
As \( VEd \leq VRdmax \), shear stress at load perimeter is within the maximum punching shear resistance. Hence satisfactory.
Ineffective perimeter due to opening

For loaded areas situated near openings the ineffective perimeter needs to be considered in the punching shear calculation (Clause 6.4.2(3)).

![Diagram of ineffective perimeter]

\[ L = 1500 \text{ mm} \]
\[ \leq 6d \]

The part of the control perimeter contained between the two tangents drawn to the outline of the opening from the centre of the loaded area considered to be ineffective is:

\[ \text{Ineffective perimeter} = \frac{2 \times d + L_3}{2} \times L_2 \times (L + L_3/2) = 197.65 \text{ mm} \]

Check shear stress at control perimeter \( u_1 \) at 2d from load face

Reinforcement ratio in x-dir \( p_{1x} = 0.012 \)
Reinforcement ratio in y-dir \( p_{1y} = 0.012 \)
As \( v_{Ed} > v_{Rdc} \) punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required

Outer control perimeter \( U_{out} = \beta \times V_{Ed} \times 1000 / (d \times v_{Rdc}) = 11416 \text{ mm} \)
Radius to \( U_{out} \) \( \text{rou}= (U_{out} - lcf) / (2 \times PI) = 1514.5 \text{ mm} \)

The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter \( U_{out} \). Radius to outer perimeter of shear reinforcement \( \text{Outper}= \text{rou} - 1.5 \times d = 1034.5 \text{ mm} \) from load face

Shear links on basic control perimeter \( u_1 \)

Spac.inside 2d control perimeter \( \text{St}=450 \text{ mm} \)
Spac.outside control perimeter \( u_1 \) \( \text{St}'=600 \text{ mm} \)
Char yield strength of reinft. \( f_{yk}=500 \text{ N/mm}^2 \)
Diameter of shear reinforcement \( \text{dial}=12 \text{ mm} \)
Spacing of links to be used \( \text{spac}=255 \text{ mm} \)
Use minimum H 12 (113.1 mm²) legs of links @ 255 mm c/c around perimeter \( u_1 \) (i.e. in the tangential direction).
Summary of punching shear links

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.

Links on perimeter 1:
Use minimum H 12 (113.1 mm²) legs of links @ 125 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 12 (113.1 mm²) legs of links @ 190 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 12 (113.1 mm²) legs of links @ 255 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeters beyond basic control perimeter u1

Links on perimeter 4:
Use minimum H 12 (113.1 mm²) legs of links @ 320 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 5:
Use minimum H 12 (113.1 mm²) legs of links @ 390 mm c/c in the tangential direction and 240 mm c/c in the radial direction.
Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel)  $g_{ams} = 1.15$
Partial safety factor (concrete)  $g_{mc} = 1.5$

Definitions:
- loaded area refers to the cross-sectional area of the column head
- load perimeter refers to the column head perimeter
- load head face refers to the column head face
- face of the load head refers to the face of the column head
- loaded area dimensions refers to column head area dimensions

For circular columns/piles
- $cx = cy = \text{diameter of loaded area}
- \text{Basic control perimeter is at } 2d \text{ from the vertical face of the load head.}
- Sr = \text{spacing of links in the radial direction.}

Design applied shear force  $V_{Ed} = 2300 \text{ kN}$
Average effective depth  $d = 320 \text{ mm}$
Projection of head from load  $1H = 500 \text{ mm}$
Load head height below slab  $h_H = 300 \text{ mm}$
Load dimension (i.e. diameter)  $c = 400 \text{ mm}$

Check shear stress at the load head perimeter

Punching shear force  $V_{Ed} = 2300 \text{ kN}$
Shear stress at load head  $v_{Ed} = \beta \cdot V_{Ed} \cdot 1000/(u_0 \cdot d) = 1.8793 \text{ N/mm}^2$
Maximum punching shear resistance  $V_{Rd,max} = 0.5 \cdot v \cdot f_{cd} = 5.5808 \text{ N/mm}^2$
As $v_{Ed} \leq V_{Rd,max}$, shear stress at load head perimeter is within the maximum punching shear resistance. Hence satisfactory.
Ineffective perimeter due to opening

For loaded areas situated near openings the ineffective perimeter needs to be considered in the punching shear calculation (Clause 6.4.2(3)).

\[
\text{Ineffective perimeter} \quad \text{ip}(3) = \frac{(2*d + L3/2) * L2}{L + L3/2} = 487.27 \text{ mm}
\]

Check shear stress at control perimeter \( u_1 \) at 2d

from load head vertical face

Reinforcement ratio in x-dir \( p_{1x} = 0.012 \)
Reinforcement ratio in y-dir \( p_{1y} = 0.012 \)
As \( v_{Ed} > v_{Rdc} \) punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required

Outer control perimeter \( U_{out} = \beta V_{Ed} * 1000 / (d * v_{Rdc}) = 11416 \text{ mm} \)
Radius to \( U_{out} \) \( r_{out} = (U_{out} - lcf) / (2 * \pi) = 1116.9 \text{ mm} \)
The outer perimeter of shear reinforcement required should be placed at a distance not greater than 1.5d from the outer control perimeter \( U_{out} \).
Radius to outer perimeter of shear reinforcement from the load head vertical face is as follows:
\( \text{Outper} = r_{out} - 1.5d = 636.94 \text{ mm} \)

Shear links on basic control perimeter \( u_1 \)

Spac.inside 2d control perimeter \( S_t = 450 \text{ mm} \)
Spac.outside control perimeter \( u_1 \) \( S_t' = 600 \text{ mm} \)
Char yield strength of reinft. \( f_yk = 500 \text{ N/mm}^2 \)
Diameter of shear reinforcement \( d_{al} = 12 \text{ mm} \)
Spacing of links to be used \( s_{pac} = 450 \text{ mm} \)
Use minimum H 12 (113.1 mm²) legs of links @ 450 mm c/c around perimeter \( u_1 \) (i.e. in the tangential direction).
Summary of punching shear links

Links on perimeter 1:
Use minimum H 12 (113.1 mm²) legs of links @ 300 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 12 (113.1 mm²) legs of links @ 380 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 12 (113.1 mm²) legs of links @ 450 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeters beyond basic control perimeter ul

Load head dimensions - circular

Internal load:

SECTION THROUGH LOAD HEAD
Location: Ex15 - As Ex13 but with edge load

Punching shear round concentrated load in accordance with EC2

Calculations are in accordance with Section 6.4 of EC2, Part 1-1.
Partial safety factor (steel)  \( g_{ams} = 1.15 \)
Partial safety factor (concrete)  \( g_{amc} = 1.5 \)

Definitions:
- loaded area refers to the cross-sectional area of the column
- load perimeter refers to the column perimeter
- load face refers to the column face
- face of the load refers to the face of the column
- loaded area dimensions refers to column area dimensions

Design applied shear force \( V_Ed = 1000 \text{ kN} \)
Average effective depth \( d = 320 \text{ mm} \)
Dimension of loaded area \( cx = 400 \text{ mm} \)
Dimension of loaded area \( cy = 550 \text{ mm} \)

Check shear stress at the load perimeter

Punching shear force \( V_Ed = 1000 \text{ kN} \)
Load dimension \( c_1 = 400 \text{ mm} \)
Load dimension \( c_2 = 550 \text{ mm} \)
Shear stress at load perimeter \( v_Ed = \beta V_Ed * 1000 / (u_o * d) = 3.2407 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_Rd_{max} = 0.5 * v * f_{cd} = 5.5808 \text{ N/mm}^2 \)
As \( v_Ed \leq v_Rd_{max} \), shear stress at load perimeter is within the maximum punching shear resistance. Hence satisfactory.
**Ineffective perimeter due to opening**

For loaded areas situated near openings the ineffective perimeter needs to be considered in the punching shear calculation (Clause 6.4.2(3)).

![Diagram of ineffective perimeter due to opening]

- L3 = 400 mm
- Loaded area dimension
- The part of the control perimeter contained between the two tangents drawn to the outline of the opening from the centre of the loaded area considered to be ineffective is:
  
  $\text{Effective perimeter} = (2d + \frac{L3}{2}) \times L2 = 197.65$ mm

**Check shear stress at control perimeter u1 at 2d from load face**

- Reinforcement ratio in x-dir $p1x = 0.012$
- Reinforcement ratio in y-dir $p1y = 0.012$
- As $vEd > vRdc$ punching shear reinforcement needs to be provided.

**Outer control perimeter where shear links are no longer required**

- Outer control perimeter $Uout = \beta \frac{VEd \times 1000}{(d \times vRdc)} = 6042.6$ mm
- Radius to $Uout = \frac{(Uout - lcf)}{(\pi)} = 1493.7$ mm
- The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter $Uout$. Radius to outer perimeter of shear reinforcement $Outer = \frac{rout - 1.5 \times d}{1013.7}$ mm from load face

**Shear links on basic control perimeter u1**

- Spac.inside 2d control perimeter $St = 450$ mm
- Spac.outside control perimeter u1 $St' = 600$ mm
- Char yield strength of reinft. $f_yk = 500$ N/mm$^2$
- Diameter of shear reinforcement $dial = 12$ mm
- Spacing of links to be used $spac = 255$ mm
- Use minimum H 12 (113.1 mm$^2$) legs of links @ 255 mm c/c around perimeter u1 (i.e. in the tangential direction).
Summary of punching shear links

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.

Links on perimeter 1:
Use minimum H 12 (113.1 mm²) legs of links @ 155 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 12 (113.1 mm²) legs of links @ 215 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 12 (113.1 mm²) legs of links @ 255 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeters beyond basic control perimeter u1

Links on perimeter 4:
Use minimum H 12 (113.1 mm²) legs of links @ 350 mm c/c in the tangential direction and 240 mm c/c in the radial direction.

Links on perimeter 5:
Use minimum H 12 (113.1 mm²) legs of links @ 410 mm c/c in the tangential direction and 240 mm c/c in the radial direction.
**Location:** Ex1 - Low moment, high axial load & shear on small section

**Design of annular short column to BS8110**

Bars are assumed located at mid-thickness of wall section. Calculations for reinforcement are in accordance with BS8110 and the IStructE/ICE 'Manual for the design of reinforced concrete building structures'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section. Partial safety factor for steel gams=1.15

<table>
<thead>
<tr>
<th>Design ultimate moment</th>
<th>M=0 kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear force</td>
<td>F=275 kN</td>
</tr>
<tr>
<td>Design ultimate axial load</td>
<td>N=2200 kN</td>
</tr>
<tr>
<td>Effective height of column</td>
<td>l=4.5 m</td>
</tr>
<tr>
<td>Diameter of cross-section</td>
<td>h=300 mm</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>h2=125 mm</td>
</tr>
<tr>
<td>Characteristic concrete strength</td>
<td>fcu=40 N/mm²</td>
</tr>
<tr>
<td>Characteristic steel strength</td>
<td>fy=500 N/mm²</td>
</tr>
<tr>
<td>Characteristic strength of links</td>
<td>fyv=500 N/mm²</td>
</tr>
<tr>
<td>Designated exposure class is XC2</td>
<td></td>
</tr>
<tr>
<td>Specified fixing tolerance</td>
<td>tol=10 mm</td>
</tr>
<tr>
<td>Main steel diameter</td>
<td>dia=25 mm</td>
</tr>
<tr>
<td>Number of bars to be provided</td>
<td>NumBar=7</td>
</tr>
<tr>
<td>Chosen diameter of links</td>
<td>Diam=8 mm</td>
</tr>
</tbody>
</table>

**Reinforcement summary**

**VERTICAL**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>7</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>3359.7 mm²</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>3436.1 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>5 %</td>
</tr>
<tr>
<td>Weight of main bars</td>
<td>26.974 kg/m</td>
</tr>
</tbody>
</table>

**LINK**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>300 mm</td>
</tr>
<tr>
<td>Weight of links (approx)</td>
<td>1.2892 kg/m</td>
</tr>
<tr>
<td>Total weight of steel</td>
<td>28.263 kg/m</td>
</tr>
</tbody>
</table>

**NOTE:** According to BS8110 Clause 3.12.7.3, circular links or spiral reinforcement passing round all bars provide adequate lateral support in circular columns.
Location: Ex2 - High moment and no axial load with mild steel

Design of annular short column to BS8110

Bars are assumed located at mid-thickness of wall section. Calculations for reinforcement are in accordance with BS8110 and the IStructE/ICE 'Manual for the design of reinforced concrete building structures'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section. Partial safety factor for steel gams=1.15

Design ultimate moment  \( M = 10000 \text{ kNm} \)
Shear force  \( F = 500 \text{ kN} \)
Design ultimate axial load  \( N = 0 \text{ kN} \)
Effective height of column  \( l = 10 \text{ m} \)
Diameter of cross-section  \( h = 2000 \text{ mm} \)
Wall thickness  \( h_2 = 325 \text{ mm} \)
Characteristic concrete strength  \( f_{cu} = 30 \text{ N/mm}^2 \)
Characteristic steel strength  \( f_y = 250 \text{ N/mm}^2 \)
Characteristic strength of links  \( f_{yv} = 500 \text{ N/mm}^2 \)
Designated exposure class is XC2
Specified fixing tolerance  \( t_{ol} = 10 \text{ mm} \)

Main steel diameter  \( d_{ia} = 40 \text{ mm} \)
Number of bars to be provided  \( n_{umBar} = 50 \)
Chosen diameter of links  \( d_{iam} = 10 \text{ mm} \)

Reinforcement summary

<table>
<thead>
<tr>
<th></th>
<th>Characteristic strength</th>
<th>Number of bars</th>
<th>Diameter of bars</th>
<th>Area of steel required</th>
<th>Area of steel provided</th>
<th>Percentage provided</th>
<th>Weight of main bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERTICAL REINFORCEMENT SUMMARY</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>250 \text{ N/mm}^2</td>
<td>50</td>
<td>40 mm</td>
<td>62382 \text{ mm}^2</td>
<td>62832 \text{ mm}^2</td>
<td>3.6739 %</td>
<td>493.23 \text{ kg/m}</td>
</tr>
<tr>
<td>LINK REINFORCEMENT SUMMARY</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>500 \text{ N/mm}^2</td>
<td></td>
<td>10 mm</td>
<td>13.819 \text{ kg/m}</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

NOTE: According to BS8110 Clause 3.12.7.3, circular links or spiral reinforcement passing round all bars provide adequate lateral support in circular columns.
Location: Ex3 - Typical jetty piles

Design of annular short column to BS8110

Bars are assumed located at mid-thickness of wall section. Calculations for reinforcement are in accordance with BS8110 and the IStructE/ICE 'Manual for the design of reinforced concrete building structures'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section. Partial safety factor for steel gams=1.15

Design ultimate moment $M=120$ kNm
Shear force $F=50$ kN
Design ultimate axial load $N=2460$ kN
Effective height of column $l=5$ m
Diameter of cross-section $h=350$ mm
Wall thickness $h_2=150$ mm
Characteristic concrete strength $f_{cu}=50$ N/mm$^2$
Characteristic steel strength $f_y=500$ N/mm$^2$
Characteristic strength of links $f_{yv}=500$ N/mm$^2$
Designated exposure class is XC4
Specified fixing tolerance $tol=10$ mm
Main steel diameter $dia=32$ mm
Number of bars to be provided $NumBar=6$
Chosen diameter of links $Diam=8$ mm

Reinforcement summary

<table>
<thead>
<tr>
<th>VERTICAL</th>
<th>Characteristic strength</th>
<th>500 N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Number of bars</td>
<td>6</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Diameter of bars</td>
<td>32 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required</td>
<td>4464.3 mm$^2$</td>
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<tr>
<td></td>
<td>Area of steel provided</td>
<td>4825.5 mm$^2$</td>
</tr>
<tr>
<td></td>
<td>Percentage provided</td>
<td>5.12 %</td>
</tr>
<tr>
<td></td>
<td>Weight of main bars</td>
<td>37.88 kg/m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LINK</th>
<th>Characteristic strength</th>
<th>500 N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>350 mm</td>
</tr>
<tr>
<td></td>
<td>Weight of links (approx)</td>
<td>1.2184 kg/m</td>
</tr>
<tr>
<td></td>
<td>Total weight of steel</td>
<td>39.098 kg/m</td>
</tr>
</tbody>
</table>

NOTE: According to BS8110 Clause 3.12.7.3, circular links or spiral reinforcement passing round all bars provide adequate lateral support in circular columns.
Location: Ex1 - Low moment, high axial load & shear on small section

Design of annular short column to EN 1992-1-1

Bars are assumed located at mid-thickness of wall section. Calculations for reinforcement are in accordance with EC2-1-1 and the IStructE/ICE 'Manual for the design of reinforced concrete building structures'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Design ultimate moment \( M_{Ed} = 0 \) kNm
Shear force \( V_{Ed} = 275 \) kN
Design ultimate axial load \( N_{Ed} = 2200 \) kN
Effective height of column \( l = 4.5 \) m
Partial safety factor for steel \( \gamma_{ams} = 1.15 \)
Partial safety factor for conc. \( \gamma_{amc} = 1.5 \)
Diameter of cross-section \( h = 300 \) mm
Wall thickness \( h_2 = 125 \) mm
Char yield strength of r'ment \( f_yk = 500 \) N/mm²
Char yield strength of links \( f_yk' = 500 \) N/mm²
Diameter of longitudinal r'ment \( dia = 25 \) mm
Diameter of column links \( dial = 8 \) mm

Slenderness - EC2 Clause 5.8.3.2

About Y-Y axis:
Column is considered to be braced about the stronger axis.
Effective height \( l_{oy} = \beta_{ty} \times l \times 1000 = 4500 \) mm

About Z-Z axis:
Column is considered to be braced about the weaker axis.
Effective height \( l_{oz} = \beta_{tz} \times l \times 1000 = 4500 \) mm

Check slenderness

For Y-Y stronger axis:
Slenderness \( l_{amy} = l_{oy} / i_{y} = 59.184 \)
Smaller initial end moment \( M_{1} = 0 \) kNm
Limiting slenderness \( l_{amy} = 20 \times A \times B \times C / n^{0.5} = 32.565 \)
Column is classified as being slender in accordance with EC2 Part 1-1.

For Z-Z weaker axis:
Slenderness \( l_{amz} = l_{oz} / i_{z} = 59.184 \)
Limiting slenderness \( l_{amz} = 20 \times A \times B \times C / n^{0.5} = 32.565 \)
Column is classified as being slender in accordance with EC2 Part 1-1.
Design moments - Clause 5.8.8.2

Imperfections eccentricity \( e_1 = \frac{\text{loy}}{400} = 11.25 \text{ mm} \)
Load \( N_{Ed} \) eccentricity \( e_0 = \frac{h}{30} = 10 \text{ mm} \)
Design moment (\( y-y \) axis) \( M_{Ed} = Mo^2 \times 10^{-6} = 44 \text{ kNm} \)
Main steel diameter \( \text{dia} = 25 \text{ mm} \)
Number of bars to be provided \( \text{NumBar} = 6 \)

Shear in column - EN 1992-1-1 Clause 6.2

Shear resistance:
\[
VR_{dc} = (CR_{dc} \times k_s \times v_{Rdc} + 0.15 \times N_{Ed} \times 1000/A_c) \times bw \times d / 1000 = 343.99 \text{ kN}
\]

SHEAR SUMMARY

<table>
<thead>
<tr>
<th></th>
<th>Shear force</th>
<th>Shear resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>275 kN</td>
<td>343.99 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.
No calculated reinforcement is required to resist shear. Provide nominal links passing round all column vertical bars.

Transverse reinforcement (links)

Maximum spacing is the minimum of the following distances:
Based on main bar diameter \( s_{mx1} = 20 \times \text{dia} = 500 \text{ mm} \)
Minimum column dimension \( s_{mx2} = h = 300 \text{ mm} \)
Maximum dimension \( s_{mx3} = 400 \text{ mm} \)
Maximum spacing of links \( s_v = 300 \text{ mm} \)
This spacing should be reduced by a factor of 0.6 within a distance equal to the larger dimension of the column cross-section above or below a beam or slab.

Reinforcement summary

<table>
<thead>
<tr>
<th>VERTICAL REINFORCEMENT SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
<td></td>
</tr>
<tr>
<td>Area of steel required</td>
<td>2919.6 mm²</td>
<td></td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>2945.2 mm²</td>
<td></td>
</tr>
<tr>
<td>Percentage provided</td>
<td>4.2857 %</td>
<td></td>
</tr>
<tr>
<td>Weight of main bars</td>
<td>23.12 kg/m</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LINK REINFORCEMENT SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
<td></td>
</tr>
<tr>
<td>Spacing</td>
<td>300 mm</td>
<td></td>
</tr>
<tr>
<td>Weight of links (approx)</td>
<td>1.2892 kg/m</td>
<td></td>
</tr>
<tr>
<td>Total weight of steel</td>
<td>24.409 kg/m</td>
<td></td>
</tr>
</tbody>
</table>

NOTE 1: Circular links or spiral reinforcement passing round all bars provide adequate lateral support in circular columns.

NOTE 2: The percentage of steel provided is 4.2857 % which is greater than the 4% maximum limit. However this is acceptable provided that the concrete can be placed and compacted sufficiently.
Location: Ex2 - High moment and no axial load with mild steel

Design of annular short column to EN 1992-1-1

Bars are assumed located at mid-thickness of wall section. Calculations for reinforcement are in accordance with EC2-1-1 and the IStructE/ICE 'Manual for the design of reinforced concrete building structures'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Design ultimate moment \( M_{Ed} = 10000 \text{ kNm} \)
Shear force \( V_{Ed} = 500 \text{ kN} \)
Design ultimate axial load \( N_{Ed} = 0 \text{ kN} \)
Effective height of column \( l = 10 \text{ m} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Diameter of cross-section \( h = 2000 \text{ mm} \)
Wall thickness \( h_2 = 325 \text{ mm} \)
Char yield strength of r'ment \( f_{yk} = 500 \text{ N/mm}^2 \)
Char yield strength of links \( f_{yk}' = 500 \text{ N/mm}^2 \)
Diameter of longitudinal r'ment \( \text{dia} = 40 \text{ mm} \)
Diameter of column links \( \text{dial} = 10 \text{ mm} \)

Slenderness - EC2 Clause 5.8.3.2

About Y-Y axis:
Column is considered to be braced about the stronger axis.
Effective height \( l_{oy} = \text{bety} \cdot l \cdot 1000 = 7500 \text{ mm} \)

About Z-Z axis:
Column is considered to be braced about the weaker axis.
Effective height \( l_{oz} = \text{betz} \cdot l \cdot 1000 = 7500 \text{ mm} \)

Check slenderness

For Y-Y stronger axis:
Slenderness \( \lambda_{my} = l_{oy} / i_{y} = 12.433 \)
Smaller initial end moment \( M_1 = 0 \text{ kNm} \)
Limiting slenderness \( \lambda_{my} = 20 \cdot A \cdot B \cdot C / n^{0.5} = 13205 \)
Column is classified as being short in accordance with EC2 Part 1-1.

For Z-Z weaker axis:
Slenderness \( \lambda_{lz} = l_{oz} / i_{z} = 12.433 \)
Limiting slenderness \( \lambda_{lz} = 20 \cdot A \cdot B \cdot C / n^{0.5} = 13205 \)
Column is classified as being short in accordance with EC2 Part 1-1.
WARNING: Check on validity of the input values

Ratio of reinforcement term       nu=1.1353
Relative axial force              n=4.1275E-6
Value of n at maximum moment     nbal=0.4
Correction factor for axial load  Kr=(nu-n)/(nu-nbal)
                                  =1.544

As this value is high, design the section as a beam.
Location: Ex3 - Typical jetty piles

Design of annular short column to EN 1992-1-1

Bars are assumed located at mid-thickness of wall section. Calculations for reinforcement are in accordance with EC2-1-1 and the IStructE/ICE 'Manual for the design of reinforced concrete building structures'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Design ultimate moment MEd=120 kNm
Shear force VEd=50 kN
Design ultimate axial load NEd=2460 kN
Effective height of column l=5 m
Partial safety factor for steel gams=1.15
Partial safety factor for conc. gamc=1.5
Diameter of cross-section h=350 mm
Wall thickness h2=150 mm
Char yield strength of r'ment fyk=500 N/mm²
Char yield strength of links fyk'=500 N/mm²
Diameter of longitudinal r'ment dia=32 mm
Diameter of column links dial=8 mm

Slenderness - EC2 Clause 5.8.3.2

About Y-Y axis:
Column is considered to be braced about the stronger axis.
Effective height loy=bety*l*1000=3750 mm

About Z-Z axis:
Column is considered to be braced about the weaker axis.
Effective height loz=betz*l*1000=3750 mm

Check slenderness

For Y-Y stronger axis:
Slenderness lamy=loy/iy=42.426
Smaller initial end moment M1=0 kNm
Limiting slenderness lamly=20*A*B*C/n^0.5=38.185
Column is classified as being slender in accordance with EC2 Part 1-1.

For Z-Z weaker axis:
Slenderness lamz=loz/iz=42.426
Limiting slenderness lamlz=20*A*B*C/n^0.5=38.185
Column is classified as being slender in accordance with EC2 Part 1-1.
Design moments - Clause 5.8.8.2

Imperfections eccentricity \( e_1 = \frac{lo_y}{400} = 9.375 \text{ mm} \)
Load \( N_{Ed} \) eccentricity \( e_0 = \frac{h}{30} = 11.667 \text{ mm} \)
Design moment (\( y-y \) axis) \( M_{Ed} = Mo^2 \times 10^{-6} = 143.06 \text{ kNm} \)
Main steel diameter \( \text{dia} = 32 \text{ mm} \)
Number of bars to be provided \( \text{NumBar} = 7 \)

Shear in column - EN 1992-1-1 Clause 6.2

Shear resistance:
\[
VR_{dc} = (CR_{dc} \times ks \times v_{Rdc} + 0.15 \times N_{Ed} \times 1000 / Ac) \times bw \times d / 1000 = 402.04 \text{ kN}
\]

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>50 \text{ kN}</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear resistance</td>
<td>402.04 \text{ kN}</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.
No calculated reinf'ment is required to resist shear. Provide nominal links passing round all column vertical bars.

Transverse reinforcement (links)

Maximum spacing is the minimum of the following distances:
Based on main bar diameter \( smx_1 = 20 \times \text{dia} = 640 \text{ mm} \)
Minimum column dimension \( smx_2 = h = 350 \text{ mm} \)
Maximum dimension \( smx_3 = 400 \text{ mm} \)
Maximum spacing of links \( sv = 350 \text{ mm} \)
This spacing should be reduced by a factor of 0.6 within a distance equal to the larger dimension of the column cross-section above or below a beam or slab.

Reinforcement summary

| VERTICAL | Characteristic strength | 500 \text{ N/mm}^2 |
| REINFORCEMENT SUMMARY | Number of bars | 7 |
| Diameter of bars | 32 \text{ mm} |
| Area of steel required | 5452.9 \text{ mm}^2 |
| Area of steel provided | 5629.7 \text{ mm}^2 |
| Percentage provided | 5.9733 % |
| Weight of main bars | 44.193 kg/m |

| LINK | Characteristic strength | 500 \text{ N/mm}^2 |
| REINFORCEMENT SUMMARY | Diameter of links | 8 \text{ mm} |
| Spacing | 350 \text{ mm} |
| Weight of links (approx) | 1.2184 kg/m |
| Total weight of steel | 45.412 kg/m |

NOTE 1: Circular links or spiral reinforcement passing round all bars provide adequate lateral support in circular columns.

NOTE 2: The percentage of steel provided is 5.9733 % which is greater than the 4% maximum limit. However this is acceptable provided that the concrete can be placed and compacted sufficiently.
**Location:** Ex4 - As Ex1 but with user defined cover

**Design of annular short column to EN 1992-1-1**

Bars are assumed located at mid-thickness of wall section. Calculations for reinforcement are in accordance with EC2-1-1 and the IStructE/ICE 'Manual for the design of reinforced concrete building structures'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Design ultimate moment \( M_{Ed} = 0 \) kNm
Shear force \( V_{Ed} = 275 \) kN
Design ultimate axial load \( N_{Ed} = 2200 \) kN
Effective height of column \( l = 4.5 \) m
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Diameter of cross-section \( h = 300 \) mm
Wall thickness \( h_{2} = 125 \) mm
Char yield strength of r’ment \( f_{yk} = 500 \) N/mm²
Char yield strength of links \( f_{yk}' = 500 \) N/mm²
Diameter of longitudinal r’ment \( d_{ia} = 25 \) mm
Diameter of column links \( d_{ial} = 8 \) mm
Nominal cover to all steel \( c = 25 \) mm

**Slenderness - EC2 Clause 5.8.3.2**

About Y-Y axis:
Column is considered to be braced about the stronger axis.
Effective height \( \ell_{oy} = b_{ey} \times l \times 1000 = 4500 \) mm

About Z-Z axis:
Column is considered to be braced about the weaker axis.
Effective height \( \ell_{oz} = b_{etz} \times l \times 1000 = 4500 \) mm

**Check slenderness**

For Y-Y stronger axis:
Slenderness \( \lambda_{my} = \ell_{oy} / i_{y} = 59.184 \)
Smaller initial end moment \( M_{1} = 0 \) kNm
Limiting slenderness \( \lambda_{my} = 20 \times A \times B \times C / n^{0.5} = 31.309 \)
Column is classified as being slender in accordance with EC2 Part 1-1.

For Z-Z weaker axis:
Slenderness \( \lambda_{mz} = \ell_{oz} / i_{z} = 59.184 \)
Limiting slenderness \( \lambda_{mz} = 20 \times A \times B \times C / n^{0.5} = 31.309 \)
Column is classified as being slender in accordance with EC2 Part 1-1.
Design moments - Clause 5.8.8.2

- Imperfections eccentricity: $e_1 = \frac{loy}{400} = 11.25$ mm
- Load NEd eccentricity: $e_0 = \frac{h}{30} = 10$ mm
- Design moment (y-y axis): $M_{Ed} = M_0^2 \times 10^{-6} = 44$ kNm
- Main steel diameter: $dia = 20$ mm
- Number of bars to be provided: $NumBar = 12$

Shear in column - EN 1992-1-1 Clause 6.2

- Shear resistance:
  
  $VR_{dc} = (CR_{dc} \times ks \times v_{Rdc} + 0.15 \times N_{Ed} \times 1000 \times Ac) \times bw \times d / 1000 = 339.77$ kN

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>275 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>339.77 kN</td>
<td></td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.
No calculated reinforcement is required to resist shear. Provide nominal links passing round all column vertical bars.

Transverse reinforcement (links)

- Maximum spacing is the minimum of the following distances:
  - Based on main bar diameter: $smx_1 = 20 \times dia = 400$ mm
  - Minimum column dimension: $smx_2 = h = 300$ mm
  - Maximum dimension: $smx_3 = 400$ mm
  - Maximum spacing of links: $sv = 300$ mm

  This spacing should be reduced by a factor of 0.6 within a distance equal to the larger dimension of the column cross-section above or below a beam or slab.

Reinforcement summary

<table>
<thead>
<tr>
<th>VERTICAL REINFORCEMENT SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
<td></td>
</tr>
<tr>
<td>Area of steel required</td>
<td>3756.3 mm²</td>
<td></td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>3769.9 mm²</td>
<td></td>
</tr>
<tr>
<td>Percentage provided</td>
<td>5.4857 %</td>
<td></td>
</tr>
<tr>
<td>Weight of main bars</td>
<td>29.594 kg/m</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LINK REINFORCEMENT SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
<td></td>
</tr>
<tr>
<td>Spacing</td>
<td>300 mm</td>
<td></td>
</tr>
<tr>
<td>Weight of links (approx)</td>
<td>1.2685 kg/m</td>
<td></td>
</tr>
<tr>
<td>Total weight of steel</td>
<td>30.862 kg/m</td>
<td></td>
</tr>
</tbody>
</table>

NOTE 1: Circular links or spiral reinforcement passing round all bars provide adequate lateral support in circular columns.

NOTE 2: The percentage of steel provided is 5.4857 % which is greater than the 4% maximum limit. However this is acceptable provided that the concrete can be placed and compacted sufficiently.
Location: Column A3 (1st-2nd floor)

Short braced rectangular column subjected to biaxial bending and thrust

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$

The calculations for uniaxial or biaxial bending and axial force are in accordance with Clause 3.8.4.5 of BS8110, and are undertaken using data similar to that used to prepare the design charts in the IStructE/ICE 'Manual for the design of reinforced concrete building structures'.

- Depth of cross-section: $h = 600$ mm
- Width of cross-section: $b = 500$ mm
- Size of main bars: $\text{dia} = 25$ mm
- Chosen size of links: $\text{dial} = 10$ mm
- Characteristic concrete strength: $f_{\text{cu}} = 30$ N/mm$^2$
- Characteristic steel strength: $f_y = 500$ N/mm$^2$
- Designated exposure class is XC2
- Specified fixing tolerance: $\text{tol} = 10$ mm
- Nominal concrete cover: $\text{cover} = 35$ mm
- Effective depth about X-X axis: $h' = 542.5$ mm
- Effective depth about Y-Y axis: $b' = 442.5$ mm
- Design ultimate axial load: $N = 1600$ kN
- Ultimate moment about X-X axis: $M_x = 300$ kNm
- Ultimate moment about Y-Y axis: $M_y = 200$ kNm
- Effective height of column: $h_e = 3$ m
- Normal-weight concrete. Critical slenderness ratio for short braced column is 15 (see Clause 3.8.1.3 of BS8110: Part 1).

Maximum allowable effective height for braced short column: $h_{\text{el}} = b' \times \text{factor}/1000 = 7.5$ m

Consider bending about X-X axis, as $M_x/h'$ is not less than $M_y/b'$

Minimum permissible eccentricity: $\text{eccmn} = 20$ mm
As this is less than $0.05 \times h'$ (see Clause 3.8.2.4), adopt this value.
Eccentricity of applied load: $\text{eccx} = 1000 \times M_x / N = 187.5$ mm
As this exceeds the min.permissible value, design for this eccentricity.
### Design of main bars

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual steel percentage required</td>
<td>0.66514 %</td>
</tr>
<tr>
<td>Required area of reinforcement</td>
<td>1995.4 mm²</td>
</tr>
<tr>
<td>Number of 25 mm bars required</td>
<td>6</td>
</tr>
<tr>
<td>Number of bars provided</td>
<td>6</td>
</tr>
<tr>
<td>MAIN Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>REINFORCEMENT Area of steel required</td>
<td>1995.4 mm²</td>
</tr>
<tr>
<td>SUMMARY Area of steel provided</td>
<td>2945.2 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.98175 %</td>
</tr>
<tr>
<td>Weight of steel provided</td>
<td>23.12 kg/m</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>6</td>
</tr>
<tr>
<td>To be placed along b-faces (spaced @ 192 mm, assume 1 layer)</td>
<td></td>
</tr>
<tr>
<td>LINK Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>REINFORCEMENT Diameter of links</td>
<td>10 mm</td>
</tr>
<tr>
<td>SUMMARY Max. longitudinal spacing</td>
<td>300 mm</td>
</tr>
<tr>
<td>Approx. weight of links</td>
<td>8.3844 kg/m</td>
</tr>
</tbody>
</table>

Clause 3.12.7.2 states that links must pass round every corner bar and each alternate bar, and also that no bar may be more than 150 mm from a bar that is restrained by a link. Thus a set of at least 2 links must be provided at not more than the spacing specified above.

NOTE: As the distance between the bars exceeds 400 mm, a check on cracking may be necessary, if the face is in tension under service loading otherwise use smaller bars, more closely spaced.

Approx. total wt. of steel 31.505 kg/m
Location: Small square column. Max. high-yield steel. High mom & load

Short braced rectangular column subjected to biaxial bending and thrust

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

The calculations for uniaxial or biaxial bending and axial force are in accordance with Clause 3.8.4.5 of BS8110, and are undertaken using data similar to that used to prepare the design charts in the IStructE/ICE 'Manual for the design of reinforced concrete building structures'.

Depth of cross-section $h=200$ mm
Width of cross section $b=200$ mm
Size of main bars $\text{dia}=16$ mm
Chosen size of links $\text{dial}=8$ mm
Characteristic concrete strength $f_{\text{cu}}=60$ N/mm$^2$
Characteristic steel strength $f_y=500$ N/mm$^2$
Designated exposure class is XC1
Specified fixing tolerance $\text{tol}=10$ mm
Nominal concrete cover $\text{cover}=25$ mm
Effective depth about X-X axis $h'=159$ mm
Effective depth about Y-Y axis $b'=159$ mm
Design ultimate axial load $N=400$ kN
Ultimate moment about X-X axis $M_x=41$ kNm
Ultimate moment about Y-Y axis $M_y=41$ kNm
Effective height of column $h_e=3$ m
Normal-weight concrete. Critical slenderness ratio for short braced column is 15 (see Clause 3.8.1.3 of BS8110: Part 1).

Maximum allowable effective height for braced short column $h_{\text{all}}=b'\times \text{factor}/1000=3$ m

Consider bending about X-X axis, as $M_x/h'$ is not less than $M_y/b'$

Minimum permissible eccentricity $\text{eccmn}_{X}=h\times 0.05=10$ mm
As this is less than 20 mm (see Clause 3.8.2.4), adopt this value.
Eccentricity of applied load $\text{eccx}=1000\times M_x/N=102.5$ mm
As this exceeds the min. permissible value, design for this eccentricity.
Design of main bars

Actual steel percentage required  \( p = 100 \times \alpha \frac{f_{cu}}{f_y} = 5.0866\% \)
Required area of reinforcement  \( A_{sc} = p \times b \times h / 100 = 2034.7 \text{ mm}^2 \)
Number of 16 mm bars required  \( n_{min} = 2 \times \text{INT}(A_{sc} / \text{area}) + 2 = 12 \)
Number of bars provided  \( n_{bars} = 12 \)

WARNING:
This exceeds the limit of 6% specified in Cl.3.12.6.2 of BS8110.
Please rerun with modified input.
Location: Large column. Minimum mild steel. Moment and low load.

Short braced rectangular column subjected to biaxial bending and thrust

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

The calculations for uniaxial or biaxial bending and axial force are in accordance with Clause 3.8.4.5 of BS8110, and are undertaken using data similar to that used to prepare the design charts in the IStructE/ICE 'Manual for the design of reinforced concrete building structures'.

Depth of cross-section \( h = 2000 \text{ mm} \)
Width of cross section \( b = 1500 \text{ mm} \)
Size of main bars \( \text{dia}=40 \text{ mm} \)
Chosen size of links \( \text{dial}=10 \text{ mm} \)
Characteristic concrete strength \( f_{cu} = 40 \text{ N/mm}^2 \)
Characteristic steel strength \( f_y = 250 \text{ N/mm}^2 \)
Nominal concrete cover \( \text{cover}=60 \text{ mm} \)
Effective depth about X-X axis \( h' = 1910 \text{ mm} \)
Effective depth about Y-Y axis \( b' = 1410 \text{ mm} \)
Design ultimate axial load \( N = 50 \text{ kN} \)
Ultimate moment about X-X axis \( M_x = 800 \text{ kNm} \)
Ultimate moment about Y-Y axis \( M_y = 1600 \text{ kNm} \)
Effective height of column \( h_e = 10 \text{ m} \)
Lightweight concrete. Critical slenderness ratio for short braced column is 10 (see Clause 5.7.2 of BS8110: Part 2).

Maximum allowable effective height for braced short column \( h_{eall} = b \times \text{factor}/1000 = 15 \text{ m} \)

Consider bending about Y-Y axis, as \( M_x/h' \) is less than \( M_y/b' \)

Minimum permissible eccentricity \( e_{ccmny} = 20 \text{ mm} \)
As this is less than \( 0.05 \times b \text{ mm} \) (see Clause 3.8.2.4), adopt this value.
Eccentricity of applied load \( e_{ccy} = 1000 \times M_y/N = 32000 \text{ mm} \)
As this exceeds the min. permissible value, design for this eccentricity.

As \( N_{ratio} \) does not exceed 0.1, the IStructE/ICE design charts do not strictly apply. Clause 3.4.4.1 states that you may, if desired, ignore the axial force and design the section as a beam resisting bending only.
Design of main bars

Actual steel percentage required \( p = 100 \times \alpha \times \frac{f_{cu}}{f_y} = 0.51204\% \)
Required area of reinforcement \( A_{sc} = \frac{p \times b \times h}{100} = 15361\text{ mm}^2 \)
Number of 40 mm bars required \( n_{min} = 2 \times \text{INT}(A_{sc}/\text{area}) + 2 = 14 \)
Number of bars provided \( n_{bars} = 14 \)

**MAIN**

- Characteristic strength: 250 N/mm²

**REINFORCEMENT**

- Area of steel required: 15361 mm²
- Percentage provided: 0.58643 %
- Weight of steel provided: 138.1 kg/m
- Diameter of bars: 40 mm
- Number of bars: 14

**SUMMARY**

- Area of steel provided: 17593 mm²
- Percentage provided: 0.58643 %
- Weight of steel provided: 138.1 kg/m
- Number of bars: 14

To be placed along h-faces (spaced @ 303 mm, assume 1 layer)

**LINK**

- Characteristic strength: 250 N/mm²

**REINFORCEMENT**

- Diameter of links: 10 mm

**SUMMARY**

- Max. longitudinal spacing: 480 mm
- Approx. weight of links: 34.113 kg/m

Clause 3.12.7.2 states that links must pass round every corner bar and each alternate bar, and also that no bar may be more than 150 mm from a bar that is restrained by a link. Thus a set of at least 4 links must be provided at not more than the spacing specified above.

NOTE: As the distance between the bars exceeds 400 mm, a check on cracking may be necessary, if the face is in tension under service loading otherwise use smaller bars, more closely spaced.

Approx. total wt. of steel 172.22 kg/m
**Location:** Ex1 - Short rectangular column biaxial bending

**Isolated rectangular section subject bending and thrust**

Design to EN 1992-1-1:2004, Eurocode 2: Design of concrete structures. The main reinforcement design is based on balancing, by iterative means, of the two derived expressions for areas of reinforcing steel - one for axial load and the other for bending. One layer of steel is assumed for the axis of bending.

Char yield strength of reinforcement $f_yk=500$ N/mm$^2$
Max. aggregate size (for bar spc.) $h_{agg}=25$ mm
Diameter of compression bars $dia=25$ mm
Diameter of link legs $dial=8$ mm
Depth to comp. reinforcement $d_2=56$ mm
Clear height between restraints $l=3.6$ m
Design ultimate axial load $N=3620$ kN
Larger initial end mt. (y-y axis) $M_{2y}=120$ kNm
Smaller initial end moment $M_{1y}=0$ kNm
Larger initial end mt. (z-z axis) $M_{2z}=120$ kNm
Smaller initial end moment $M_{1z}=0$ kNm
About Y-Y axis:
Column is considered to be braced about the stronger axis.
Fixity condition at top (1-3) $cty=2$
Fixity condition at bottom (1-3) $cby=2$
About Z-Z axis:
Column is considered to be braced about the weaker axis.
Fixity condition at top (1-3) $ctz=2$
Fixity condition at bottom (1-3) $cbz=2$

**Axial load and moment considerations**

Effective/overall-depth ratio $d_{2h}=d_2/h=0.14$
Axial load ratio $N_{rat}=N_{Ed}/(b*h*f_{ck})=0.60333$
Moment ratio $M_{rat}=M_{Ed}/(b*h^2*f_{ck})=0.061539$

**Longitudinal reinforcement**

Chart data for rectangular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'.
Area of steel required \( A_s = \frac{f_{ck}}{f_{yk}} \times b \times h = 2818.4 \text{ mm}^2 \)
Number of bars provided \( n_{bars} = 8 \)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear column height</td>
<td>3.6 m</td>
</tr>
<tr>
<td>Slenderness (Y-Y axis)</td>
<td>21.2</td>
</tr>
<tr>
<td>Column is short (Y-Y)</td>
<td></td>
</tr>
<tr>
<td>Slenderness (Z-Z axis)</td>
<td>26.5</td>
</tr>
<tr>
<td>Column is short (Z-Z)</td>
<td></td>
</tr>
<tr>
<td>Design axial load</td>
<td>3620 kN</td>
</tr>
<tr>
<td>Design bending moments:</td>
<td></td>
</tr>
<tr>
<td>Y-Y axis</td>
<td>147.69 kNm</td>
</tr>
<tr>
<td>Z-Z axis</td>
<td>147.69 kNm</td>
</tr>
<tr>
<td>Designed area of steel</td>
<td>2818.4 mm²</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>3927 mm²</td>
</tr>
<tr>
<td>Minimum area of steel</td>
<td>832.6 mm²</td>
</tr>
<tr>
<td>Maximum area of steel</td>
<td>8000 mm²</td>
</tr>
<tr>
<td>Link diameter</td>
<td>8 mm</td>
</tr>
<tr>
<td>Maximum link spacing</td>
<td>400 mm</td>
</tr>
<tr>
<td>Resistance moment (Y-Y)</td>
<td>288.94 kNm</td>
</tr>
<tr>
<td>Resistance moment (Z-Z)</td>
<td>222.04 kNm</td>
</tr>
<tr>
<td>Biaxial bending factor</td>
<td>( 0.90113 \leq 1 )</td>
</tr>
</tbody>
</table>
Location: Ex2 - Bending about a single axis

Isolated rectangular section subject bending and thrust

The main reinforcement design is based on balancing, by iterative means, of the two derived expressions for areas of reinforcing steel - one for axial load and the other for bending.
One layer of steel is assumed for the axis of bending.

Char yield strength of reinf' ment f_yk=500 N/mm²
Max.aggregate size (for bar spc.) hagg=25 mm
Diameter of compression bars dia=32 mm
Diameter of link legs dial=8 mm
Depth to comp. reinforcement d2=70 mm
Clear height between restraints l=5.2 m
Design ultimate axial load N=1650 kN
Larger initial end mt.(positive) M2=100 kNm
Smaller initial end moment M1=0 kNm
About Y-Y axis:
Column is considered to be braced about the stronger axis.
Fixity condition at top (1-3) cty=1
Fixity condition at bottom (1-3) cby=1
About Z-Z axis:
Column is considered to be braced about the weaker axis.
Fixity condition at top (1-3) ctz=3
Fixity condition at bottom (1-3) cbz=3

Axial load and moment considerations

Effective/overall-depth ratio d2h=d2/h=0.2
Axial load ratio Nrat=NEd/(b*h*fck)=0.44898
Moment ratio Mrat=MEd/(b*h^2*fck)=0.090253

Longitudinal reinforcement

Chart data for rectangular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'.
Area of steel required: $A_s = \frac{f_{ck}}{f_{yk}} b h = 1516.7 \text{ mm}^2$
Number of bars provided: $n_{bars} = 4$

**DESIGN SUMMARY**
- Clear column height: $5.2 \text{ m}$
- Slenderness (Y-Y axis): $38.6$
- Slenderness (Z-Z axis): $51.467$
- Column is short
- Design axial load: $1650 \text{ kN}$
- Design bending moment: $116.09 \text{ kNm}$
- Designed area of steel: $1516.7 \text{ mm}^2$
- Area of steel provided: $3217 \text{ mm}^2$
- Minimum area of steel: $379.5 \text{ mm}^2$
- Maximum area of steel: $4900 \text{ mm}^2$
- Link diameter: $8 \text{ mm}$
- Maximum link spacing: $350 \text{ mm}$

**NOTE:** As the column is slender about Z-Z, the weaker axis, biaxial bending is recommended. Repeat calculations taking a small value or zero moment about the weaker axis.
Location: Column D1 (1st to 2nd floor)

Slender rectangular column subjected to uniaxial bending and thrust

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Additional moments are calculated in accordance with Clause 3.8.3 of BS8110, and section design is undertaken using data similar to that used to prepare the design charts in the IStructE/ICE 'Manual for the design of reinforced concrete building structures'.

Depth of cross-section $h=500$ mm
Width of cross-section $b=300$ mm
Size of main bars $\text{dia}=25$ mm
Chosen size of links $\text{dial}=8$ mm
Characteristic concrete strength $f_{cu}=30$ N/mm$^2$
Characteristic steel strength $f_y=500$ N/mm$^2$
Designated exposure class is XC2
Specified fixing tolerance $\text{tol}=10$ mm
Nominal concrete cover $\text{cover}=35$ mm
Effective depth about $X-X$ axis $h'=444.5$ mm
Effective depth about $Y-Y$ axis $b'=244.5$ mm
Clear height between restraints $l_o=5.6$ m
Design ultimate axial load $N=1000$ kN
Larger initial end mt.(positive) $M_2=100$ kNm
Smaller initial end moment $M_1=-60$ kNm
About $X-X$ axis:
Fixity condition at top (1-4) $\text{ctx}=1$
Fixity condition at bottom (1-3) $\text{cbx}=1$
and column is unbraced about $X-X$ axis.
About $Y-Y$ axis:
Fixity condition at top (1-4) $\text{cty}=2$
Fixity condition at bottom (1-3) $\text{cby}=3$
and column is braced about $Y-Y$ axis.
Area assumed for this calculation $A_{se}=600$ mm$^2$
Design section for $N=1000$ kN
and moment about major axis $M_x=178.62$ kNm
Number of bars provided $n_{bars}=4$

| MAIN | Characteristic strength $500$ N/mm$^2$ |
| REINFORCEMENT | Area of steel required $739.58$ mm$^2$ |
| SUMMARY | Area of steel provided $1963.5$ mm$^2$ |
| | Percentage provided $1.309$ % |
| | Weight of steel provided $15.413$ kg/m |
| | Diameter of bars $25$ mm |
| | Number of bars $4$ |

SCALE 5.48 Office 1007 Proforma 91
<table>
<thead>
<tr>
<th><strong>LINK</strong></th>
<th><strong>REINFORCEMENT</strong></th>
<th><strong>SUMMARY</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td></td>
<td>Max. longitudinal spacing</td>
<td>300 mm</td>
</tr>
<tr>
<td></td>
<td>Approx. weight of links</td>
<td>1.8623 kg/m</td>
</tr>
<tr>
<td></td>
<td>Approx. total wt. of steel</td>
<td>17.276 kg/m</td>
</tr>
</tbody>
</table>
Location: Small column with high moments and maximum reinforcement.

Slender rectangular column subjected to uniaxial bending and thrust

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Additional moments are calculated in accordance with Clause 3.8.3 of BS8110, and section design is undertaken using data similar to that used to prepare the design charts in the IStructE/ICE 'Manual for the design of reinforced concrete building structures'.

Depth of cross-section $h=200$ mm
Width of cross-section $b=200$ mm
Size of main bars $\text{dia}=16$ mm
Chosen size of links $\text{dial}=8$ mm
Characteristic concrete strength $f_{cu}=60$ N/mm$^2$
Characteristic steel strength $f_y=500$ N/mm$^2$
Designated exposure class is XC1
Specified fixing tolerance $\text{tol}=10$ mm
Nominal concrete cover $\text{cover}=25$ mm
Effective depth about X-X axis $h'=159$ mm
Effective depth about Y-Y axis $b'=159$ mm
Clear height between restraints $l_0=4$ m
Design ultimate axial load $N=500$ kN
Larger initial end mt. (positive) $M_2=70$ kNm
Smaller initial end moment $M_1=-50$ kNm
About X-X axis:
Fixity condition at top (1-4) $c_{tx}=1$
Fixity condition at bottom (1-3) $c_{bx}=1$
and column is braced about X-X axis.
About Y-Y axis:
Fixity condition at top (1-4) $c_{ty}=2$
Fixity condition at bottom (1-3) $c_{by}=3$
and column is braced about Y-Y axis.
Area assumed for this calculation $A_{se}=160$ mm$^2$
Design section for
N=500 kN
and moment about minor axis $M_y=70$ kNm
Number of bars provided $n_{bars}=10$

**MAIN**
Characteristic strength 500 N/mm$^2$

**REINFORCEMENT**
Area of steel required 1834.3 mm$^2$

**SUMMARY**
Area of steel provided 2010.6 mm$^2$
Percentage provided 5.0265 %
Weight of steel provided 15.783 kg/m
Diameter of bars 16 mm
Number of bars 10

to be placed along h-faces (spaced at 29 mm, assuming 1 layer)
(Spacing is close, hence pairs/bundles or several layers may be required).
<table>
<thead>
<tr>
<th>LINK</th>
<th>REINFORCEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. longitudinal spacing</td>
</tr>
<tr>
<td>Approx. weight of links</td>
</tr>
</tbody>
</table>

Clause 3.12.7.2 states that links must pass round every corner bar and each alternate bar, and also that no bar may be more than 150 mm from a bar that is restrained by a link. Thus a set of at least 2 links must be provided at not more than the spacing specified above.

Approx. total wt. of steel 18.674 kg/m
Location: Large, long column. Mild steel. Axial load predominates.

Slender rectangular column subjected to uniaxial bending and thrust

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Additional moments are calculated in accordance with Clause 3.8.3 of BS8110, and section design is undertaken using data similar to that used to prepare the design charts in the IStructE/ICE 'Manual for the design of reinforced concrete building structures'.

Depth of cross-section $h=2000$ mm
Width of cross-section $b=1500$ mm
Size of main bars $\text{dia}=40$ mm
Chosen size of links $\text{dial}=10$ mm
Characteristic concrete strength $f_{\text{cu}}=40$ N/mm$^2$
Characteristic steel strength $f_y=250$ N/mm$^2$
Nominal concrete cover $\text{cover}=60$ mm
Effective depth about X-X axis $h'=1910$ mm
Effective depth about Y-Y axis $b'=1410$ mm
Clear height between restraints $l_0=25$ m
Design ultimate axial load $N=15000$ kN
Larger initial end mt. (positive) $M_2=2500$ kNm
Smaller initial end moment $M_1=-1800$ kNm
About X-X axis:
Fixity condition at top (1-4) $c_x=3$
Fixity condition at bottom (1-3) $c_b=1$
and column is unbraced about X-X axis.
About Y-Y axis:
Fixity condition at top (1-4) $c_y=3$
Fixity condition at bottom (1-3) $c_y=2$
and column is braced about Y-Y axis.
Area assumed for this calculation $A_{\text{se}}=12000$ mm$^2$
Design section for $N=15000$ kN
and moment about major axis $M_x=8500$ kNm
Number of bars provided $n_{\text{bars}}=10$

### MAIN
- Characteristic strength 250 N/mm$^2$

### REINFORCEMENT
- Area of steel required 12000 mm$^2$
  (represents minimum of 0.4%)
  (less theoretically needed.)
- Area of steel provided 12566 mm$^2$
- Percentage provided 0.41888 %
- Weight of steel provided 98.646 kg/m
- Diameter of bars 40 mm
- Number of bars 10

**SUMMARY**

**to be placed along b-faces (spaced at 330 mm, assuming 1 layer)**

SCALE 5.48 Office 1007 Proforma 91
Clause 3.12.7.2 states that links must pass round every corner bar and each alternate bar, and also that no bar may be more than 150 mm from a bar that is restrained by a link. Thus a set of at least 3 links must be provided at not more than the spacing specified above. As the distance between the bars exceeds 400 mm, a check on cracking may be necessary, if the face is in tension under service loading – otherwise use smaller bars, more closely spaced.

Approx. total wt. of steel 124.23 kg/m
Location: Ex1 - Slender column uniaxial bending

Isolated rectangular section subject bending and thrust

Design to BS EN 1992-1-1:2004
Eurocode 2: Design of concrete structures.
The main reinforcement design is based on balancing, by iterative means, of the two derived expressions for areas of reinforcing steel - one for axial load and the other for bending.
One layer of steel is assumed for the axis of bending.

Char yield strength of reinforcement $f_{yk}=500$ N/mm²
Max.aggregate size (for bar spc.) $h_{agg}=25$ mm
Diameter of compression bars $dia=25$ mm
Diameter of link legs $dial=8$ mm
Depth to comp. reinforcement $d2=61$ mm
Clear height between restraints $l=3.6$ m
Design ultimate axial load (kN) $N_{E,d}=1620$ kN
Larger initial end mt.(positive) $M_2=38.5$ kNm
Smaller initial end moment $M_1=0$ kNm
About $Y-Y$ axis:
Column is considered to be braced about the stronger axis.
Fixity condition at top (1-3) $c_{ty}=2$
Fixity condition at bottom (1-3) $c_{by}=2$
About $Z-Z$ axis:
Column is considered to be braced about the weaker axis.
Fixity condition at top (1-3) $c_{tz}=2$
Fixity condition at bottom (1-3) $c_{bz}=2$

Axial load and moment considerations

Effective/overall-depth ratio $d_{2h}=d_2/h=0.20333$
Axial load ratio $N_{rat}=N_{E,d}/(b*h*f_{ck})=0.6$
Moment ratio $M_{rat}=M_{E,d}/(b*h^2*f_{ck})=0.068765$

Longitudinal reinforcement

Chart data for rectangular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'.
Area of steel required          $A_s = \frac{\alpha v f_{ck}}{f_{yk} b h} = 1485.8 \text{ mm}^2$
Number of bars provided         $n_{bars} = 4$

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear column height</td>
<td>3.6 m</td>
</tr>
<tr>
<td>Slenderness (Y-Y axis)</td>
<td>35.334</td>
</tr>
<tr>
<td>Slenderness (Z-Z axis)</td>
<td>35.334</td>
</tr>
<tr>
<td>Column is slender</td>
<td></td>
</tr>
<tr>
<td>Design axial load</td>
<td>1620 kN</td>
</tr>
<tr>
<td>Design bending moment</td>
<td>55.7 kNm</td>
</tr>
<tr>
<td>Designed area of steel</td>
<td>1485.8 mm$^2$</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>1963.5 mm$^2$</td>
</tr>
<tr>
<td>Minimum area of steel</td>
<td>372.6 mm$^2$</td>
</tr>
<tr>
<td>Maximum area of steel</td>
<td>3600 mm$^2$</td>
</tr>
<tr>
<td>Link diameter</td>
<td>8 mm</td>
</tr>
<tr>
<td>Maximum link spacing</td>
<td>300 mm</td>
</tr>
</tbody>
</table>
**Location:** Ex2 - Bending about a single axis

**Isolated rectangular section subject bending and thrust**

Design to BS EN 1992-1-1:2004

Eurocode 2: Design of concrete structures.

The main reinforcement design is based on balancing, by iterative means, of the two derived expressions for areas of reinforcing steel - one for axial load and the other for bending.

One layer of steel is assumed for the axis of bending.

Char yield strength of reinforcement $f_{yk}=500 \text{ N/mm}^2$

Max. aggregate size (for bar spc.) $h_{agg}=25 \text{ mm}$

Diameter of compression bars $d_{ia}=32 \text{ mm}$

Diameter of link legs $d_{ial}=8 \text{ mm}$

Depth to comp. reinforcement $d_{2}=70 \text{ mm}$

Clear height between restraints $l=5.2 \text{ m}$

Design ultimate axial load (kN) $N_{Ed}'=1650 \text{ kN}$

Larger initial end mt. (positive) $M_2=100 \text{ kNm}$

Smaller initial end moment $M_1=0 \text{ kNm}$

About Y-Y axis:

Column is considered to be braced about the stronger axis.

Fixity condition at top (1-3) $c_{ty}=1$

Fixity condition at bottom (1-3) $c_{by}=1$

About Z-Z axis:

Column is considered to be braced about the weaker axis.

Fixity condition at top (1-3) $c_{tz}=3$

Fixity condition at bottom (1-3) $c_{bz}=3$

**Axial load and moment considerations**

Effective/overall-depth ratio $d_{2h}=d_{2}/h=0.2$

Axial load ratio $N_{rat}=N_{Ed}/(b\cdot h\cdot f_{ck})=0.44898$

Moment ratio $M_{rat}=M_{Ed}/(b\cdot h^2\cdot f_{ck})=0.090253$

**Longitudinal reinforcement**

Chart data for rectangular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'.
Area of steel required $A_s = \frac{f_{ck}}{f_{yk}} b h = 1516.7 \text{ mm}^2$
Number of bars provided $n_{bars} = 4$

**DESIGN SUMMARY**
- Clear column height 5.2 m
- Slenderness (Y-Y axis) 38.6
- Slenderness (Z-Z axis) 51.467
- Column is short
- Design axial load 1650 kN
- Design bending moment 116.09 kNm
- Designed area of steel 1516.7 mm$^2$
- Area of steel provided 3217 mm$^2$
- Minimum area of steel 379.5 mm$^2$
- Maximum area of steel 4900 mm$^2$
- Link diameter 8 mm
- Maximum link spacing 350 mm

**NOTE:** As the column is slender about Z-Z, the weaker axis, biaxial bending is recommended. Repeat calculations taking a small value or zero moment about the weaker axis.
Slender rectangular column subjected to biaxial bending and thrust

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Additional moments are calculated in accordance with Clause 3.8.3 of BS8110, and section design is undertaken using data similar to that used to prepare the design charts in the IStructE/ICE 'Manual for the design of reinforced concrete building structures'.

Depth of cross-section \( h = 500 \text{ mm} \)
Width of cross-section \( b = 300 \text{ mm} \)
Size of main bars \( \text{dia} = 25 \text{ mm} \)
Chosen size of links \( \text{dial} = 8 \text{ mm} \)
Characteristic concrete strength \( f_{cu} = 30 \text{ N/mm}^2 \)
Characteristic steel strength \( f_y = 500 \text{ N/mm}^2 \)
Designated exposure class is XC2
Specified fixing tolerance \( \text{tol} = 10 \text{ mm} \)
Nominal concrete cover \( \text{cover} = 35 \text{ mm} \)
Effective depth about X-X axis \( h' = 444.5 \text{ mm} \)
Effective depth about Y-Y axis \( b' = 244.5 \text{ mm} \)
Clear height between restraints \( l_0 = 6 \text{ m} \)
Design ultimate axial load \( N = 1000 \text{ kN} \)
Larger initial end moment (positive) \( M_{2x} = 100 \text{ kNm} \)
Smaller initial end moment \( M_{1x} = -60 \text{ kNm} \)
Larger initial end moment (positive) \( M_{2y} = 80 \text{ kNm} \)
Smaller initial end moment \( M_{1y} = -50 \text{ kNm} \)

About X-X axis:
Fixity condition at top (1-4) \( c_{tx} = 2 \)
Fixity condition at bottom (1-3) \( c_{bx} = 3 \)
and column is unbraced about X-X axis.
About Y-Y axis:
Fixity condition at top (1-4) \( c_{ty} = 1 \)
Fixity condition at bottom (1-3) \( c_{by} = 1 \)
and column is braced about Y-Y axis.

Area assumed for this calculation \( A_{se} = 600 \text{ mm}^2 \)
minor-axis design moment \( M_y = M_{2y} = 80 \text{ kNm} \)
major-axis design moment \( M_x = M_{2x} + M_{add} = 216.64 \text{ kNm} \)
Number of bars provided: nbars=6

<table>
<thead>
<tr>
<th>MAIN</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Area of steel required</td>
<td>2446.2 mm²</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Area of steel provided</td>
<td>2945.2 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided</td>
<td>1.9635 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided</td>
<td>23.12 kg/m</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
<td>6</td>
</tr>
</tbody>
</table>

To be placed along b-faces (spaced @ 94 mm, assume 1 layer)

<table>
<thead>
<tr>
<th>LINK</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Max.longitudinal spacing</td>
<td>300 mm</td>
</tr>
<tr>
<td></td>
<td>Approx.weight of links</td>
<td>1.8623 kg/m</td>
</tr>
<tr>
<td></td>
<td>Approx.total wt.of steel</td>
<td>24.982 kg/m</td>
</tr>
</tbody>
</table>
Location: Small column with high moments and maximum reinforcement.

Slender rectangular column subjected to biaxial bending and thrust

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Additional moments are calculated in accordance with Clause 3.8.3 of BS8110, and section design is undertaken using data similar to that used to prepare the design charts in the IStructE/ICE 'Manual for the design of reinforced concrete building structures'.

Depth of cross-section $h=200$ mm
Width of cross-section $b=200$ mm
Size of main bars $\text{dia}=16$ mm
Chosen size of links $\text{dial}=8$ mm
Characteristic concrete strength $f_{cu}=60$ N/mm$^2$
Characteristic steel strength $f_y=500$ N/mm$^2$
Designated exposure class is XC1
Specified fixing tolerance $\text{tol}=10$ mm
Nominal concrete cover $\text{cover}=25$ mm
Effective depth about X-X axis $h'=159$ mm
Effective depth about Y-Y axis $b'=159$ mm
Clear height between restraints $\text{lo}=4$ m
Design ultimate axial load $N=500$ kN
Larger initial end mt (positive) $M_{2x}=40$ kNm
Smaller initial end moment $M_{1x}=-20$ kNm
Larger initial end mt (positive) $M_{2y}=40$ kNm
Smaller initial end moment $M_{1y}=-20$ kNm
About X-X axis:
Fixity condition at top (1-4) $\text{ctx}=1$
Fixity condition at bottom (1-3) $\text{cbx}=2$
and column is braced about X-X axis.
About Y-Y axis:
Fixity condition at top (1-4) $\text{cty}=1$
Fixity condition at bottom (1-3) $\text{cby}=2$
and column is braced about Y-Y axis.
Area assumed for this calculation $A_{se}=160$ mm$^2$
minor-axis design moment $M_y=M_{2y}=40$ kNm
major-axis design moment $M_x=M_{2x}=40$ kNm
### MAIN
- **Characteristic strength**: 500 N/mm²

### REINFORCEMENT
- **Area of steel required**: 1850.9 mm²

### SUMMARY
- **Area of steel provided**: 2010.6 mm²
- **Percentage provided**: 5.0265 %
- **Weight of steel provided**: 15.783 kg/m
- **Diameter of bars**: 16 mm
- **Number of bars**: 10

To be placed along b-faces (spaced @ 29 mm, assume 1 layer)
Spacing is close, so pairs/bundles or several layers may be needed

### LINK
- **Characteristic strength**: 500 N/mm²

### REINFORCEMENT
- **Diameter of links**: 8 mm

### SUMMARY
- **Max.longitudinal spacing**: 190 mm
- **Approx.weight of links**: 2.8907 kg/m

Clause 3.12.7.2 states that links must pass round every corner bar and each alternate bar, and also that no bar may be more than 150 mm from a bar that is restrained by a link. Thus a set of at least 2 links must be provided at not more than the spacing specified above.

**Approx.total wt.of steel**: 18.674 kg/m
Location: Large, long column. Mild steel. Axial load predominates.

Slender rectangular column subjected to biaxial bending and thrust

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

Additional moments are calculated in accordance with Clause 3.8.3 of BS8110, and section design is undertaken using data similar to that used to prepare the design charts in the IStructE/ICE 'Manual for the design of reinforced concrete building structures'.

Depth of cross-section \( h = 2000 \text{ mm} \)
Width of cross-section \( b = 1500 \text{ mm} \)
Size of main bars \( \text{dia} = 40 \text{ mm} \)
Chosen size of links \( \text{dia} = 10 \text{ mm} \)
Characteristic concrete strength \( f_{cu} = 40 \text{ N/mm}^2 \)
Characteristic steel strength \( f_y = 250 \text{ N/mm}^2 \)
Nominal concrete cover \( \text{cover} = 60 \text{ mm} \)
Effective depth about X-X axis \( h' = 1910 \text{ mm} \)
Effective depth about Y-Y axis \( b' = 1410 \text{ mm} \)
Clear height between restraints \( l_0 = 40 \text{ m} \)
Design ultimate axial load \( N = 15000 \text{ kN} \)
Larger initial end mt (positive) \( M_{2x} = 2500 \text{ kNm} \)
Smaller initial end moment \( M_{1x} = -1800 \text{ kNm} \)
Larger initial end mt (positive) \( M_{2y} = 2000 \text{ kNm} \)
Smaller initial end moment \( M_{1y} = -1400 \text{ kNm} \)

About X-X axis:
Fixity condition at top (1-4) \( c_{tx} = 3 \)
Fixity condition at bottom (1-3) \( c_{bx} = 1 \)
and column is unbraced about X-X axis.

About Y-Y axis:
Fixity condition at top (1-4) \( c_{ty} = 3 \)
Fixity condition at bottom (1-3) \( c_{by} = 2 \)
and column is braced about Y-Y axis.

Area assumed for this calculation \( A_{se} = 12000 \text{ mm}^2 \)
minor-axis design moment \( M_y = M'_{b'} = 8020 \text{ kNm} \)
major-axis design moment \( M_x = M_{2x} + M_{add} = 17860 \text{ kNm} \)
Number of bars provided \( nbars = 68 \)

<table>
<thead>
<tr>
<th>MAIN</th>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Area of steel required</td>
<td>84363 mm²</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Area of steel provided</td>
<td>85451 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided</td>
<td>2.8484 %</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided</td>
<td>670.79 kg/m</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>40 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
<td>68</td>
</tr>
</tbody>
</table>

To be placed along b-faces (spaced @ 40 mm, assume 1 layer)  
Spacing is close, so pairs/bundles or several layers may be needed

<table>
<thead>
<tr>
<th>LINK</th>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of links</td>
<td>10 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Max.longitudinal spacing</td>
<td>480 mm</td>
</tr>
<tr>
<td></td>
<td>Approx.weight of links</td>
<td>76.754 kg/m</td>
</tr>
</tbody>
</table>

Clause 3.12.7.2 states that links must pass round every corner bar and each alternate bar, and also that no bar may be more than 150 mm from a bar that is restrained by a link. Thus a set of at least 9 links must be provided at not more than the spacing specified above.

As the distance between the bars exceeds 400 mm, check on cracking may be necessary, if the face is in tension under service loading - otherwise use smaller bars, more closely spaced.

\[
\text{Approx.total wt.of steel} = 747.55 \text{ kg/m}
\]
Location: Ex1 - Biaxial bending example

Isolated rectangular section subject bending and thrust

Design to EN 1992-1-1:2004, Eurocode 2: Design of concrete structures. The main reinforcement design is based on balancing, by iterative means, of the two derived expressions for areas of reinforcing steel - one for axial load and the other for bending. One layer of steel is assumed for the axis of bending.

Char yield strength of reinf. $f_{yk}=500 \text{ N/mm}^2$
Max.aggregate size (for bar spc.) $h_{agg}=25 \text{ mm}$
 Diameter of compression bars $d_{ia}=32 \text{ mm}$
 Diameter of link legs $d_{ial}=8 \text{ mm}$
Depth to comp. reinforcement $d_{2}=69 \text{ mm}$
Clear height between restraints $l=3.2 \text{ m}$
Design ultimate axial load $N=1650 \text{ kN}$
Larger initial end mt. (y-y axis) $M_{2y}=146.1 \text{ kNm}$
Smaller initial end moment $M_{1y}=0 \text{ kNm}$
Larger initial end mt. (z-z axis) $M_{2z}=114.5 \text{ kNm}$
Smaller initial end moment $M_{1z}=0 \text{ kNm}$
About Y-Y axis: Column is considered to be braced.
About Z-Z axis: Column is considered to be braced.

Axial load and moment considerations

Effective/overall-depth ratio $d_{2h}=d_{2}/h=0.19714$
Axial load ratio $N_{rat}=N_{Ed}/(b*h*f_{ck})=0.44898$
Moment ratio $M_{rat}=M_{Ed}/(b*h^2*f_{ck})=0.12231$

Longitudinal reinforcement

Chart data for rectangular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'.
Area of steel required $A_{s}=v_{a}l*f_{ck}/f_{yk}*b*h=2642.9 \text{ mm}^2$
Number of bars provided $n_{bars}=8$
**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear column height</td>
<td>3.2 m</td>
</tr>
<tr>
<td>Slenderness (Y-Y axis)</td>
<td>26.921</td>
</tr>
<tr>
<td>Column is short (Y-Y)</td>
<td></td>
</tr>
<tr>
<td>Slenderness (Z-Z axis)</td>
<td>26.921</td>
</tr>
<tr>
<td>Column is short (Z-Z)</td>
<td></td>
</tr>
<tr>
<td>Design axial load</td>
<td>1650 kN</td>
</tr>
<tr>
<td>Design bending moments:</td>
<td></td>
</tr>
<tr>
<td>Y-Y axis</td>
<td>157.32 kNm</td>
</tr>
<tr>
<td>Z-Z axis</td>
<td>125.72 kNm</td>
</tr>
<tr>
<td>Designed area of steel</td>
<td>2642.9 mm²</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>6434 mm²</td>
</tr>
<tr>
<td>Minimum area of steel</td>
<td>379.5 mm²</td>
</tr>
<tr>
<td>Maximum area of steel</td>
<td>4900 mm²</td>
</tr>
<tr>
<td>Link diameter</td>
<td>8 mm</td>
</tr>
<tr>
<td>Maximum link spacing</td>
<td>350 mm</td>
</tr>
<tr>
<td>Resistance moment (Y-Y)</td>
<td>318.89 kNm</td>
</tr>
<tr>
<td>Resistance moment (Z-Z)</td>
<td>318.89 kNm</td>
</tr>
<tr>
<td>Biaxial bending factor</td>
<td>0.75655</td>
</tr>
</tbody>
</table>

**NOTE:** Designed steel exceeds 4% of gross area. However the maximum area can be increased provided that the concrete can be placed and compacted sufficiently.
Location: Ex2 - Bending about a single axis

Isolated rectangular section subject bending and thrust

Design to EN 1992-1-1:2004, Eurocode 2: Design of concrete structures. The main reinforcement design is based on balancing, by iterative means, of the two derived expressions for areas of reinforcing steel - one for axial load and the other for bending. One layer of steel is assumed for the axis of bending.

Char yield strength of reinft. $f_{yk}=500 \text{ N/mm}^2$
Max.aggregate size (for bar spc.) $h_{agg}=25 \text{ mm}$
Diameter of compression bars $d_{ia}=32 \text{ mm}$
Diameter of link legs $d_{ial}=8 \text{ mm}$
Depth to comp. reinforcement $d_2=69 \text{ mm}$
Clear height between restraints $l=5.2 \text{ m}$
Design ultimate axial load $N=1650 \text{ kN}$
Larger initial end mt.(positive) $M_2=100 \text{ kNm}$
Smaller initial end moment $M_1=0 \text{ kNm}$
About Y-Y axis:
Column is considered to be braced.
About Z-Z axis:
Column is considered to be braced.

Axial load and moment considerations

Effective/overall-depth ratio $d_{2h}=d_2/h=0.19714$
Axial load ratio $N_{rat}=N_{Ed}/(b*h*f_{ck})=0.44898$
Moment ratio $M_{rat}=M_{Ed}/(b*h^2*f_{ck})=0.090253$

Longitudinal reinforcement

Chart data for rectangular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'.
Area of steel required $A_s=\text{val}*f_{ck}/f_{yk}*b*h=1507 \text{ mm}^2$
Number of bars provided $n_{bars}=4$
### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear column height</td>
<td>5.2 m</td>
</tr>
<tr>
<td>Slenderness (Y-Y axis)</td>
<td>38.6</td>
</tr>
<tr>
<td>Slenderness (Z-Z axis)</td>
<td>51.467</td>
</tr>
<tr>
<td>Column is short</td>
<td></td>
</tr>
<tr>
<td>Design axial load</td>
<td>1650 kN</td>
</tr>
<tr>
<td>Design bending moment</td>
<td>116.09 kNm</td>
</tr>
<tr>
<td>Designed area of steel</td>
<td>1507 mm²</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>3217 mm²</td>
</tr>
<tr>
<td>Minimum area of steel</td>
<td>379.5 mm²</td>
</tr>
<tr>
<td>Maximum area of steel</td>
<td>4900 mm²</td>
</tr>
<tr>
<td>Link diameter</td>
<td>8 mm</td>
</tr>
<tr>
<td>Maximum link spacing</td>
<td>350 mm</td>
</tr>
</tbody>
</table>

**NOTE:** As the column is slender about Z-Z, the weaker axis, biaxial bending is recommended. Repeat calculations taking a small value or zero moment about the weaker axis.

Design of circular section under bending and axial load

Design to BS8110(1997), partial safety factor for steel $\gamma_S = 1.15$

Calculations for reinforcement are in accordance with BS 8110. Solution is obtained by trial-and-adjustment procedure with check calculations printed out beneath. The concrete section is split into eight strips of equal depth. Eight steel bars arranged as shown are assumed, giving four steel areas.

Design axial load $N = 1$ kN
Design moment $M = 128$ kNm
Section diameter $D = 300$ mm
Size of links $d_{\text{dial}} = 8$ mm
Characteristic concrete strength $f_{\text{cu}} = 50$ N/mm$^2$
Characteristic steel strength $f_y = 500$ N/mm$^2$
Nominal cover to all steel $c = 25$ mm
SUMMARY ( $f_{cu}=50 \text{ N/mm}^2$ : $f_{y}=500 \text{ N/mm}^2$ )

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

<table>
<thead>
<tr>
<th>Diameter of section</th>
<th>300 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover to all reinforcement</td>
<td>25 mm</td>
</tr>
<tr>
<td>Size of main bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Number of main bars</td>
<td>30</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>3392.9 mm$^2$</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>4.8 %</td>
</tr>
<tr>
<td>Size of links provided</td>
<td>8 mm</td>
</tr>
<tr>
<td>Comp. strain at outer conc. fibre</td>
<td>3500 microstrain</td>
</tr>
<tr>
<td>Comp. strain at steel area As1</td>
<td>2055.4 microstrain</td>
</tr>
<tr>
<td>Tensile strain at steel area As4</td>
<td>4189.1 microstrain</td>
</tr>
<tr>
<td>Depth of concrete in compression</td>
<td>114.96 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>1</td>
<td>108.86</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>128</td>
<td>131.81</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Location: Large column. Mild steel. Low moment. High axial load.

Design of circular section under bending and axial load

Design to BS8110(1997), partial safety factor for steel $\gamma_S = 1.15$

Calculations for reinforcement are in accordance with BS 8110. Solution is obtained by trial- and-adjustment procedure with check calculations printed out beneath. The concrete section is split into eight strips of equal depth. Eight steel bars arranged as shown are assumed, giving four steel areas.

Design axial load $N = 10000$ kN
Design moment $M = 100$ kNm
Section diameter $D = 1500$ mm
Size of links $d_{\text{bar}} = 10$ mm
Characteristic concrete strength $f_{\text{cu}} = 35$ N/mm²
Characteristic steel strength $f_y = 250$ N/mm²
Nominal cover to all steel $c = 25$ mm

SUMMARY ( $f_{\text{cu}} = 35$ N/mm² : $f_y = 250$ N/mm² )

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

Diameter of section $1500$ mm
Cover to all reinforcement $25$ mm
Size of main bars $12$ mm
Number of main bars $64$
Area of steel provided $7238.2$ mm²
Percentage of steel provided $0.4096$
Size of links provided $10$ mm
Comp. strain at outer conc.fibre $3500$ microstrain
Comp. strain at steel area $A_{\text{S1}}$ $3300.2$ microstrain
Comp. strain at steel area $A_{\text{S4}}$ $544.51$ microstrain
Depth of concrete in compression $1500$ mm

Axial load in kN

<table>
<thead>
<tr>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10000</td>
<td>28100</td>
<td>180.9</td>
</tr>
</tbody>
</table>

Resistance moment in kNm

<table>
<thead>
<tr>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>572.84</td>
<td>186.4</td>
</tr>
</tbody>
</table>

Section reinforced with the minimum permissible steel area of 0.4% can support the specified loading shown, and is accepted as the design.
Location: Medium-sized column (High-yield steel, Moderate mom, Low ld)

Design of circular section under bending and axial load

Design to BS8110(1997), partial safety factor for steel $\gamma_S=1.15$

Calculations for reinforcement are in accordance with BS 8110. Solution is obtained by trial-and-adjustment procedure with check calculations printed out beneath. The concrete section is split into eight strips of equal depth. Eight steel bars arranged as shown are assumed, giving four steel areas.

Design axial load $N=10$ kN
Design moment $M=300$ kNm
Section diameter $D=450$ mm
Size of links $d=8$ mm
Characteristic concrete strength $f_{cu}=40$ N/mm$^2$
Characteristic steel strength $f_y=500$ N/mm$^2$
Nominal cover to all steel $c=25$ mm
SUMMARY (fcu=40 N/mm² : fy=500 N/mm²)

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

<table>
<thead>
<tr>
<th>Specification</th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of section</td>
<td>450 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cover to all reinforcement</td>
<td>25 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Size of main bars</td>
<td>12 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of main bars</td>
<td>40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>4523.9 mm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>2.8444 %</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Size of links provided</td>
<td>8 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Comp. strain at outer conc. fibre</td>
<td>3500 microstrain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Comp. strain at steel area As1</td>
<td>2253.2 microstrain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile strain at steel area As4</td>
<td>5808 microstrain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth of concrete in compression</td>
<td>149.22 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Axial load in kN</td>
<td>10</td>
<td>177.72</td>
<td>1677.2</td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>300</td>
<td>301.56</td>
<td>0.5</td>
</tr>
</tbody>
</table>

The above result is judged acceptable.

Design of circular section under bending and axial load

Design to BS8110(1997), partial safety factor for steel $\gamma_S=1.15$

Calculations for reinforcement are in accordance with BS 8110 Solution is obtained by trial-and-adjustment procedure with check calculations printed out beneath. The concrete section is split into eight strips of equal depth. Eight steel bars arranged as shown are assumed, giving four steel areas.

Design axial load $N=500$ kN
Design moment $M=420$ kNm
Section diameter $D=500$ mm
Size of links $d=8$ mm
Characteristic concrete strength $f_{cu}=40$ N/mm$^2$
Characteristic steel strength $f_y=250$ N/mm$^2$
Nominal cover to all steel $c=25$ mm
**SUMMARY (fcu=40 N/mm\(^2\) : fy=250 N/mm\(^2\))**

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of section</td>
<td>500 mm</td>
</tr>
<tr>
<td>Cover to all reinforcement</td>
<td>25 mm</td>
</tr>
<tr>
<td>Size of main bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Number of main bars</td>
<td>90</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>10179 mm(^2)</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>5.184 %</td>
</tr>
<tr>
<td>Size of links provided</td>
<td>8 mm</td>
</tr>
<tr>
<td>Comp. strain at outer conc. fibre</td>
<td>3500 microstrain</td>
</tr>
<tr>
<td>Comp. strain at steel area As1</td>
<td>2468.4 microstrain</td>
</tr>
<tr>
<td>Tensile strain at steel area As4</td>
<td>4836.3 microstrain</td>
</tr>
<tr>
<td>Depth of concrete in compression</td>
<td>186.81 mm</td>
</tr>
<tr>
<td>Axial load in kN</td>
<td>Specified: 500</td>
</tr>
<tr>
<td></td>
<td>Calculated: 622.03</td>
</tr>
<tr>
<td></td>
<td>Excess (%): 24.4</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>Specified: 420</td>
</tr>
<tr>
<td></td>
<td>Calculated: 424.4</td>
</tr>
<tr>
<td></td>
<td>Satisfactory: 1</td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Location: Small column (High-yield steel, High mom, Low axial load)

Design of circular section under bending and axial load to EC2

Design ultimate axial load \( N_{Ed} = 1 \text{ kN} \)
Design ultimate moment \( M_{Ed} = 128 \text{ kNm} \)
Section diameter \( D = 350 \text{ mm} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of longitudinal reinft. \( d_{ia} = 16 \text{ mm} \)
Diameter of column links \( d_{il} = 8 \text{ mm} \)

**SUMMARY** ( \( f_{ck} = 50 \text{ N/mm}^2 : f_{yk} = 500 \text{ N/mm}^2 \) )

(\( \text{Note: 'microstrain' indicates true strain multiplied by 1,000,000.} \))

- Diameter of section: 350 mm
- Cover to all reinforcement: 45 mm
- Diameter of longitudinal reinft.: 16 mm
- Number of main bars: 14
- Area of steel provided: 2814.9 mm²
- Percentage of steel provided: 2.9257 %
- Size of links provided: 8 mm
- Comp. strain at outer conc. fibre: 3500 microstrain
- Comp. strain at steel area As1: 1414.2 microstrain
- Tensile strain at steel area As4: 4891.3 microstrain
- Depth of concrete in compression: 116.92 mm

<table>
<thead>
<tr>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load (kN)</td>
<td>1</td>
<td>94.331</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance moment (kNm)</td>
<td>128</td>
<td>128.86</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Location: Large column (Mild steel, Low moment, High axial load)

Design of circular section under bending and axial load to EC2

Design ultimate axial load $N_{Ed}=10000$ kN
Design ultimate moment $M_{Ed}=100$ kNm
Section diameter $D=1500$ mm
Partial safety factor for steel $\gamma_{ms}=1.15$
Char yield strength of reinft. $f_{yk}=500$ N/mm²
Diameter of column links $d_{al}=10$ mm
Nominal cover to all steel $c=25$ mm

SUMMARY ($f_{ck}=28$ N/mm² : $f_{yk}=500$ N/mm²)

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

<table>
<thead>
<tr>
<th>Specification</th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load (kN)</td>
<td>10000</td>
<td>26383</td>
<td>163.8</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance moment (kNm)</td>
<td>499.5</td>
<td>1326.1</td>
<td>165.4</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Section reinforced with the minimum permissible steel area of 0.2 % can support the specified loading shown, and is accepted as the design.
Location: Medium-sized column (High-yield steel, Moderate mom, Low ld)

Design of circular section under bending and axial load to EC2

Design ultimate axial load \( N_{Ed} = 10 \text{ kN} \)
Design ultimate moment \( M_{Ed} = 300 \text{ kNm} \)
Section diameter \( D = 450 \text{ mm} \)
Partial safety factor for steel \( \gamma_{Ms} = 1.15 \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of column links \( d_{al} = 8 \text{ mm} \)
Nominal cover to all steel \( c = 25 \text{ mm} \)

SUMMARY (\( f_{ck} = 32 \text{ N/mm}^2 : f_{yk} = 500 \text{ N/mm}^2 \))

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

Design axial load (kN) | Specified | Calculated | Excess (%) |
------------------------|-----------|------------|------------|
Satisfactory            | 10        | 115.23     | 1052.2     |
Resistance moment (kNm) | 300       | 306.96     | 2.3        |
Satisfactory            |           |            |            |

The above result is judged acceptable.
Location: Medium-sized column (Mild steel, High mom, Moderate load)

Design of circular section under bending and axial load to EC2

Design ultimate axial load \( N_{Ed} = 500 \, \text{kN} \)
Design ultimate moment \( M_{Ed} = 420 \, \text{kNm} \)
Section diameter \( D = 500 \, \text{mm} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Char yield strength of reinft. \( f_{yk} = 500 \, \text{N/mm}^2 \)
Diameter of column links \( \text{d}_{\text{al}} = 8 \, \text{mm} \)
Nominal cover to all steel \( c = 25 \, \text{mm} \)

**SUMMARY** (\( f_{ck} = 32 \, \text{N/mm}^2 \) : \( f_{yk} = 500 \, \text{N/mm}^2 \))

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

Diameter of section \( 500 \, \text{mm} \)
Cover to all reinforcement \( 25 \, \text{mm} \)
Diameter of longitudinal reinft. \( 12 \, \text{mm} \)
Number of main bars \( 46 \)
Area of steel provided \( 5202.5 \, \text{mm}^2 \)
Percentage of steel provided \( 2.6496 \% \)
Size of links provided \( 8 \, \text{mm} \)
Comp. strain at outer conc. fibre \( 3500 \, \text{microstrain} \)
Comp. strain at steel area As1 \( 2556.2 \, \text{microstrain} \)
Tensile strain at steel area As4 \( 4126.8 \, \text{microstrain} \)
Depth of concrete in compression \( 204.19 \, \text{mm} \)

<table>
<thead>
<tr>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load (kN)</td>
<td>500</td>
<td>606.74</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance moment (kNm)</td>
<td>420</td>
<td>428.06</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Location: Column C2 (Ground-1st floor)

Analysis of circular section subjected to service bending moment and axial load, including crack-width calculation

(no tension stiffening)

Service-load calculations are based on the assumptions given in BS8110 (1985). Any tension stiffening is ignored. Solution is found by trial and adjustment, check calculations being printed out below. The concrete section is split into eight strips of equal depth. Eight steel reinforcing bars arranged as shown are assumed, paired to give four areas.

Diameter of section \( D = 800 \text{ mm} \)
Characteristic concrete strength \( f_{cu} = 30 \text{ N/mm}^2 \)
Characteristic steel strength \( f_y = 500 \text{ N/mm}^2 \)
Size of links \( \text{dial} = 8 \text{ mm} \)
Designated exposure class is XC2
Specified fixing tolerance \( \text{tol} = 10 \text{ mm} \)
Nominal cover to all steel \( \text{cover} = 35 \text{ mm} \)
Size of main bars \( \text{diac} = 25 \text{ mm} \)
Number of bars provided \( \text{num} = 8 \)
Determination of response of section

Applied axial service load \( N = 800 \text{ kN} \)
Applied service moment \( M = 500 \text{ kNm} \)

SUMMARY \( \text{fcu} = 30 \text{ N/mm}^2, \text{fy} = 500 \text{ N/mm}^2 \) 

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

<table>
<thead>
<tr>
<th>Diameter of section</th>
<th>800 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover to all reinforcement</td>
<td>35 mm</td>
</tr>
<tr>
<td>Size of main bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>Number of main bars</td>
<td>8</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>0.78125 %</td>
</tr>
<tr>
<td>Comp. strain at outer conc. fibre</td>
<td>859.87 microstrain</td>
</tr>
<tr>
<td>Strain at steel area As1</td>
<td>579.32 microstrain</td>
</tr>
<tr>
<td>Strain at steel area As4</td>
<td>-1605.9 microstrain</td>
</tr>
<tr>
<td>Depth of concrete in compression</td>
<td>250.48 mm</td>
</tr>
</tbody>
</table>

Forces and moments

<table>
<thead>
<tr>
<th>Forces and moments</th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>800</td>
<td>806.37</td>
<td>0.7</td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>500</td>
<td>502.02</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Crack width between two bottom bars is 0.59233 mm
Location: Ex1 - Column C2 (Ground-1st floor)

Analysis of circular section subjected to service bending moment and axial load, including crack-width calculation

(no tension stiffening)

Service-load calculations are based on the assumptions given in BS EN 1992-1-1:2004. Any tension stiffening is ignored. Solution is found by trial and adjustment, check calculations being printed out below. The concrete section is split into eight strips of equal depth. Eight steel reinforcing bars arranged as shown are assumed, paired to give four areas.

Diameter of section \( D = 800 \text{ mm} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Diameter of longitudinal reinft. \( \text{dia} = 25 \text{ mm} \)
Diameter of column links \( \text{dial} = 8 \text{ mm} \)
Number of bars provided \( \text{num} = 8 \)
### Determination of response of section

<table>
<thead>
<tr>
<th>Applied axial service load</th>
<th>N=800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied service moment</td>
<td>M=500 kNm</td>
</tr>
</tbody>
</table>

**SUMMARY ( fck=25 N/mm² : fyk=500 N/mm² )**

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

<table>
<thead>
<tr>
<th>Diameter of section</th>
<th>800 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover to all reinforcement</td>
<td>40 mm</td>
</tr>
<tr>
<td>Diameter of longitudinal reinft.</td>
<td>25 mm</td>
</tr>
<tr>
<td>Number of main bars</td>
<td>8</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>0.78125 %</td>
</tr>
<tr>
<td>Comp. strain at outer conc. fibre</td>
<td>1076 microstrain</td>
</tr>
<tr>
<td>Strain at steel area As1</td>
<td>738.85 microstrain</td>
</tr>
<tr>
<td>Strain at steel area As4</td>
<td>-1710.7 microstrain</td>
</tr>
<tr>
<td>Depth of concrete in compression</td>
<td>275.56 mm</td>
</tr>
<tr>
<td>Crack width between two bottom bars is</td>
<td>0.64624 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Forces and moments</th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>800</td>
<td>800.9</td>
<td>0.1</td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>500</td>
<td>500.29</td>
<td>0</td>
</tr>
</tbody>
</table>
Location: Column C2 (1st to 2nd floor)

Analysis of circular section subjected to service bending moment
and axial load, including crack-width assessment
(with tension stiffening)

Diameter of section D=800 mm
Characteristic concrete strength fcu=30 N/mm²
Characteristic steel strength fy=500 N/mm²
Size of links diac=8 mm
Designated exposure class is XC2
Specified fixing tolerance tol=10 mm
Nominal cover to all steel cover=35 mm
Size of main bars diac=25 mm
Number of bars provided num=8
Applied axial service load N=800 kN
Applied service moment M=500 kNm

SUMMARY (fcu=30 N/mm² : fy=500 N/mm² )

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

Diameter of section 800 mm
Cover to all reinforcement 35 mm
Size of main bars 25 mm
Number of main bars 8
Percentage of steel provided 0.78125 %
Comp.strain at outer conc.fibre 752.02 microstrain
Strain at steel area As1 529.93 microstrain
Strain at steel area As4 -1199.9 microstrain
Depth of concrete in compression 276.73 mm
Crack width between two bottom bars is 0.44061 mm

Forces and moments

<table>
<thead>
<tr>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>800</td>
<td>804.44</td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>500</td>
<td>501.27</td>
</tr>
</tbody>
</table>
Location: Small column: high steel proportion, load and moment

Analysis of circular section subjected to service bending moment
and axial load, including crack-width assessment

(with tension stiffening)

Service-load calculations are based on the assumptions given in BS8110 (1985), with tension stiffening as given in Part 2. Solution is found by trial and adjustment, check calculations being printed out below. The concrete section is split into eight strips of equal depth. Eight steel reinforcing bars arranged as shown are assumed, paired to give four areas.

Diameter of section $D=200$ mm
Characteristic concrete strength $f_{cu}=60$ N/mm$^2$
Characteristic steel strength $f_y=500$ N/mm$^2$
Size of links $d_{ia}=6$ mm
Designated exposure class is XC1
Specified fixing tolerance $t_{ol}=10$ mm
Nominal cover to all steel $c=25$ mm
Size of main bars $d_{ia}=16$ mm
Number of bars provided $n=8$
Applied axial service load $N=75$ kN
Applied service moment $M=29$ kNm

SUMMARY ($f_{cu}=60$ N/mm$^2 : f_y=500$ N/mm$^2$ )

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)
Diameter of section 200 mm
Cover to all reinforcement 25 mm
Size of main bars 16 mm
Number of main bars 8
Percentage of steel provided 5.12 %
Comp. strain at outer conc. fibre 1623.2 microstrain
Strain at steel area As1 703.45 microstrain
Strain at steel area As4 -1671.9 microstrain
Depth of concrete in compression 77.023 mm
Crack width between two bottom bars is 0.27558 mm

Forces and moments

<table>
<thead>
<tr>
<th></th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>75</td>
<td>75.837</td>
<td>1.1</td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>29</td>
<td>29.06</td>
<td>0.2</td>
</tr>
</tbody>
</table>
Location: Large column: minimum mild steel: axial load only

Analysis of circular section subjected to service bending moment and axial load, including crack-width assessment

(with tension stiffening)

Service-load calculations are based on the assumptions given in BS8110 (1985), with tension stiffening as given in Part 2. Solution is found by trial and adjustment, check calculations being printed out below. The concrete section is split into eight strips of equal depth. Eight steel reinforcing bars arranged as shown are assumed, paired to give four areas.

Diameter of section \(D=2000 \text{ mm}\)
Characteristic concrete strength \(f_{cu}=40 \text{ N/mm}^2\)
Characteristic steel strength \(f_y=250 \text{ N/mm}^2\)
Size of links \(\text{dial}=10 \text{ mm}\)
Designated exposure class is XC4
Specified fixing tolerance \(\text{tol}=10 \text{ mm}\)
Nominal cover to all steel \(\text{cover}=35 \text{ mm}\)
Size of main bars \(\text{diac}=40 \text{ mm}\)
Number of bars provided \(\text{num}=12\)
Applied axial service load \(N=85000 \text{ kN}\)
Applied service moment \(M=0 \text{ kNm}\)

SUMMARY \((f_{cu}=40 \text{ N/mm}^2 : f_y=250 \text{ N/mm}^2)\)

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

Diameter of section \(2000 \text{ mm}\)
Cover to all reinforcement \(35 \text{ mm}\)
Size of main bars \(40 \text{ mm}\)
Number of main bars \(12\)
Percentage of steel provided \(0.48 \%\)
Comp. strain at outer conc. fibre \(2012.9 \text{ microstrain}\)
Strain at steel area As1 \(1934.7 \text{ microstrain}\)
Strain at steel area As4 \(942 \text{ microstrain}\)
Entire section is in compression, therefore no cracking occurs.
Crack width between two bottom bars is 0 mm

Forces and moments

<table>
<thead>
<tr>
<th></th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>85000</td>
<td>84954</td>
<td>0</td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>1700</td>
<td>1700</td>
<td>0</td>
</tr>
</tbody>
</table>
Location: As Example No.2 but with tensile axial load

Analysis of circular section subjected to service bending moment
and axial load, including crack-width assessment

(with tension stiffening)

Service-load calculations are based on the assumptions given in BS8110 (1985), with tension stiffening as given in Part 2. Solution is found by trial and adjustment, check calculations being printed out below. The concrete section is split into eight strips of equal depth. Eight steel reinforcing bars arranged as shown are assumed, paired to give four areas.

Diameter of section                   $D=200$ mm
Characteristic concrete strength      $f_{cu}=60$ N/mm$^2$
Characteristic steel strength         $f_y=500$ N/mm$^2$
Size of links                         $\text{dial}=6$ mm
Designated exposure class is XC1
Specified fixing tolerance            $\text{tol}=10$ mm
Nominal cover to all steel            $\text{cover}=25$ mm
Size of main bars                     $\text{diac}=16$ mm
Number of bars provided               $\text{num}=8$
Applied axial service load            $N=-75$ kN
Applied service moment                $M=29$ kNm

SUMMARY ($f_{cu}=60$ N/mm$^2$ : $f_y=500$ N/mm$^2$ )

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

Diameter of section                   200 mm
Cover to all reinforcement            25 mm
Size of main bars                     16 mm
Number of main bars                   8
Percentage of steel provided           5.12 %
Comp. strain at outer conc. fibre     1624.4 microstrain
Strain at steel area As1              560.66 microstrain
Strain at steel area As4              -2186.6 microstrain
Depth of concrete in compression      66.646 mm
As strain es4 exceeds 0.8*$f_y/E_s$, Clause 3.8.3 of BS8110: Part 2 does not apply. Cracking may be critical.

Forces and moments
<table>
<thead>
<tr>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>-75</td>
<td>-74.47</td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>29</td>
<td>29.039</td>
</tr>
</tbody>
</table>
Location: Ex1 - Column C2 (1st to 2nd floor)

Analysis of circular section subjected to service bending moment and axial load, including crack-width assessment

(with tension stiffening)

Service-load calculations with tension stiffening are based on the assumptions given in EC2.

Solution is found by trial and adjustment, check calculations being printed out below. The concrete section is split into eight strips of equal depth. Eight steel reinforcing bars arranged as shown are assumed, paired to give four areas.

Diameter of section \( D = 800 \text{ mm} \)
Char yield strength of reinft. \( f_yk = 500 \text{ N/mm}^2 \)
Partial safety factor for steel \( \gamma_{m} = 1.15 \)
Diameter of longitudinal reinft. \( \text{dia} = 25 \text{ mm} \)
Diameter of column links \( \text{dia} = 8 \text{ mm} \)
Number of bars provided \( \text{num} = 8 \)
Applied axial service load \( N = 800 \text{ kN} \)
Applied service moment \( M = 500 \text{ kNm} \)

**SUMMARY ( \( fck=25 \text{ N/mm}^2 \ : \ fyk=500 \text{ N/mm}^2 \ ))**

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

Diameter of section \( 800 \text{ mm} \)
Cover to all reinforcement \( 40 \text{ mm} \)
Diameter of longitudinal reinft. \( 25 \text{ mm} \)
Number of main bars \( 8 \)
Percentage of steel provided \( 0.78125 \% \)
Comp.strain at outer conc.fibre \( 961.89 \) microstrain
Strain at steel area As1 \( 687.03 \) microstrain
Strain at steel area As4 \( -1309.9 \) microstrain
Depth of concrete in compression \( 302.16 \text{ mm} \)
Crack width between two bottom bars is \( 0.49308 \text{ mm} \)

<table>
<thead>
<tr>
<th>Forces and moments</th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>800</td>
<td>803.32</td>
<td>0.4</td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>500</td>
<td>500.97</td>
<td>0.1</td>
</tr>
</tbody>
</table>
Location: Ex2 - Small column: high steel proportion, load and moment

Analysis of circular section subjected to service bending moment
and axial load, including crack-width assessment

(with tension stiffening)

Diameter of section               $D=250$ mm
Char yield strength of reinft.    $f_{yk}=500$ N/mm²
Partial safety factor for steel   $g_{ams}=1.15$
Diameter of longitudinal reinft.  $d_{ia}=16$ mm
Diameter of column links          $d_{ial}=8$ mm
Number of bars provided           $n_{um}=8$
Applied axial service load        $N=75$ kN
Applied service moment           $M=29$ kN

SUMMARY ( $f_{ck}=50$ N/mm² : $f_{yk}=500$ N/mm² )

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)
Diameter of section               250 mm
Cover to all reinforcement         35 mm
Diameter of longitudinal reinft.   16 mm
Number of main bars               8
Percentage of steel provided       3.2768 %
Comp.strain at outer conc.fibre    943.4 microstrain
Strain at steel area $A_{s1}$      346.65 microstrain
Strain at steel area $A_{s4}$      -1094.2 microstrain
Depth of concrete in compression   89.53 mm
Crack width between two bottom bars is 0.24327 mm

Forces and moments     Specified    Calculated    Excess (%) 
Axial load in kN       75           75.752        1
Resistance moment in kNm 29           29.067        0.2
Location: Ex3 - Large column: minimum mild steel: axial load only

Analysis of circular section subjected to service bending moment and axial load, including crack-width assessment

(with tension stiffening)

Service-load calculations with tension stiffening are based on the assumptions given in EC2.

Solution is found by trial and adjustment, check calculations being printed out below. The concrete section is split into eight strips of equal depth. Eight steel reinforcing bars arranged as shown are assumed, paired to give four areas.

Diameter of section $D=2000$ mm
Char yield strength of reinft. $f_{yk}=500$ N/mm$^2$
Partial safety factor for steel $\gamma_{S}=1.15$
Diameter of longitudinal reinft. $\text{dia}=40$ mm
Diameter of column links $\text{dial}=10$ mm
Number of bars provided $\text{num}=25$
Applied axial service load $N=85000$ kN
Applied service moment $M=0$ kNm

SUMMARY ($f_{ck}=32$ N/mm$^2$ : $f_{yk}=500$ N/mm$^2$)

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

Diameter of section $2000$ mm
Cover to all reinforcement $50$ mm
Diameter of longitudinal reinft. $40$ mm
Number of main bars $25$
Percentage of steel provided $1\%$
Comp.strain at outer conc.fibre $2193.4$ microstrain
Strain at steel area As1 $2087.6$ microstrain
Strain at steel area As4 $888.5$ microstrain
Entire section is in compression, therefore no cracking occurs.
Crack width between two bottom bars is 0 mm

Forces and moments

<table>
<thead>
<tr>
<th></th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>85000</td>
<td>85003</td>
<td>0</td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>5661</td>
<td>5661.2</td>
<td>0</td>
</tr>
</tbody>
</table>
Location: Ex4 - As Example No.2 but with tensile axial load

Analysis of circular section subjected to service bending moment and axial load, including crack-width assessment

(with tension stiffening)

Service-load calculations with tension stiffening are based on the assumptions given in EC2.

Solution is found by trial and adjustment, check calculations being printed out below. The concrete section is split into eight strips of equal depth. Eight steel reinforcing bars arranged as shown are assumed, paired to give four areas.

Diameter of section D=250 mm
Char yield strength of reinft. fyk=500 N/mm²
Partial safety factor for steel gams=1.15
Diameter of longitudinal reinft. dia=16 mm
Diameter of column links dial=8 mm
Number of bars provided num=8
Applied axial service load N=-75 kN
Applied service moment M=29 kNm

SUMMARY ( fck=50 N/mm² : fyk=500 N/mm² )

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>-75</td>
<td>-74.573</td>
<td>0.5</td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>29</td>
<td>29.039</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Diameter of section 250 mm
Cover to all reinforcement 35 mm
Diameter of longitudinal reinft. 16 mm
Number of main bars 8
Percentage of steel provided 3.2768 %
Comp.strain at outer conc.fibre 992.86 microstrain
Strain at steel area As1 226.96 microstrain
Strain at steel area As4 -1622.2 microstrain
Depth of concrete in compression 73.415 mm
Crack width between two bottom bars is 0.3478 mm
Location: Typical column section reinforced with high-yield steel

Design of rectangular section for uniaxial bending and thrust

(using parabolic-rectangular stress-strain curve for concrete)

Design to BS8110(1997), partial safety factor for steel gammaS=1.15

Calculations based on assumptions given in BS 8110. Solution is found by trial and adjustment, with check calculations printed out beneath.

The concrete section is divided into eight strips of equal depth. Two symmetrically-placed layers of steel are assumed. (Only positive values of moment are valid.)

Design axial load \( N = 5000 \) kN
Design moment \( M = 1500 \) kNm
Section depth \( h = 900 \) mm
Section breadth \( b = 600 \) mm
Size of links \( \text{dial}=8 \) mm
Characteristic concrete strength \( f_{cu} = 30 \) N/mm²
Characteristic steel strength \( f_y = 500 \) N/mm²
Designated exposure class is XC2
Specified fixing tolerance \( \text{tol}=10 \) mm
Nominal cover to all steel \( \text{cover}=35 \) mm

Distances to centroids of strips
- \( d_1 = 0.4375 \times h = 393.75 \) mm
- \( d_2 = 0.3125 \times h = 281.25 \) mm
- \( d_3 = 0.1875 \times h = 168.75 \) mm
- \( d_4 = 0.0625 \times h = 56.25 \) mm
- \( d_5 = -0.0625 \times h = -56.25 \) mm
- \( d_6 = -0.1875 \times h = -168.75 \) mm
- \( d_7 = -0.3125 \times h = -281.25 \) mm
- \( d_8 = -0.4375 \times h = -393.75 \) mm

The forces in the strips are found by integrating the parabolic stress-strain curve. If a strip is partly in compression, the force is still assumed to act at the centroid positions as calculated above.
Calculation of strains and forces in concrete strips

Max. stress on stress-strain curve $f_u=0.444*f_{cu}=13.32 \text{ N/mm}^2$.

As strain at bottom $eb$ exceeds $e_1$, rectangular part of curve applies and
force on strip $f=A*f_u/1E3=899.1 \text{ kN}$

As strain at bottom $eb$ exceeds $e_1$, rectangular part of curve applies and
force on strip $f=A*f_u/1E3=899.1 \text{ kN}$

As strain at bottom $eb$ exceeds $e_1$, rectangular part of curve applies and
force on strip $f=A*f_u/1E3=899.1 \text{ kN}$

As strain at top of strip $et$ exceeds $e_1$, but strain at bottom $eb$ is
less than $e_1$, both parabolic and rectangular parts of curve apply. Thus
stress at strip bottom $f_{b}=f_u*(2*eb/e_1-eb^2/e_1^2)$

$=13.298 \text{ N/mm}^2$, and

force on strip $f=A*f_u*(1-0.333*(1-f_{b}/f_u)*(e_1-eb)/(et-eb))/1E3$

$=899.06 \text{ kN}$

As strain at top of strip $et$ does not exceed $e_1$, parabolic part of
stress-strain curve applies, and

force on strip $f=A*f_u*((et+eb)/e_1-(et^2+et*eb+eb^2)/e_1^2/3)$

$=777.21 \text{ kN}$

Strain at strip bottom is negative; only part of strip in compression.

As strain at top of strip $et$ does not exceed $e_1$,

force on strip $f=A*et/(et-eb)*f_u*(et/e_1-((et/e_1)^2)/3)/1E3$

$=200.59 \text{ kN}$

Size of main bars $diac=25$

---

**SUMMARY ( $f_{cu}=30 \text{ N/mm}^2$ : $f_y=500 \text{ N/mm}^2$ )**

(Notes: 'microstrain' indicates true strain multiplied by 1,000,000.)

<table>
<thead>
<tr>
<th>Section dimensions</th>
<th>900 mm deep by 600 mm wide</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover to all reinforcement</td>
<td>35 mm</td>
</tr>
<tr>
<td>Size of main bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>Number of main bars</td>
<td>14</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>6872.2 mm$^2$</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>1.2726 %</td>
</tr>
<tr>
<td>Size of links provided</td>
<td>8 mm</td>
</tr>
<tr>
<td>Comp. strain at outer conc. fibre</td>
<td>3500 microstrain</td>
</tr>
<tr>
<td>Comp. strain at steel area As1</td>
<td>3195.3 microstrain</td>
</tr>
<tr>
<td>Tensile strain at steel area As2</td>
<td>1136.4 microstrain</td>
</tr>
<tr>
<td>Depth of concrete in compression</td>
<td>637.51 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Axial load in kN</th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5000</td>
<td></td>
<td>5287.2</td>
<td>5.7</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Resistance moment in kNm | 1500 | 1629.1 | 8.6 |
| Satisfactory            |      |        |     |

The above result is judged acceptable.
Location: Small column section, low axial load & substantial moment

Design of rectangular section for uniaxial bending and thrust

(using parabolic-rectangular stress-strain curve for concrete)

Design to BS8110(1997), partial safety factor for steel $\gamma_s = 1.15$

Calculations based on assumptions given in BS 8110. Solution is found by trial and adjustment, with check calculations printed out beneath. The concrete section is divided into eight strips of equal depth. Two symmetrically-placed layers of steel are assumed. (Only positive values of moment are valid.)

Design axial load $N = 250 \text{ kN}$
Design moment $M = 450 \text{ kNm}$
Section depth $h = 500 \text{ mm}$
Section breadth $b = 400 \text{ mm}$
Size of links $\text{dial} = 8 \text{ mm}$
Characteristic concrete strength $f_{cu} = 60 \text{ N/mm}^2$
Characteristic steel strength $f_y = 500 \text{ N/mm}^2$
Nominal cover to all steel $\text{cover} = 30 \text{ mm}$

**Distances to centroids of strips**

\[
\begin{align*}
\text{d1} &= 0.4375h = 218.75 \text{ mm} \\
\text{d2} &= 0.3125h = 156.25 \text{ mm} \\
\text{d3} &= 0.1875h = 93.75 \text{ mm} \\
\text{d4} &= 0.0625h = 31.25 \text{ mm} \\
\text{d5} &= -0.0625h = -31.25 \text{ mm} \\
\text{d6} &= -0.1875h = -93.75 \text{ mm} \\
\text{d7} &= -0.3125h = -156.25 \text{ mm} \\
\text{d8} &= -0.4375h = -218.75 \text{ mm}
\end{align*}
\]

The forces in the strips are found by integrating the parabolic stress-strain curve. If a strip is partly in compression, the force is still assumed to act at the centroid positions as calculated above.
Calculation of strains and forces in concrete strips

Max. stress on stress-strain curve \( f_u = 0.444 \times f_{cu} = 26.64 \, \text{N/mm}^2 \).

As strain at top of strip \( e_t \) exceeds \( e_1 \), but strain at bottom \( e_b \) is less than \( e_1 \), both parabolic and rectangular parts of curve apply. Thus

stress at strip bottom \( f_b = f_u \times \left(2 \times e_b / e_1 - e_b^2 / e_1^2\right) \)

= 19.632 \, \text{N/mm}^2, \text{ and}

force on strip \( f = A \times f_u \times \left(1 - 0.333 \times (1 - f_b / f_u) \times (e_1 - e_b) / (e_t - e_b)\right) / 1E3 \)

= 649.55 \, \text{kN}

Strain at strip bottom is negative; only part of strip in compression.

As strain at top of strip \( e_t \) does not exceed \( e_1 \),

force on strip \( f = A \times e_t / (e_t - e_b) \times f_u \times (e_t / e_1 - ((e_t / e_1)^2)/3) / 1E3 \)

= 72.78 \, \text{kN}

Size of main bars \( d_{ac} = 25 \)

SUMMARY (\( f_{cu} = 60 \, \text{N/mm}^2 \) : \( f_y = 500 \, \text{N/mm}^2 \))

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

<table>
<thead>
<tr>
<th>Section dimensions</th>
<th>500 mm deep by 400 mm wide</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover to all reinforcement</td>
<td>30 mm</td>
</tr>
<tr>
<td>Size of main bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>Number of main bars</td>
<td>10</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>4908.7 mm²</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>2.4544 %</td>
</tr>
<tr>
<td>Size of links provided</td>
<td>8 mm</td>
</tr>
<tr>
<td>Comp. strain at outer conc. fibre</td>
<td>3500 microstrain</td>
</tr>
<tr>
<td>Comp. strain at steel area As1</td>
<td>1269.4 microstrain</td>
</tr>
<tr>
<td>Tensile strain at steel area As2</td>
<td>16354 microstrain</td>
</tr>
<tr>
<td>Depth of concrete in compression</td>
<td>79.239 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>250</td>
<td>278.33</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>450</td>
<td>490.66</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Location: Substantial column section reinforced with mild steel

Design of rectangular section for uniaxial bending and thrust
(using parabolic-rectangular stress-strain curve for concrete)

Design to BS8110(1997), partial safety factor for steel gammaS=1.15

Calculations based on assumptions given in BS 8110. Solution is found by trial and adjustment, with check calculations printed out beneath. The concrete section is divided into eight strips of equal depth. Two symmetrically-placed layers of steel are assumed. (Only positive values of moment are valid.)

Design axial load \( N=200 \text{ kN} \)
Design moment \( M=1000 \text{ kNm} \)
Section depth \( h=1500 \text{ mm} \)
Section breadth \( b=1000 \text{ mm} \)
Size of links \( \text{dial}=16 \text{ mm} \)
Characteristic concrete strength \( f_{cu}=25 \text{ N/mm}^2 \)
Characteristic steel strength \( f_{y}=250 \text{ N/mm}^2 \)
Designated exposure class is XC1
Specified fixing tolerance \( \text{tol}=10 \text{ mm} \)
Nominal cover to all steel \( \text{cover}=25 \text{ mm} \)

Distances to centroids of strips
\[
\begin{align*}
d_1 &= 0.4375 \times h = 656.25 \text{ mm} \\
d_2 &= 0.3125 \times h = 468.75 \text{ mm} \\
d_3 &= 0.1875 \times h = 281.25 \text{ mm} \\
d_4 &= 0.0625 \times h = 93.75 \text{ mm} \\
d_5 &= -0.0625 \times h = -93.75 \text{ mm} \\
d_6 &= -0.1875 \times h = -281.25 \text{ mm} \\
d_7 &= -0.3125 \times h = -468.75 \text{ mm} \\
d_8 &= -0.4375 \times h = -656.25 \text{ mm}
\end{align*}
\]

The forces in the strips are found by integrating the parabolic stress-strain curve. If a strip is partly in compression, the force is still assumed to act at the centroid positions as calculated above.

Calculation of strains and forces in concrete strips

Max. stress on stress-strain curve \( f_{u}=0.444 \times f_{cu}=11.1 \text{ N/mm}^2 \).
Strain at strip bottom is negative; only part of strip in compression. As strain at top of strip exceeds \( e_l \), force on strip \( f=A \times e_t/(e_t-\text{eb}) \times f_{u}(1-0.333 \times e_l/e_t)/1E3=645.77 \text{ kN} \)
Size of main bars \( \text{dias}=16 \)
SUMMARY \( (f_{cu}=25 \text{ N/mm}^2 : f_{y}=250 \text{ N/mm}^2) \)

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

<table>
<thead>
<tr>
<th>Section dimensions</th>
<th>1500 mm deep by 1000 mm wide</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover to all reinforcement</td>
<td>25 mm</td>
</tr>
<tr>
<td>Size of main bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of main bars</td>
<td>30</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>6031.9 mm²</td>
</tr>
<tr>
<td>Percentage of steel provided</td>
<td>0.40212 %</td>
</tr>
<tr>
<td>Size of links provided</td>
<td>16 mm</td>
</tr>
<tr>
<td>Comp.strain at outer conc.fibre</td>
<td>1289.2 microstrain</td>
</tr>
<tr>
<td>Comp.strain at steel area As1</td>
<td>570.24 microstrain</td>
</tr>
<tr>
<td>Tensile strain at steel area As2</td>
<td>20000 microstrain</td>
</tr>
<tr>
<td>Depth of concrete in compression</td>
<td>87.866 mm</td>
</tr>
</tbody>
</table>

specified | calculated | excess (%) |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>200</td>
<td>334.09</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>1000</td>
<td>1124.5</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Section reinforced with the min permissible steel area of 0.4% can support the specified loading shown, and is accepted as the design.
Location: Typical column section reinforced with high-yield steel

Design of rectangular section for uniaxial bending and thrust

(using parabolic-rectangular stress-strain curve for concrete)

Calculations based on assumptions given in EC2. Solution is found by trial and adjustment, with check calculations printed out beneath. The concrete section is divided into eight strips of equal depth. Two symmetrically-placed layers of steel are assumed (only positive values of moment are valid).

Design axial load $N=5000$ kN
Design moment $M=1500$ kNm
Section depth $h=900$ mm
Section breadth $b=600$ mm
Partial safety factor for steel $\gamma_{ms}=1.15$
Characteristic steel strength $f_{yk}=500$ N/mm$^2$
Diameter of longitudinal reinft. $dia=25$ mm
Diameter of column links $dial=8$ mm
Diameter of longitudinal reint. $diac=25$

**SUMMARY** ($f_{ck}=25$ N/mm$^2$ : $f_{yk}=500$ N/mm$^2$ )

<table>
<thead>
<tr>
<th>Specification</th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>5000</td>
<td>5309.7</td>
<td>6.1</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>1500</td>
<td>1623.5</td>
<td>8.2</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Location: Small column section, low axial load & substantial moment

Design of rectangular section for uniaxial bending and thrust

(using parabolic-rectangular stress-strain curve for concrete)

Calculations based on assumptions given in EC2. Solution is found by trial and adjustment, with check calculations printed out beneath. The concrete section is divided into eight strips of equal depth. Two symmetrically-placed layers of steel are assumed (only positive values of moment are valid).

Design axial load N=175 kN
Design moment M=450 kNm
Section depth h=500 mm
Section breadth b=400 mm
Partial safety factor for steel gams=1.15
Characteristic steel strength fyk=500 N/mm²
Diameter of column links dial=8 mm
Nominal cover to all steel cover=30 mm
Diameter of longitudinal reint. diac=25

SUMMARY ( fck=50 N/mm² : fyk=500 N/mm² )

(Note: 'microstrain' indicates true strain multiplied by 1,000,000.)

Section dimensions 500 mm deep by 400 mm wide
Cover to all reinforcement 30 mm
Diameter of longitudinal reintft. 25 mm
Number of main bars 10
Area of steel provided 4908.7 mm²
Percentage of steel provided 2.4544 %
Size of links provided 8 mm
Comp.strain at outer conc.fibre 3500 microstrain
Comp.strain at steel area As1 1225.1 microstrain
Tensile strain at steel area As2 16749 microstrain
Depth of concrete in compression 77.696 mm

Axial load in kN Specified Calculated Excess (%) 175 204.53 16.8
Satisfactory
Resistance moment in kNm 450 476.42 5.8
Satisfactory

The above result is judged acceptable.
Location: Substantial column section reinforced with mild steel

Design of rectangular section for uniaxial bending and thrust
(using parabolic-rectangular stress-strain curve for concrete)

Calculations based on assumptions given in EC2. Solution is found by trial and adjustment, with check calculations printed out beneath. The concrete section is divided into eight strips of equal depth. Two symmetrically-placed layers of steel are assumed (only positive values of moment are valid).

Design axial load \( N = 200 \, \text{kN} \)
Design moment \( M = 10 \, \text{kNm} \)
Section depth \( h = 1500 \, \text{mm} \)
Section breadth \( b = 1000 \, \text{mm} \)
Partial safety factor for steel \( \gamma_{am} = 1.15 \)
Characteristic steel strength \( f_{yk} = 500 \, \text{N/mm}^2 \)
Diameter of longitudinal reinft. \( d_{ia} = 20 \, \text{mm} \)
Diameter of column links \( d_{ial} = 8 \, \text{mm} \)
Diameter of longitudinal reint. \( d_{ia} = 20 \)

**SUMMARY** \(( f_{ck} = 25 \, \text{N/mm}^2 : f_{yk} = 500 \, \text{N/mm}^2 \) \)

<table>
<thead>
<tr>
<th>Specification</th>
<th>Specified</th>
<th>Calculated</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>200</td>
<td>19686</td>
<td>9743.1</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance moment in kNm</td>
<td>10</td>
<td>996.44</td>
<td>9864.4</td>
</tr>
<tr>
<td>Satisfactory</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Section reinforced with the min permissible steel area of 0.2% can support the specified loading shown, and is accepted as the design.
Location: Column with biaxial bending and substantial axial load

Design of rectangular section under biaxial bending and axial load

(rigorous procedure using trial and error)

Design to BS8110(1997), partial safety factor for steel γ_s=1.15
Calculations are based on assumptions given in BS 8110. Solution is by trial and error, with check calculations printed out below.
The concrete section is divided into 25 elements of equal area.
Moments must be applied to give tension in lower left corner, with dominant moment about X-X axis.

Design axial load N=3000 kN
Design moment about X-X axis M_x=300 kNm
Design moment about Y-Y axis M_y=250 kNm
Depth of section h=500 mm
Breadth of section b=600 m
Size of links dia=8 mm
Characteristic conc strength f_{cu}=30 N/mm²
Characteristic steel strength f_y=500 N/mm²
Designated exposure class is XC2
Specified fixing tolerance tol=10 mm
Nominal cover to all steel cover=35 mm
Element strains, forces, and contributions to mts about x-x and y-y

\[
e = 0.98101 \times 10^{-3}, 0.0014763, 0.0019715, 0.0024668, 0.002962
\]
\[
f = 158.66, 159.84, 159.84, 159.84, 159.84
\]
\[
m_x = 31.732, 31.968, 31.968, 31.968, 31.968
\]
\[
m_y = -38.078, -19.181, 0, 19.181, 38.362
\]

Calculation of strains and forces in steel areas

Steel arrangement:

Type 2

Steel area provided
\[A_{stot} = \text{num} \times \pi \times \text{diac}^2 / 4 = 4092.2 \text{ mm}^2\]
Steel percentage provided
\[\text{perct} = 100 \times A_{stot} / (h \times b) = 1.3641\%\]
Distances to Bar 1
\[x = -242.27 \text{ mm} = -192.27 \text{ mm}\]
Strain at bar
\[e = 0.0035 - \text{curvx} \times (h/2 - y) - \text{curvy} \times (b/2 - x) - 0.0013062\]
Steel is elastic, force
\[f = A_{sb} \times e \times 200 = -178.17 \text{ kN}\]
Similar calculations for remaining bars are summarised as follows:

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Strain</th>
<th>Force (kN)</th>
<th>mx (kNm)</th>
<th>my (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.0013062</td>
<td>-178.17</td>
<td>34.256</td>
<td>43.165</td>
</tr>
<tr>
<td>2</td>
<td>0.92674E-3</td>
<td>126.41</td>
<td>24.305</td>
<td>-30.626</td>
</tr>
<tr>
<td>3</td>
<td>0.0019266</td>
<td>262.8</td>
<td>50.528</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>0.0029265</td>
<td>296.54</td>
<td>57.014</td>
<td>71.84</td>
</tr>
<tr>
<td>5</td>
<td>0.69354E-3</td>
<td>94.603</td>
<td>-18.189</td>
<td>22.919</td>
</tr>
<tr>
<td>6</td>
<td>-0.30632E-3</td>
<td>-41.784</td>
<td>8.0337</td>
<td>0</td>
</tr>
</tbody>
</table>

Summing forces and moments for all elements and bars gives:

- Calculated axial force \( N_c = 3177.9 \text{ kN} \)
- Moment about \( x-x \) axis \( M_{cx} = 317.62 \text{ kNm} \)
- Moment about \( y-y \) axis \( M_{cy} = 263.54 \text{ kNm} \)

**SUMMARY** (\( f_{cu} = 30 \text{ N/mm}^2 \), \( f_y = 500 \text{ N/mm}^2 \))

(Note: 'microstrain' = true strain multiplied by 1,000,000.)

Steel percentage required 1.3641 %
Steel percentage provided 1.3641 %
Provide the equivalent of 6 No. 29.468 mm bars
Total steel area required 4092.2 mm²
Total steel area provided 4092.2 mm²
Cover to all reinforcement 35 mm
Strain at corner Bar 1 -1306.2 microstrain (tension -ve)
Strain at top right corner 3500 microstrain
Strain at top left corner 1023.7 microstrain

Forces and moments:
<table>
<thead>
<tr>
<th>Required</th>
<th>Provided</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN 3000</td>
<td>3177.9</td>
<td>5.9</td>
</tr>
<tr>
<td>Moment about ( X-X ) axis in kNm 300</td>
<td>317.62</td>
<td>5.8</td>
</tr>
<tr>
<td>Moment about ( Y-Y ) axis in kNm 250</td>
<td>263.54</td>
<td>5.4</td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Design of rectangular section under biaxial bending and axial load

(rigorous procedure using trial and error)

Design to BS8110(1997), partial safety factor for steel $\gamma_S=1.15$
Calculations are based on assumptions given in BS 8110. Solution is by trial and error, with check calculations printed out below.
The concrete section is divided into 25 elements of equal area.

Moments must be applied to give tension in lower left corner, with dominant moment about X-X axis.

- Design axial load $N=1000$ kN
- Design moment about X-X axis $M_x=200$ kNm
- Design moment about Y-Y axis $M_y=150$ kNm
- Depth of section $h=500$ mm
- Breadth of section $b=500$ m
- Size of links $dial=8$ mm
- Characteristic conc strength $f_{cu}=50$ N/mm$^2$
- Characteristic steel strength $f_y=250$ N/mm$^2$
- Designated exposure class is XC3
- Specified fixing tolerance $tol=10$ mm
- Nominal cover to all steel $cover=30$ mm
Element strains, forces, and contributions to mts about x-x and y-y

<table>
<thead>
<tr>
<th>e</th>
<th>f</th>
<th>mx</th>
<th>my</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.60871E-3</td>
<td>0.18522E-3</td>
<td>0.97915E-3</td>
<td>0.0017731</td>
</tr>
<tr>
<td>55.383</td>
<td>202.89</td>
<td>222</td>
<td>222</td>
</tr>
<tr>
<td>-0.0016808</td>
<td>-0.88684E-3</td>
<td>-92.913E-6</td>
<td>0.70102E-3</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>167.81</td>
<td>222</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>16.781</td>
<td>22.2</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>16.781</td>
<td>44.4</td>
</tr>
</tbody>
</table>

Calculation of strains and forces in steel areas

Steel arrangement:

```
Type 4          2
3          6
1          7
```

Steel area provided: \( A_{stot} = \frac{\pi \times d_{ac}^2}{4} = 1870.2 \text{ mm}^2 \)

Steel percentage provided: \( \text{perct} = 100 \times A_{stot} / (h \times b) = 0.74807\% \)

Distances to Bar 1: \( x = -203.37 \text{ mm} = -203.37 \text{ mm} \)

Strain at bar: \( e = 0.0035 - \text{curv}_x \times (h/2 - y) - \text{curv}_y \times (b/2 - x) = -0.0049599 \)

Steel is plastic, force: \( f = -A_{sb} \times f_y / (\gamma_S \times 1E3) = -50.82 \text{ kN} \)
Similar calculations for remaining bars are summarised as follows:

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Strain</th>
<th>Force (kN)</th>
<th>mx (kNm)</th>
<th>my (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.00495999</td>
<td>-50.82</td>
<td>10.335</td>
<td>10.335</td>
</tr>
<tr>
<td>2</td>
<td>-0.0027796</td>
<td>-50.82</td>
<td>0</td>
<td>10.335</td>
</tr>
<tr>
<td>3</td>
<td>-0.59933E-3</td>
<td>-28.021</td>
<td>-5.6988</td>
<td>5.6988</td>
</tr>
<tr>
<td>4</td>
<td>0.0010153</td>
<td>47.471</td>
<td>9.6543</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0.00263</td>
<td>50.82</td>
<td>10.335</td>
<td>10.335</td>
</tr>
<tr>
<td>6</td>
<td>0.44967E-3</td>
<td>21.024</td>
<td>0</td>
<td>4.2758</td>
</tr>
<tr>
<td>7</td>
<td>-0.0017306</td>
<td>-50.82</td>
<td>10.335</td>
<td>-10.335</td>
</tr>
<tr>
<td>8</td>
<td>-0.0033453</td>
<td>-50.82</td>
<td>10.335</td>
<td>0</td>
</tr>
</tbody>
</table>

Summing forces and moments for all elements and bars gives

Calculated axial force \( N_c = 1094.9 \text{ kN} \)
Moment about x-x axis \( M_{cx} = 224.73 \text{ kNm} \)
Moment about y-y axis \( M_{cy} = 175.85 \text{ kNm} \)

**SUMMARY** (\( f_{cu} = 50 \text{ N/mm}^2 \) : \( f_y = 250 \text{ N/mm}^2 \))

(Note: 'microstrain' = true strain multiplied by 1,000,000.)

Steel percentage required \( 0.74807 \% \)
Steel percentage provided \( 0.74807 \% \)
Provide the equivalent of 8 No. 17.252 mm bars
Total steel area required \( 1870.2 \text{ mm}^2 \)
Total steel area provided \( 1870.2 \text{ mm}^2 \)
Cover to all reinforcement \( 30 \text{ mm} \)
Strain at corner Bar 1 \(-4959.9 \text{ microstrain (tension -ve)} \)
Strain at top right corner \( 3500 \text{ microstrain} \)
Strain at top left corner \( -469.65 \text{ microstrain} \)
Forces and moments: Required Provided Excess (%)
Axial load in kN 1000 1094.9 9.4
Moment about X-X axis in kNm 200 224.73 12.3
Moment about Y-Y axis in kNm 150 175.85 17.2

The above result is judged acceptable.
Location: Column with low axial load

Design of rectangular section under biaxial bending and axial load

(rigorous procedure using trial and error)

Design to BS 8110(1997), partial safety factor for steel $\gamma_S=1.15$
Calculations are based on assumptions

<table>
<thead>
<tr>
<th>Y</th>
<th>Ulit. conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Y</td>
<td>governed by conc.</td>
</tr>
<tr>
<td>X</td>
<td>or steel strain</td>
</tr>
</tbody>
</table>

The concrete section is divided into 25 elements of equal area.

Moments must be applied to give tension in lower left corner, with dominant moment about X-X axis.

- Design axial load $N=100 \text{ kN}$
- Design moment about X-X axis $M_x=350 \text{ kNm}$
- Design moment about Y-Y axis $M_y=350 \text{ kNm}$
- Depth of section $h=550 \text{ mm}$
- Breadth of section $b=550 \text{ m}$
- Size of links $dial=10 \text{ mm}$
- Characteristic conc strength $f_{cu}=35 \text{ N/mm}^2$
- Characteristic steel strength $f_{y}=500 \text{ N/mm}^2$
- Nominal cover to all steel $cover=30 \text{ mm}$
Element strains, forces, and contributions to mts about x-x and y-y

<table>
<thead>
<tr>
<th>e=-0.0012505 -0.30037E-3 0.64973E-3 0.0015998 0.0025499</th>
<th>f=0          0     151.7      188.03     188.03</th>
</tr>
</thead>
<tbody>
<tr>
<td>mx=0          0     0           33.375      41.367      41.367</td>
<td></td>
</tr>
<tr>
<td>my=0          0     0           0           20.684      41.367</td>
<td></td>
</tr>
<tr>
<td>----------------------------------------------------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td>-0.0022005 -0.0012505 -0.30037E-3 0.64973E-3 0.0015998</td>
<td></td>
</tr>
<tr>
<td>----------------------------------------------------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td>0             0     0           151.7     188.03</td>
<td></td>
</tr>
<tr>
<td>0             0     0           16.687     20.684</td>
<td></td>
</tr>
<tr>
<td>----------------------------------------------------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td>0             0     0           0           16.687     41.367</td>
<td></td>
</tr>
</tbody>
</table>

Calculation of strains and forces in steel areas

Steel arrangement:

```
Type 4          2
1 o o o 6   (bar dia =32.407 mm
    and Asb =824.83 mm²)
```  

Steel area provided $\text{Astot} = \text{num} \times \pi \times \text{diac}^2 / 4 = 6598.6 \text{ mm}^2$

Steel percentage provided $\text{perct} = 100 \times \text{Astot} / (h \times b) = 2.1814 \%$

Distances to Bar 1 $x=-218.8 \text{ mm}\ y=-218.8 \text{ mm}$

Strain at bar $e=0.0035 -\text{curvx} \times (h/2-y) -\text{curvy} \times (b/2-x) = -0.00503$

Steel is plastic, force $f=-\text{Asb} \times \text{fy} / (\text{gammaS} \times 1 \times 10^3) = -358.62 \text{ kN}$
Similar calculations for remaining bars are summarised as follows:

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Strain</th>
<th>Force (kN)</th>
<th>mx (kNm)</th>
<th>my (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.00503</td>
<td>-358.62</td>
<td>78.465</td>
<td>78.465</td>
</tr>
<tr>
<td>2</td>
<td>-0.0031402</td>
<td>-358.62</td>
<td>0</td>
<td>78.465</td>
</tr>
<tr>
<td>3</td>
<td>-0.0012505</td>
<td>-206.28</td>
<td>-45.134</td>
<td>45.134</td>
</tr>
<tr>
<td>4</td>
<td>0.63933E-3</td>
<td>105.47</td>
<td>23.076</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0.0025291</td>
<td>358.62</td>
<td>78.465</td>
<td>78.465</td>
</tr>
<tr>
<td>6</td>
<td>0.63933E-3</td>
<td>105.47</td>
<td>0</td>
<td>23.076</td>
</tr>
<tr>
<td>7</td>
<td>-0.0012505</td>
<td>-206.28</td>
<td>45.134</td>
<td>-45.134</td>
</tr>
<tr>
<td>8</td>
<td>-0.0031402</td>
<td>-358.62</td>
<td>78.465</td>
<td>0</td>
</tr>
</tbody>
</table>

Summing forces and moments for all elements and bars gives:

Calculated axial force \( N_c = 100.34 \) kN
Moment about x-x axis \( M_{cx} = 411.95 \) kNm
Moment about y-y axis \( M_{cy} = 411.95 \) kNm

**SUMMARY** (\( f_{cu} = 35 \) N/mm² : \( f_y = 500 \) N/mm²)

(Note: 'microstrain' = true strain multiplied by 1,000,000.)

4

Section chosen

<table>
<thead>
<tr>
<th></th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>o</td>
<td>o</td>
<td>o</td>
</tr>
<tr>
<td>(same bars in all faces)</td>
<td>2</td>
<td>o</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>o</td>
<td>o</td>
<td>7</td>
</tr>
</tbody>
</table>

Depth of section \( h = 550 \) mm
Breadth of section \( b = 550 \) mm
Equiv. bar diameter = 32.407 mm

Steel percentage required 2.1814 %
Steel percentage provided 2.1814 %

Provide the equivalent of 8 No. 32.407 mm bars

Total steel area required 6598.6 mm²
Total steel area provided 6598.6 mm²

Cover to all reinforcement 30 mm

Strain at corner Bar 1 -5030 microstrain (tension -ve)
Strain at top right corner 3500 microstrain
Strain at top left corner -1250.5 microstrain

Forces and moments:

<table>
<thead>
<tr>
<th></th>
<th>Required</th>
<th>Provided</th>
<th>Excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>100</td>
<td>100.34</td>
<td>0.3</td>
</tr>
<tr>
<td>Moment about X-X axis in kNm</td>
<td>350</td>
<td>411.95</td>
<td>17.7</td>
</tr>
<tr>
<td>Moment about Y-Y axis in kNm</td>
<td>350</td>
<td>411.95</td>
<td>17.7</td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Location: Column with biaxial bending and substantial axial load

Design of rectangular section under biaxial bending and axial load

(rigorous procedure using trial and error)

Calculations are based on assumptions given in EC2. Solution is by trial and error, with check calculations printed out below. The concrete section is divided into 25 elements of equal area.

Moments must be applied to give tension in lower left corner, with dominant moment about Y-Y axis.

Design axial load \( M = 3000 \text{ kN} \)
Design moment about Y-Y axis \( M_y = 300 \text{ kNm} \)
Design moment about Z-Z axis \( M_z = 250 \text{ kNm} \)
Depth of section \( h = 500 \text{ mm} \)
Breadth of section \( b = 600 \text{ m} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Characteristic steel strength \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of longitudinal reinft. \( \text{dia} = 32 \text{ mm} \)
Diameter of column links \( \text{dial} = 8 \text{ mm} \)
Calculation of strains and forces in steel areas

Steel arrangement:

Type 4

Steel area provided

Steel percentage provided

Distances to Bar 1

Strain at bar

Steel is elastic, force
Similar calculations for remaining bars are summarised as follows:

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Strain</th>
<th>Force (kN)</th>
<th>my (kNm)</th>
<th>mz (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.0010517</td>
<td>-132.75</td>
<td>24.137</td>
<td>30.774</td>
</tr>
<tr>
<td>2</td>
<td>58.476E-6</td>
<td>7.3808</td>
<td>0</td>
<td>-1.711</td>
</tr>
<tr>
<td>3</td>
<td>0.0011687</td>
<td>147.51</td>
<td>26.821</td>
<td>-34.197</td>
</tr>
<tr>
<td>4</td>
<td>0.0020035</td>
<td>252.87</td>
<td>45.979</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0.0028383</td>
<td>274.39</td>
<td>49.891</td>
<td>63.611</td>
</tr>
<tr>
<td>6</td>
<td>0.0017281</td>
<td>218.11</td>
<td>0</td>
<td>50.564</td>
</tr>
<tr>
<td>7</td>
<td>0.61785E-3</td>
<td>77.984</td>
<td>-14.179</td>
<td>18.079</td>
</tr>
<tr>
<td>8</td>
<td>-0.21694E-3</td>
<td>-27.382</td>
<td>4.9788</td>
<td>0</td>
</tr>
</tbody>
</table>

Summing forces and moments for all elements and bars gives

Calculated axial force \( N_c = 3108.5 \text{ kN} \)
Moment about y-y axis \( M_{cy} = 315.5 \text{ kNm} \)
Moment about z-z axis \( M_{cz} = 263.12 \text{ kNm} \)

SUMMARY (\( f_{ck} = 25 \text{ N/mm}^2 \): \( f_{yk} = 500 \text{ N/mm}^2 \))

(Note: 'microstrain' = true strain multiplied by 1,000,000.)

Steel percentage required \( 1.6829 \% \)
Steel percentage provided \( 1.6829 \% \)
Provide the equivalent of 8 No. 28.347 mm diameter bars
Total steel area required \( 5048.7 \text{ mm}^2 \)
Total steel area provided \( 5048.7 \text{ mm}^2 \)
Cover to all reinforcement \( 46 \text{ mm} \)
Strain at corner Bar 1 \( -1051.7 \text{ microstrain (tension -ve)} \)
Strain at top right corner \( 3500 \text{ microstrain} \)
Strain at top left corner \( 1339.5 \text{ microstrain} \)
Forces and moments: 
<table>
<thead>
<tr>
<th></th>
<th>required</th>
<th>provided</th>
<th>excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>3000</td>
<td>3108.5</td>
<td>3.6</td>
</tr>
<tr>
<td>Moment about Y-Y axis in kNm</td>
<td>300</td>
<td>315.5</td>
<td>5.1</td>
</tr>
<tr>
<td>Moment about Z-Z axis in kNm</td>
<td>250</td>
<td>263.12</td>
<td>5.2</td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Location: Square column reinforced with mild steel

Design of rectangular section under biaxial bending and axial load

(rigorous procedure using trial and error)

Calculations are based on assumptions given in EC2. Solution is by trial and error, with check calculations printed out below. The concrete section is divided into 25 elements of equal area. Moments must be applied to give tension in lower left corner, with dominant moment about Y-Y axis.

<table>
<thead>
<tr>
<th>5.</th>
<th>2 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.</td>
<td>. 6</td>
</tr>
<tr>
<td>25.</td>
<td>. 20</td>
</tr>
</tbody>
</table>

Design axial load \( N = 1000 \text{ kN} \)
Design moment about Y-Y axis \( M_y = 200 \text{ kNm} \)
Design moment about Z-Z axis \( M_z = 150 \text{ kNm} \)
Depth of section \( h = 500 \text{ mm} \)
Breadth of section \( b = 500 \text{ m} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Characteristic steel strength \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of longitudinal reinft. \( \text{dia} = 16 \text{ mm} \)
Diameter of column links \( \text{dial} = 8 \text{ mm} \)
Calculation of strains and forces in steel areas

Steel arrangement:

```
  4
  3  o  o  o  5
  2  o  o  6  bar dia =15.261 mm
  1  o  o  7  and Asb =182.91 mm²
```

Steel area provided $A_{stot}=\text{num} \times \pi \times \text{diac}^2/4=1463.3 \text{ mm}^2$
Steel percentage provided $\text{perct}=100 \times A_{stot}/(h \times b)=0.5853 \%$
Distances to Bar 1 $y=-179.37 \text{ mm} \quad z=-179.37 \text{ mm}$
Strain at bar $e=0.0035 - \text{curvy} \times (h/2-z) - \text{curvz} \times (b/2-y)$
$=-0.0036977$
Steel is plastic, force $f=-A_{sb} \times f_{yk}/(\gamma_m \times 1E3)=-79.525 \text{ kN}$
Similar calculations for remaining bars are summarised as follows:

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Strain</th>
<th>Force (kN)</th>
<th>my (kNm)</th>
<th>mz (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.0036977</td>
<td>-79.525</td>
<td>14.264</td>
<td>14.264</td>
</tr>
<tr>
<td>2</td>
<td>-0.0019826</td>
<td>-72.527</td>
<td>0</td>
<td>13.009</td>
</tr>
<tr>
<td>3</td>
<td>-0.26754E-3</td>
<td>-9.7869</td>
<td>-1.7555</td>
<td>1.7555</td>
</tr>
<tr>
<td>4</td>
<td>0.0010242</td>
<td>37.468</td>
<td>6.7206</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0.002316</td>
<td>79.525</td>
<td>14.264</td>
<td>14.264</td>
</tr>
<tr>
<td>6</td>
<td>0.60093E-3</td>
<td>21.983</td>
<td>0</td>
<td>3.9431</td>
</tr>
<tr>
<td>7</td>
<td>-0.0011141</td>
<td>-40.757</td>
<td>7.3105</td>
<td>-7.3105</td>
</tr>
<tr>
<td>8</td>
<td>-0.0024059</td>
<td>-79.525</td>
<td>14.264</td>
<td>0</td>
</tr>
</tbody>
</table>

Summing forces and moments for all elements and bars gives:

- Calculated axial force \( N_c = 1206.5 \text{ kN} \)
- Moment about y-y axis \( M_{cy} = 244.68 \text{ kNm} \)
- Moment about z-z axis \( M_{cz} = 184.39 \text{ kNm} \)

**SUMMARY** (\( f_{ck}=40 \text{ N/mm}^2 \) : \( f_{yk}=500 \text{ N/mm}^2 \))

(Note: 'microstrain' = true strain multiplied by 1,000,000.)

- Steel percentage required \( 0.5853 \% \)
- Steel percentage provided \( 0.5853 \% \)
- Provide the equivalent of 8 No. 15.261 mm diameter bars
- Total steel area required \( 1463.3 \text{ mm}^2 \)
- Total steel area provided \( 1463.3 \text{ mm}^2 \)
- Cover to all reinforcement \( 55 \text{ mm} \)
- Strain at corner Bar 1 \(-3697.7 \text{ microstrain (tension -ve)}\)
- Strain at top right corner \(3500 \text{ microstrain} \)
- Strain at top left corner \(-100.86 \text{ microstrain} \)

Forces and moments:

<table>
<thead>
<tr>
<th></th>
<th>required</th>
<th>provided</th>
<th>excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>1000</td>
<td>1206.5</td>
<td>20.6</td>
</tr>
<tr>
<td>Moment about Y-Y axis in kNm</td>
<td>200</td>
<td>244.68</td>
<td>22.3</td>
</tr>
<tr>
<td>Moment about Z-Z axis in kNm</td>
<td>150</td>
<td>184.39</td>
<td>22.9</td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Location: Column with low axial load

Design of rectangular section under biaxial bending and axial load

(rigorous procedure using trial and error)

Calculations are based on assumptions given in EC2. Solution is by trial and error, with check calculations printed out below. The concrete section is divided into 25 elements of equal area.

Moments must be applied to give tension in lower left corner, with dominant moment about Y-Y axis.

Design axial load \( N = 100 \text{ kN} \)
Design moment about Y-Y axis \( M_y = 350 \text{ kNm} \)
Design moment about Z-Z axis \( M_z = 350 \text{ kNm} \)
Depth of section \( h = 550 \text{ mm} \)
Breadth of section \( b = 550 \text{ m} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Characteristic steel strength \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of column links \( d_{al} = 10 \text{ mm} \)
Nominal cover to all steel \( c_{cover} = 30 \text{ mm} \)
Calculation of strains and forces in steel areas

Steel arrangement:

```
   4
  3 o o o
   5
  2 o o o
   6
1 o o o
   7
  8
```

Type 4

Bar dia = 32.472 mm
And Asb = 828.17 mm^2

Steel area provided

\[ Astot = \text{num} \times \pi \times \text{diac}^2 / 4 = 6625.3 \text{ mm}^2 \]

Steel percentage provided

\[ \text{perct} = 100 \times \frac{\text{Astot}}{(h \times b)} = 2.1902 \% \]

Distances to Bar 1

\[ y = -218.76 \text{ mm} \quad z = -218.76 \text{ mm} \]

Strain at bar

\[ e = 0.0035 - \text{curvy} \times (h/2-z) - \text{curvz} \times (b/2-y) = -0.0044479 \]

Steel is plastic, force

\[ f = -\text{Asb} \times \text{fyk} / (gams \times 1E3) = -360.07 \text{ kN} \]
Similar calculations for remaining bars are summarised as follows:

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Strain</th>
<th>Force (kN)</th>
<th>my (kN)</th>
<th>mz (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.0044479</td>
<td>-360.07</td>
<td>78.771</td>
<td>78.771</td>
</tr>
<tr>
<td>2</td>
<td>-0.0026872</td>
<td>-360.07</td>
<td>0</td>
<td>78.771</td>
</tr>
<tr>
<td>3</td>
<td>-0.92653E-3</td>
<td>-153.47</td>
<td>-33.573</td>
<td>33.573</td>
</tr>
<tr>
<td>4</td>
<td>0.83413E-3</td>
<td>138.16</td>
<td>30.224</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0.0025948</td>
<td>360.07</td>
<td>78.771</td>
<td>78.771</td>
</tr>
<tr>
<td>6</td>
<td>0.83413E-3</td>
<td>138.16</td>
<td>0</td>
<td>30.224</td>
</tr>
<tr>
<td>7</td>
<td>-0.92653E-3</td>
<td>-153.47</td>
<td>33.573</td>
<td>-33.573</td>
</tr>
<tr>
<td>8</td>
<td>-0.0026872</td>
<td>-360.07</td>
<td>78.771</td>
<td>0</td>
</tr>
</tbody>
</table>

Summing forces and moments for all elements and bars gives

- Calculated axial force: \( N_c = 111.31 \text{ kN} \)
- Moment about y-y axis: \( M_{cy} = 389.63 \text{ kNm} \)
- Moment about z-z axis: \( M_{cz} = 389.63 \text{ kNm} \)

**SUMMARY** (\( f_{ce} = 25 \text{ N/mm}^2 \); \( f_{yk} = 500 \text{ N/mm}^2 \))

(Note: 'microstrain' = true strain multiplied by 1,000,000.)

- Steel percentage required: 2.1902 %
- Steel percentage provided: 2.1902 %
- Provide the equivalent of 8 No. 32.472 mm diameter bars
- Total steel area required: 6625.3 mm²
- Total steel area provided: 6625.3 mm²
- Cover to all reinforcement: 30 mm
- Strain at corner Bar 1: -4447.9 microstrain (tension -ve)
- Strain at top right corner: 3500 microstrain
- Strain at top left corner: -926.53 microstrain

Forces and moments:

<table>
<thead>
<tr>
<th></th>
<th>required</th>
<th>provided</th>
<th>excess (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load in kN</td>
<td>100</td>
<td>111.31</td>
<td>11.3</td>
</tr>
<tr>
<td>Moment about Y-Y axis in kNm</td>
<td>350</td>
<td>389.63</td>
<td>11.3</td>
</tr>
<tr>
<td>Moment about Z-Z axis in kNm</td>
<td>350</td>
<td>389.63</td>
<td>11.3</td>
</tr>
</tbody>
</table>

The above result is judged acceptable.
Location: Typical circular column

Design of circular column section to BS8110

Design to BS8110(1997), partial safety factor for steel gammaS=1.15

Calculations for reinforcement are in accordance with BS 8110 and the IStructE/ICE 'Manual for the design of reinforced concrete building structures'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Design ultimate moment \( M=91 \) kNm
Shear force \( F=0 \) kN
Design ultimate axial load \( N=2000 \) kN
Clear height \( l_0=3 \) m
Fixity condition at top (1-4) \( \text{FixT}=1 \)
Fixity condition at bottom (1-3) \( \text{FixB}=1 \)
 Diameter of cross-section \( h=350 \) mm
Concrete grade \( f_{cu}=35 \) N/mm\(^2\)
Characteristics steel strength \( f_y=500 \) N/mm\(^2\)
Designated exposure class is XC2
Specified fixing tolerance \( \text{tol}=10 \) mm
Cover to all reinforcement \( \text{cover}=35 \) mm
Effective depth \( h'=290 \) mm
Main steel diameter \( \text{dia}=16 \) mm
Number of bars to be provided \( \text{NumBar}=18 \)
Chosen diameter of links \( \text{Diam}=8 \) mm

**SUMMARY OF REINFORCEMENT**

<table>
<thead>
<tr>
<th>VERTICAL REINFORCEMENT SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
<td></td>
</tr>
<tr>
<td>Area of steel required</td>
<td>3590.9 mm(^2)</td>
<td></td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>3619.1 mm(^2)</td>
<td></td>
</tr>
<tr>
<td>Percentage provided</td>
<td>3.7616 %</td>
<td></td>
</tr>
<tr>
<td>Volume of main bars</td>
<td>3619.1 cm(^3)/m</td>
<td></td>
</tr>
<tr>
<td>Weight of main bars</td>
<td>28.41 kg/m</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LINK REINFORCEMENT SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
<td></td>
</tr>
<tr>
<td>Spacing</td>
<td>190 mm</td>
<td></td>
</tr>
<tr>
<td>Volume of links (approx)</td>
<td>258.11 cm(^3)/m</td>
<td></td>
</tr>
<tr>
<td>Weight of links (approx)</td>
<td>2.0262 kg/m</td>
<td></td>
</tr>
<tr>
<td>Total weight of steel</td>
<td>30.436 kg/m</td>
<td></td>
</tr>
</tbody>
</table>
According to Cl 3.12.7.3 of BS8110, circular links or spiral reinforcement passing round all bars provide adequate lateral support in circular columns.
Location: Small-diameter circular column subjected to heavy load

Design of circular column section to BS8110

Design to BS8110(1997), partial safety factor for steel $\gamma_S=1.15$

Calculations for reinforcement are in accordance with BS 8110 and the IStructE/ICE 'Manual for the design of reinforced concrete building structures'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Design ultimate moment $M=125$ kNm
Shear force $F=100$ kN
Design ultimate axial load $N=1750$ kN
Clear height $l_0=0$ m
Fixity condition at top (1-4) $FixT=1$
Fixity condition at bottom (1-3) $FixB=1$
Diameter of cross-section $h=300$ mm
Concrete grade $f_{cu}=60$ N/mm$^2$
Characteristic steel strength $f_y=500$ N/mm$^2$
Designated exposure class is XC1
Specified fixing tolerance $tol=10$ mm
Cover to all reinforcement $cover=25$ mm
Effective depth $h'=259.5$ mm
Main steel diameter $dia=20$ mm
Number of bars to be provided $NumBar=12$
Chosen diameter of links $Diam=8$ mm

WARNING:
Cover is less than minimum specified. For cover of 25 mm, main steel of 20 mm and $h'$ equal to 259.5 mm, diameter of links cannot exceed 5.5 mm.
Reduce effective depth to 257 mm (maximum).
Location: Massive lightly-loaded column reinforced with mild steel

Design of circular column section to BS8110

Design to BS8110(1997), partial safety factor for steel $\gamma_S = 1.15$

Calculations for reinforcement are in accordance with BS 8110 and the IStructE/ICE 'Manual for the design of reinforced concrete building structures'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Summary of reinforcement

<table>
<thead>
<tr>
<th>Vertical Reinforcement Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
</tr>
<tr>
<td>Number of bars</td>
</tr>
<tr>
<td>Diameter of bars</td>
</tr>
<tr>
<td>Area of steel required</td>
</tr>
<tr>
<td>Area of steel provided</td>
</tr>
<tr>
<td>Percentage provided</td>
</tr>
<tr>
<td>Volume of main bars</td>
</tr>
<tr>
<td>Weight of main bars</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Link Reinforcement Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
</tr>
<tr>
<td>Diameter of links</td>
</tr>
<tr>
<td>Spacing</td>
</tr>
<tr>
<td>Volume of links (approx)</td>
</tr>
<tr>
<td>Weight of links (approx)</td>
</tr>
</tbody>
</table>

Total weight of steel 114.69 kg/m
According to Cl 3.12.7.3 of BS8110, circular links or spiral reinforcement passing round all bars provide adequate lateral support in circular columns.
Location: EX1 - Typical circular column

Design of circular column section to EC2

Reinforcement calculations are in accordance with EC2 Part 1-1 and the IStructE 'Manual for the design of concrete building structures to Eurocode 2'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Larger initial end moment (+ve) $M_2=110$ kNm
Smaller initial end moment $M_1=0$ kNm
Design ultimate axial load $N_{Ed}=2000$ kN
Design ultimate shear force $V_{Ed}=0$ kN
Diameter of cross-section $h=375$ mm
Partial safety factor for steel $\gamma_{ms}=1.15$
Partial safety factor for conc. $\gamma_{mc}=1.5$
Clear height of column $l=3$ m
Char yield strength of reinforcement $f_{yk}=500$ N/mm$^2$
Diameter of longitudinal reinft. $d_{ia}=20$ mm
Diameter of column links $d_{il}=8$ mm
Effective depth $h'=300$ mm

Slenderness - EC2 Clause 5.8.3.2

About Y-Y axis:
Column is considered to be braced about the stronger axis.
Effective height $\ell_{oy}=b_{ey}*l*1000=2250$ mm

About Z-Z axis:
Column is considered to be braced about the weaker axis.
Effective height $\ell_{oz}=b_{ez}*l*1000=2250$ mm

Check slenderness

For Y-Y stronger axis:
Slenderness $\lambda_{my}=\ell_{oy}/i_{y}=24$
Limiting slenderness $\lambda_{my}'=20*A*B*C/n^{0.5}=39.743$
Column is classified as being short in accordance with EC2 Part 1-1.

For Z-Z weaker axis:
Slenderness $\lambda_{mz}=\ell_{oz}/i_{z}=24$
Limiting slenderness $\lambda_{mz}'=20*A*B*C/n^{0.5}=39.743$
Column is classified as being short in accordance with EC2 Part 1-1.
Design moments - Clause 5.8.8.2

Design moment (y-y axis) \( M_{Ed} = Mo \times 10^{-6} = 121.25 \text{ kNm} \)

Axial load and moment considerations

Effective/overall-depth ratio \( d' = h'/h = 0.8 \)
Axial load ratio \( N_{rat} = N_{Ed} \times 1000 / (h^2 \times f_{ck}) = 0.50794 \)
Moment ratio \( M_{rat} = M_{Ed} \times 10^6 / (h^3 \times f_{ck}) = 0.082116 \)

Longitudinal reinforcement

Chart data for circular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'. The program initially evaluates the area of reinforcement taking \( K = 1 \). The value of \( N_u \) and hence a new value of \( K \) is then determined leading to a reduced area of reinforcement. This process is then repeated until the minimum area of reinforcement is found. For further guidance refer to the IStructE Manual to EC2, Section 5.5.4.3.

Calculated area of reinforcement \( A_{Sc} = val \times f_{ck} / f_{yk} \times h^2 = 4181.2 \text{ mm}^2 \)
Main steel diameter \( \text{dia} = 20 \text{ mm} \)
Number of bars to be provided \( \text{NumBar} = 14 \)

Transverse reinforcement (links)

Chosen diameter of links \( \text{dia} = 8 \text{ mm} \)
Maximum spacing \( smx = 375 \text{ mm} \)

SUMMARY OF REINFORCEMENT

<table>
<thead>
<tr>
<th>VERTICAL REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
<td></td>
</tr>
<tr>
<td>Area of steel required</td>
<td>4181.2 mm²</td>
<td></td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>4398.2 mm²</td>
<td></td>
</tr>
<tr>
<td>Percentage provided</td>
<td>3.9822 %</td>
<td></td>
</tr>
<tr>
<td>Weight of main bars</td>
<td>34.526 kg/m</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LINK REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
<td></td>
</tr>
<tr>
<td>Spacing</td>
<td>375 mm</td>
<td></td>
</tr>
<tr>
<td>Weight of links (approx)</td>
<td>7.9931 kg/m</td>
<td></td>
</tr>
<tr>
<td>Total weight of steel</td>
<td>42.519 kg/m</td>
<td></td>
</tr>
</tbody>
</table>

NOTE: Circular links or spiral reinforcement passing round all bars provide adequate lateral support in circular columns.

Check for biaxial bending - EC2 Clause 5.8.9

Eccentricity due to imperfections about the weaker axis will be ignored as this was considered about the stronger axis. The calculations below will consider first order and second order moments about z-z axis only.
Design ultimate moment (if any) \( Mo_{2z} = 0 \) kNm  
Column is not slender about \( z-z \) axis and second order moments are not required to be evaluated.

Design ultimate moment (\( z-z \) axis) \( M_{Edz} = Mo_{2z} \times 10^6 = 40E6 \) Nmm

### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear column height</td>
<td>3 m</td>
</tr>
<tr>
<td>Slenderness (Y-Y axis)</td>
<td>24</td>
</tr>
<tr>
<td>Slenderness (Z-Z axis)</td>
<td>24</td>
</tr>
<tr>
<td>Design axial load</td>
<td>2000 kN</td>
</tr>
<tr>
<td>Design bending moments:</td>
<td></td>
</tr>
<tr>
<td>Y-Y axis</td>
<td>121.25 kNm</td>
</tr>
<tr>
<td>Z-Z axis</td>
<td>40 kNm</td>
</tr>
<tr>
<td>Resistance moment (Y-Y)</td>
<td>146.87 kNm</td>
</tr>
<tr>
<td>Resistance moment (Z-Z)</td>
<td>146.87 kNm</td>
</tr>
<tr>
<td>Biaxial bending factor</td>
<td>0.75568</td>
</tr>
</tbody>
</table>
Location: Ex2 - Small-diameter circular column subj. to heavy load

Design of circular column section to EC2

Reinforcement calculations are in accordance with EC2 Part 1-1 and the IStructE 'Manual for the design of concrete building structures to Eurocode 2'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Larger initial end moment (+ve) \( M_2 = 125 \text{ kNm} \)
Smaller initial end moment \( M_1 = 0 \text{ kNm} \)
Design ultimate axial load \( N_{Ed} = 1750 \text{ kN} \)
Design ultimate shear force \( V_{Ed} = 100 \text{ kN} \)
Diameter of cross-section \( h = 400 \text{ mm} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Clear height of column \( l = 4 \text{ m} \)
Char yield strength of reinft. \( f_yk = 500 \text{ N/mm}^2 \)
Diameter of longitudinal reinft. \( \text{dia} = 20 \text{ mm} \)
Diameter of column links \( \text{dial} = 8 \text{ mm} \)
Effective depth \( h' = 350 \text{ mm} \)

Slenderness - EC2 Clause 5.8.3.2

About Y-Y axis:
Column is considered to be braced about the stronger axis.
Effective height \( l_{oy} = 3000 \text{ mm} \)

About Z-Z axis:
Column is considered to be braced about the weaker axis.
Effective height \( l_{oz} = 3000 \text{ mm} \)

Check slenderness

For Y-Y stronger axis:
Slenderness \( \lambda_{my} = \frac{l_{oy}}{i_y} = 30 \)
Limiting slenderness \( \lambda_{my} = 43.126 \)
Column is classified as being short in accordance with EC2 Part 1-1.

For Z-Z weaker axis:
Slenderness \( \lambda_{mz} = \frac{l_{oz}}{i_z} = 30 \)
Limiting slenderness \( \lambda_{mz} = 43.126 \)
Column is classified as being short in accordance with EC2 Part 1-1.
Design moments - Clause 5.8.8.2

Design moment (y-y axis) \( M_{Ed} = Mo^2 \times 10^{-6} = 138.13 \text{ kNm} \)

**Axial load and moment considerations**

Effective/overall-depth ratio \( d'h = h'/h = 0.875 \)
Axial load ratio \( N_{rat} = N_{Ed} \times 1000 / (h^2 \times f_{ck}) = 0.21875 \)
Moment ratio \( M_{rat} = M_{Ed} \times 10^6 / (h^3 \times f_{ck}) = 0.043164 \)

**Longitudinal reinforcement**

Chart data for circular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'. The program initially evaluates the area of reinforcement taking \( K = 1 \). The value of \( N_u \) and hence a new value of \( K \) is then determined leading to a reduced area of reinforcement. This process is then repeated until the minimum area of reinforcement is found. For further guidance refer to the IStructE Manual to EC2, Section 5.5.4.3.

Calculated area of reinforcement \( A_{sc} = \text{val} \times f_{ck} / f_{yk} \times h^2 = 0 \text{ mm}^2 \)
Main steel diameter \( \text{dia} = 12 \text{ mm} \)
Number of bars to be provided \( \text{NumBar} = 6 \)

**Shear in column**

Shear resistance:
\[ VR_{dc} = (CR_{dc} \times ks \times v_{Rdc} + 0.15 \times N_{Ed} \times 1000 / Ac) \times bw \times d / 1000 = 377.82 \text{ kN} \]

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>100 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>377.82 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

**Transverse reinforcement (links)**

Chosen diameter of links \( \text{dial} = 8 \text{ mm} \)
Maximum spacing \( \text{smx} = 240 \text{ mm} \)

**WARNING:**
The durability check needs to be repeated. Please set the diameter of longitudinal reinforcement to 12 mm.
Location: Ex3 - Massive lightly-loaded column reinf. with mild steel

Design of circular column section to EC2

Reinforcement calculations are in accordance with EC2 Part 1-1 and the IStructE 'Manual for the design of concrete building structures to Eurocode 2'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Larger initial end moment (+ve) \( M_2 = 10 \text{ kNm} \)
Smaller initial end moment \( M_1 = 0 \text{ kNm} \)
Design ultimate axial load \( N_{Ed} = 100 \text{ kN} \)
Design ultimate shear force \( V_{Ed} = 1000 \text{ kN} \)
Diameter of cross-section \( h = 2000 \text{ mm} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Clear height of column \( l = 20 \text{ m} \)
Char yield strength of reinf'ment \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of longitudinal reinf.t. \( \text{dia} = 20 \text{ mm} \)
Diameter of column links \( \text{dial} = 10 \text{ mm} \)
Effective depth \( h' = 1900 \text{ mm} \)

Slenderness - EC2 Clause 5.8.3.2

About Y-Y axis:
Column is considered to be braced about the stronger axis.
Effective height \( l_{oy} = \text{bety}*l*1000 = 15000 \text{ mm} \)

About Z-Z axis:
Column is considered to be braced about the weaker axis.
Effective height \( l_{oz} = \text{betz}*l*1000 = 15000 \text{ mm} \)

Check slenderness

For Y-Y stronger axis:
Slenderness \( l_{amy} = l_{oy}/iy = 30 \)
Limiting slenderness \( l_{amy} = 20*A*B*C/n^{0.5} = 514.27 \)
Column is classified as being short in accordance with EC2 Part 1-1.

For Z-Z weaker axis:
Slenderness \( l_{amz} = l_{oz}/iz = 30 \)
Limiting slenderness \( l_{amz} = 20*A*B*C/n^{0.5} = 514.27 \)
Column is classified as being short in accordance with EC2 Part 1-1.
WARNING: Check on validity of the input values

- Ratio of reinforcement term: \( n = 1.0246 \)
- Relative axial force: \( n = 0.0022469 \)
- Value of \( n \) at maximum moment: \( n_{bal} = 0.4 \)
- Correction factor for axial load: \( K_r = \frac{(n - n)}{(n - n_{bal})} = 1.6369 \)

As this value is high, design the section as a beam.
Design of circular column section to EC2

Reinforcement calculations are in accordance with EC2 Part 1-1 and the IStructE 'Manual for the design of concrete building structures to Eurocode 2'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Larger initial end moment (+ve) \( M_2 = 40 \) kNm
Smaller initial end moment \( M_1 = 0 \) kNm
Design ultimate axial load \( N_{Ed} = 1700 \) kN
Design ultimate shear force \( V_{Ed} = 0 \) kN
Diameter of cross-section \( h = 350 \) mm
Partial safety factor for steel \( \gamma_{ams} = 1.15 \)
Partial safety factor for conc. \( \gamma_{amc} = 1.5 \)
Clear height of column \( l = 5 \) m
Char yield strength of reinforcement \( f_{yk} = 500 \) N/mm²
Diameter of longitudinal reinf. \( \text{dia} = 20 \) mm
Diameter of column links \( \text{dial} = 8 \) mm
Nominal cover to all steel \( \text{cover} = 35 \) mm
Effective depth \( h' = 295 \) mm

Slenderness - EC2 Clause 5.8.3.2

About Y-Y axis:
Column is considered to be braced about the stronger axis.
Effective height \( l_{oy} = \beta_{e,y} \times l \times 1000 = 3750 \) mm

About Z-Z axis:
Column is considered to be braced about the weaker axis.
Effective height \( l_{oz} = \beta_{e,z} \times l \times 1000 = 3750 \) mm

Check slenderness

For Y-Y stronger axis:
Slenderness \( \lambda_y = l_{oy}/l_y = 42.857 \)
Limiting slenderness \( \lambda_{ly} = 20 \times A \times B \times C/n^{0.5} = 36.187 \)
Column is classified as being slender in accordance with EC2 Part 1-1.

For Z-Z weaker axis:
Slenderness \( \lambda_z = l_{oz}/l_z = 42.857 \)
Limiting slenderness \( \lambda_{lz} = 20 \times A \times B \times C/n^{0.5} = 36.187 \)
Column is classified as being slender in accordance with EC2 Part 1-1.
Design moments - Clause 5.8.8.2

Design moment (y-y axis)  \( M_{Ed} = M_{o2} \times 10^{-6} = 55.938 \text{ kNm} \)

Axial load and moment considerations

- Effective/overall-depth ratio  \( d'h = h'/h = 0.84286 \)
- Axial load ratio  \( N_{rat} = N_{Ed} \times 1000/(h^2 \times f_{ck}) = 0.43367 \)
- Moment ratio  \( M_{rat} = M_{Ed} \times 10^6/(h^3 \times f_{ck}) = 0.040771 \)

Longitudinal reinforcement

Chart data for circular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'. The program initially evaluates the area of reinforcement taking \( K=1 \). The value of \( N_u \) and hence a new value of \( K \) is then determined leading to a reduced area of reinforcement. This process is then repeated until the minimum area of reinforcement is found. For further guidance refer to the IStructE Manual to EC2, Section 5.5.4.3.

- Calculated area of reinforcement  \( A_{sc} = val \times f_{ck}/f_{yk} \times h^2 = 1891.7 \text{ mm}^2 \)
- Main steel diameter  \( \text{dia} = 12 \text{ mm} \)
- Number of bars to be provided  \( \text{NumBar} = 17 \)

Transverse reinforcement (links)

- Chosen diameter of links  \( \text{dial} = 8 \text{ mm} \)
- Maximum spacing  \( \text{smx} = 240 \text{ mm} \)

WARNING:
The durability check needs to be repeated. Please set the diameter of longitudinal reinforcement to 12 mm.
Location: Ex5

Design of circular column section to EC2

Reinforcement calculations are in accordance with EC2 Part 1-1 and the IStructE 'Manual for the design of concrete building structures to Eurocode 2'. Min. steel = six bars arranged symmetrically. Any greater number of bars (eight shown) increases resistance moment of section.

Larger initial end moment (+ve) \( M_2 = 40 \text{ kNm} \)
Smaller initial end moment \( M_1 = 0 \text{ kNm} \)
Design ultimate axial load \( N_{Ed} = 1720 \text{ kN} \)
Design ultimate shear force \( V_{Ed} = 0 \text{ kN} \)
Diameter of cross-section \( h = 350 \text{ mm} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Clear height of column \( l = 4 \text{ m} \)
Char yield strength of reinf'ment \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of longitudinal reinft. \( d = 20 \text{ mm} \)
Diameter of column links \( d_{al} = 8 \text{ mm} \)
Nominal cover to all steel \( c = 35 \text{ mm} \)
Effective depth \( h' = 295 \text{ mm} \)

Slenderness - EC2 Clause 5.8.3.2

About Y-Y axis:
Column is considered to be braced about the stronger axis.
Effective height \( l_{oy} = \beta_{ty} l \times 1000 = 3000 \text{ mm} \)

About Z-Z axis:
Column is considered to be braced about the weaker axis.
Effective height \( l_{oz} = \beta_{tz} l \times 1000 = 4000 \text{ mm} \)

Check slenderness

For Y-Y stronger axis:
Slenderness \( l_{amy} = l_{oy}/\gamma_{iy} = 34.286 \)
Limiting slenderness \( l_{amy} = 20*A*B*C/n^0.5 = 35.976 \)
Column is classified as being short in accordance with EC2 Part 1-1.

For Z-Z weaker axis:
Slenderness \( l_{amz} = l_{oz}/\gamma_{iz} = 45.714 \)
Limiting slenderness \( l_{amz} = 20*A*B*C/n^0.5 = 35.976 \)
Column is classified as being slender in accordance with EC2 Part 1-1.
Design moments - Clause 5.8.8.2

Design moment (y-y axis) \[ M_{Ed} = M_0 \times 10^{-6} = 52.9 \text{ kNm} \]

Axial load and moment considerations

Effective/overall-depth ratio \[ d'h = h'/h = 0.84286 \]
Axial load ratio \[ N_{rat} = N_{Ed} \times 1000 / (h^2 \times f_{ck}) = 0.43878 \]
Moment ratio \[ M_{rat} = M_{Ed} \times 10^6 / (h^3 \times f_{ck}) = 0.038557 \]

Longitudinal reinforcement

Chart data for circular columns is taken from 'The Concrete Centre' publication entitled 'Worked Examples to Eurocode 2: Volume 1'. The program initially evaluates the area of reinforcement taking \( K = 1 \). The value of \( N_u \) and hence a new value of \( K \) is then determined leading to a reduced area of reinforcement. This process is then repeated until the minimum area of reinforcement is found. For further guidance refer to the IStructE Manual to EC2, Section 5.5.4.3.

Calculated area of reinforcement \[ A_{sc} = \frac{va_1 \times f_{ck}}{f_{yk} \times h^2} = 1125.4 \text{ mm}^2 \]
Main steel diameter \[ d_{ia} = 12 \text{ mm} \]
Number of bars to be provided \[ N_{umb} = 10 \]

Transverse reinforcement (links)

Chosen diameter of links \[ d_{ia} = 8 \text{ mm} \]
Maximum spacing \[ s_{mx} = 240 \text{ mm} \]

WARNING:
The durability check needs to be repeated. Please set the diameter of longitudinal reinforcement to 12 mm.
Location: Typical problem

Design to BS8110(1997), partial safety factor for steel $\gamma_{S}=1.15$

---

**Effective span**  
$L=5\ m$

**Dead load factor**  
$\gamma_{d}=1.4$

**Imposed load factor**  
$\gamma_{i}=1.6$

**Distance from left support**  
$L_{c}(1)=1.4\ m$

**Characteristic dead load**  
$G_{k}(1)=6.4\ kN$

**Characteristic imposed load**  
$Q_{k}(1)=0\ kN$

**Distance from left support to start**  
$L_{a}(1)=0\ m$

**Distance from left support to end**  
$L_{b}(1)=1.4\ m$

**Characteristic dead load**  
$G_{k}(1)=8\ kN/m$

**Characteristic imposed load**  
$Q_{k}(1)=10\ kN/m$

**Distance from left support to start**  
$L_{a}(2)=1.4\ m$

**Distance from left support to end**  
$L_{b}(2)=5\ m$

**Characteristic dead load**  
$G_{k}(2)=12\ kN/m$

**Characteristic imposed load**  
$Q_{k}(2)=15\ kN/m$

**Overall depth of concrete section**  
$h=400\ mm$

**Breadth of concrete section**  
$b=200\ mm$

**Eff.depth of concrete section**  
$d=357.5\ mm$

**Depth to comp. reinforcement**  
$d'=40\ mm$

**Cover to all reinforcement**  
$cover=20\ mm$

**Characteristic concrete strength**  
$f_{cu}=40\ N/mm^2$

**Characteristic strength of links**  
$f_{y}=500\ N/mm^2$

**Diameter of tension reinforcement**  
$dia=25\ mm$

**Diameter of links**  
$dial=10\ mm$

**Characteristics**

- **TENSION**
  - Characteristic strength  
    $500\ N/mm^2$

- **REINFORCEMENT**
  - Diameter of bars  
    $25\ mm$

- **SUMMARY**
  - Number of bars  
    $2$
  - Area required  
    $979.97\ mm^2$
  - Area provided  
    $981.75\ mm^2$

See BS 8110 (Anchorage & tension lap)

- Part 1:  
  - 1.4 * tension lap  
    $860\ mm$
  - Cl.3.12.8.13 (2.0 * tension lap  
    $1730\ mm$

**Shear reinforcement**

- Design shear stress  
  $v=\frac{V*1000}{(b*v*d)}=1.4243\ N/mm^2$

- Longit. bars effective for shear  
  $n_{bars}=2$

As characteristic concrete strength exceeds 25 N/mm$^2$, therefore increase $v_{c}$ according to footnote in Table 3.8.

- Modified design shear stress  
  $v_{c}=v_{c}*(\frac{40}{25})^{(1/3)}=0.84499\ N/mm^2$

Design shear stress of 1.4243 N/mm$^2$ exceeds critical stress of 0.84499 N/mm$^2$, so links to resist balance of shear forces are necessary.

- Number of shear legs  
  $n_{legs}=2$

Provide links to take shear force in accordance with Table 3.7.
Spacing of links along member

\[ sv = \frac{fyv}{\gamma_S A_{sv}} (bv^2 (v-v_c)) \]

= 589.42 mm

Limit spacing to \( sv = 0.75d = 268.13 \text{ mm} \) (to Clause 3.4.5.5)

<table>
<thead>
<tr>
<th>SHEAR</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
<td>10 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of legs</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>260 mm</td>
</tr>
<tr>
<td></td>
<td>Ensure that no longitudinal bar is more than 150 mm from vertical leg</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DEFLECTION</th>
<th>Permissible span</th>
<th>5.3962 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Actual span</td>
<td>5 m</td>
</tr>
</tbody>
</table>

As actual span does not exceed permissible, deflection is satisfactory.
Location: Heavily-loaded long span with triangular load, etc

Design to BS8110(1997), partial safety factor for steel gammaS=1.15

---

Effective span $L=12$ m
Dead load factor $\gamma_d=1.4$
Imposed load factor $\gamma_i=1.6$
Distance from left support to start $L_u(1)=2$ m
Distance from left support to end $L_{bu}(1)=10.5$ m
Characteristic dead load $G_{ku}(1)=8$ kN/m
Characteristic imposed load $Q_{ku}(1)=12$ kN/m
Overall depth of concrete section $h=600$ mm
Breadth of concrete section $b=500$ mm
Effective depth of concrete section $d=357.5$ mm
Depth to comp. reinforcement $d'=50$ mm
Cover to all reinforcement $\text{cover}=20$ mm
Characteristic concrete strength $f_{cu}=60$ N/mm²
Characteristic strength of links $f_y=500$ N/mm²
Diameter of tension reinforcement $\text{dia}=25$ mm
Diameter of links $\text{dial}=10$ mm

**TENSION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
</table>

**REINFORCEMENT**

<table>
<thead>
<tr>
<th>Diameter of bars</th>
<th>25 mm</th>
</tr>
</thead>
</table>

**SUMMARY**

<table>
<thead>
<tr>
<th>Area required</th>
<th>3899.9 mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area provided</td>
<td>3927 mm²</td>
</tr>
</tbody>
</table>

See BS 8110

<table>
<thead>
<tr>
<th>Cl.3.12.8.13</th>
<th>( Anchorage &amp; tension lap</th>
</tr>
</thead>
<tbody>
<tr>
<td>Part 1</td>
<td>( 1.4 * tension lap</td>
</tr>
<tr>
<td></td>
<td>1000 mm</td>
</tr>
<tr>
<td></td>
<td>( 2.0 * tension lap</td>
</tr>
<tr>
<td></td>
<td>1430 mm</td>
</tr>
</tbody>
</table>

**Shear reinforcement**

Design shear stress $v=V*1000/(b\gamma_S d)=0.7529$ N/mm²
Long. bars effective for shear $n_bars=4$
As characteristic concrete strength exceeds 25 N/mm²
therefore increase vc according to footnote in Table 3.8.
Modified design shear stress $vc=vc*{(40/25)^{1/3}}=0.78442$ N/mm²
Design shear stress of 0.7529 N/mm² does not exceed critical shear stress of 0.78442 N/mm², so only nominal links need be provided.

Number of shear legs $n_legs=2$
Provide minimum links to meet requirements of Table 3.7.
Spacing of links along member $sv=f_y/\gamma_S As/(0.4*bv)=341.48$ mm
Limit spacing to $sv=0.75*d=268.13$ mm (to Clause 3.4.5.5)
SHEAR
Characteristic strength 500 N/mm²

REINFORCEMENT
Diameter of bars 10 mm
Number of legs 2
Spacing 260 mm
Ensure that no longitudinal bar is more than 150 mm from vertical leg

SUMMARY
Number of legs 2
Spacing 260 mm

DEFLECTION
Permissible span 4.1082 m

SUMMARY
Actual span 12 m

WARNING:
As actual span > permissible, deflection criteria are violated.
**Location:** Lightly loaded beam with concentrated load only

Design to BS8110(1997), partial safety factor for steel $\gamma_S = 1.15$

---

**Effective span**  $L = 7 \text{ m}$

**Dead load factor**  $\gamma_d = 1.4$

**Imposed load factor**  $\gamma_i = 1.6$

**Distance from left support**  $L_c(1) = 1.4 \text{ m}$

**Characteristic dead load**  $G_{kc}(1) = 4 \text{ kN}$

**Characteristic imposed load**  $Q_{kc}(1) = 8 \text{ kN}$

**Overall depth of concrete section**  $h = 400 \text{ mm}$

**Breadth of concrete section**  $b = 250 \text{ mm}$

**Depth to comp. reinforcement**  $d' = 40 \text{ mm}$

**Cover to all reinforcement**  $\text{cover} = 20 \text{ mm}$

**Characteristic concrete strength**  $f_{cu} = 25 \text{ N/mm}^2$

**Characteristic strength of links**  $f_y = 500 \text{ N/mm}^2$

**Diameter of tension reinforcement**  $d_{ea} = 20 \text{ mm}$

**Diameter of links**  $d_{al} = 10 \text{ mm}$

---

### TENSION

**Characteristic strength**  $500 \text{ N/mm}^2$

### REINFORCEMENT

**Diameter of bars**  $20 \text{ mm}$

### SUMMARY

**Number of bars**  $1$

**Area required**  $139.56 \text{ mm}^2$

**Area provided**  $314.16 \text{ mm}^2$

See BS 8110 (Anchorage & tension lap)

: Part 1:  $1.4 \times \text{ tension lap}$  $870 \text{ mm}$

Cl.3.12.8.13  $2.0 \times \text{ tension lap}$  $1220 \text{ mm}$

### Shear reinforcement

**Design shear stress**  $\nu = \frac{V \times 1000}{(b \times d)} = 0.1647 \text{ N/mm}^2$

**Longit. bars effective for shear**  $n_{bars} = 2$

**Design shear stress of 0.1647 N/mm$^2$ does not exceed critical shear stress of 0.57796 N/mm$^2$, so only nominal links need be provided.**

**Number of shear legs**  $n_{legs} = 2$

**Spacing of links along member**  $s_v = \frac{f_y \times \gamma_S \times A_{sv}}{(0.4 \times b \times d)} = 682.95 \text{ mm}$

**Limit spacing to**  $s_v = 0.75 \times d = 268.13 \text{ mm}$ (to Clause 3.4.5.5)

### SHEAR

**Characteristic strength**  $500 \text{ N/mm}^2$

### REINFORCEMENT

**Diameter of bars**  $10 \text{ mm}$

### SUMMARY

**Number of legs**  $2$

**Spacing**  $260 \text{ mm}$

**Ensure that no longitudinal bar is more than 150 mm from vertical leg**
DEFLECTION

SUMMARY

Permissible span  14.3 m
Actual span  7 m

As actual span does not exceed permissible, deflection is satisfactory.
Location: Slab example

Design to BS8110(1997), partial safety factor for steel \( \gamma_S = 1.15 \)

<table>
<thead>
<tr>
<th>L</th>
<th>Effective span</th>
<th>L=4.5 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load factor</td>
<td>gADM=1.4</td>
<td></td>
</tr>
<tr>
<td>Imposed load factor</td>
<td>gIM=1.6</td>
<td></td>
</tr>
<tr>
<td>Dist. from left support to start</td>
<td>Lau(1)=0 m</td>
<td></td>
</tr>
<tr>
<td>Distance from left support to end</td>
<td>Lbu(1)=4.5 m</td>
<td></td>
</tr>
<tr>
<td>Characteristic dead load</td>
<td>GkU(1)=6 kN/m</td>
<td></td>
</tr>
<tr>
<td>Characteristic imposed load</td>
<td>QkU(1)=5 kN/m</td>
<td></td>
</tr>
<tr>
<td>Overall depth of concrete section</td>
<td>h=225 mm</td>
<td></td>
</tr>
<tr>
<td>Breadth of concrete section</td>
<td>b=1000 mm</td>
<td></td>
</tr>
<tr>
<td>Eff. depth of concrete section</td>
<td>d=190 mm</td>
<td></td>
</tr>
<tr>
<td>Depth to comp. reinforcement</td>
<td>d'=35 mm</td>
<td></td>
</tr>
<tr>
<td>Cover to all reinforcement</td>
<td>cover=20 mm</td>
<td></td>
</tr>
<tr>
<td>Characteristic concrete strength</td>
<td>fcu=35 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Characteristic strength of links</td>
<td>fyv=500 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Diameter of tension reinforcement</td>
<td>dia=12 mm</td>
<td></td>
</tr>
</tbody>
</table>

**TENSION**

| Characteristic strength | 500 N/mm² |
| Diameter of bars | 12 mm |
| Bar cross-centres | 200 mm |
| Area required | 528.97 mm²/m |
| Area provided | 565.49 mm²/m |

See BS 8110 Part 1:
- Anchorage & tension lap: 450 mm
- 1.4 * tension lap: 640 mm
- 2.0 * tension lap: 910 mm

**DISTRIBUTION**

| Characteristic strength | 500 N/mm² |
| Diameter of bars | 8 mm |
| Spacing of bars | 150 mm |
| Area of steel required | 292.5 mm²/m |
| Area of steel provided | 335 mm²/m |

**Shear reinforcement**

Design shear stress \( v = \frac{V*1000}{(b*v*d)} = 0.19421 \) N/mm²

Longit. bars effective for shear \( n_bars=3 \)

As characteristic concrete strength exceeds 25 N/mm², therefore increase \( v_c \) according to footnote in Table 3.8.

Modified design shear stress \( v_c = v * (fcu/25)^{(1/3)} = 0.47958 \) N/mm²

As design shear stress of 0.19421 N/mm² does not exceed critical shear stress of 0.47958 N/mm², no shear reinforcement is required.
As actual span does not exceed permissible, deflection is satisfactory.
### Location: Typical problem

![Diagram of a beam/slab with labeled dimensions](image)

- **Effective span** \( L = 5 \text{ m} \)
- **Dead load factor** \( \gamma_d = 1.25 \)
- **Imposed load factor** \( \gamma_i = 1.5 \)
- **Distance from left support** \( L_c(1) = 1.4 \text{ m} \)
- **Characteristic dead load** \( G_{kc}(1) = 6.4 \text{ kN} \)
- **Characteristic imposed load** \( Q_{kc}(1) = 0 \text{ kN} \)
- **Distance from left support to start** \( L_{au}(1) = 0 \text{ m} \)
- **Distance from left support to end** \( L_{bu}(1) = 1.4 \text{ m} \)
- **Characteristic dead load** \( G_{ku}(1) = 8 \text{ kN/m} \)
- **Characteristic imposed load** \( Q_{ku}(1) = 10 \text{ kN/m} \)
- **Distance from left support to start** \( L_{au}(2) = 1.4 \text{ m} \)
- **Distance from left support to end** \( L_{bu}(2) = 5 \text{ m} \)
- **Characteristic dead load** \( G_{ku}(2) = 12 \text{ kN/m} \)
- **Characteristic imposed load** \( Q_{ku}(2) = 15 \text{ kN/m} \)
- **Overall depth of concrete section** \( h = 400 \text{ mm} \)
- **Breadth of concrete section** \( b = 200 \text{ mm} \)

### Materials

- **Char yield strength of reinf'ment** \( f_y = 500 \text{ N/mm}^2 \)
- **Max.aggregate size (for bar spc.)** \( h_{agg} = 20 \text{ mm} \)
- **Diameter of tension bars** \( \text{dia} = 25 \text{ mm} \)
- **Diameter of link legs** \( \text{dial} = 8 \text{ mm} \)
- **Effective depth of section** \( d = 345 \text{ mm} \)

### DESIGN

<table>
<thead>
<tr>
<th>Overall depth</th>
<th>400 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective depth</td>
<td>345 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>200 mm</td>
</tr>
</tbody>
</table>

### SUMMARY

| Parameter K | 0.1634 |
| Parameter K' | 0.1684 |
| Lever arm ratio z/d | 0.8254 |

### FLEXURE

| Steel area required | 942.3 \text{ mm}^2 |
| Steel area provided | 981.7 \text{ mm}^2 |
| Diameter of bars | 25 mm |
| Number of bars | 2 |
| Steel percentage req. | 1.366 % |
| Minimum area of steel | 103.9 \text{ mm}^2 |
| Maximum area of steel | 3200 \text{ mm}^2 |

### TENSION REINFORCEMENT

### Shear check

- **Design shear stress** \( v_{Ed} = \frac{V_{Ed} \times 10^3}{(0.9 \times b \times d)} = 1.506 \text{ N/mm}^2 \)
- **Concrete strut capacity** \( v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck} / 250) = 3.274 \text{ N/mm}^2 \)
- **Angle of inclination of strut** \( \theta = 22^\circ \)
- **Number of shear legs** \( n = 2 \)
- **Chosen spacing** \( s = 250 \text{ mm} \)
<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force</td>
<td>93.54 kN</td>
</tr>
<tr>
<td>Design shear stress</td>
<td>1.506 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.274 N/mm²</td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.2772</td>
</tr>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>Area of links</td>
<td>100 mm²</td>
</tr>
<tr>
<td>Maximum spacing (based on 0.75d)</td>
<td>258.8 mm</td>
</tr>
<tr>
<td>Actual spacing</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual l/d ratio</td>
<td>14.49</td>
</tr>
<tr>
<td>Basic l/d ratio</td>
<td>14.3</td>
</tr>
<tr>
<td>Span factor</td>
<td>1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DEFLECTION</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Flange beam factor F1</td>
<td>1</td>
</tr>
<tr>
<td>Long spans factor F2</td>
<td>1</td>
</tr>
<tr>
<td>Steel stress factor F3</td>
<td>1.042</td>
</tr>
<tr>
<td>Allowable l/d ratio</td>
<td>14.89</td>
</tr>
</tbody>
</table>
Location: Heavily-loaded long span with triangular load, etc

![Diagram of beam/slab with dimensions and reinforcement details]

- Effective span: \( L = 12 \) m
- Dead load factor: \( \gamma_{d} = 1.25 \)
- Imposed load factor: \( \gamma_{i} = 1.5 \)
- Distance from left support to start: \( L_{a}(1) = 2 \) m
- Distance from left support to end: \( L_{b}(1) = 10.5 \) m
- Characteristic dead load: \( G_{k}(1) = 8 \) kN/m
- Characteristic imposed load: \( Q_{k}(1) = 12 \) kN/m
- Overall depth of concrete section: \( h = 600 \) mm
- Breadth of concrete section: \( b = 500 \) mm

**Materials**

- Char yield strength of reinf'ment: \( f_{yk} = 500 \) N/mm²
- Max.aggregate size (for bar spc.): \( h_{agg} = 20 \) mm
- Diameter of tension bars: \( \text{dia} = 25 \) mm
- Diameter of link legs: \( \text{dial} = 10 \) mm
- Effective depth of section: \( d = 357.5 \) mm
- Diameter of compression bars: \( \text{diac} = 25 \) mm
- Depth to compression steel: \( d_{2} = 55 \) mm

**DESIGN**

- Overall depth: 600 mm
- Effective depth: 357.5 mm
- Width of section: 500 mm

**SUMMARY**

- Parameter K: 0.2401
- Parameter K': 0.1684
- Lever arm ratio z/d: 0.8185

**FLEXURE**

- Steel area provided: 3927 mm²
- Diameter of bars: 25 mm
- Number of bars: 8
- Steel area required: 3582 mm²
- Steel percentage req.: 2.004 %
- Minimum area of steel: 269.2 mm²
- Maximum area of steel: 12000 mm²

**TENSION REINFORCEMENT**

- Steel area provided: 1473 mm²
- Diameter of bars: 25 mm
- Number of bars: 3
- Steel area required: 1045 mm²
- Minimum area required: 600 mm²

**COMPRESSION REINFORCEMENT**

- Steel area provided: 1473 mm²
- Diameter of bars: 25 mm
- Number of bars: 3
- Steel area required: 1045 mm²
- Minimum area required: 600 mm²

**Shear check**

- Design shear stress: \( v_{Ed} = \frac{V_{Ed} \times 10^{3}}{(0.9 \times b \times d)} = 0.7705 \) N/mm²
- Concrete strut capacity: \( v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck} / 250) = 3.274 \) N/mm²
- Angle of inclination of strut: \( \theta' = 22° \)
- Number of shear legs: \( n_{SL} = 2 \)
- Chosen spacing: \( s = 250 \) mm
Reinforced concrete design to BS8110 & Eurocode 2
Simply supported rectangular beam/slab with genera
Date: 02/12/19
Ref No: SC100 EC

Shear Summary

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force</td>
<td>124 kN</td>
</tr>
<tr>
<td>Design shear stress</td>
<td>0.7705 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.274 N/mm²</td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.4382</td>
</tr>
<tr>
<td>Diameter of links</td>
<td>10 mm</td>
</tr>
<tr>
<td>Area of links</td>
<td>157 mm²</td>
</tr>
<tr>
<td>Maximum spacing (based on 0.75d)</td>
<td>268.1 mm</td>
</tr>
<tr>
<td>Actual spacing</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

WARNING:
Actual ratio exceeds allowable ratio (33.57 > 9.362)
Please revise input values.
Location: Lightly loaded beam with concentrated load only

![Diagram of beam with dimensions](image)

- Effective span: \( L = 7 \text{ m} \)
- Dead load factor: \( \gamma_{\text{d}} = 1.25 \)
- Imposed load factor: \( \gamma_{\text{i}} = 1.5 \)
- Distance from left support: \( L_{c(1)} = 1.4 \text{ m} \)
- Characteristic dead load: \( G_{kc(1)} = 4 \text{ kN} \)
- Characteristic imposed load: \( Q_{kc(1)} = 8 \text{ kN} \)
- Overall depth of concrete section: \( h = 400 \text{ mm} \)
- Breadth of concrete section: \( b = 250 \text{ mm} \)

### Materials
- Char yield strength of reinforcement: \( f_{yk} = 500 \text{ N/mm}^2 \)
- Max. aggregate size (for bar spc.): \( h_{agg} = 20 \text{ mm} \)
- Diameter of tension bars: \( \text{dia} = 20 \text{ mm} \)
- Diameter of link legs: \( \text{dial} = 8 \text{ mm} \)
- Effective depth of section: \( d = 357.5 \text{ mm} \)

### Design
- Overall depth: 400 mm
- Effective depth: 357.5 mm
- Width of section: 250 mm

### Flexure
- Parameter K: 0.0199
- Parameter K': 0.1684
- Lever arm ratio \( z/d \): 0.95

### Tension Reinforcement
- Steel area required: 134.6 \( \text{mm}^2 \)
- Steel area provided: 628.3 \( \text{mm}^2 \)
- Diameter of bars: 20 mm
- Number of bars: 2
- Steel percentage req.: 0.1506%
- Minimum area of steel: 134.6 \( \text{mm}^2 \)
- Maximum area of steel: 4000 \( \text{mm}^2 \)

### Shear Check
- Design shear stress: \( v_{Ed} = V_{Ed} 	imes 10^3 / (0.9 \times b \times d) = 0.1691 \text{ N/mm}^2 \)
- Concrete strut capacity: \( v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 3.274 \text{ N/mm}^2 \)
- Angle of inclination of strut: \( \theta' = 22^\circ \)
- Number of shear legs: \( n_{sl} = 2 \)
- Chosen spacing: \( s = 250 \text{ mm} \)

### Shear Summary
- Design shear force: 13.6 kN
- Design shear stress: 0.1691 \( \text{N/mm}^2 \)
- Concrete strut capacity: 3.274 \( \text{N/mm}^2 \)
- Area/spacing ratio: 0.2191
- Diameter of links: 8 mm
- Area of links: 100 \( \text{mm}^2 \)
- Maximum spacing: 268.1 mm
- (based on 0.75d)
- Actual spacing: 250 mm
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Actual l/d ratio</th>
<th>19.58</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Basic l/d ratio</td>
<td>115.9</td>
</tr>
<tr>
<td></td>
<td>Span factor</td>
<td>1</td>
</tr>
<tr>
<td>DEFLECTION</td>
<td>Flange beam factor F1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Long spans factor F2</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Steel stress factor F3</td>
<td>4.668</td>
</tr>
<tr>
<td></td>
<td>Allowable l/d ratio</td>
<td>541</td>
</tr>
</tbody>
</table>
Location: Slab example

- Effective span: L = 4.5 m
- Dead load factor: \( \gamma_{md} = 1.4 \)
- Imposed load factor: \( \gamma_{mi} = 1.6 \)
- Distance from left support to start: \( L_{au(1)} = 0 \) m
- Distance from left support to end: \( L_{bu(1)} = 4.5 \) m
- Characteristic dead load: \( G_{ku(1)} = 6 \) kN/m
- Characteristic imposed load: \( Q_{ku(1)} = 5 \) kN/m

Overall depth of concrete section: \( h = 225 \) mm

Materials

- Char yield strength of reinforcement: \( \gamma_{yk} = 500 \) N/mm²
- Max. aggregate size (for bar spc.): \( h_{ag} = 20 \) mm
- Diameter of tension bars: \( d_{ia} = 12 \) mm
- Effective depth of section: \( d = 190 \) mm

Design

Overall depth: 225 mm
Effective depth: 190 mm

Flexure

Parameter \( K \): 0.0383
Parameter \( K' \): 0.1684
Lever arm ratio \( z/d \): 0.95
Steel area (tension): 529 mm²/m
Steel percentage req.: 0.2784%
Minimum area of steel: 286.2 mm²/m
Maximum area of steel: 9000 mm²/m
Distribution steel: 286.2 mm²/m

Chosen spacing of tension bars: \( p_{ch} = 200 \) mm
Diameter of distribution bars: \( d_{iad} = 10 \) mm
Chosen spacing of distn. bars: \( p_{chd} = 250 \) mm

Tension reinforcement

Diameter of bars: 12 mm
Spacing of bars: 200 mm
Area of steel required: 529 mm²/m
Area of steel provided: 565 mm²/m

Distribution reinforcement

Diameter of bars: 10 mm
Spacing of bars: 250 mm
Area of steel required: 286.2 mm²/m
Area of steel provided: 314 mm²/m

Shear check

Term for shear resistance: \( v_{Rdc} = (\rho_1 \gamma_{kc})^{(1/3)} = 1.646 \)
Shear resistance: \( v_{Rdc} = C_{Rdc} \cdot k_s \cdot v_{Rdc} \cdot b \cdot w / d / 1000 = 75.06 \) kN
Minimum stress: \( v_{min} = 0.035 \cdot k_s \cdot 1.5 \cdot f_{ck} \cdot 0.5 = 0.5422 \) N/mm²
Minimum shear resistance: \( v_{Rdm} = v_{min} \cdot b \cdot w / d / 1000 = 103 \) kN
Modified shear resistance: \( v_{Rdc} = v_{Rdm} = 103 \) kN
### SHEAR SUMMARY

<table>
<thead>
<tr>
<th></th>
<th>Design shear force</th>
<th>36.9 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design shear resistance</td>
<td>103 kN</td>
</tr>
</tbody>
</table>

### DESIGN SUMMARY

<table>
<thead>
<tr>
<th></th>
<th>Actual l/d ratio</th>
<th>23.68</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic l/d ratio</td>
<td>43.84</td>
<td></td>
</tr>
<tr>
<td>Span factor</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

### DEFLECTION

<table>
<thead>
<tr>
<th>Factor</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flange beam factor F1</td>
<td>1</td>
</tr>
<tr>
<td>Long spans factor F2</td>
<td>1</td>
</tr>
<tr>
<td>Steel stress factor F3</td>
<td>1.068</td>
</tr>
<tr>
<td>Allowable l/d ratio</td>
<td>46.83</td>
</tr>
</tbody>
</table>

**NOTE:** If the percentage reinforcement is less than 0.4% care should be taken with the EC2 values as the deflection equations used give unrealistically high values.

Table 7.4N gives span/effective depth ratios for C30/37 concrete and reinforcement percentage, p=0.5% as:
- Simply supported spans: 20
- End spans: 26
- Interior spans: 30
Location: Beam B1-B2

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Rectangular beam/slab section

Calculations for reinforcement are in accordance with BS8110.

Normal-weight concrete only.

Moment before redistribution $M_{befor}=181.42$ kNm
Shear force due to ultimate loads $V=50$ kN
Axial load due to ultimate loads $N=0$ kN
Length of beam/slab $len=5$ m
Overall depth of concrete section $h=400$ mm
Breadth of concrete section $b=200$ mm
Cover to all reinforcement $cover=20$ mm
Eff.depth of concrete section $d=357.5$ mm
Depth to compression steel $d'=40$ mm
BM at section after redistrib. $M=127$ kNm
Characteristic concrete strength $f_{cu}=40$ N/mm$^2$
Characteristic strength of links $f_{yv}=500$ N/mm$^2$
Diameter of tension bars $dia=25$ mm
Diameter of links $dial=10$ mm
Diameter of compression bars $diac=12$ mm

<table>
<thead>
<tr>
<th>COMPRESSION</th>
<th>Characteristic strength $500$ N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars $12$ mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars $2$</td>
</tr>
<tr>
<td></td>
<td>Area required $160$ mm$^2$</td>
</tr>
<tr>
<td></td>
<td>Area provided $226.19$ mm$^2$</td>
</tr>
</tbody>
</table>

See BS 8110 (Anchorage length $330$ mm)
Cl.3.12.8.15 (Lap length $410$ mm)

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength $500$ N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars $25$ mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars $2$</td>
</tr>
<tr>
<td></td>
<td>Area required $953.04$ mm$^2$</td>
</tr>
<tr>
<td></td>
<td>Area provided $981.75$ mm$^2$</td>
</tr>
</tbody>
</table>

See BS 8110 (Anchorage & tension lap $860$ mm)
Part 1 Clause $1.4$ * tension lap $1210$ mm
3.12.8.13 $2.0$ * tension lap $1730$ mm

Shear reinforcement

No. of long. bars eff. for shear $nbars=2$
Design shear stress of $0.6993$ N/mm$^2$ does not exceed critical shear stress of $0.84499$ N/mm$^2$, so only nominal links need be provided.
<table>
<thead>
<tr>
<th>SHEAR</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
<td>10 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of legs</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>140 mm</td>
</tr>
<tr>
<td></td>
<td>Ensure that no longitudinal bar is more than 150 mm from vertical leg</td>
<td></td>
</tr>
</tbody>
</table>
Location: Small beam with high moment and shear

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$

Moment before redistribution $M_{\text{befor}} = 250 \text{ kNm}$
Shear force due to ultimate loads $V = 200 \text{ kN}$
Axial load due to ultimate loads $N = 50 \text{ kN}$
Length of beam/slab $l_{\text{en}} = 6 \text{ m}$
Overall depth of concrete section $h = 300 \text{ mm}$
Breadth of concrete section $b = 200 \text{ mm}$
Cover to all reinforcement $c_{\text{over}} = 20 \text{ mm}$
Effective depth of concrete section $d = 252 \text{ mm}$
Depth to compression steel $d' = 41 \text{ mm}$
BM at section after redistribution $M = 200 \text{ kNm}$
Characteristic concrete strength $f_{\text{cu}} = 40 \text{ N/mm}^2$
Characteristic strength of links $f_{\text{yv}} = 500 \text{ N/mm}^2$
Diameter of tension bars $d_{\text{ia}} = 40 \text{ mm}$
Diameter of links $d_{\text{ial}} = 8 \text{ mm}$
Diameter of compression bars $d_{\text{iac}} = 25 \text{ mm}$

<table>
<thead>
<tr>
<th>COMPRESSION</th>
<th>REINFORCEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>Diameter of bars</td>
</tr>
<tr>
<td>Number of bars</td>
<td>Area required</td>
</tr>
<tr>
<td>Area provided</td>
<td>See BS 8110 (Anchorage length)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>See BS 8110 (Anchorage length)</td>
</tr>
<tr>
<td>Cl.3.12.8.15 (Lap length)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TENSION</th>
<th>REINFORCEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>Diameter of bars</td>
</tr>
<tr>
<td>Number of bars</td>
<td>Area required</td>
</tr>
<tr>
<td>Area provided</td>
<td>See BS 8110</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>See BS 8110</td>
</tr>
<tr>
<td>Part 1 Clause</td>
</tr>
<tr>
<td>3.12.8.13</td>
</tr>
</tbody>
</table>

Shear reinforcement

No. of long bars eff. for shear $n_{\text{bars}} = 2$

Design shear stress of $3.9683 \text{ N/mm}^2$ exceeds critical stress of $1.1966 \text{ N/mm}^2$, so links to resist balance of shear forces are necessary.
### SHEAR
- Characteristic strength: 500 N/mm²

### REINFORCEMENT
- Diameter of bars: 8 mm
- Number of legs: 2
- Spacing: 70 mm

### SUMMARY
Ensure that no longitudinal bar is more than 150 mm from vertical leg

### DEFLECTION
- Permissible span: 5.9363 m

### SUMMARY
- Actual span: 6 m

**WARNING:**
As actual span > permissible, deflection criteria are violated.
Location: Deep, narrow beam with minimum moment and shear

Design to BS8110 (1997) with partial safety factor for steel $\gamma_S = 1.15$

Rectangular beam/slab section

Calculations for reinforcement are in accordance with BS8110.

Normal-weight concrete only.

Moment before redistribution $M_{\text{befor}} = 10 \text{ kNm}$
Shear force due to ultimate loads $V = 5 \text{ kN}$
Axial load due to ultimate loads $N = 0 \text{ kN}$
Length of beam/slab $\text{len} = 6.75 \text{ m}$
Overall depth of concrete section $h = 1500 \text{ mm}$
Breadth of concrete section $b = 200 \text{ mm}$
Cover to all reinforcement $\text{cover} = 35 \text{ mm}$
Eff. depth of concrete section $d = 1440 \text{ mm}$
Depth to compression steel $d' = 60 \text{ mm}$
BM at section after redistrib. $M = 10 \text{ kNm}$
Characteristic concrete strength $f_{\text{cu}} = 40 \text{ N/mm}^2$
Characteristic strength of links $f_{\text{yv}} = 250 \text{ N/mm}^2$
Diameter of tension bars $\text{dia} = 25 \text{ mm}$
Diameter of links $\text{dial} = 10 \text{ mm}$

TENSION
REINFORCEMENT
SUMMARY
Characteristic strength $500 \text{ N/mm}^2$
Diameter of bars $25 \text{ mm}$
Number of bars $2$
Area required $390 \text{ mm}^2$
Area provided $981.75 \text{ mm}^2$
See BS 8110
Part 1 Clause 1.4 * tension lap $860 \text{ mm}$
3.12.8.13 2.0 * tension lap $1730 \text{ mm}$

Shear reinforcement

No. of long. bars eff. for shear $n_{\text{bars}} = 2$
Design shear stress of $0.017361 \text{ N/mm}^2$ does not exceed critical shear stress of $0.51637 \text{ N/mm}^2$, so only nominal links need be provided.

SHEAR
REINFORCEMENT
SUMMARY
Characteristic strength $250 \text{ N/mm}^2$
Diameter of bars $10 \text{ mm}$
Number of legs $2$
Spacing $420 \text{ mm}$
Ensure that no longitudinal bar is more than $150 \text{ mm}$ from vertical leg
Diameter provided                 dias=16 mm

<table>
<thead>
<tr>
<th>SIDE BAR</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars/side</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
<td>200 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DEFLECTION</th>
<th>Permissible span</th>
<th>57.6 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Actual span</td>
<td>6.75 m</td>
</tr>
</tbody>
</table>

As actual span does not exceed permissible, deflection is OK.
Location: Deep, heavily loaded slab

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$

---

**Rectangular beam/slab section**

Calculations for reinforcement are in accordance with BS8110.

Normal-weight concrete only.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment before redistribution ($M_{befor}$)</td>
<td>1000 kNm</td>
</tr>
<tr>
<td>Shear force due to ultimate loads ($V$)</td>
<td>340 kN</td>
</tr>
<tr>
<td>Axial load due to ultimate loads ($N$)</td>
<td>0 kN</td>
</tr>
<tr>
<td>Length of beam/slab ($l_{en}$)</td>
<td>4.75 m</td>
</tr>
<tr>
<td>Overall depth of concrete section ($h$)</td>
<td>400 mm</td>
</tr>
<tr>
<td>Breadth of concrete section ($b$)</td>
<td>1000 mm</td>
</tr>
<tr>
<td>Cover to all reinforcement ($cover$)</td>
<td>30 mm</td>
</tr>
<tr>
<td>Eff.depth of concrete section ($d$)</td>
<td>340 mm</td>
</tr>
<tr>
<td>Depth to compression steel ($d'$)</td>
<td>60 mm</td>
</tr>
<tr>
<td>BM at section after redistrib. ($M$)</td>
<td>900 kNm</td>
</tr>
<tr>
<td>Characteristic concrete strength ($f_{cu}$)</td>
<td>60 N/mm²</td>
</tr>
<tr>
<td>Characteristic strength of links ($f_{yv}$)</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of tension bars ($dia$)</td>
<td>32 mm</td>
</tr>
<tr>
<td>Diameter of links ($dial$)</td>
<td>10 mm</td>
</tr>
</tbody>
</table>

**TENSION**

- Diameter of bars: 32 mm

**REINFORCEMENT**

- Number of bars: 10
- Area required: 7376.9 mm²
- Area provided: 8042.5 mm²

See BS 8110 Part 1 Clause 1.4 * tension lap: 900 mm

- 3.12.8.13 2.0 * tension lap: 1270 mm

**DISTRIBUTION**

- Diameter of distribution bars ($diamn$): 12 mm

**REINFORCEMENT**

- Diameter of bars: 12 mm
- Spacing of bars: 200 mm
- Area of steel required: 520 mm²/m
- Area of steel provided: 565 mm²/m

**Shear reinforcement**

- No. of long. bars eff. for shear: 10

As design shear stress of 1 N/mm² does not exceed critical shear stress of 1.0257 N/mm², no shear reinforcement is required.
<table>
<thead>
<tr>
<th>DEFLECTION</th>
<th>Permissible span</th>
<th>6.0263 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Actual span</td>
<td>4.75 m</td>
</tr>
</tbody>
</table>

As actual span does not exceed permissible, deflection is OK.
Location: Default example

Rectangular beam/slab design

Calculations are based on EN1992-1-1:2004 Eurocode 2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.45*d.

Moment before redistribution \( M_{bef} = 221 \text{ kNm} \)
Shear force due to ultimate loads \( V_{Ed} = 102 \text{ kN} \)
Axial load due to ultimate loads \( N_{Ed} = 0 \text{ kN} \)
Length of beam/slab \( L = 5 \text{ m} \)
Overall depth of concrete section \( h = 500 \text{ mm} \)
Breadth of concrete section \( b = 250 \text{ mm} \)
The member containing the section being considered is supported at both ends and is continuous at one or both ends.
Section considered has a hogging moment (support condition).
Char yield strength of reinft \( f_{yk} = 500 \text{ N/mm}^2 \)
Max. aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( d_{ia} = 25 \text{ mm} \)
Diameter of link legs \( d_{ial} = 10 \text{ mm} \)
BM after redistribution \( M = 221 \text{ kNm} \)
Effective depth \( d = 415 \text{ mm} \)
Diameter of compression bars \( d_{iac} = 20 \text{ mm} \)
Depth to compression steel \( d_2 = 55 \text{ mm} \)

**DESIGN**

Overall depth 500 mm
Effective depth 415 mm
Width of section 250 mm

**SUMMARY**

Parameter K 0.1711
Parameter K' 0.1684
Lever arm ratio \( z/d \) 0.8185

**FLEXURE**

Steel area provided 1963 mm\(^2\)
Diameter of bars 25 mm
Number of bars 4
Steel area required 1495 mm\(^2\)
Steel percentage req. 1.441 %
Minimum area of steel 156.3 mm\(^2\)
Maximum area of steel 5000 mm\(^2\)

**TENSION REINFORCEMENT**

Steel area provided 628.3 mm\(^2\)
Diameter of bars 20 mm
Number of bars 2
Steel area required 22.23 mm\(^2\)
Minimum area required 250 mm\(^2\)
**Shear reinforcement**

- **Design shear stress**  
  \( \nu_{Ed} = \frac{V_{Ed} \times 10^3}{(0.9 \times b \times w \times d)} = 1.092 \text{ N/mm}^2 \)

- **Concrete strut capacity**  
  \( \nu_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 3.274 \text{ N/mm}^2 \)

- **Angle of inclination of strut**  
  \( \theta' = 22^\circ \)

- **Number of shear legs**  
  \( n_{sl} = 2 \)

- **Chosen spacing**  
  \( s = 300 \text{ mm} \)

**SHEAR SUMMARY**

- **Design shear force**  
  102 kN

- **Design shear stress**  
  1.092 N/mm\(^2\)

- **Concrete strut capacity**  
  3.274 N/mm\(^2\)

- **Area/spacing ratio**  
  0.2512

- **Diameter of links**  
  10 mm

- **Area of links**  
  157 mm\(^2\)

- **Maximum spacing**  
  311.3 mm

  (based on 0.75d)

- **Actual spacing**  
  300 mm
Location: Slab arrangement

Rectangular beam/slab design

Calculations are based on EN1992-1-1:2004 Eurocode 2:Design of concrete structures and assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.45*d.

Moment before redistribution \( M_{bef} = 200 \text{ kNm} \)
Shear force due to ultimate loads \( V_{Ed} = 100 \text{ kN} \)
Axial load due to ultimate loads \( N_{Ed} = 0 \text{ kN} \)
Length of beam/slab \( L = 6 \text{ m} \)
Overall depth of concrete section \( h = 400 \text{ mm} \)
Breadth of concrete section \( b = 1000 \text{ mm} \)

The member containing the section being considered is supported at both ends and is continuous at one or both ends.

Section considered has a hogging moment (support condition).
Char yield strength of reinft \( f_{yk} = 500 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( hagg = 20 \text{ mm} \)
Diameter of tension bars \( \text{dia} = 25 \text{ mm} \)
BM after redistribution \( M = 221 \text{ kNm} \)
Effective depth \( d = 415 \text{ mm} \)

| DESIGN                         | Parameter K | 0.0428 |
| SUMMARY                        | Parameter K' | 0.1684 |
| FLEXURE                        | Lever arm ratio z/d | 0.95 |
|                               | Steel area (tension) | 1289 mm$^2$/m |
|                               | Steel percentage req. | 0.3107 % |
|                               | Minimum area of steel | 625.1 mm$^2$/m |
|                               | Maximum area of steel | 16000 mm$^2$/m |
|                               | Distribution steel | 625.1 mm$^2$/m |
| Chosen spacing of tension bars | pch = 350 mm |
| Diameter of distribution bars  | diad = 12 mm |
| Chosen spacing of distn. bars  | pchd = 150 mm |

| TENSION REINFORCEMENT          | Diameter of bars | 25 mm |
|                                | Spacing of bars | 350 mm |
|                                | Area of steel required | 1289 mm$^2$/m |
|                                | Area of steel provided | 1402 mm$^2$/m |

| DISTRIBUTION REINFORCEMENT     | Diameter of bars | 12 mm |
|                                | Spacing of bars | 150 mm |
|                                | Area of steel required | 625.1 mm$^2$/m |
|                                | Area of steel provided | 753 mm$^2$/m |
### Shear reinforcement

<table>
<thead>
<tr>
<th>Term for shear resistance</th>
<th>vRdc=(ρ1*fck)^(1/3)=2.164</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>VRdc=CRdc<em>ks</em>vRdc<em>bw</em>d/1000=182.6 kN</td>
</tr>
<tr>
<td>Minimum stress</td>
<td>vmin=0.035<em>ks^1.5</em>fck^0.5=0.4227 N/mm²</td>
</tr>
<tr>
<td>Minimum shear resistance</td>
<td>VRdm=vmin<em>bw</em>d/1000=175.4 kN</td>
</tr>
</tbody>
</table>

**SHEAR SUMMARY**

- Design shear force: 100 kN
- Design shear resistance: 182.6 kN
Location: Columns C2-C6, D2-D6, E4 & E5

Design to BS8110(1997), partial safety factor for steel $\gamma_S = 1.15$

Biaxially bent stocky column

Calculations for reinforcement are in accordance with BS 8110 Clause 3.8.4.5 and ISE 'Manual for the design of reinforced concrete building structures'. For effective height of column see Clause 3.8.1.6 of BS8110.

Design ultimate moment about z-axis $M_z = 160$ kNm
Shear force in y-direction $F_y = 100$ kN
Design ultimate axial load $N = 2460$ kN
Effective height of column $h = 2.5$ m
Depth of column cross-section $h' = 450$ mm
Width of column cross-section $b = 250$ mm
Design ultimate moment about y-axis $M_y = 0$ kNm
Cover to all reinforcement $cover = 20$ mm
Effective depth in y-direction $h' = 394$ mm
Effective depth in z-direction $b' = 194$ mm
Characteristic strength of concrete $f_{cu} = 50$ N/mm²
Bar diameter for main steel $dia = 32$ mm

SUMMARY OF REINFORCEMENT REQUIRED

<table>
<thead>
<tr>
<th>VERTICAL REINFORCEMENT SUMMARY</th>
<th>Characteristic strength $500$ N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>4</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>$32$ mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LINK REINFORCEMENT SUMMARY</th>
<th>Diameter of links $8$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing</td>
<td>$250$ mm</td>
</tr>
</tbody>
</table>

Links round every corner bar and each alternate bar. Included angle must not exceed 135 deg. No bar must be more than 150 mm from a bar restrained by a link. See Clause 3.12.7.2.

Due to the column being lightly loaded sufficient longitudinal steel must be provided to control cracking.
Location: Small square column. Max. high-yield steel. High mom & load

Design to BS8110(1997), partial safety factor for steel gammaS=1.15

Biaxially bent stocky column

Calculations for reinforcement are in accordance with BS 8110 Clause 3.8.4.5 and ISE 'Manual for the design of reinforced concrete building structures'. For effective height of column see Clause 3.8.1.6 of BS8110.

Design ult. moment about z-axis $M_z=38 \text{ kNm}$
Shear force in y-direction $F_y=10 \text{ kN}$
Design ultimate axial load $N=400 \text{ kN}$
Effective height of column $l=2.5 \text{ m}$
Depth of column cross-section $h=200 \text{ mm}$
Width of column cross-section $b=200 \text{ mm}$
Design ult. moment about y-axis $M_y=38 \text{ kNm}$
Shear force in z-direction $F_z=0 \text{ kN}$
Cover to all reinforcement $\text{cover}=20 \text{ mm}$
Effective depth in y-direction $h'=150 \text{ mm}$
Effective depth in z-direction $b'=150 \text{ mm}$
Characteristic strength of conc. $f_{cu}=60 \text{ N/mm}^2$
Bar diameter for main steel $\text{dia}=16 \text{ mm}$

SUMMARY OF REINFORCEMENT REQUIRED

<table>
<thead>
<tr>
<th>VERTICAL</th>
<th>Characteristic strength</th>
<th>500 N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Number of bars</td>
<td>12</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>LINK</td>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>REINFORCEMENT</td>
<td>Spacing</td>
<td>190 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Links round every corner bar and each alternate bar. Included angle must not exceed 135 deg. No bar must be more than 150 mm from a bar restrained by a link. See Clause 3.12.7.2.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Due to the column being lightly loaded sufficient longitudinal steel must be provided to control cracking.</td>
<td></td>
</tr>
</tbody>
</table>
Location: Large column. Min. high-yield steel. Moment and low load.

Design to BS8110(1997), partial safety factor for steel $\gamma_S = 1.15$

![Biaxially bent stocky column](image)

Calculations for reinforcement are in accordance with BS 8110 Clause 3.8.4.5 and ISE 'Manual for the design of reinforced concrete building structures'. For effective height of column see Clause 3.8.1.6 of BS8110.

**SUMMARY OF REINFORCEMENT REQUIRED**

<table>
<thead>
<tr>
<th>VERTICAL</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Number of bars</td>
<td>10</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Diameter of bars</td>
<td>40 mm</td>
</tr>
<tr>
<td>LINK</td>
<td>Diameter of links</td>
<td>10 mm</td>
</tr>
<tr>
<td>REINFORCEMENT</td>
<td>Spacing</td>
<td>480 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Links round every corner bar and each alternate bar. Included angle must not exceed 135 deg. No bar must be more than 150 mm from a bar restrained by a link. See Clause 3.12.7.2.</td>
<td></td>
</tr>
</tbody>
</table>

Due to the column being lightly loaded sufficient longitudinal steel must be provided to control cracking.
Location: Biaxial bending example

Isolated rectangular section subject bending and thrust

Design is based on EN 1992-1-1:2004, Eurocode 2: Design of concrete structures. The main reinforcement design is based on balancing, by iterative means, of the two derived expressions for areas of reinforcing steel - one for axial load and the other for bending. One layer of steel is assumed for the axis of bending.

Design ult.moment about y-axis \( M_{yy} = 146.1 \text{ kNm} \)
Shear force in z-direction \( V_{Ed} = 0 \text{ kN} \)
Design ultimate axial load \( N_{Ed} = 1650 \text{ kN} \)
Effective height of column \( l = 3.2 \text{ m} \)
Depth of column cross-section \( h = 350 \text{ mm} \)
Width of column cross-section \( b = 350 \text{ mm} \)
Char yield strength of reinf'ment \( f_{yk} = 500 \text{ N/mm}^2 \)
Max.aggregate size (for bar spc.) \( h_{agg} = 25 \text{ mm} \)
Diameter of compression bars \( \text{dia} = 32 \text{ mm} \)
Diameter of link legs \( \text{dial} = 8 \text{ mm} \)
Depth to comp. reinforcement \( d_2 = 88 \text{ mm} \)
Smaller initial end moment \( M_{1y} = 0 \text{ kNm} \)
Larger initial end mt. (z-z axis) \( M_{2z} = 114.5 \text{ kNm} \)
Smaller initial end moment \( M_{1z} = 0 \text{ kNm} \)

About Y-Y axis:
Column under consideration is braced.

About Z-Z axis:
Column under consideration is braced.

Number of bars provided \( n_{bars} = 8 \)
### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear column height</td>
<td>3.2 m</td>
</tr>
<tr>
<td>Slenderness (y-y axis)</td>
<td>26.921</td>
</tr>
<tr>
<td>Column is short (y-y)</td>
<td></td>
</tr>
<tr>
<td>Slenderness (z-z axis)</td>
<td>26.921</td>
</tr>
<tr>
<td>Column is short (z-z)</td>
<td></td>
</tr>
<tr>
<td>Design axial load</td>
<td>1650 kN</td>
</tr>
<tr>
<td>Design bending moments:</td>
<td></td>
</tr>
<tr>
<td>Y-Y axis</td>
<td>159.3 kNm</td>
</tr>
<tr>
<td>Z-Z axis</td>
<td>127.7 kNm</td>
</tr>
<tr>
<td>Designed area of steel</td>
<td>3047.1 mm²</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>6434 mm²</td>
</tr>
<tr>
<td>Minimum area of steel</td>
<td>379.5 mm²</td>
</tr>
<tr>
<td>Maximum area of steel</td>
<td>4900 mm²</td>
</tr>
<tr>
<td>Link diameter</td>
<td>8 mm</td>
</tr>
<tr>
<td>Maximum link spacing</td>
<td>350 mm</td>
</tr>
<tr>
<td>Resistance moment (y-y)</td>
<td>256.51 kNm</td>
</tr>
<tr>
<td>Resistance moment (z-z)</td>
<td>256.51 kNm</td>
</tr>
<tr>
<td>Biaxial bending factor</td>
<td>0.99851</td>
</tr>
</tbody>
</table>

Designed steel exceeds 4% of gross area. However the maximum area can be increased provided that the concrete can be placed and compacted sufficiently.
Design of flanged beam-and-slab section to IStructE Manual

Effective width of flange = b mm

Flanged beam section

Calculations for reinforcement are in accordance with IStructE 'Manual for the design of reinforced concrete building structures' and based on simplified rectangular concrete stress-block.

BM before redistribution Mbefor=252 kNm
Ultimate shear force V=84 kN
Axial load due to ultimate loads N=0 kN
Length of flanged beam len=10 m
Overall depth of section h=500 mm
Actual flange width ba=2000 mm
Flange thickness hf=100 mm
Breadth of rib bw=300 mm
Ribbed slab=(1) flanged beam=(0) ribslab=0
Characteristic concrete strength fcu=35 N/mm²
Char.strength of long'¹ bars fy=500 N/mm²
High-yield steel reinforcement.
Bond type (0,1 or 2) Type=2
Characteristic strength of links fyv=500 N/mm²
Diameter of longitudinal bars dia=25 mm
Diameter of links dial=8 mm
Designated exposure condition is Moderate.
From Table 18 of IStructE Manual, minimum cover to all steel required for fcu=35 N/mm² and exposure condition 2 is 35 mm.
From Table 17 of IStructE Manual, minimum cover to main bars needed for continuous beam with a fire period of 2 hour/s is 50 mm.
As actual rib width of 300 mm exceeds minimum width of 150 mm specified in Table 17, cover may be decreased by 15 mm (Cl.4.4.2.1).
Thus min.cover to main bars to meet fire reqs. cover2=cover2-minus=35 mm
Thus minimum permissible cover to all steel is 35 mm
Concrete cover to all steel cover=35 mm
BM after redistribution M=252 kNm

TENSION

REINFORCEMENT

SUMMARY

Characteristic strength 500 N/mm²
Diameter of bars 25 mm
Number of bars 3
Area required 1386.5 mm²
Area provided 1472.6 mm²
Percentage provided (of rib area) 0.98175 %
See Manual
: Clause : 1.4 * tension lap 1150 mm
: 4.12.3 : 2.0 * tension lap 1630 mm

Weight of steel provided 11.56 kg/m
Shear reinforcement

Design shear stress \( v = \frac{V}{1000 \cdot (b \cdot d)} = 0.62992 \text{ N/mm}^2 \)
Size of tension bars at support \( d_{\text{ias}} = 25 \text{ mm} \)
No. of tension bars at support \( n_{\text{bars}} = 3 \)
From Table 27 of IStructE Manual, design concrete shear stress \( v_c = \text{TABLE 27 for percent}=1.1043, d=444.5 \)
\( = 0.68878 \text{ N/mm}^2 \)
As characteristic concrete strength is not equal to 30 N/mm², modify tabulated value of \( v_c \) according to footnote to Table 27.
Modification factor \( \text{factor} = \text{TABLE 27.1 for } f_{cu}=35 \)
\( = 1.053 \)
Modified concrete shear stress \( v_c = v_c \cdot \text{factor} = 0.72528 \text{ N/mm}^2 \)
Area of 2 legs of a link \( A_{sv} = n_{\text{legs}} \cdot \pi \cdot d_{\text{ias}}^2 / 4 = 100.53 \text{ mm}^2 \)
Provide minimum links as specified in Table 28 of Manual.
Spacing of links along member \( s_v = A_{sv} \cdot 0.87 \cdot f_{yv} / (0.4 \cdot b) = 364.42 \text{ mm} \)
Limit spacing to \( s_v = 0.75 \cdot d = 333.38 \text{ mm} \) (to Clause 4.4.5.3)

<table>
<thead>
<tr>
<th>SHEAR</th>
<th>Characteristic strength 500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 8 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of legs 2</td>
</tr>
<tr>
<td></td>
<td>Spacing 330 mm</td>
</tr>
<tr>
<td></td>
<td>Approx. weight of links 1.5974 kg/m</td>
</tr>
<tr>
<td></td>
<td>Ensure that no longitudinal bar is more than 150 mm from vertical leg.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DEFLECTION</th>
<th>Permissible span 13.58 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Actual span 10 m</td>
</tr>
</tbody>
</table>
Location: Ribbed L-slab: low mom/high shear/mild steel/min exposure

Design of flanged beam-and-slab section to IStructE Manual

Effective width of flange = b mm

![Diagram of flanged beam section]

Flanged beam section

Calculations for reinforcement are in accordance with IStructE 'Manual for the design of reinforced concrete building structures' and based on simplified rectangular concrete stress-block.

BM before redistribution: M_{befor}=30 kNm
Ultimate shear force: V=204 kN
Axial load due to ultimate loads: N=100 kN
Length of flanged beam: len=6 m
Overall depth of section: h=250 mm
Actual flange width: b_a=1000 mm
Flange thickness: h_f=100 mm
Breadth of rib: b_w=300 mm
Ribbed slab=(1) flanged beam=(0) ribslab=1
Characteristic concrete strength: f_{cu}=30 N/mm²
Char. strength of long’l bars: f_y=250 N/mm²
Mild steel reinforcement.
Bond type (0,1 or 2): Type=0
Characteristic strength of links: f_{yv}=250 N/mm²
Diameter of longitudinal bars: d_{ia}=25 mm
Diameter of links: d_{il}=8 mm
Designated exposure condition is Mild.
From Table 18 of IStructE Manual, minimum cover to all steel required for f_{cu}=30 N/mm² and exposure condition 1 is 25 mm.
From Table 17 of Manual, minimum cover for main bars with simply-supported ribbed slab with a fire period of 1 hour/s is 25 mm.
Thus minimum permissible cover to all steel is 25 mm
Concrete cover to all steel: c_{over}=25 mm

### TENSION

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
</tbody>
</table>

### REINFORCEMENT

<table>
<thead>
<tr>
<th>Number of bars</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area required</td>
<td>717.53 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>981.75 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>1.309 %</td>
</tr>
</tbody>
</table>

### SUMMARY

<table>
<thead>
<tr>
<th>See Manual</th>
<th>Anchorage &amp; tension lap</th>
<th>660 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>: Clause :</td>
<td>1.4 * tension lap</td>
<td>920 mm</td>
</tr>
<tr>
<td>: 4.12.3 :</td>
<td>2.0 * tension lap</td>
<td>1300 mm</td>
</tr>
</tbody>
</table>

Weight of steel provided: 7.7067 kg/m
Shear reinforcement

Design shear stress \( v = \frac{V}{1000 (bv \cdot d)} = 3.3252 \text{ N/mm}^2 \)

Longit. bars effective for shear \( n_{bars} = 2 \)

From Table 27 of IStructE Manual,

design concrete shear stress \( v_c = \text{TABLE 27 for percent}=1.6002, d=204.5 \text{ mm} \)

= \( 0.92429 \text{ N/mm}^2 \)

As \( v \) exceeds \( v_c \), shear reinforcement must be provided in a ribbed slab (see Clause 4.2.6.3).

Area of 2 legs of a link \( A_{sv} = n_{legs} \cdot \pi \cdot d_{ial}^2 / 4 = 100.53 \text{ mm}^2 \)

Provide links to resist shear force in accordance with Clause 4.2.6.3.

Spacing of links along member \( s_v = A_{sv} \cdot 0.87 \cdot f_{yv} / (bv \cdot (v - v_c)) = 30.357 \text{ mm} \)

Increase spacing to about one-half of the maximum distance permitted and increase the number of link legs correspondingly.

Increase number of link legs to \( n_{legs} = \text{INT}(n_{legs} \cdot 0.375 \cdot d / s_v) + 1 = 6 \)

at revised spacing of \( s_v = 0.375 \cdot d = 76.688 \text{ mm} \)

<table>
<thead>
<tr>
<th>SHEAR</th>
<th>Characteristic strength 250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 8 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of legs 6</td>
</tr>
<tr>
<td></td>
<td>Spacing 70 mm</td>
</tr>
<tr>
<td></td>
<td>Approx. weight of links 14.182 kg/m</td>
</tr>
</tbody>
</table>

Ensure that no longitudinal bar is more than 150 mm from vertical leg.

| DEFLECTION | Permissible span 6.6184 m |
| SUMMARY    | Actual span 6 m |
Location: Large T-beam: high moment: low shear: severe exposure

Design of flanged beam-and-slab section to IStructE Manual

Effective width of flange = b mm

Flanged beam section

Calculations for reinforcement are in accordance with IStructE 'Manual for the design of reinforced concrete building structures' and based on simplified rectangular concrete stress-block.

BM before redistribution
M_{befor}=10000\ \text{kNm}

Ultimate shear force
V=1000\ \text{kN}

Axial load due to ultimate loads
N=0\ \text{kN}

Length of flanged beam
len=24\ \text{m}

Overall depth of section
h=2000\ \text{mm}

Actual flange width
ba=10000\ \text{mm}

Flange thickness
hf=250\ \text{mm}

Breadth of rib
bw=500\ \text{mm}

Ribbed slab=(1) flanged beam=(0) ribslab=0

Characteristic concrete strength
f_{cu}=50\ \text{N/mm}^2

Char.strength of long’l bars
f_{y}=500\ \text{N/mm}^2

High-yield steel reinforcement.

Bond type (0,1 or 2)
Type=2

Characteristic strength of links
f_{yv}=500\ \text{N/mm}^2

Diameter of longitudinal bars
dia=40\ \text{mm}

Diameter of links
dial=10\ \text{mm}

Designated exposure condition is Very Severe.

From Table 18 of IStructE Manual, minimum cover to all steel required for f_{cu}=40\ \text{N/mm}^2 and exposure condition 4 is 50 mm.

From Table 17 of IStructE Manual, minimum cover to main bars needed for continuous beam with a fire period of 4 hour/s is 70 mm.

As actual rib width of 500 mm exceeds minimum width of 240 mm specified in Table 17, cover may be decreased by 15 mm (Cl.4.4.2.1).

Thus min.cover to main bars to meet fire reqs. cover2=cover2-minus=55 mm

Thus minimum permissible cover to all steel is 50 mm

Concrete cover to all steel
cover=50\ \text{mm}

BM after redistribution
M=10000\ \text{kNm}

TENSION
Characteristic strength 500\ \text{N/mm}^2

REINFORCEMENT
Diameter of bars 40\ \text{mm}

Number of bars 11

Area required 12737\ \text{mm}^2

Area provided 13823\ \text{mm}^2

Percentage provided
(of rib area) 1.3823\%

SUMMARY
Weight of steel provided 108.51\ \text{kg/m}

See Manual

: Clause : 1.4 * tension lap 1660\ \text{mm}

: 4.12.3 : 2.0 * tension lap 2360\ \text{mm}
Shear reinforcement

Design shear stress \( v = \frac{V \times 1000}{(b \times d)} = 1.0417 \text{ N/mm}^2 \)

Size of tension bars at support \( d_{\text{ias}} = 40 \text{ mm} \)

No. of tension bars at support \( n_{\text{bars}} = 11 \)

From Table 27 of IStructE Manual,

design concrete shear stress \( v_c = \text{TABLE 27 for } \% = 1.4399, d = 1920 \)= 0.74918 \text{ N/mm}^2

As characteristic concrete strength is not equal to 30 \text{ N/mm}^2,
modify tabulated value of \( v_c \) according to footnote to Table 27.
Modification factor \( \text{factor = TABLE 27.1 for } f_{cu} = 50 \)= 1.1

Modified concrete shear stress \( v_c = v_c \times \text{factor} = 0.8241 \text{ N/mm}^2 \)

Area of 2 legs of a link \( A_{sv} = n_{legs} \times \pi \times d_{\text{ial}}^2 / 4 = 157.08 \text{ mm}^2 \)

Provide minimum links as specified in Table 28 of Manual.
Spacing of links along member \( s_v = A_{sv} \times 0.87 \times f_{yv} / (0.4 \times b_v) = 341.65 \text{ mm} \)

Increase spacing to 300 mm together with a corresponding increase
in the number of link legs to \( n_{legs} = \lfloor n_{legs} \times 300 / s_v \rfloor + 1 = 2 \)
at revised spacing of \( s_v = 300 \text{ mm} \)

---

**DEFECTION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>( 500 \text{ N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>10 mm</td>
</tr>
<tr>
<td>Number of legs</td>
<td>2</td>
</tr>
<tr>
<td>Spacing</td>
<td>300 mm</td>
</tr>
<tr>
<td>Approx. weight of links</td>
<td>9.4941 kg/m</td>
</tr>
<tr>
<td>Ensure that no longitudinal bar is more than 150 mm from vertical leg.</td>
<td></td>
</tr>
</tbody>
</table>

---

**SUMMARY**

<table>
<thead>
<tr>
<th>Permissible span</th>
<th>25.53 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual span</td>
<td>24 m</td>
</tr>
</tbody>
</table>
**Location: Ex1 - Default example**

**Bending in flanged beams**

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Moment before redistribution \( M_{bef} = 600 \text{ kNm} \)
Shear force due to ultimate loads \( V_{Ed} = 267 \text{ kN} \)
Axial load due to ultimate loads \( N_{Ed} = 0 \text{ kN} \)
Length of beam/slab \( L = 8.2 \text{ m} \)
Overall depth of section \( h = 650 \text{ mm} \)
Actual flange width \( b_a = 3350 \text{ mm} \)
Flange thickness \( h_f = 200 \text{ mm} \)
Breadth of rib \( b_w = 350 \text{ mm} \)

Section being analysed is considered as non-continuous.
Char yield strength of reinforcement \( f_{yk} = 500 \text{ N/mm}^2 \)
Max. aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( d_{ia} = 25 \text{ mm} \)
Diameter of link legs \( d_{ial} = 8 \text{ mm} \)
BM after redistribution \( M = 600 \text{ kNm} \)
Effective depth of section \( d = 580 \text{ mm} \)

**DESIGN**

- Overall depth: 650 mm
- Effective depth: 580 mm
- Flange width: 2590 mm
- Web width: 350 mm

**FLEXURE**

- Parameter K: 0.023
- Parameter K': 0.1684
- Lever arm ratio z/d: 0.95

**TENSION REINFORCEMENT**

- Steel area required: 2505 mm²
- Steel area provided: 2945 mm²
- Diameter of bars: 25 mm
- Number of bars: 6
- Steel percentage req.: 0.1667 %
- Minimum area of steel: 305.8 mm²
- Maximum area of steel: 67340 mm²

**Shear check**

- Design shear stress \( V_{Ed} = V_{Ed} * 10^3 / (0.9 * bw * d) = 1.461 \text{ N/mm}^2 \)
- Concrete strut capacity \( V_{Rdm} = 0.124 * f_{ck} * (1 - f_{ck} / 250) = 3.274 \text{ N/mm}^2 \)
- Angle of inclination of strut \( \theta' = 22^\circ \)
- Number of shear legs \( n_{sl} = 2 \)
- Chosen spacing \( s = 200 \text{ mm} \)
### SHEAR SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force</td>
<td>267 kN</td>
</tr>
<tr>
<td>Design shear stress</td>
<td>1.461 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.274 N/mm²</td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.4706</td>
</tr>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>Area of links</td>
<td>100 mm²</td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>213 mm</td>
</tr>
<tr>
<td>Actual spacing</td>
<td>200 mm</td>
</tr>
</tbody>
</table>

### Deflection

Actual span to depth ratio: \( l'/d = \frac{L}{1000} \div d = 14.14 \)

Allowable span/depth ratio: \( l'/da = k \times N \times F1 \times F2 \times F3 = 79.14 \)

Absolute value of span/depth: \( l'/da = 40 \times k = 40 \)

### DESIGN SUMMARY

- **Actual 1/d ratio**: 14.14
- **Basic 1/d ratio**: 98.54

### DEFLECTION

- **Span factor**: 1
- **Flange beam factor F1**: 0.8
- **Long spans factor F2**: 0.8537
- **Steel stress factor F3**: 1.176
- **Allowable 1/d ratio**: 40
Location: Ex2 - Hogging moment

Bending in flanged beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Moment before redistribution $M_{bef}=600\ \text{kNm}$
Shear force due to ultimate loads $V_{Ed}=267\ \text{kN}$
Axial load due to ultimate loads $N_{Ed}=0\ \text{kN}$
Length of beam/slab $L=8.2\ \text{m}$
Overall depth of section $h=650\ \text{mm}$
Actual flange width $b_a=3350\ \text{mm}$
Flange thickness $h_f=200\ \text{mm}$
Breadth of rib $b_w=350\ \text{mm}$
Section being analysed is considered as continuous.
Section considered has a hogging moment.
Char yield strength of reinforcment $f_{yk}=500\ \text{N/mm}^2$
Max.aggregate size (for bar spc.) $h_{agg}=20\ \text{mm}$
Diameter of tension bars $d_{ia}=25\ \text{mm}$
Diameter of link legs $d_{ia l}=8\ \text{mm}$
BM after redistribution $M=600\ \text{kNm}$
Effective depth of section $d=580\ \text{mm}$
Diameter of compression bars $d_{iac}=20\ \text{mm}$
Depth to compression steel $d_2=55\ \text{mm}$

DESIGN

<table>
<thead>
<tr>
<th>Overall depth</th>
<th>650 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective depth</td>
<td>580 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>350 mm</td>
</tr>
</tbody>
</table>

SUMMARY

<table>
<thead>
<tr>
<th>Overall depth</th>
<th>650 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective depth</td>
<td>580 mm</td>
</tr>
</tbody>
</table>

FLEXURE

Parameter K | 0.1699 |
Parameter K' | 0.1684 |
Lever arm ratio $z/d$ | 0.8185 |

TENSION REINFORCEMENT

| Steel area provided | 2945 mm$^2$ |
| Diameter of bars | 25 mm |
| Number of bars | 6 |
| Steel area required | 2905 mm$^2$ |
| Steel percentage req. | 1.431 % |
| Minimum area of steel | 305.8 mm$^2$ |
| Maximum area of steel | 9100 mm$^2$ |

COMPRESSION REINFORCEMENT

| Steel area provided | 628.3 mm$^2$ |
| Diameter of bars | 20 mm |
| Number of bars | 2 |
| Steel area required | 22.68 mm$^2$ |
| Minimum area required | 455 mm$^2$ |
Shear check

Design shear stress: $v_{Ed} = V_{Ed} \times 10^3 / (0.9 \times bw \times d) = 1.461 \text{ N/mm}^2$
Concrete strut capacity: $v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 3.274 \text{ N/mm}^2$
Angle of inclination of strut: $\theta' = 22^\circ$
Number of shear legs: $n_{sl} = 2$
Chosen spacing: $s = 200 \text{ mm}$

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Design shear force</th>
<th>267 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear stress</td>
<td>1.461 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.274 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.4706</td>
<td></td>
</tr>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
<td></td>
</tr>
<tr>
<td>Area of links</td>
<td>100 mm$^2$</td>
<td></td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>213 mm</td>
<td></td>
</tr>
<tr>
<td>Actual spacing</td>
<td>200 mm</td>
<td></td>
</tr>
</tbody>
</table>
Location: Typical flanged beam

Design of flanged beam-and-slab section to BS8110

Effective width of flange = b mm
Design to BS8110(1997), partial safety factor for steel gammaS=1.15

Flanged beam section

Calculations for reinforcement are in accordance with Clauses 3.4.4.4 and 3.4.4.5 of BS8110: Part 1 and are based on the simplified concrete stress block.

BM before redistribution Mbefor=165 kNm
Ultimate shear force V=110.1 kN
Axial load due to ultimate loads N=0 kN
Length of flanged beam len=6 m
Overall depth of section h=350 mm
Actual flange width ba=2000 mm
Flange thickness hf=100 mm
Breadth of rib bw=250 mm
Characteristic concrete strength fcu=30 N/mm²
Char.strength of long'l bars fy=500 N/mm²
High-yield steel reinforcement.
Bond type (0,1 or 2) Type=2
Characteristic strength of links fyv=500 N/mm²
Diameter of longitudinal bars dia=25 mm
Diameter of links dial=8 mm
Designated exposure class is XC2
Specified fixing tolerance tol=10 mm
From Table 3.4 of BS8110, minimum cover needed to all steel for continuous beam with a fire period of 2 hour/s is 30 mm
Thus minimum permissible cover to all steel is 35 mm
Nominal concrete cover cover=35 mm
From Figure 3.2 of BS8110, minimum possible rib width complying with a fire period of 2 hour/s is 125 mm
BM after redistribution M=165 kNm

**TENSION**

**REINFORCEMENT**

**SUMMARY**

Characteristic strength 500 N/mm²
Diameter of bars 25 mm
Number of bars 3
Area required 1384.8 mm²
Area provided 1472.6 mm²
Percentage of rib prov. 1.683 %
Weight of bars provided 11.56 kg/m

See BS 8110

: Part 1 :
1.4 * tension lap 1310 mm
2.0 * tension lap 1870 mm

SCALE 5.48 Office 1007 Proforma 107
Shear reinforcement

It is assumed that at least nominal shear steel will be provided.
Size of tension bars at support  dias=25 mm
No. of tension bars at support  nbarss=3

<table>
<thead>
<tr>
<th>SHEAR</th>
<th>Characteristic strength 500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 8 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of legs 2</td>
</tr>
<tr>
<td></td>
<td>Spacing 220 mm</td>
</tr>
<tr>
<td></td>
<td>Approx. weight of links 1.6787 kg/m</td>
</tr>
<tr>
<td></td>
<td>Ensure that no longitudinal bar is more than 150 mm from vertical leg.</td>
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</table>

<table>
<thead>
<tr>
<th>DEFLECTION</th>
<th>Permissible span 6.5248 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Actual span 6 m</td>
</tr>
</tbody>
</table>
Location: Small cantilevered L-beam: low mom/high shear/mild steel

Design of flanged beam-and-slab section to BS8110

Effective width of flange = b mm
Design to BS8110(1997), partial safety factor for steel \( \gamma_S = 1.15 \)

Flanged beam section

Calculations for reinforcement are in accordance with Clauses 3.4.4.4 and 3.4.4.5 of BS8110: Part 1 and are based on the simplified concrete stress block.

BM before redistribution \( M_{befor} = 30 \text{ kNm} \)
Ultimate shear force \( V = 204 \text{ kN} \)
Axial load due to ultimate loads \( N = 100 \text{ kN} \)
Length of flanged beam \( l_{en} = 3 \text{ m} \)
Overall depth of section \( h = 250 \text{ mm} \)
Actual flange width \( b_a = 1000 \text{ mm} \)
Flange thickness \( h_f = 100 \text{ mm} \)
Breadth of rib \( b_w = 300 \text{ mm} \)
Characteristic concrete strength \( f_{cu} = 25 \text{ N/mm}^2 \)
Char. strength of long’l bars \( f_y = 250 \text{ N/mm}^2 \)
Mild steel reinforcement.
Bnd type (0,1 or 2) \( \text{Type} = 0 \)
Characteristic strength of links \( f_{yv} = 250 \text{ N/mm}^2 \)
Diameter of longitudinal bars \( \text{dia} = 25 \text{ mm} \)
Diameter of links \( \text{dia} = 8 \text{ mm} \)
Designated exposure class is XC1
Specified fixing tolerance \( \text{tol} = 10 \text{ mm} \)
From Table 3.4 of BS8110, minimum cover needed to all steel for continuous beam with a fire period of 1 hour/s is 20 mm
Thus minimum permissible cover to all steel is 25 mm
Nominal concrete cover \( \text{cover} = 25 \text{ mm} \)
From Figure 3.2 of BS8110, minimum possible rib width complying with a fire period of 1 hour/s is 125 mm
BM after redistribution \( M = 30 \text{ kNm} \)

**TENSION**
- Characteristic strength \( 250 \text{ N/mm}^2 \)

**REINFORCEMENT**
- Diameter of bars \( 25 \text{ mm} \)
- Number of bars \( 2 \)
- Area required \( 723.28 \text{ mm}^2 \)
- Area provided \( 981.75 \text{ mm}^2 \)
- Percentage of rib prov. \( 1.309 \% \)
- Weight of bars provided \( 7.7067 \text{ kg/m} \)

**SUMMARY**
- See BS 8110
- **Anchorage & tension lap** 720 mm
  - Part 1: \( 1.4 \times \text{tension lap} = 1010 \text{ mm} \)
  - Cl.3.12.8.13 \( 2.0 \times \text{tension lap} = 1430 \text{ mm} \)
Shear reinforcement

It is assumed that at least nominal shear steel will be provided.
Size of tension bars at support   diam=25 mm
No. of tension bars at support    nbarss=2

<table>
<thead>
<tr>
<th>SHEAR</th>
<th>REINFORCEMENT</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>250 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>8 mm</td>
<td></td>
</tr>
<tr>
<td>Number of legs</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Spacing</td>
<td>70 mm</td>
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</tr>
<tr>
<td>Approx. weight of links</td>
<td>14.182 kg/m</td>
<td></td>
</tr>
<tr>
<td>Ensure that no longitudinal bar is more than 150 mm from vertical leg.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DEFLECTION</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permissible span</td>
<td>8.5841 m</td>
</tr>
<tr>
<td>Actual span</td>
<td>3 m</td>
</tr>
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</table>
Location: Large continuous T-beam: high mom/low shr/high-yield steel

Design of flanged beam-and-slab section to BS8110

Effective width of flange = b mm
Design to BS8110(1997), partial safety factor for steel \( \gamma_S = 1.15 \)

![Flanged beam section](image)

Calculations for reinforcement are in accordance withClauses 3.4.4.4 and 3.4.4.5 of BS8110: Part 1 and are based on the simplified concrete stress block.

BM before redistribution \( M_{befor} = 10000 \text{ kNm} \)
Ultimate shear force \( V = 1000 \text{ kN} \)
Axial load due to ultimate loads \( N = 0 \text{ kN} \)
Length of flanged beam \( l = 26.75 \text{ m} \)
Overall depth of section \( h = 2250 \text{ mm} \)
Actual flange width \( b_a = 10000 \text{ mm} \)
Flange thickness \( h_f = 250 \text{ mm} \)
Breadth of rib \( b_w = 500 \text{ mm} \)
Characteristic concrete strength \( f_{cu} = 50 \text{ N/mm}^2 \)
Char. strength of long'1 bars \( f_y = 500 \text{ N/mm}^2 \)
High-yield steel reinforcement.
Bond type (0,1 or 2) \( \text{Type} = 2 \)
Characteristic strength of links \( f_{yv} = 500 \text{ N/mm}^2 \)
Diameter of longitudinal bars \( d = 40 \text{ mm} \)
Diameter of links \( d = 10 \text{ mm} \)
Designated exposure class is XD1
Specified fixing tolerance \( t = 10 \text{ mm} \)
From Table 3.4 of BS8110, minimum cover needed to all steel for continuous beam with a fire period of 4 hour/s is 50 mm
Thus minimum permissible cover to all steel is 50 mm
Additional measures are necessary to reduce the risks of spalling (see Section 4 of BS8110: Part 2: 1985).
Nominal concrete cover \( c = 50 \text{ mm} \)
From Figure 3.2 of BS8110, minimum possible rib width complying with a fire period of 4 hour/s is 175 mm
BM after redistribution \( M = 10000 \text{ kNm} \)

**TENSION**

| Characteristic strength | 500 N/mm² |

**REINFORCEMENT**

| Diameter of bars | 40 mm |
| Number of bars | 9 |
| Area required | 1157 mm² |
| Area provided | 11310 mm² |
| Percentage of rib prov. | 1.0053 % |
| Weight of bars provided | 88.781 kg/m |

**SUMMARY**

- See BS 8110: Part 1: 1.4 * tension lap 1700 mm
- Cl.3.12.8.13: 2.0 * tension lap 2430 mm

See BS 8110: Anchorage & tension lap 1220 mm
Shear reinforcement

It is assumed that at least nominal shear steel will be provided.
Size of tension bars at support  dias=40 mm
No.of tension bars at support  nbarss=9

<table>
<thead>
<tr>
<th>SHEAR</th>
<th>REINFORCEMENT</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
<td>Diameter of bars</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Number of legs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Spacing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>300 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Approx.weight of links</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10.522 kg/m</td>
</tr>
<tr>
<td></td>
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<td>Ensure that no longitudinal bar is more than 150 mm from vertical leg.</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>DEFLECTION</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permissible span</td>
<td>24.159 m</td>
</tr>
<tr>
<td>Actual span</td>
<td>26.75 m</td>
</tr>
</tbody>
</table>
Location: Ex1 - Default example

Bending in flanged beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45\(*d\).

Moment before redistribution \(M_{bef}=600\ \text{kNm}\)
Shear force due to ultimate loads \(V_{Ed}=267\ \text{kN}\)
Axial load due to ultimate loads \(N_{Ed}=0\ \text{kN}\)
Length of beam/slab \(L=8.2\ \text{m}\)
Overall depth of section \(h=650\ \text{mm}\)
Actual flange width \(b_a=3350\ \text{mm}\)
Flange thickness \(h_f=200\ \text{mm}\)
Breadth of rib \(b_w=350\ \text{mm}\)

Section being analysed is considered as non-continuous.
Char yield strength of reinforcement \(f_{yk}=500\ \text{N/mm}^2\)
Max. aggregate size (for bar spc.) \(h_{agg}=20\ \text{mm}\)
Diameter of tension bars \(d_{ia}=25\ \text{mm}\)
Diameter of link legs \(d_{ial}=8\ \text{mm}\)
BM after redistribution \(M=600\ \text{kNm}\)
Effective depth of section \(d=580\ \text{mm}\)

Section being considered is a T-beam.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth</th>
<th>650 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth</td>
<td>580 mm</td>
</tr>
<tr>
<td></td>
<td>Flange width</td>
<td>2590 mm</td>
</tr>
<tr>
<td></td>
<td>Web width</td>
<td>350 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K</td>
<td>0.023</td>
</tr>
<tr>
<td></td>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio z/d</td>
<td>0.95</td>
</tr>
<tr>
<td>TENSION REINFORCEMENT</td>
<td>Steel area required</td>
<td>2505 mm²</td>
</tr>
<tr>
<td></td>
<td>Steel area provided</td>
<td>2945 mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Steel percentage req.</td>
<td>0.1667 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel</td>
<td>305.8 mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel</td>
<td>67340 mm²</td>
</tr>
</tbody>
</table>

Shear check

Design shear stress \(v_{Ed}=V_{Ed}*10^3/(0.9*b_w*d)=1.461\ \text{N/mm}^2\)
Concrete strut capacity \(v_{Rdm}=0.124*f_{ck}*(1-f_{ck}/250)=3.274\ \text{N/mm}^2\)
Angle of inclination of strut \(\theta'=22^\circ\)
Number of shear legs \(n_{sl}=2\)
Chosen spacing \(s=200\ \text{mm}\)
### SHEAR SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force</td>
<td>267 kN</td>
</tr>
<tr>
<td>Design shear stress</td>
<td>1.461 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.274 N/mm²</td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.4706</td>
</tr>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>Area of links</td>
<td>100 mm²</td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>213 mm</td>
</tr>
<tr>
<td>Actual spacing</td>
<td>200 mm</td>
</tr>
</tbody>
</table>

### Deflection

- Actual span to depth ratio: \( l'/d = \frac{L}{1000/d} = 14.14 \)
- Allowable span/depth ratio: \( l'/da = k*N*F1*F2*F3 = 79.14 \)
- Absolute value of span/depth: \( l'/da = 40*k = 40 \)

### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual 1/d ratio</td>
<td>14.14</td>
</tr>
<tr>
<td>Basic 1/d ratio</td>
<td>98.54</td>
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</tbody>
</table>

### DEFLECTION

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span factor</td>
<td>1</td>
</tr>
<tr>
<td>Flange beam factor F1</td>
<td>0.8</td>
</tr>
<tr>
<td>Long spans factor F2</td>
<td>0.8537</td>
</tr>
<tr>
<td>Steel stress factor F3</td>
<td>1.176</td>
</tr>
<tr>
<td>Allowable 1/d ratio</td>
<td>40</td>
</tr>
</tbody>
</table>
Location: Ex2 - Hogging moment

Bending in flanged beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.

Moment before redistribution \( M_{bef} = 600 \text{ kNm} \)
Shear force due to ultimate loads \( V_{Ed} = 267 \text{ kN} \)
Axial load due to ultimate loads \( N_{Ed} = 0 \text{ kN} \)
Length of beam/slab \( L = 8.2 \text{ m} \)
Overall depth of section \( h = 650 \text{ mm} \)
Actual flange width \( b_a = 3350 \text{ mm} \)
Flange thickness \( h_f = 200 \text{ mm} \)
Breadth of rib \( b_w = 350 \text{ mm} \)

Section being analysed is considered as continuous. Section considered has a hogging moment.
Char yield strength of reinforcement \( f_yk = 500 \text{ N/mm}^2 \)
Max. aggregate size (for bar spc.) \( h_{agg} = 20 \text{ mm} \)
Diameter of tension bars \( \text{dia} = 25 \text{ mm} \)
Diameter of link legs \( \text{dial} = 8 \text{ mm} \)
BM after redistribution \( M = 600 \text{ kNm} \)
Effective depth of section \( d = 580 \text{ mm} \)
Diameter of compression bars \( \text{diac} = 20 \text{ mm} \)
Depth to compression steel \( d_2 = 55 \text{ mm} \)

**DESIGN**

- Overall depth: 650 mm
- Effective depth: 580 mm
- Width of section: 350 mm

**SUMMARY**

- Parameter \( K \): 0.1699
- Parameter \( K' \): 0.1684
- Lever arm ratio \( z/d \): 0.8185

**FLEXURE**

Steel area provided: 2945 mm²
Diameter of bars: 25 mm
Number of bars: 6

**TENSION REINFORCEMENT**

Steel area provided: 2905 mm²
Steel percentage req.: 1.431 %
Minimum area of steel: 305.8 mm²
Maximum area of steel: 9100 mm²

**COMPRESSION REINFORCEMENT**

Steel area provided: 628.3 mm²
Diameter of bars: 20 mm
Number of bars: 2

Steel area required: 22.68 mm²
Minimum area required: 455 mm²
Shear check

Design shear stress
\[ v_{Ed} = \frac{V_{Ed} \times 10^3}{0.9 \times bw \times d} = 1.461 \, \text{N/mm}^2 \]

Concrete strut capacity
\[ v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 3.274 \, \text{N/mm}^2 \]

Angle of inclination of strut
\[ \theta' = 22^\circ \]

Number of shear legs
\[ n_{sl} = 2 \]

Chosen spacing
\[ s = 200 \, \text{mm} \]

**SHEAR SUMMARY**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force</td>
<td>267 kN</td>
</tr>
<tr>
<td>Design shear stress</td>
<td>1.461 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>3.274 N/mm²</td>
</tr>
<tr>
<td>Area/spacing ratio</td>
<td>0.4706</td>
</tr>
<tr>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td>Area of links</td>
<td>100 mm²</td>
</tr>
<tr>
<td>Maximum spacing</td>
<td>213 mm</td>
</tr>
<tr>
<td>Actual spacing</td>
<td>200 mm</td>
</tr>
</tbody>
</table>
Location: East stairwell edge beams

Rectangular section – torsion design

Calculations for torsion reinforcement in accordance with BS8110 : Part 2 (Issue 2) Clause 2.4.4.

Design to BS8110(1997), partial safety factor for steel $\gamma_S=1.15$

Torsional moment due to ult.loads $T=15$ kNm
Shear force due to ultimate loads $V=125$ kN
Overall depth of section $D=350$ mm
Breadth of concrete section $b_v=300$ mm
Concrete grade Grade=30
Diameter of closed link $d_{\text{link}}=10$ mm
Diameter of longitudinal reinforcement to be used when designing torsion steel $d_{\text{long}}=20$ mm
Nominal concrete cover $\text{cover}=50$ mm
Char.strength of longitud.bars $f_y=500$ N/mm²
Characteristic strength of links $f_{yv}=250$ N/mm²
Area tens.steel prov.for bending $A_{sb}=942$ mm²

<table>
<thead>
<tr>
<th>TORSION</th>
<th>Diam.of longitudinal bars</th>
<th>20 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Number of longitudinal bars</td>
<td>4</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Diameter of links</td>
<td>10 mm</td>
</tr>
<tr>
<td></td>
<td>Spacing of links</td>
<td>80 mm</td>
</tr>
</tbody>
</table>

SCALE 5.48        Office 1007                     Proforma 108
Location: Edge beams with low torsion & large bars

Rectangular section – torsion design

Calculations for torsion reinforcement in accordance with BS8110 : Part 2 (Issue 2) Clause 2.4.4.

Design to BS8110(1997), partial safety factor for steel gammaS=1.15

Torsional moment due to ult.loads T=5 kNm
Shear force due to ultimate loads V=125 kN
Overall depth of section D=350 mm
Breadth of concrete section bv=300 mm
Concrete grade Grade=30
Diameter of closed link dlink=10 mm
Diameter of longitudinal reinforcement to be used when designing torsion steel dlong=32 mm
Nominal concrete cover cover=50 mm
Characteristic strength of longitudinal bars fy=500 N/mm²
Characteristic strength of links fyv=250 N/mm²
Area tens. steel prov. for bending Asb=942 mm²

<table>
<thead>
<tr>
<th>TORSION</th>
<th>REINFORCEMENT</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diam. of longitudinal bars</td>
<td>Number of longitudinal bars</td>
<td>Diameter of links</td>
</tr>
<tr>
<td>32 mm</td>
<td>4</td>
<td>10 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Spacing of links</td>
</tr>
<tr>
<td></td>
<td></td>
<td>120 mm</td>
</tr>
</tbody>
</table>
Location: Edge beams with high torsion and severe exposure

Rectangular section – torsion design

Calculations for torsion reinforcement in accordance with BS8110: Part 2 (Issue 2) Clause 2.4.4.

Design to BS8110(1997), partial safety factor for steel γ_s=1.15

Torsional moment due to ult.loads T=20 kNm
Shear force due to ultimate loads V=125 kN
Overall depth of section D=350 mm
Breadth of concrete section bv=350 mm
Concrete grade Grade=30
Diameter of closed link dlink=10 mm
Diameter of longitudinal reinforcement to be used when designing torsion steel dlong=16 mm
Nominal concrete cover cover=50 mm
Char.strength of longitud.bars fy=500 N/mm²
Characteristic strength of links fyv=250 N/mm²
Area tens.steel prov.for bending Asb=942 mm²

TORSION
Diam.of longitudinal bars 16 mm

REINFORCEMENT
Number of longitudinal bars 4

SUMMARY
Diameter of links 10 mm
Spacing of links 70 mm
Location: Ex1 - East stairwell edge beams

Rectangular section — torsion design to EC2

Calculations for torsion reinforcement in accordance with EC2 Part 1-1: General rules and rules for buildings.

- $d =$ effective depth

Design torsion moment $\text{TEd} = 13 \text{kNm}$
Design shear force $\text{VED} = 125 \text{kN}$
Overall depth of section $h = 350 \text{mm}$
Breadth of concrete section $b = 300 \text{mm}$
Char.strength of longitud.bars $f_{yk} = 500 \text{N/mm}^2$
Partial safety factor for steel $\gamma_{ms} = 1.15$
Diameter of closed link $d_{al} = 8 \text{mm}$
Diameter of long. reinf'ment $d_{long} = 20 \text{mm}$
Nominal cover (to links) $c_{ov} = 30 \text{mm}$

Design for shear using the "Variable Strut Inclination Method"

- $\text{comp} = \cot(\theta) = 1/\tan(\theta) = 2.4751$
- Maximum shear resistance: $\text{VR}_{d_{max}} = 0.36 * b * d * (1 - f_{ck}/250) * f_{ck}/((\text{comp} + 1/\text{comp}) * 1000) = 254.89 \text{kN}$
- $\text{Asw}/s$ based on area of two legs $\text{AswsS} = \text{VED} * 1000 / (0.78 * d * f_{yk} * \text{comp})$
  $= 0.42879$

Convert the rectangular section to an equivalent hollow box section

- Effective thickness $\text{tef} = h * b / (2 * (h + b)) = 80.769 \text{mm}$
- Limiting thickness $\text{tef} = 2 * (c_{ov} + d_{al} + d_{long}/2) = 96 \text{mm}$
- Area within centre line $\text{Ak} = (b - \text{tef}) * (h - \text{tef}) = 51816 \text{mm}^2$
- Perimeter of centreline $\text{uk} = 2 * (b + h - 2 * \text{tef}) = 916 \text{mm}$

Crushing limit for combined shear and torsion

The following check is in accordance with EN 1992-2 Clause 6.3.2(104).
- Effective thickness $\text{tef} = 96 \text{mm}$
- Area within centre line $\text{Ak}' = (b - \text{tef}) * (h - \text{tef}) = 51816 \text{mm}^2$
- Max torsional moment resistance $\text{TR}_{d_{max}} = 2 * v_1 * f_{ck}/1.5 * \text{Ak}' * \text{tef} * \text{comp1}$
  $/1E6 = 31.099 \text{kNm}$
- Interaction condition $\text{ratio} = \text{TEd} / \text{TR}_{d_{max}} + \text{VED} / \text{VR}_{d_{max}} = 0.90842$
- Therefore concrete section is adequate.

Total amount of link reinforcement (shear and torsion)

The area $\text{Asw}$ below is based on the area of two legs of the stirrup.
- $\text{Asw}/s$ required $\text{AswsS} + \text{AswsT} = 0.66182$
- Round link spacing down to $s_{v} = 10 \times \text{INT}(s_{v}/10) = 110 \text{mm}$
- $\text{Asw}/s$ provided $\text{Aswp} = \text{TABLE 4.4 for dial}=8, \text{sv}=110$
  $= 0.9256$

SCALE 5.48 Office 1007 Proforma 108
Links should be of closed type fully anchored by means of laps or hooked ends, as per EC2 Clause 9.2.3(1) Figure 9.6.

**Area of additional longitudinal reinforcement required for torsion**

Diameter long.tension reinf'ment \( ltbars = 16 \text{ mm} \)

Area of longitudinal bars provided for torsion \( \text{Aspro}=nbars \times (\pi \times ltbars^2/4) = 804.25 \text{ mm}^2 \)

Area of longitudinal bars reqd. \( 653.8 \text{ mm}^2 \)

This torsional steel is additional to that required for bending and shear. Additional longitudinal reinforcement required at the level of the tension steel may be provided by using larger bars than those required for bending alone.

<table>
<thead>
<tr>
<th>TORSION</th>
<th>Diam. of longitudinal bars</th>
<th>16 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Number of longitudinal bars</td>
<td>4</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td></td>
<td>Spacing of shear and torsion</td>
<td>110 mm</td>
</tr>
</tbody>
</table>
Location: Ex2 - Edge beams with low torsion & large bars

Rectangular section - torsion design to EC2

Calculations for torsion reinforcement in accordance with EC2 Part 1-1: General rules and rules for buildings.

d = effective depth

Design torsion moment

Design shear force

Overall depth of section

Breadth of concrete section

Char. strength of longitud. bars

Partial safety factor for steel

Diameter of closed link

Diameter of long. reinf.'ment

Nominal cover (to links)

Design for shear using the "Variable Strut Inclination Method"

Maximum shear resistance:

Asw/s based on area of two legs

Convert the rectangular section to an equivalent hollow box section

Crushing limit for combined shear and torsion

Total amount of link reinforcement (shear and torsion)
Links should be of closed type fully anchored by means of laps or hooked ends, as per EC2 Clause 9.2.3(1) Figure 9.6.

**Area of additional longitudinal reinforcement required for torsion**

<table>
<thead>
<tr>
<th>TORSION</th>
<th>Diam. of longitudinal bars</th>
<th>12 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Number of longitudinal bars</td>
<td>4</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td></td>
<td>Spacing of shear and torsion</td>
<td>100 mm</td>
</tr>
</tbody>
</table>

Area of longitudinal bars provided for torsion

\[ \text{Area of longitudinal bars} = \pi \times 12^2 / 4 = 452.39 \, \text{mm}^2 \]

Area of longitudinal bars regd.

\[ \text{Area of longitudinal bars regd.} = 265.73 \, \text{mm}^2 \]

This torsional steel is additional to that required for bending and shear. Additional longitudinal reinforcement required at the level of the tension steel may be provided by using larger bars than those required for bending alone.
Location: Ex3 - Edge beams with high torsion and severe exposure

Rectangular section – torsion design to EC2

Calculations for torsion reinforcement in accordance with EC2 Part 1-1: General rules and rules for buildings.

\[ d = \text{effective depth} \]

Design torsion moment \( T_{Ed} = 20 \text{ kNm} \)
Design shear force \( V_{Ed} = 125 \text{ kN} \)
Overall depth of section \( h = 350 \text{ mm} \)
Breadth of concrete section \( b = 350 \text{ mm} \)
Char. strength of longitud. bars \( f_{yk} = 500 \text{ N/mm}^2 \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
 Diameter of closed link \( d_{all} = 8 \text{ mm} \)
 Diameter of long. reinf' ment \( d_{long} = 16 \text{ mm} \)
Nominal cover (to links) \( \text{cover} = 30 \text{ mm} \)

Design for shear using the "Variable Strut Inclination Method"

\[ \text{comp} = \cot(\theta) = 1/\tan(\theta) = 2.4751 \]
Maximum shear resistance:
\[ V_{Rdmax} = 0.36b'd'(1-f_{ck}/250)f_{ck}/((\text{comp}+1/\text{comp})*1000) = 299.34 \text{ kN} \]
Asw/s based on area of two legs
\[ A_{wsS} = \frac{V_{Ed} * 1000}{(0.78 * d * f_{yk} * \text{comp})} = 0.42597 \]

Convert the rectangular section to an equivalent hollow box section

Effective thickness \( tef = h*b/(2*(h+b)) = 87.5 \text{ mm} \)
Limiting thickness \( tef = 2*(\text{cover}+d_{all}+d_{long}/2) = 92 \text{ mm} \)
Area within centre line \( A_k = (b-tef) * (h-tef) = 66564 \text{ mm}^2 \)
Perimeter of centreline \( u_k = 2*(b+h-2*tef) = 1032 \text{ mm} \)

Crushing limit for combined shear and torsion

The following check is in accordance with EN 1992-2 Clause 6.3.2(104).
Effective thickness \( tef = 92 \text{ mm} \)
Area within centre line \( A_k' = (b-tef) * (h-tef) = 66564 \text{ mm}^2 \)
Max torsional moment resistance \( T_{Rdmax} = 2*v_1*f_{ck}/1.5*A_k'*t_ef*comp1 /1E6 = 38.286 \text{ kNm} \)
Interaction condition \( \text{ratio} = T_{Ed}/T_{Rdmax}+V_{Ed}/V_{Rdmax} = 0.93997 \)
Therefore concrete section is adequate.

Total amount of link reinforcement (shear and torsion)

The area Asw below is based on the area of two legs of the stirrup.
Asw/s required \( A_{wsS} + A_{wsT} = 0.70504 \)
Round link spacing down to \( sv = 10*\text{INT}(sv/10) = 120 \text{ mm} \)
Asw/s provided \( A_{ws} = \text{TABLE 4.4 for dial=8, } sv=120 = 0.8452 \)
Links should be of closed type fully anchored by means of laps or hooked ends, as per EC2 Clause 9.2.3(1) Figure 9.6.

**Area of additional longitudinal reinforcement required for torsion**

Diameter long.tension reinf'ment  
ltbars=16 mm

Area of longitudinal bars provided for torsion  
Aspro=nbars*(PI*ltbars^2/4)=1206.4 mm²

Area of longitudinal bars reqd.  
882.15 mm²

This torsional steel is additional to that required for bending and shear. Additional longitudinal reinforcement required at the level of the tension steel may be provided by using larger bars than those required for bending alone.

<table>
<thead>
<tr>
<th>TORSION</th>
<th>Diam.of longitudinal bars</th>
<th>16 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Number of longitudinal bars</td>
<td>6</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Diameter of links</td>
<td>8 mm</td>
</tr>
<tr>
<td></td>
<td>Spacing of shear and torsion</td>
<td>120 mm</td>
</tr>
</tbody>
</table>
Location: Ex4 - Reinforced Concrete Design to EC2 by B Mosley Ex 7.9

Rectangular section – torsion design to EC2

Calculations for torsion reinforcement in accordance with EC2 Part 1-1: General rules and rules for buildings.

\[ d = \text{effective depth} \]

Design torsion moment \( T_{Ed} = 24 \text{ kNm} \)
Design shear force \( V_{Ed} = 308 \text{ kN} \)
Overall depth of section \( h = 600 \text{ mm} \)
Breadth of concrete section \( b = 300 \text{ mm} \)
Char. strength of longitud. bars \( f_{yk} = 500 \text{ N/mm}^2 \)
Partial safety factor for steel \( \gamma_m = 1.15 \)
Diameter of closed link \( d_{al} = 8 \text{ mm} \)
Diameter of long. reinf’ment \( d_{long} = 32 \text{ mm} \)
Nominal cover (to links) \( c = 35 \text{ mm} \)

Design for shear using the "Variable Strut Inclination Method"

\[ \text{compute} \quad comp = \cot(\theta) = 1/\tan(\theta) = 2.4751 \]
Maximum shear resistance:
\[ V_{Rdmax} = 0.36 \times b \times d \times (1 - f_{ck}/250) \times f_{ck}/((\text{comp} + 1/\text{comp}) \times 1000) = 535.75 \text{ kN} \]
Asw/s based on area of two legs
\[ A_{sw} = \frac{V_{Ed} \times 1000}{(0.78 \times d \times f_{yk} \times \text{comp})} = 0.58979 \]

Convert the rectangular section to an equivalent hollow box section

Effective thickness \( t_{ef} = h \times b / (2 \times (h + b)) = 100 \text{ mm} \)
Limiting thickness \( t_{ef} = 2 \times (c + d_{al} + d_{long}/2) = 118 \text{ mm} \)
Area within centre line \( A_k = (b - t_{ef}) \times (h - t_{ef}) = 87724 \text{ mm}^2 \)
Perimeter of centreline \( u_k = 2 \times (b + h - 2 \times t_{ef}) = 1328 \text{ mm} \)

Crushing limit for combined shear and torsion

The following check is in accordance with EN 1992-2 Clause 6.3.2(104).
Effective thickness \( t_{ef} = 118 \text{ mm} \)
Area within centre line \( A_k' = (b - t_{ef}) \times (h - t_{ef}) = 87724 \text{ mm}^2 \)
Max torsional moment resistance \( T_{Rdmax} = 2 \times v_1 \times f_{ck} / 1.5 \times A_k' \times t_{ef} \times \text{comp1} / 1E6 = 75.934 \text{ kNm} \)
Interaction condition \( \text{ratio} = T_{Ed} / T_{Rdmax} + V_{Ed} / V_{Rdmax} = 0.89095 \)
Therefore concrete section is adequate.

Total amount of link reinforcement (shear and torsion)

The area Asw below is based on the area of two legs of the stirrup.
Asw/s required \( A_{sws} = A_{swsS} + A_{swsT} = 0.8439 \)
Round link spacing down to \( sv = 10 \times \text{INT}(sv/10) = 120 \text{ mm} \)
Asw/s provided \( A_{swp} = \text{TABLE 4.4 for dial}=8, sv=120 \times 0.8452 \)
Links should be of closed type fully anchored by means of laps or hooked ends, as per EC2 Clause 9.2.3(1) Figure 9.6.

**Area of additional longitudinal reinforcement required for torsion**

Diameter long. tension reinforc'ment  \( ltbars = 16 \text{ mm} \)

Area of longitudinal bars provided for torsion  \( \text{Aspro} = \text{nbars} \times (\pi \times ltbars^2 / 4) = 1260.4 \text{ mm}^2 \)

Area of longitudinal bars reqd.  \( 1033.6 \text{ mm}^2 \)

This torsional steel is additional to that required for bending and shear. Additional longitudinal reinforcement required at the level of the tension steel may be provided by using larger bars than those required for bending alone.

<table>
<thead>
<tr>
<th>TORSION</th>
<th>Diam. of longitudinal bars 16 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Number of longitudinal bars 6</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Diameter of links 8 mm</td>
</tr>
<tr>
<td></td>
<td>Spacing of shear and torsion 120 mm</td>
</tr>
<tr>
<td></td>
<td>links</td>
</tr>
</tbody>
</table>
Location: Ex1 - Factory at Erewhon

Design of ground-supported concrete slabs (pallet loading)


<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>CBR of subgrade</td>
<td>CBR=10 %</td>
</tr>
<tr>
<td>Type of sub-base</td>
<td>Granular material</td>
</tr>
<tr>
<td>Sub-base thickness</td>
<td>sbb=150 mm</td>
</tr>
<tr>
<td>Loading category (1-5)</td>
<td>Lcat=2</td>
</tr>
<tr>
<td>Specified concrete grade</td>
<td>fcu=40 N/mm²</td>
</tr>
<tr>
<td>28-day modulus of rupture</td>
<td>mrupt=4.77 MN/m²</td>
</tr>
<tr>
<td>90-day modulus of rupture</td>
<td>Amrupt=mrupt<em>1.1</em>1000</td>
</tr>
<tr>
<td>90-day modulus of rupture</td>
<td>Amrupt=1.2*Amrupt=6296.4 kN/m²</td>
</tr>
<tr>
<td>Static modulus of elasticity</td>
<td>E=31 kN/mm²</td>
</tr>
<tr>
<td>Leg load</td>
<td>Lload=4 tonnes</td>
</tr>
<tr>
<td>Depth of rack</td>
<td>d=900 mm</td>
</tr>
<tr>
<td>Leg spacing centre to centre</td>
<td>Lcc=2700 mm</td>
</tr>
<tr>
<td>Dist. from edge to first rack leg</td>
<td>Defr=150 mm</td>
</tr>
<tr>
<td>Dist. between double rack legs</td>
<td>Dbr=300 mm</td>
</tr>
<tr>
<td>Aisle width</td>
<td>A=1500 mm</td>
</tr>
<tr>
<td>Length of baseplate (in X-dir.)</td>
<td>L=100 mm</td>
</tr>
<tr>
<td>Breadth of baseplate (in Y-dir.)</td>
<td>b=100 mm</td>
</tr>
</tbody>
</table>

Capacity of forklift truck        fcap=2.5 tonnes
Number of wheels per axle         naxle=2
Thousands of load repetitions     nrpt=120
Radius of equivalent circ. area   Rw=100 mm
Wheel spacing centre-to-centre    Wcc=1500 mm
STRESS CALCULATIONS

Rack positioned at edge of slab - Arrangement 1

900 300 900
R3 R1 R4

150

All dimensions are in millimetres
R1 and R2 have double legs
R3 and R4 have single legs

Leg stresses from racks:
1. Edge stress beneath double-leg loaded point R1:
2. Internal stress beneath double-leg loaded point R2:
3. Edge stress beneath single-leg loaded point R3:
4. Edge stress beneath single-leg loaded point R4:

Total stress at R1
\[ T_{qe1} = (q_{e1} + q_{i21} + q_{e31} + q_{e41}) \times 0.85 \]
\[ = 4763.9 \text{ kN/m}^2 \]

where 0.85 is the load-transfer factor at the edge.

Rack positioned at edge of slab - Arrangement 2

2700 2700
R3 R1 R4

150

300

900

All dimensions are in millimetres
R1, R3 and R4 have double legs
R2 has single leg

Leg stresses from racks:
1. Edge stress beneath double-leg loaded point R1:
2. Internal stress beneath single-leg loaded point R2:
3. Edge stress beneath double-leg loaded point R3:
4. Edge stress beneath double-leg loaded point R4:

Total stress at R1
\[ T_{qe2} = (q_{e1} + q_{i21} + q_{e31} + q_{e41}) \times 0.85 \]
\[ = 5769.3 \text{ kN/m}^2 \]

where 0.85 is the load-transfer factor at the edge.
Rack positioned at corner of slab - Arrangement 1

![Diagram of Rack Arrangement 1]

All dimensions are in millimetres
R1 and R4 have double legs
R2 and R3 have single legs
Critical position is at a radius of 838.49 mm from corner

Leg stresses from racks:
1. Corner stress beneath double-leg loaded point R1:
2. Edge stress beneath single-leg loaded point R2:
3. Edge stress beneath single-leg loaded point R3:
4. Edge stress beneath double-leg loaded point R4:
where 0.7 is the factor for load transfer at the corner.

Rack positioned at corner of slab - Arrangement 2

![Diagram of Rack Arrangement 2]

All dimensions are in millimetres
R1, R2 and R4 have double legs
Critical position is at a radius of 838.49 mm from corner

Leg stresses from racks:
1. Corner stress beneath double-leg loaded point R1:
2. Edge stress beneath double-leg loaded point R2:
3. Edge stress beneath double-leg loaded point R3:
where 0.7 is the factor for load transfer at the corner.
Rack positioned internally

Leg stresses from racks:
1. Internal stress beneath double-leg loaded point R1:
2. Internal stress beneath single-leg loaded point R2:
3. Internal stress beneath single-leg loaded point R3:
4. Internal stress beneath double-leg loaded point R4:
5. Internal stress beneath double-leg loaded point R5:

Orientated about X-X axis:
Stress at R1 due to R2  \( q_{i21x} = q_{i2} \cdot P_{21x}/100 = -169.82 \text{ kN/m}^2 \)
Stress at R1 due to R3  \( q_{i31x} = q_{i3} \cdot P_{31x}/100 = -169.82 \text{ kN/m}^2 \)
Stress at R1 due to R4  \( q_{i41x} = q_{i4} \cdot P_{41x}/100 = 0 \text{ kN/m}^2 \)
Stress at R1 due to R5  \( q_{i51x} = q_{i5} \cdot P_{51x}/100 = 0 \text{ kN/m}^2 \)

Orientated about Y-Y axis:
Stress at R1 due to R2  \( q_{i21y} = q_{i2} \cdot P_{21y}/100 = 189.59 \text{ kN/m}^2 \)
Stress at R1 due to R3  \( q_{i31y} = q_{i3} \cdot P_{31y}/100 = 189.59 \text{ kN/m}^2 \)
Stress at R1 due to R4  \( q_{i41y} = q_{i4} \cdot P_{41y}/100 = -140.99 \text{ kN/m}^2 \)
Stress at R1 due to R5  \( q_{i51y} = q_{i5} \cdot P_{51y}/100 = -140.99 \text{ kN/m}^2 \)
WHEEL LOADING

Wheel stresses from forklift or other trucks:
1. Edge stress due to wheel load:
2. Corner stress due to wheel load:
3. Internal stress due to rack leg beneath double-leg point R1:

Wheel stress at the edge

\[
\begin{align*}
W2 & \quad 500 \quad W1 \\
\text{W1 and W2 are wheel loads} \\
\text{R1 is a double rack leg} \\
\text{All dimensions are in millimetres}
\end{align*}
\]

Total stress at W1
\[
T_{qwe} = (q_{ew} + q_{w1} + q_{1w1}) \times 0.85 = 3923.7 \text{ kN/m}^2
\]
where 0.85 is the load-transfer factor at the edge.

Wheel stress at the corner

\[
\begin{align*}
W1 & \quad / \quad W2 \\
627.75 & \quad / \quad / \\
\text{W1 and W2 are wheel loads} \\
\text{All dimensions are in millimetres}
\end{align*}
\]
where 0.7 is the load-transfer factor at the edge.
SUMMARY OF RESULTS

Concrete grade specified  
Allowable 90-day modulus of rupture 
Thus the calculated stresses are as follows:
Edge stress in slab due to rack 1  
Edge stress in slab due to rack 2  
Corner stress in slab due to rack 1  
Corner stress in slab due to rack 2  
Internal stress in slab due to rack 
Edge stress in slab due to wheel  
Corner stress in slab due to wheel  
Minimum suitable slab thickness 

h

h

Concrete slab 
Slip membrane 
Sub-base 
Subgrade
Location: Ex2 - High leg load

Design of ground-supported concrete slabs (pallet loading)


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<thead>
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<tr>
<td>Loading category (1-5)</td>
<td>Lcat=2</td>
</tr>
<tr>
<td>Specified concrete grade</td>
<td>fcu=40 N/mm²</td>
</tr>
<tr>
<td>28-day modulus of rupture</td>
<td>mrupt=4.77 MN/m²</td>
</tr>
<tr>
<td>90-day modulus of rupture</td>
<td>Amrupt=mrupt<em>1.1</em>1000 =5247 kN/m² (allowable)</td>
</tr>
<tr>
<td>90-day modulus of rupture</td>
<td>Amrupt=1.2*Amrupt=6296.4 kN/m²</td>
</tr>
<tr>
<td>Static modulus of elasticity</td>
<td>E=31 kN/mm²</td>
</tr>
<tr>
<td>Leg load</td>
<td>Lload=30 tonnes</td>
</tr>
<tr>
<td>Depth of rack</td>
<td>d=900 mm</td>
</tr>
<tr>
<td>Leg spacing centre to centre</td>
<td>Lcc=2700 mm</td>
</tr>
<tr>
<td>Dist. from edge to first rack leg</td>
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<td>A=1500 mm</td>
</tr>
<tr>
<td>Length of baseplate (in X-dir.)</td>
<td>L=100 mm</td>
</tr>
<tr>
<td>Breadth of baseplate (in Y-dir.)</td>
<td>b=100 mm</td>
</tr>
</tbody>
</table>

WARNING:
The load is too high for an unreinforced slab. Instead, design as a reinforced raft slab. This is beyond the scope of this proforma.
Location: Ex1 - Ground-supported steel fibre-reinforced slab

Design of ground-supported concrete slabs to TR34 (4th edition)

This proforma is for the design of ground-supported concrete floors for industrial buildings. The design method employed here is as per TR34 (4th edition) entitled 'Concrete Industrial Ground Floors' by The Concrete Society (March 2016 publication).

Assume a slab thickness           h=175 mm
Type of sub-base                  Granular material
Type of sub-base to be well graded granular material such as Type 1 or Type 2 complying with and laid in accordance with the Highway Agency Specification for Highway Works. All fill material should be checked for content of potentially expansive materials or reactions with lime and/or cement.
Sub-base thickness                sbb=150 mm
Modulus of subgrade reaction      k=0.05 N/mm²

Back-to-back pallet racking

Length of baseplate (in X-dir.)   Lr=100 mm
Breadth of baseplate (in Y-dir.)  br=100 mm
Radius of relative stiffness:

\[ l = \frac{Ecm \times h^3}{12 \times (1-u^2) \times k} \times 0.25 = 746.33 \text{ mm} \]

Dist.from edge to first rack leg  Defr=0 mm
Dist.between double rack legs     Dbr=250 mm
Reduced design load               Dldr=0.8xDld=115.2 kN

CASE 4: Check for punching shear
Punching shear may be critical for the edge loading condition and will be considered. The shear stress will be checked at the face of the loaded area and on a perimeter at a distance 2d from the loaded area.

\[ P1 = 72 \text{ kN} \]
\[ 2d = 262.5 \text{ mm} \]

PLAN SHOWING PUNCHING SHEAR PERIMETER

(1) At the face of the loaded area
Shear stress \( v_{Ed} = \frac{Dld*1000}{(u_o*d)} = 1.9948 \) 
Shear strength \( v_{Rd} = 0.5*k2*f_{cd} = 5.5808 \text{ N/mm}^2 \)
As \( v_{Rd} \geq v_{Ed} \) (5.5808 N/mm\(^2\) \geq 1.9948 N/mm\(^2\)), shear stress is OK.

(2) At the critical perimeter
Shear capacity for fibre-reinforced concrete:
The expression below is based on RILEM guidance which states that the presence of steel fibres will increase the design shear capacity over that of the plain concrete (an equivalent flexural strength ratio of \( Re3 = 0.5 \) will be assumed).
\[
0.035*k1^{1.5}*f_{ck}^{0.5} + 0.12*Re3*f_{ctk}/2 = 0.68848 \text{ N/mm}^2
\]
Slab load capacity \( P_p = \frac{v_{Rd}*u_1*d}{1000} = 124.22 \text{ kN} \)
Reduced design load \( Dldr = 0.8*Dld = 115.2 \text{ kN} \)
As \( Dldr \leq P_p \) (115.2 kN \leq 124.22 kN), this loading case is OK.

**General storage loading - random loading in kN/m**
Uniformly distributed load \( udl = 30 \text{ kN/m}^2 \) (unfactored)
Maximum UDL permitted \( w = 5.95*\lambda_2*M_n = 79.436 \text{ kN/m}^2 \)
As \( udl \leq w \) (30 kN/m\(^2\) \leq 79.436 kN/m\(^2\)), this loading case is OK.

**Internal wall loading - line load in kN/m**
Line load from wall (unfactored) \( ll = 30 \text{ kN/m} \)
For line loads remote from joints or slab edges:
Maximum line load permitted \( Plin = 4*\lambda_2*M_n = 55.793 \text{ kN/m} \)
As \( ll \leq Plin \) (30 kN/m \leq 55.793 kN/m), this loading case is OK.

For line loads adjacent to a free edge:
Maximum line load permitted \( Plin = 3*\lambda_2*M_n = 41.845 \text{ kN/m} \)
As \( ll \leq Plin \) (30 kN/m \leq 41.845 kN/m), this loading case is OK.

**Mezzanine floor**

---

Distance between columns \( x_1 = 5 \text{ m} \)
Distance between columns \( y_1 = 4 \text{ m} \)
Design column load (internal) \( W_{ci} = (x_1*y_1)*w_t = 183.75 \text{ kN} \)
Design column load (free edge) \( W_{ce} = (x_1*y_1)/2*w_t = 91.875 \text{ kN} \)
Length of baseplate (in X-dir.) \( L_m = 250 \text{ mm} \)
Breadth of baseplate (in Y-dir.) \( b_m = 250 \text{ mm} \)

CASE 1: Internal loading
Load capacity (rat=a1/l) \[ P_{ui} = P_{ui1} + \frac{(P_{ui2} - P_{ui1}) \times rat}{0.2} = 284.57 \text{ kN} \]
As \( W_{ci} \leq P_{ui} \ (183.75 \text{ kN/m} \leq 284.57 \text{ kN/m}) \), this loading case is OK.

CASE 2: Free edge loading
Load capacity (TR34 Eq. 23) \[ P_{ue1} = \frac{P_{i} \times (M_{n} + M_{p})}{2} + 2 \times M_{n} = 63.481 \text{ kN} \]
Load capacity (TR34 Eq. 24) \[ P_{ue2} = \frac{P_{i} \times (M_{n} + M_{p}) + 4 \times M_{n}}{1 - \frac{2 \times a_{1}}{3 \times l}} = 145.26 \text{ kN} \]
Load capacity (rat=a1/l) \[ P_{ue} = P_{ue1} + \frac{(P_{ue2} - P_{ue1}) \times rat}{0.2} = 140.76 \text{ kN} \]
As \( W_{ce} \leq P_{ue} \ (91.875 \text{ kN/m} \leq 140.76 \text{ kN/m}) \), this loading case is OK.

**Forklift truck loading**

Maximum wheel load (unfactored) \( W_{l} = 40 \text{ kN} \)
Design load per wheel \[ D_{lw} = \gamma_{D} \times W_{l} = 64 \text{ kN} \]

CASE 1: Internal loading
Load capacity (rat2=a2/l) \[ P_{u} = P_{u1} + \frac{(P_{u2} - P_{u1}) \times rat2}{0.2} = 181.28 \text{ kN} \]
As \( D_{lw} \leq P_{u} \ (64 \text{ kN} \leq 181.28 \text{ kN}) \), internal loading case is OK.

CASE 2: Loading at joints
Load capacity (a2/l=0) \[ P_{u1} = \frac{P_{i} \times (M_{p} + M_{n})}{2} + 2 \times M_{n} = 63.481 \text{ kN} \]
Load capacity (a2/l\leq0.2) \[ P_{u2} = \frac{P_{i} \times (M_{p} + M_{n}) + 4 \times M_{n}}{1 - \frac{2 \times a_{2}}{3 \times l}} = 132.38 \text{ kN} \]
Load capacity (rat2=a2/l) \[ P_{u} = P_{u1} + \frac{(P_{u2} - P_{u1}) \times rat2}{0.2} = 84.638 \text{ kN} \]
As \( D_{lw} \leq P_{u} \ (64 \text{ kN} \leq 84.638 \text{ kN}) \), this loading case is OK.

CASE 3: Corner loading
It is assumed that the corner loading is not relevant provided adequate load transfer is provided and the edge condition has been considered.

**Relative position of truck wheel and racking leg**

<table>
<thead>
<tr>
<th>racking leg</th>
<th>---o---</th>
<th>Design leg load =72 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>H</td>
</tr>
<tr>
<td>truck wheel</td>
<td>---o---</td>
<td>Design truck wheel load =64 kN</td>
</tr>
</tbody>
</table>

**EQUIVALENT LOADED AREAS FOR RACKING LEGS AND FORK LIFT TRUCK WHEELS**

Distance from wheel to baseplate \( H = 300 \text{ mm} \)

CASE 1: Internal loading
Load capacity (Rat=aeq/l) \[ P_{u} = P_{u1} + \frac{(P_{u2} - P_{u1}) \times Rat}{0.2} = 244.5 \text{ kN} \]
As \( D_{ld} \leq P_{u} \ (144 \text{ kN} \leq 244.5 \text{ kN}) \), internal loading case is OK.

CASE 2: Free edge loading (at a joint)
Load capacity (a/l=0) \[ P_{u1} = \frac{P_{i} \times (M_{p} + M_{n})}{2} + 2 \times M_{n} = 63.481 \text{ kN} \]
Load capacity (a/l\geq0.2) \[ P_{u2} = \frac{P_{i} \times (M_{p} + M_{n}) + 4 \times M_{n}}{1 - \frac{2 \times a}{3 \times l}} = 133.04 \text{ kN} \]
Load capacity (Rat=aeq/l) \[ P_{u} = P_{u1} + \frac{(P_{u2} - P_{u1}) \times Rat}{0.2} = 115.33 \text{ kN} \]
Reduced design load \( D_{ldr} = 0.8 \times D_{ld} = 115.2 \text{ kN} \)

As \( D_{ldr} \leq P_u \) (115.2 kN \( \leq \) 115.33 kN), this loading case is OK.

**Deflection check - Serviceability Limit State (SLS)**

The deflection expression below is by 'Westergaard' and can be used to obtain an approximate quantification of the slab deflection under a concentrated load \( P \).

\[
\text{Deflection (edge condition)} \quad \text{DEL} = \frac{c_1 \times P \times 1000}{k \times l^2} = 1.9044 \text{ mm}
\]

**SUMMARY OF RESULTS**

<table>
<thead>
<tr>
<th>Ground-supported slab</th>
<th>Steel-fibre-reinforced</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum suitable slab thickness</td>
<td>175 mm</td>
</tr>
<tr>
<td>Char cylinder compressive strength</td>
<td>32 N/mm²</td>
</tr>
<tr>
<td>Char cube compressive strength</td>
<td>40 N/mm²</td>
</tr>
<tr>
<td>Modulus of subgrade reaction</td>
<td>0.05 N/mm³</td>
</tr>
<tr>
<td>Type of sub-base</td>
<td>Granular material</td>
</tr>
<tr>
<td>Sub-base thickness</td>
<td>150 mm</td>
</tr>
<tr>
<td>Distance between double rack legs</td>
<td>250 mm</td>
</tr>
<tr>
<td>Racking leg baseplate</td>
<td>100 mm x 100 mm</td>
</tr>
<tr>
<td>Mezzanine column base plates</td>
<td>250 mm x 250 mm</td>
</tr>
<tr>
<td>Unfactored racking single leg load</td>
<td>60 kN</td>
</tr>
<tr>
<td>Unfactored UDL storage loading</td>
<td>30 kN/m²</td>
</tr>
<tr>
<td>Unfactored internal line loading</td>
<td>30 kN/m</td>
</tr>
<tr>
<td>Unfactored mezz. slab permanent load</td>
<td>1.25 kN/m²</td>
</tr>
<tr>
<td>Unfactored mezz. slab variable load</td>
<td>5 kN/m²</td>
</tr>
<tr>
<td>Unfactored wheel load</td>
<td>40 kN</td>
</tr>
</tbody>
</table>

The design calculations assume 20% load-transfer across the joint. Ensure this can be accommodated by checking with the designer of the load transfer system.
Design of ground-supported concrete slabs to TR34 (4th edition)

This proforma is for the design of ground-supported concrete floors for industrial buildings. The design method employed here is as per TR34 (4th edition) entitled 'Concrete Industrial Ground Floors' by The Concrete Society (March 2016 publication).

Assume a slab thickness $h=175$ mm
Type of sub-base Granular material
Type of sub-base to be well graded granular material such as Type 1 or Type 2 complying with and laid in accordance with the Highway Agency Specification for Highway Works. All fill material should be checked for content of potentially expansive materials or reactions with lime and/or cement.
Sub-base thickness $s_{bb}=150$ mm
Modulus of subgrade reaction $k=0.05$ N/mm³

Back-to-back pallet racking

Length of baseplate (in X-dir.) $L_r=150$ mm
Breadth of baseplate (in Y-dir.) $b_r=150$ mm
Radius of relative stiffness:

$$l=(E_{cm}*h^3/(12*(1-u^2)*k))^{0.25}=746.33 \text{ mm}$$

Dist. from edge to first rack leg $D_{efr}=0$ mm
Dist. between double rack legs $D_{br}=300$ mm
Reduced design load $D_{ldr}=0.8*D_{ld}=115.2$ kN

CASE 4: Check for punching shear
Punching shear may be critical for the edge loading condition and will be considered. The shear stress will be checked at the face of the loaded area and on a perimeter at a distance $2d$ from the loaded area.

Punching shear: $P_1=72$ kN
Perimeter: $2d=262.5$ mm

(1) At the face of the loaded area
Shear stress \[ v_{Ed} = \frac{Dld \times 1000}{(uo \times d)} = 1.4629 \]
Shear strength \[ v_{Rd} = 0.5 \times k_2 \times f_{cd} = 5.5808 \text{ N/mm}^2 \]
As \[ v_{Rd} \geq v_{Ed} \ (5.5808 \text{ N/mm}^2 \geq 1.4629 \text{ N/mm}^2) \], shear stress is OK.

(2) At the critical perimeter
Shear capacity for fibre-reinforced concrete:
The expression below ignores the steel fibres in the concrete and assumes plain concrete (EC2 Part 1-1, expression 6.3N).
\[ v_{Rd} = 0.035 \times k_1^{1.5} \times f_{ck}^{0.5} = 0.56 \text{ N/mm}^2 \]
Slab load capacity \[ P_p = v_{Rd} \times u_1 \times d / 1000 = 115.74 \text{ kN} \]
Reduced design load \[ Dldr = 0.8 \times Dld = 115.2 \text{ kN} \]
As \[ Dldr \leq P_p \ (115.2 \text{ kN} \leq 115.74 \text{ kN}) \], this loading case is OK.

**General storage loading - random loading in kN/m²**

Uniformly distributed load \[ udl = 30 \text{ kN/m}^2 \] (unfactored)
Maximum UDL permitted \[ w = 5.95 \times \lambda^2 \times M_n = 79.436 \text{ kN/m}^2 \]
As \[ udl \leq w \ (30 \text{ kN/m}^2 \leq 79.436 \text{ kN/m}^2) \], this loading case is OK.

**Internal wall loading - line load in kN/m**

Line load from wall (unfactored) \[ ll = 30 \text{ kN/m} \]
For line loads remote from joints or slab edges:
Maximum line load permitted \[ Plin = 4 \times \lambda \times M_n = 55.793 \text{ kN/m} \]
As \[ ll \leq Plin \ (30 \text{ kN/m} \leq 55.793 \text{ kN/m}) \], this loading case is OK.

For line loads adjacent to a free edge:
Maximum line load permitted \[ Plin = 3 \times \lambda \times M_n = 41.845 \text{ kN/m} \]
As \[ ll \leq Plin \ (30 \text{ kN/m} \leq 41.845 \text{ kN/m}) \], this loading case is OK.

**Mezzanine floor**

\[\begin{align*}
&| & | & | & \text{slab edge} \\
| & -0- & -0- & | \\
| & \cdot & \cdot & | \\
| & \cdot & \cdot & | \\
| & -0- & -0- & | \\
& \text{Partial plan} \\
& \text{on mezzanine} \\
& \text{columns} \\
\end{align*}\]

Distance between columns \[ x_1 = 5 \text{ m} \]
Distance between columns \[ y_1 = 4 \text{ m} \]
Design column load (internal) \[ W_{ci} = (x_1 \times y_1) \times w_t = 183.75 \text{ kN} \]
Design column load (free edge) \[ W_{ce} = (x_1 \times y_1) / 2 \times w_t = 91.875 \text{ kN} \]
Length of baseplate (in X-dir.) \[ L_m = 250 \text{ mm} \]
Breadth of baseplate (in Y-dir.) \[ b_m = 250 \text{ mm} \]

**CASE 1: Internal loading**
Load capacity (rat = a1/l) \[ P_{ui} = P_{ui1} + (P_{ui2} - P_{ui1}) \times \text{rat} / 0.2 = 284.57 \text{ kN} \]
As \[ W_{ci} \leq P_{ui} \ (183.75 \text{ kN/m} \leq 284.57 \text{ kN/m}) \], this loading case is OK.
CASE 2: Free edge loading

Load capacity (TR34 Eq. 23)
$$P_{ue1} = \left( \pi \left( \frac{M_n + M_p}{2} \right) + 2M_n \right) = 63.481 \text{ kN}$$

Load capacity (TR34 Eq. 24)
$$P_{ue2} = \left( \pi \left( \frac{M_n + M_p + 4M_n}{1 - 2a_1/(3l)} \right) \right) = 145.26 \text{ kN}$$

Load capacity (rat=a1/l)
$$P_{ue} = P_{ue1} + \left( P_{ue2} - P_{ue1} \right) \cdot \frac{rat}{0.2} = 140.76 \text{ kN}$$

As $W_{ce} \leq P_{ue}$ (91.875 kN/m \leq 140.76 kN/m), this loading case is OK.

Forklift truck loading

Maximum wheel load (unfactored) $w_l = 40 \text{ kN}$

Design load per wheel $D_{lw} = \gamma D \cdot w_l = 64 \text{ kN}$

CASE 1: Internal loading

Load capacity (rat2=a2/l)
$$P_{u} = P_{u1} + \left( P_{u2} - P_{u1} \right) \cdot \frac{rat2}{0.2} = 181.28 \text{ kN}$$

As $D_{lw} \leq P_{u}$ (64 kN \leq 181.28 kN), internal loading case is OK.

CASE 2: Loading at joints

Load capacity (a2/l=0)
$$P_{u1} = \left( \pi \left( \frac{M_p + M_n}{2} \right) + 2M_n \right) = 63.481 \text{ kN}$$

Load capacity (a2/l\geq0.2)
$$P_{u2} = \left( \pi \left( \frac{M_p + M_n + 4M_n}{1 - 2a_2/(3l)} \right) \right) = 132.38 \text{ kN}$$

Load capacity (rat2=a2/l)
$$P_{u} = P_{u1} + \left( P_{u2} - P_{u1} \right) \cdot \frac{rat2}{0.2} = 84.638 \text{ kN}$$

As $D_{lw} \leq P_{u}$ (64 kN \leq 84.638 kN), this loading case is OK.

CASE 3: Corner loading

It is assumed that the corner loading is not relevant provided adequate load transfer is provided and the edge condition has been considered.

Relative position of truck wheel and racking leg

$$\text{Distance from wheel to baseplate } H = 300 \text{ mm}$$

CASE 1: Internal loading

Load capacity (Rat=aeq/l)
$$P_{u} = P_{u1} + \left( P_{u2} - P_{u1} \right) \cdot \frac{Rat}{0.2} = 263.43 \text{ kN}$$

As $D_{ld} \leq P_{u}$ (144 kN \leq 263.43 kN), internal loading case is OK.

CASE 2: Free edge loading (at a joint)

Load capacity (a/l=0)
$$P_{u1} = \left( \pi \left( \frac{M_p + M_n}{2} \right) + 2M_n \right) = 63.481 \text{ kN}$$

Load capacity (a/l\geq0.2)
$$P_{u2} = \left( \pi \left( \frac{M_p + M_n + 4M_n}{1 - 2a/(3l)} \right) \right) = 134.82 \text{ kN}$$

Load capacity (Rat=aeq/l)
$$P_{u} = P_{u1} + \left( P_{u2} - P_{u1} \right) \cdot \frac{Rat}{0.2} = 125.27 \text{ kN}$$

Reduced design load $D_{ldr} = 0.8 \cdot D_{ld} = 115.2 \text{ kN}$

As $D_{ldr} \leq P_{u}$ (115.2 kN \leq 125.27 kN), this loading case is OK.
Deflection check - Serviceability Limit State (SLS)

The deflection expression below is by 'Westergaard' and can be used to obtain an approximate quantification of the slab deflection under a concentrated load P.

Deflection (edge condition) \( DEL = \frac{c_1 \cdot P \cdot 1000}{k \cdot l^2} = 1.9044 \text{ mm} \)

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<tr>
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<td>Racking leg baseplate</td>
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The design calculations assume 20% load-transfer across the joint. Ensure this can be accommodated by checking with the designer of the load transfer system.
Design of ground-supported concrete slabs to TR34 (4th edition)

This proforma is for the design of ground-supported concrete floors for industrial buildings. The design method employed here is as per TR34 (4th edition) entitled 'Concrete Industrial Ground Floors' by The Concrete Society (March 2016 publication).

Assume a slab thickness \( h = 150 \text{ mm} \)
Type of sub-base Granular material
Type of sub-base to be well graded granular material such as Type 1 or Type 2 complying with and laid in accordance with the Highway Agency Specification for Highway Works. All fill material should be checked for content of potentially expansive materials or reactions with lime and/or cement.

Sub-base thickness \( s_{bb} = 150 \text{ mm} \)
Modulus of subgrade reaction \( k = 0.05 \text{ N/mm}^3 \)

Pallet racking (single leg)

Length of baseplate (in X-dir.) \( L_r = 100 \text{ mm} \)
Breadth of baseplate (in Y-dir.) \( b_r = 100 \text{ mm} \)
Radius of relative stiffness:
\[
l = \left( \frac{E_{cm} \cdot h^3}{12 \cdot (1 - u^2) \cdot k} \right)^{0.25} = 664.85 \text{ mm}
\]
Dist. from edge to first rack leg \( D_{efr} = 0 \text{ mm} \)
Reduced design load \( D_{ldr} = 0.8 \cdot S_{ld} = 57.6 \text{ kN} \)

CASE 4: Check for punching shear
Punching shear may be critical for the edge loading condition and will be considered. The shear stress will be checked at the face of the loaded area and on a perimeter at a distance 2\( d \) from the loaded area.

\[
v_{Ed} = \frac{S_{ld} \cdot 1000}{(u_0 \cdot d)} = 2.1333
\]

(1) At the face of the loaded area

Plan showing punching shear perimeter
Shear strength  \[ v_{Rd} = 0.5 \times k_2 \times f_{cd} = 5.5808 \text{ N/mm}^2 \]
As \( v_{Rd} \geq v_{Ed} \) (5.5808 N/mm² ≥ 2.1333 N/mm²), shear stress is OK.

(2) At the critical perimeter

Shear capacity for fibre-reinforced concrete:

\[ v_{Rd} = (0.035 \times k_1 \times 1.5 \times f_{ck} \times 0.5 + 0.12 \times R_3 \times f_{ctk}/2) = 0.68848 \text{ N/mm}^2 \]

Slab load capacity  \[ P_p = v_{Rd} \times u_1 \times d/1000 = 77.985 \text{ kN} \]
As \( S_{ld} \leq P_p \) (72 kN ≤ 77.985 kN), this loading case is OK.

**General storage loading - random loading in kN/m²**

Uniformly distributed load  \[ u_{dl} = 30 \text{ kN/m}^2 \] (unfactored)
Maximum UDL permitted  \[ w = 5.95 \times \lambda_{m}^2 \times M_n = 73.544 \text{ kN/m}^2 \]
As \( u_{dl} \leq w \) (30 kN/m² ≤ 73.544 kN/m²), this loading case is OK.

**Internal wall loading - line load in kN/m**

Line load from wall (unfactored)  \[ l_l = 30 \text{ kN/m} \]

For line loads remote from joints or slab edges:
Maximum line load permitted  \[ P_{lin} = 4 \times \lambda_{m} \times M_n = 46.014 \text{ kN/m} \]
As \( l_l \leq P_{lin} \) (30 kN/m ≤ 46.014 kN/m), this loading case is OK.

For line loads adjacent to a free edge:
Maximum line load permitted  \[ P_{lin} = 3 \times \lambda_{m} \times M_n = 34.511 \text{ kN/m} \]
As \( l_l \leq P_{lin} \) (30 kN/m ≤ 34.511 kN/m), this loading case is OK.

**Mezzanine floor**

\[ | O | O | \text{slab edge} \]
\[ | O | O | y_1 \]
\[ | . | . | \]
\[ | . | . | \]
\[ Y | O | O | \text{Partial plan} \]
\[ x_1 \text{ on mezzanine} \]
\[ \text{columns} \]

Distance between columns  \[ x_1 = 5 \text{ m} \]
Distance between columns  \[ y_1 = 4 \text{ m} \]
Design column load (internal)  \[ W_{ci} = (x_1 \times y_1) \times w_t = 183.75 \text{ kN} \]
Design column load (free edge)  \[ W_{ce} = (x_1 \times y_1)/2 \times w_t = 91.875 \text{ kN} \]
Length of baseplate (in X-dir.)  \[ L_m = 250 \text{ mm} \]
Breadth of baseplate (in Y-dir.)  \[ B_m = 250 \text{ mm} \]

**CASE 1: Internal loading**

Load capacity (rat=a1/l)  \[ P_{ui} = P_{ui1} + (P_{ui2} - P_{ui1}) \times \text{rat}/0.2 = 224.23 \text{ kN} \]
As \( W_{ci} \leq P_{ui} \) (183.75 kN/m ≤ 224.23 kN/m), this loading case is OK.

**CASE 2: Free edge loading**

Load capacity (TR34 Eq. 23)  \[ P_{ue1} = \left( \frac{P_{ui} \times (M_n + M_p)}{2} \right) + 2 \times M_n = 46.639 \text{ kN} \]
Load capacity (TR34 Eq. 24) \[ P_{ue2} = \frac{\pi (M_n + M_p) + 4M_n}{1 - \frac{2a_1}{3l}} = 108.64 \text{ kN} \]

Load capacity \( \text{rat} = \frac{a_1}{l} \) \[ P_u = P_{ue1} + \frac{(P_{ue2} - P_{ue1}) \cdot \text{rat}}{0.2} = 112.41 \text{ kN} \]  
As \( W_{ce} \leq P_u \) (91.875 kN/m \leq 112.41 kN/m), this loading case is OK.

Forklift truck loading

Maximum wheel load (unfactored) \( w_l = 40 \text{ kN} \)  
Design load per wheel \( D_{lw} = \gamma D \cdot w_l = 64 \text{ kN} \)

CASE 1: Internal loading
Load capacity \( \text{rat} = \frac{a_2}{l} \) \[ P_u = P_{u1} + \frac{(P_{u2} - P_{u1}) \cdot \text{rat}}{0.2} = 137.32 \text{ kN} \]  
As \( D_{lw} \leq P_u \) (64 kN \leq 137.32 kN), internal loading case is OK.

CASE 2: Loading at joints
Load capacity \( \frac{a_2}{l} = 0 \) \[ P_{u1} = \frac{\pi (M_p + M_n) + 2M_n}{2} = 46.639 \text{ kN} \]  
Load capacity \( \frac{a_2}{l} = 0.2 \) \[ P_{u2} = \frac{\pi (M_p + M_n) + 4M_n}{1 - \frac{2a_2}{3l}} = 97.771 \text{ kN} \]  
Load capacity \( \text{rat} = \frac{a_2}{l} \) \[ P_u = P_{u1} + \frac{(P_{u2} - P_{u1}) \cdot \text{rat}}{0.2} = 64.264 \text{ kN} \]  
As \( D_{lw} \leq P_u \) (64 kN \leq 64.264 kN), this loading case is OK.

CASE 3: Corner loading
It is assumed that the corner loading is not relevant provided adequate load transfer is provided and the edge condition has been considered.

Deflection check - Serviceability Limit State (SLS)

The deflection expression below is by 'Westergaard' and can be used to obtain an approximate quantification of the slab deflection under a concentrated load \( P \).  
Deflection (edge condition) \[ \text{DEL} = c_1 \cdot P \cdot 1000 / (k \cdot l^2) = 1.2499 \text{ mm} \]

SUMMARY OF RESULTS

<table>
<thead>
<tr>
<th>Ground-supported slab</th>
<th>Steel-fibre-reinforced</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum suitable slab thickness</td>
<td>150 mm</td>
</tr>
<tr>
<td>Char cylinder compressive strength</td>
<td>32 N/mm²</td>
</tr>
<tr>
<td>Char cube compressive strength</td>
<td>40 N/mm²</td>
</tr>
<tr>
<td>Modulus of subgrade reaction</td>
<td>0.05 N/mm³</td>
</tr>
<tr>
<td>Type of sub-base</td>
<td>Granular material</td>
</tr>
<tr>
<td>Sub-base thickness</td>
<td>150 mm</td>
</tr>
<tr>
<td>Racking leg baseplate</td>
<td>100 mm x 100 mm</td>
</tr>
<tr>
<td>Mezzanine column base plates</td>
<td>250 mm x 250 mm</td>
</tr>
<tr>
<td>Unfactored racking single leg load</td>
<td>60 kN</td>
</tr>
<tr>
<td>Unfactored UDL storage loading</td>
<td>30 kN/m²</td>
</tr>
<tr>
<td>Unfactored internal line loading</td>
<td>30 kN/m</td>
</tr>
<tr>
<td>Unfactored mezz. slab permanent load</td>
<td>1.25 kN/m²</td>
</tr>
<tr>
<td>Unfactored mezz. slab variable load</td>
<td>5 kN/m²</td>
</tr>
<tr>
<td>Unfactored wheel load</td>
<td>40 kN</td>
</tr>
</tbody>
</table>

The design calculations assume 20% load-transfer across the joint. Ensure this can be accommodated by checking with the designer of the load transfer system.
Location: Ex4 - Ground-supported fabric-reinforced slab

Design of ground-supported concrete slabs to TR34 (4th edition)

This proforma is for the design of ground-supported concrete floors for industrial buildings. The design method employed here is as per TR34 (4th edition) entitled 'Concrete Industrial Ground Floors' by The Concrete Society (March 2016 publication).

Assume a slab thickness $h=200$ mm
Type of sub-base Granular material
Type of sub-base to be well graded granular material such as Type 1 or Type 2 complying with and laid in accordance with the Highway Agency Specification for Highway Works. All fill material should be checked for content of potentially expansive materials or reactions with lime and/or cement.
Sub-base thickness $s_{bb}=150$ mm
Modulus of subgrade reaction $k=0.05$ N/mm\(^3\)

Back-to-back pallet racking

Length of baseplate (in X-dir.) $L_r=100$ mm
Breadth of baseplate (in Y-dir.) $b_r=100$ mm
Radius of relative stiffness:

$$l=(E_{cm}h^3/(12(1-u^2)*k))^{0.25}=824.95 \text{ mm}$$

Dist. from edge to first rack leg $D_{efr}=0$ mm
Dist. between double rack legs $D_{br}=250$ mm
Characteristic strength of steel $f_{yk}=500$ N/mm\(^2\)
Effective depth of fabric-r'ment $d_1=150$ mm
Area of bottom fabric reinf'ment $A_{s}=142$ mm\(^2\)
Load-transfer across the joint $l_{tr}=24\%$
Reduced design load $D_{ldr}=D_{ld}-l_{tr}/100*D_{ld}=109.44$ kN

CASE 4: Check for punching shear
Punching shear may be critical for the edge loading condition and will be considered. The shear stress will be checked at the face of the loaded area and on a perimeter at a distance 2d from the loaded area.
punching shear perimeter

P1=72 kN
2d=300 mm
Lr=100 mm
L1=350 mm
Defr=0 mm
br=100 mm

PLAN SHOWING PUNCHING SHEAR PERIMETER

(1) At the face of the loaded area
Shear stress \( v_{Ed} = \frac{Dld \times 1000}{(u_0 \times d)} = 1.7455 \)
Shear strength \( v_{Rd} = 0.5 \times k_2 \times f_{cd} = 5.5808 \, N/mm^2 \)
As \( v_{Rd} \geq v_{Ed} \) (5.5808 N/mm\(^2\) \geq 1.7455 N/mm\(^2\)), shear stress is OK.

(2) At the critical perimeter
Shear capacity for fabric-reinforced concrete:
\[ v_{Rd} = (0.18/gamc) \times k_1 \times ((100 \times p_1 \times f_{ck})^{0.33} = 0.34598 \, N/mm^2 \]
Limiting capacity \( v_{Rd} = 0.035 \times k_1^{1.5} \times f_{ck}^{0.5} = 0.56 \, N/mm^2 \)
Slab load capacity \( P_p = v_{Rd} \times u_1 \times d / 1000 = 125.37 \, kN \)
Reduced design load \( Dldr = Dld - ltr / 100 \times Dld = 109.44 \, kN \)
As \( Dldr \leq P_p \) (109.44 kN \leq 125.37 kN), this loading case is OK.

General storage loading - random loading in kN/m\(^2\)
Uniformly distributed load \( u_{dl} = 30 \, kN/m^2 \) (unfactored)
Maximum UDL permitted \( w = 5.95 \times \lambda_1^2 \times M_n = 84.921 \, kN/m^2 \)
As \( u_{dl} \leq w \) (30 kN/m\(^2\) \leq 84.921 kN/m\(^2\)), this loading case is OK.

Internal wall loading - line load in kN/m
Line load from wall (unfactored) \( ll = 30 \, kN/m \)
For line loads remote from joints or slab edges:
Maximum line load permitted \( Plin = 4 \times \lambda_1 \times M_n = 65.928 \, kN/m \)
As \( ll \leq Plin \) (30 kN/m \leq 65.928 kN/m), this loading case is OK.

For line loads adjacent to a free edge:
Maximum line load permitted \( Plin = 3 \times \lambda_1 \times M_n = 49.446 \, kN/m \)
As \( ll \leq Plin \) (30 kN/m \leq 49.446 kN/m), this loading case is OK.

Mezzanine floor
Distance between columns \( x_1 = 5 \text{ m} \)
Distance between columns \( y_1 = 4 \text{ m} \)
Design column load (internal) \( W_{CI} = (x_1 \times y_1) \times w_t = 183.75 \text{ kN} \)
Design column load (free edge) \( W_{CE} = (x_1 \times y_1) / 2 \times w_t = 91.875 \text{ kN} \)
Length of baseplate (in X-dir.) \( L_m = 250 \text{ mm} \)
Breadth of baseplate (in Y-dir.) \( B_m = 250 \text{ mm} \)

**CASE 1: Internal loading**

Load capacity (rat = a1/l) \( P_{UI} = P_{UI1} + (P_{UI2} - P_{UI1}) \times \text{rat}/0.2 = 342.43 \text{ kN} \)
As \( W_{CI} \leq P_{UI} \ (183.75 \text{ kN/m} \leq 342.43 \text{ kN/m} ) \), this loading case is OK.

**CASE 2: Free edge loading**

Load capacity (TR34 Eq. 23) \( P_{UE1} = (P_I \times (M_n + M_p)/2) + 2 \times M_n = 81.784 \text{ kN} \)
Load capacity (TR34 Eq. 24) \( P_{UE2} = (P_I \times (M_n + M_p) + 4 \times M_n)/(1 - 2 \times a_1/(3 \times l)) = 184.61 \text{ kN} \)
Load capacity (rat = a1/l) \( P_{UE} = P_{UE1} + (P_{UE2} - P_{UE1}) \times \text{rat}/0.2 = 169.69 \text{ kN} \)
As \( W_{CE} \leq P_{UE} \ (91.875 \text{ kN/m} \leq 169.69 \text{ kN/m} ) \), this loading case is OK.

**Forklift truck loading**

Maximum wheel load (unfactored) \( w_l = 40 \text{ kN} \)
Design load per wheel \( D_{lw} = \gamma_D \times w_l = 64 \text{ kN} \)

**CASE 1: Internal loading**

Load capacity (rat2 = a2/l) \( P_u = P_{u1} + (P_{u2} - P_{u1}) \times \text{rat2}/0.2 = 225.28 \text{ kN} \)
As \( D_{lw} \leq P_u \ (64 \text{ kN} \leq 225.28 \text{ kN} ) \), internal loading case is OK.

**CASE 2: Loading at joints**

Load capacity \( (a_2 / l = 0) \) \( P_{u1} = (P_I \times (M_p + M_n)/2) + 2 \times M_n = 81.784 \text{ kN} \)
Load capacity \( (a_2 / l > 0.2) \) \( P_{u2} = (P_I \times (M_p + M_n) + 4 \times M_n)/(1 - 2 \times a_2/(3 \times l)) = 169.86 \text{ kN} \)
Load capacity (rat2 = a2/l) \( P_u = P_{u1} + (P_{u2} - P_{u1}) \times \text{rat2}/0.2 = 106.25 \text{ kN} \)
As \( D_{lw} \leq P_u \ (64 \text{ kN} \leq 106.25 \text{ kN} ) \), this loading case is OK.

**CASE 3: Corner loading**

It is assumed that the corner loading is not relevant provided adequate load transfer is provided and the edge condition has been considered.

**Relative position of truck wheel and racking leg**

- - - - - - - - - Design leg load = 72 kN
- - - - - - - - - Design truck wheel load = 64 kN

**Equivalent loaded areas for racking legs and forklift truck wheels**

Distance from wheel to baseplate \( H = 300 \text{ mm} \)
CASE 1: Internal loading
Load capacity (\(\text{Rat}=\text{aeq}/l\)) \(\text{Pu}=\text{Pu}_1+(\text{Pu}_2-\text{Pu}_1)*\text{Rat}/0.2=297.76\ \text{kN}\)
As \(\text{Dld} \leq \text{Pu}\ (144\ \text{kN} \leq 297.76\ \text{kN})\), internal loading case is OK.

CASE 2: Free edge loading (at a joint)
Load capacity \((a/l=0)\) \(\text{Pu}_1=(\pi*(\text{M}_{p}+\text{M}_{n})/2)+2*\text{M}_{n}=81.784\ \text{kN}\)
Load capacity \((a/l \geq 0.2)\) \(\text{Pu}_2=(\pi*(\text{M}_{p}+\text{M}_{n})+4*\text{M}_{n})/(1-2*a/(3*l))\)
\(=170.62\ \text{kN}\)
Reduced design load \(\text{Dldr}=\text{Dld}-\text{ltr}/100*\text{Dld}=109.44\ \text{kN}\)
As \(\text{Dldr} \leq \text{Pu}\ (109.44\ \text{kN} \leq 141.69\ \text{kN})\), this loading case is OK.

**Deflection check - Serviceability Limit State (SLS)**

The deflection expression below is by 'Westergaard' and can be used to obtain an approximate quantification of the slab deflection under a concentrated load \(P\).
Deflection (edge condition) \(\text{DEL}=c_1*P*1000/(k*l^2)=1.5588\ \text{mm}\)

**SUMMARY OF RESULTS**

<table>
<thead>
<tr>
<th>Ground-supported slab</th>
<th>Fabric-reinforced</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of fabric-reinforcement</td>
<td>142 mm²</td>
</tr>
<tr>
<td>Minimum suitable slab thickness</td>
<td>200 mm</td>
</tr>
<tr>
<td>Characteristic strength of steel</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Char cylinder compressive strength</td>
<td>32 N/mm²</td>
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<tr>
<td>Char cube compressive strength</td>
<td>40 N/mm²</td>
</tr>
<tr>
<td>Modulus of subgrade reaction</td>
<td>0.05 N/mm³</td>
</tr>
<tr>
<td>Type of sub-base</td>
<td>Granular material</td>
</tr>
<tr>
<td>Sub-base thickness</td>
<td>150 mm</td>
</tr>
<tr>
<td>Distance between double rack legs</td>
<td>250 mm</td>
</tr>
<tr>
<td>Racking leg baseplate</td>
<td>100 mm x 100 mm</td>
</tr>
<tr>
<td>Mezzanine column base plates</td>
<td>250 mm x 250 mm</td>
</tr>
<tr>
<td>Unfactored racking single leg load</td>
<td>60 kN</td>
</tr>
<tr>
<td>Unfactored UDL storage loading</td>
<td>30 kN/m²</td>
</tr>
<tr>
<td>Unfactored internal line loading</td>
<td>30 kN/m</td>
</tr>
<tr>
<td>Unfactored mezz. slab permanent load</td>
<td>1.25 kN/m²</td>
</tr>
<tr>
<td>Unfactored mezz. slab variable load</td>
<td>5 kN/m²</td>
</tr>
<tr>
<td>Unfactored wheel load</td>
<td>40 kN</td>
</tr>
</tbody>
</table>

Design calculations assume 24% load-transfer across the joint.
Location: Ex5 - Ground-supported fabric-reinforced slab

Design of ground-supported concrete slabs to TR34 (4th edition)

This proforma is for the design of ground-supported concrete floors for industrial buildings. The design method employed here is as per TR34 (4th edition) entitled 'Concrete Industrial Ground Floors' by The Concrete Society (March 2016 publication).

Assume a slab thickness $h=225$ mm
Type of sub-base Granular material
Type of sub-base to be well graded granular material such as Type 1 or Type 2 complying with and laid in accordance with the Highway Agency Specification for Highway Works. All fill material should be checked for content of potentially expansive materials or reactions with lime and/or cement.
Sub-base thickness $s_{bb}=200$ mm
Modulus of subgrade reaction $k=0.05$ N/mm²

Pallet racking (single leg)

Length of baseplate (in X-dir.) $L_r=100$ mm
Breadth of baseplate (in Y-dir.) $b_r=100$ mm
Radius of relative stiffness: $l=(E_c m h^3/(12*(1-u^2)*k))^{0.25}=913.55$ mm
Dist. from edge to first rack leg $D_{efr}=100$ mm
Characteristic strength of steel $f_{yk}=500$ N/mm²
Effective depth of fabric-reinforcement $d_1=175$ mm
Area of bottom fabric reinforcement $A_s=393$ mm²

CASE 4: Check for punching shear
Punching shear may be critical for the edge loading condition and will be considered. The shear stress will be checked at the face of the loaded area and on a perimeter at a distance $2d$ from the loaded area.

P1=72 kN
$2d=337.5$ mm
$L_r=100$ mm
$b_r=100$ mm
$D_{efr}=100$ mm

PLAN SHOWING PUNCHING SHEAR PERIMETER

(1) At the face of the loaded area
Shear stress
\[ v_{Ed} = \frac{Sld \times 1000}{(u_o \times d)} = 0.82286 \]
Shear strength
\[ v_{Rd} = 0.5 \times k_2 \times f_{cd} = 6.72 \text{ N/mm}^2 \]
As \( v_{Rd} \geq v_{Ed} \) (6.72 N/mm\(^2\) \geq 0.82286 N/mm\(^2\)), shear stress is OK.

(2) At the critical perimeter
Shear capacity for fabric-reinforced concrete:
\[ v_{Rd} = \frac{(0.18/\gamma_c) \times k_1 \times (100 \times p_1 \times f_{ck})^{0.33}}{0.33} = 0.49527 \text{ N/mm}^2 \]
Limiting capacity
\[ v_{Rd} = 0.035 \times k_1^{1.5} \times f_{ck}^{0.5} = 0.6261 \text{ N/mm}^2 \]
Slab load capacity
\[ P_p = \frac{v_{Rd} \times u_1 \times d}{1000} = 175.26 \text{ kN} \]
As \( Sld \leq P_p \) (72 kN \leq 175.26 kN), this loading case is OK.

General storage loading - random loading in kN/m\(^2\)

Uniformly distributed load
\[ u_{dl} = 30 \text{ kN/m}^2 \text{ (unfactored)} \]
Maximum UDL permitted
\[ w = 5.95 \times \lambda m^2 \times M_n = 98.865 \text{ kN/m}^2 \]
As \( u_{dl} \leq w \) (30 kN/m\(^2\) \leq 98.865 kN/m\(^2\)), this loading case is OK.

Internal wall loading - line load in kN/m

Line load from wall (unfactored)
\[ l_l = 30 \text{ kN/m} \]
For line loads remote from joints or slab edges:
Maximum line load permitted
\[ P_{lin} = 4 \times \lambda m \times M_p = 88.853 \text{ kN/m} \]
As \( l_l \leq P_{lin} \) (30 kN/m \leq 88.853 kN/m), this loading case is OK.
For line loads adjacent to a free edge:
Maximum line load permitted
\[ P_{lin} = 3 \times \lambda m \times M_p = 66.64 \text{ kN/m} \]
As \( l_l \leq P_{lin} \) (30 kN/m \leq 66.64 kN/m), this loading case is OK.

Mezzanine floor

Distance between columns
\[ x_1 = 5 \text{ m} \]
Distance between columns
\[ y_1 = 4 \text{ m} \]
Design column load (internal)
\[ W_{ci} = (x_1 \times y_1) \times w_t = 183.75 \text{ kN} \]
Design column load (free edge)
\[ W_{ce} = (x_1 \times y_1) / 2 \times w_t = 91.875 \text{ kN} \]
Length of baseplate (in X-dir.)
\[ L_m = 250 \text{ mm} \]
Breadth of baseplate (in Y-dir.)
\[ b_m = 250 \text{ mm} \]

CASE 1: Internal loading
Load capacity (rat=a1/l)
\[ P_{ui} = P_{ui1} + (P_{ui2} - P_{ui1}) \times \text{rat} / 0.2 = 648.08 \text{ kN} \]
As \( W_{ci} \leq P_{ui} \) (183.75 kN/m \leq 648.08 kN/m), this loading case is OK.

CASE 2: Free edge loading
Load capacity (TR34 Eq. 23) \[ P_{ue1} = \frac{\pi}{4} \left( M_n + M_p \right) + 2M_n = 141.65 \text{ kN} \]
Load capacity (TR34 Eq. 24) \[ P_{ue2} = \frac{\pi}{4} \left( M_n + M_p + 4M_n \right) / \left( 1 - 2a_1 / (3l) \right) = 315.82 \text{ kN} \]
Load capacity (rat=a1/l) \[ P_{ue} = P_{ue1} + \frac{P_{ue2} - P_{ue1}}{0.2} \times \text{rat} = 276.1 \text{ kN} \]
As \( W_{ce} \lesssim P_{ue} \) (91.875 kN/m \lesssim 276.1 kN/m), this loading case is OK.

**Forklift truck loading**

Maximum wheel load (unfactored) \( W_l \) = 40 kN
Design load per wheel \( D_{lw} = \gamma_D \times W_l = 64 \text{ kN} \)

CASE 1: Internal loading
Load capacity (rat2=a2/l) \[ P_u = P_{u1} + \frac{P_{u2} - P_{u1}}{0.2} \times \text{rat} = 439.81 \text{ kN} \]
As \( D_{lw} \lesssim P_u \) (64 kN \lesssim 439.81 kN), internal loading case is OK.

CASE 2: Loading at joints
Load capacity (a2/l=0) \[ P_{u1} = \frac{\pi}{4} \left( M_p + M_n \right) + 2M_n = 141.65 \text{ kN} \]
Load capacity (a2/l\geq0.2) \[ P_{u2} = \frac{\pi}{4} \left( M_p + M_n + 4M_n \right) / \left( 1 - 2a_2 / (3l) \right) = 293.11 \text{ kN} \]
Load capacity (rat2=a2/l) \[ P_u = P_{u1} + \frac{P_{u2} - P_{u1}}{0.2} \times \text{rat} = 179.65 \text{ kN} \]
As \( D_{lw} \lesssim P_u \) (64 kN \lesssim 179.65 kN), this loading case is OK.

CASE 3: Corner loading
Load capacity (rat2=a2/l) \[ P_u = P_{u1} + \frac{P_{u2} - P_{u1}}{0.2} \times \text{rat} = 69.422 \text{ kN} \]
As \( D_{lw} \lesssim P_u \) (64 kN \lesssim 69.422 kN), corner loading case is OK.

**Relative position of truck wheel and racking leg**

\[
\begin{array}{c}
\text{racking leg} \quad -o-\quad \text{Design leg load} = 72 \text{ kN} \\
\quad \quad \quad \quad \quad \quad \quad \quad \quad \quad \text{H} \\
\text{truck wheel} \quad -o- \quad \text{Design truck wheel load} = 64 \text{ kN} \\
\end{array}
\]

EQUIVALENT LOADED AREAS FOR RACKING LEGS AND FORK LIFT TRUCK WHEELS

Distance from wheel to baseplate \( H = 300 \text{ mm} \)

CASE 1: Internal loading
Load capacity (Rat=aeq/l) \[ P_u = P_{u1} + \frac{P_{u2} - P_{u1}}{0.2} \times \text{Rat} = 569.97 \text{ kN} \]
As \( D_{ld} \lesssim P_u \) (144 kN \lesssim 569.97 kN), internal loading case is OK.

CASE 2: Free edge loading (at a joint)
Load capacity (a/l=0) \[ P_{u1} = \frac{\pi}{4} \left( M_p + M_n \right) / 2 + 2M_n = 141.65 \text{ kN} \]
Load capacity (a/l\geq0.2) \[ P_{u2} = \frac{\pi}{4} \left( M_p + M_n + 4M_n \right) / \left( 1 - 2a / (3l) \right) = 294.29 \text{ kN} \]
Load capacity (Rat=aeq/l) \[ P_u = P_{u1} + \frac{P_{u2} - P_{u1}}{0.2} \times \text{Rat} = 234.6 \text{ kN} \]
As \( D_{ld} \lesssim P_u \) (144 kN \lesssim 234.6 kN), this loading case is OK.
Deflection check - Serviceability Limit State (SLS)

The deflection expression below is by 'Westergaard' and can be used to obtain an approximate quantification of the slab deflection under a concentrated load P.

\[
\text{Deflection (edge condition)} \quad \text{DEL} = c_1 \frac{P \times 1000}{k \times l^2} = 0.66202 \text{ mm}
\]

**SUMMARY OF RESULTS**

<table>
<thead>
<tr>
<th>Ground-supported slab</th>
<th>Fabric-reinforced</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of fabric-reinforcement</td>
<td>393 mm²</td>
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<tr>
<td>Minimum suitable slab thickness</td>
<td>225 mm</td>
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<tr>
<td>Characteristic strength of steel</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Char cylinder compressive strength</td>
<td>40 N/mm²</td>
</tr>
<tr>
<td>Char cube compressive strength</td>
<td>50 N/mm²</td>
</tr>
<tr>
<td>Modulus of subgrade reaction</td>
<td>0.05 N/mm³</td>
</tr>
<tr>
<td>Type of sub-base</td>
<td>Granular material</td>
</tr>
<tr>
<td>Sub-base thickness</td>
<td>200 mm</td>
</tr>
<tr>
<td>Racking leg baseplate</td>
<td>100 mm x 100 mm</td>
</tr>
<tr>
<td>Mezzanine column base plates</td>
<td>250 mm x 250 mm</td>
</tr>
<tr>
<td>Unfactored racking single leg load</td>
<td>60 kN</td>
</tr>
<tr>
<td>Unfactored UDL storage loading</td>
<td>30 kN/m²</td>
</tr>
<tr>
<td>Unfactored internal line loading</td>
<td>30 kN/m</td>
</tr>
<tr>
<td>Unfactored mezzanine slab permanent load</td>
<td>1.25 kN/m²</td>
</tr>
<tr>
<td>Unfactored mezzanine slab variable load</td>
<td>5 kN/m²</td>
</tr>
<tr>
<td>Unfactored wheel load</td>
<td>40 kN</td>
</tr>
</tbody>
</table>
Location: Ex6 - Fabric-reinforced slab (TR34 Appendix E, 3rd Ed)

Design of ground-supported concrete slabs to TR34 (4th edition)

This proforma is for the design of ground-supported concrete floors for industrial buildings. The design method employed here is as per TR34 (4th edition) entitled 'Concrete Industrial Ground Floors' by The Concrete Society (March 2016 publication).

Assume a slab thickness h=175 mm
Type of sub-base Granular material
Type of sub-base to be well graded granular material such as Type 1 or Type 2 complying with and laid in accordance with the Highway Agency Specification for Highway Works. All fill material should be checked for content of potentially expansive materials or reactions with lime and/or cement.
Sub-base thickness sbb=150 mm
Modulus of subgrade reaction k=0.05 N/mm³

Back-to-back pallet racking

Length of baseplate (in X-dir.) Lr=100 mm
Breadth of baseplate (in Y-dir.) br=100 mm
Radius of relative stiffness:

\[ l = \left( \frac{Ecm \cdot h^3}{12 \cdot (1-u^2) \cdot k} \right)^{0.25} = 746.33 \text{ mm} \]

Dist. from edge to first rack leg Defr=80 mm
Dist. between double rack legs Db=250 mm
Characteristic strength of steel f_yk=500 N/mm²
Effective depth of fabric-reinforcement d=135 mm
Area of bottom fabric reinforcement A_s=142 mm²
Load-transfer across the joint ltr=24 %
Reduced design load Dldr=Dld-ltr/100*Dld=109.44 kN

CASE 4: Check for punching shear
Punching shear may be critical for the edge loading condition and will be considered. The shear stress will be checked at the face of the loaded area and on a perimeter at a distance 2d from the loaded area.
punching shear perimeter

slab edge

PLAN SHOWING PUNCHING SHEAR PERIMETER

(1) At the face of the loaded area

Shear stress \( v_{Ed} = D_{ld} \times 1000 / (u_o \times d) = 1.5023 \)

Shear strength \( v_{Rd} = 0.5 \times k_2 \times f_{cd} = 5.5808 \text{ N/mm}^2 \)

As \( v_{Rd} \geq v_{Ed} \) (5.5808 N/mm^2 \geq 1.5023 N/mm^2), shear stress is OK.

(2) At the critical perimeter

Shear capacity for fabric-reinforced concrete:

\( v_{Rd} = (0.18 / \gamma_c) \times k_1 \times (100 \times p_1 \times f_{ck})^{0.33} = 0.35822 \text{ N/mm}^2 \)

Limiting capacity \( v_{Rd} = 0.035 \times k_1 \times 1.5 \times f_{ck} \times 0.5 = 0.56 \text{ N/mm}^2 \)

Slab load capacity \( P_p = v_{Rd} \times u_1 \times d / 1000 = 117.8 \text{ kN} \)

Reduced design load \( D_{ldr} = D_{ld} - l_{tr} / 100 \times D_{ld} = 109.44 \text{ kN} \)

As \( D_{ldr} \leq P_p \) (109.44 kN \leq 117.8 kN), this loading case is OK.

General storage loading - random loading in kN/m²

Uniformly distributed load \( u_{dl} = 30 \text{ kN/m}² \) (unfactored)

Maximum UDL permitted \( w = 5.95 \times \lambda_1 \times 2 \times M_n = 79.436 \text{ kN/m}² \)

As \( u_{dl} \leq w \) (30 kN/m² \leq 79.436 kN/m²), this loading case is OK.

Internal wall loading - line load in kN/m

Line load from wall (unfactored) \( l_l = 30 \text{ kN/m} \)

For line loads remote from joints or slab edges:

Maximum line load permitted \( P_{lin} = 4 \times \lambda_1 \times M_n = 55.793 \text{ kN/m} \)

As \( l_l \leq P_{lin} \) (30 kN/m \leq 55.793 kN/m), this loading case is OK.

For line loads adjacent to a free edge:

Maximum line load permitted \( P_{lin} = 3 \times \lambda_1 \times M_n = 41.845 \text{ kN/m} \)

As \( l_l \leq P_{lin} \) (30 kN/m \leq 41.845 kN/m), this loading case is OK.

Mezzanine floor

<table>
<thead>
<tr>
<th>-o-0</th>
<th>-o-0</th>
</tr>
</thead>
<tbody>
<tr>
<td>.</td>
<td>.</td>
</tr>
<tr>
<td>-o-0</td>
<td>-o-0</td>
</tr>
</tbody>
</table>

Partial plan on mezzanine columns

- SCALE 5.48
- Office 1007
- Proforma 109
Distance between columns \( x_1 = 5 \) m
Distance between columns \( y_1 = 4 \) m
Design column load (internal) \( W_{ci} = (x_1 \times y_1) \times wt = 183.75 \) kN
Design column load (free edge) \( W_{ce} = (x_1 \times y_1) / 2 \times wt = 91.875 \) kN
Length of baseplate (in X-dir.) \( L_m = 250 \) mm
Breadth of baseplate (in Y-dir.) \( b_m = 250 \) mm

**CASE 1: Internal loading**
Load capacity (rat=\( a_1 \)/\( l \)) \( P_{ui} = P_{ui1} + (P_{ui2} - P_{ui1}) \times \text{rat}/0.2 = 292.8 \) kN
As \( W_{ci} \leq P_{ui} \) (183.75 kN/m \( \leq 292.8 \) kN/m), this loading case is OK.

**CASE 2: Free edge loading**
Load capacity (TR34 Eq. 23) \( P_{ue1} = (P_1 \times (M_n + M_p) / 2) + 2 \times M_n = 64.473 \) kN
Load capacity (TR34 Eq. 24) \( P_{ue2} = (P_1 \times (M_n + M_p) + 4 \times M_n) / (1 - 2 \times a_1 / (3 \times l)) = 147.53 \) kN
Load capacity (rat=\( a_1 \)/\( l \)) \( P_{ue} = P_{ue1} + (P_{ue2} - P_{ue1}) \times \text{rat}/0.2 = 142.96 \) kN
As \( W_{ce} \leq P_{ue} \) (91.875 kN/m \( \leq 142.96 \) kN/m), this loading case is OK.

**Forklift truck loading**
Maximum wheel load (unfactored) \( w_l = 40 \) kN
Design load per wheel \( D_{lw} = \gamma_D \times w_l = 64 \) kN

**CASE 1: Internal loading**
Load capacity (rat2=\( a_2 \)/\( l \)) \( P_{u} = P_{u1} + (P_{u2} - P_{u1}) \times \text{rat}/0.2 = 186.52 \) kN
As \( D_{lw} \leq P_{u} \) (64 kN \( \leq 186.52 \) kN), internal loading case is OK.

**CASE 2: Loading at joints**
Load capacity (\( a_2 \)/\( l = 0 \)) \( P_{u1} = (P_1 \times (M_p + M_n) / 2) + 2 \times M_n = 64.473 \) kN
Load capacity (\( a_2 \)/\( l \geq 0.2 \)) \( P_{u2} = (P_1 \times (M_p + M_n) + 4 \times M_n) / (1 - 2 \times a_2 / (3 \times l)) = 134.45 \) kN
Load capacity (rat2=\( a_2 \)/\( l \)) \( P_{u} = P_{u1} + (P_{u2} - P_{u1}) \times \text{rat}/0.2 = 85.961 \) kN
As \( D_{lw} \leq P_{u} \) (64 kN \( \leq 85.961 \) kN), this loading case is OK.

**CASE 3: Corner loading**
It is assumed that the corner loading is not relevant provided adequate load transfer is provided and the edge condition has been considered.

**Relative position of truck wheel and racking leg**

```
        |---o---|
rack leg --- o ---
        |   H   |
        |---o---|
truck wheel --- o ---
```

Design leg load =72 kN
Design truck wheel load =64 kN

**Equivalent Loaded Areas for Racking Legs and Fork Lift Truck Wheels**

Distance from wheel to baseplate \( H = 300 \) mm
CASE 1: Internal loading
Load capacity (\(\text{Rat}=\text{aeq}/l\)) \(P_u=Pu_1+(Pu_2-Pu_1)\times\text{Rat}/0.2=251.56 \text{ kN}\)
As \(D_{ld} \leq P_u \ (144 \text{ kN} \leq 251.56 \text{ kN})\), internal loading case is OK.

CASE 2: Free edge loading (at a joint)
Load capacity (\(a/1=0\)) \(P_u_1=(\pi*(M_p+M_n)/2)+2*M_n=64.473 \text{ kN}\)
Load capacity (\(a/l\geq0.2\)) \(P_u_2=(\pi*(M_p+M_n)+4*M_n)/(1-2*a/(3*l))=135.12 \text{ kN}\)
Reduced design load \(D_{ldr}=D_{ld}-ltr/100*D_{ld}=109.44 \text{ kN}\)
As \(D_{ldr} \leq P_u \ (109.44 \text{ kN} \leq 117.13 \text{ kN})\), this loading case is OK.

**Deflection check - Serviceability Limit State (SLS)**

The deflection expression below is by 'Westergaard' and can be used to obtain an approximate quantification of the slab deflection under a concentrated load \(P\).

\[
\text{Deflection (edge condition)} \quad \text{DEL}=c_1*P*1000/(k*l^2)=1.9044 \text{ mm}
\]

**SUMMARY OF RESULTS**

<table>
<thead>
<tr>
<th>Ground-supported slab</th>
<th>Fabric-reinforced</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of fabric-reinforcement</td>
<td>142 mm²</td>
</tr>
<tr>
<td>Minimum suitable slab thickness</td>
<td>175 mm</td>
</tr>
<tr>
<td>Characteristic strength of steel</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Char cylinder compressive strength</td>
<td>32 N/mm²</td>
</tr>
<tr>
<td>Char cube compressive strength</td>
<td>40 N/mm²</td>
</tr>
<tr>
<td>Modulus of subgrade reaction</td>
<td>0.05 N/mm³</td>
</tr>
<tr>
<td>Type of sub-base</td>
<td>Granular material</td>
</tr>
<tr>
<td>Sub-base thickness</td>
<td>150 mm</td>
</tr>
<tr>
<td>Distance between double rack legs</td>
<td>250 mm</td>
</tr>
<tr>
<td>Racking leg baseplate</td>
<td>100 mm x 100 mm</td>
</tr>
<tr>
<td>Mezzanine column base plates</td>
<td>250 mm x 250 mm</td>
</tr>
<tr>
<td>Unfactored racking single leg load</td>
<td>60 kN</td>
</tr>
<tr>
<td>Unfactored UDL storage loading</td>
<td>30 kN/m²</td>
</tr>
<tr>
<td>Unfactored internal line loading</td>
<td>30 kN/m</td>
</tr>
<tr>
<td>Unfactored mezz. slab permanent load</td>
<td>1.25 kN/m²</td>
</tr>
<tr>
<td>Unfactored mezz. slab variable load</td>
<td>5 kN/m²</td>
</tr>
<tr>
<td>Unfactored wheel load</td>
<td>40 kN</td>
</tr>
</tbody>
</table>

Design calculations assume 24% load-transfer across the joint.
Location: Typical problem

Design of r.c. pad footing with shear checks

Design to BS8110(1997), partial safety factor for steel \( \gamma_S = 1.15 \)

All calculations to requirements in Clauses 3.4.4.4 and 3.11 of BS8110: Part 1.
Moment & shear are positive in directions shown.

Ultimate axial force in column: \( N = 1606 \text{ kN} \)
Ultimate moment in column: \( M = 163.6 \text{ kNm} \)
Ultimate shear force at base top: \( V = 73 \text{ kN} \)
Column dimension in X-direction: \( cx = 450 \text{ mm} \)
Column dimension in Y-direction: \( cy = 450 \text{ mm} \)
Base length [in bending (X) dir.]: \( lx = 3.6 \text{ m} \)
Width of base (i.e. in Y-dir.): \( ly = 2.8 \text{ m} \)
Note: A separate check should be made to ensure the resistance of the base against sliding due to this shear force.

Characteristic steel strength: \( f_y = 500 \text{ N/mm}^2 \)
Characteristic concrete strength: \( f_{cu} = 35 \text{ N/mm}^2 \)
Overall thickness of base: \( h = 500 \text{ mm} \)
Designated exposure class is XC2
Specified fixing tolerance: \( \text{tol} = 10 \text{ mm} \)
Cover to reinforcement: \( \text{cover} = 35 \text{ mm} \)
Diameter of X-direction bars $\text{dia}_x=16\text{ mm}$  
As actual shear stress ($v=0.45759\text{ N/mm}^2$) does not exceed design stress ($v_c=0.60348\text{ N/mm}^2$), no shear steel is needed across Y-direction faces.  
As actual shear stress ($v=0.45759\text{ N/mm}^2$) does not exceed design stress ($v_c=0.58906\text{ N/mm}^2$), no shear steel is needed across Y-direction faces.  

Diameter of Y-direction bars $\text{dia}_y=12\text{ mm}$  
As actual shear stress ($v=0.26974\text{ N/mm}^2$) does not exceed design stress ($v_c=0.5382\text{ N/mm}^2$), no shear steel is needed across X-direction faces.  
As actual shear stress ($v=0.26974\text{ N/mm}^2$) does not exceed design stress ($v_c=0.50729\text{ N/mm}^2$), no shear steel is needed across X-direction faces.  

### SUMMARY OF REINFORCEMENT REQUIRED IN X-DIRECTION

Since base width (2.8 m) exceeds (3cy/2+9dx/2)=2.7315 m, according to Cl.3.11.3.2, two-thirds of steel must be concentrated within central zone extending 1/2d from each column face (i.e. total width =1.821 m).

<table>
<thead>
<tr>
<th>CENTRAL ZONE</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm$^2$</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of bars required</td>
<td>12</td>
</tr>
<tr>
<td>Alternate bars curtailed at 930 mm from either side of column face.</td>
<td></td>
</tr>
<tr>
<td>Max spacing of bars</td>
<td>160 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>2241.7 mm$^2$</td>
</tr>
<tr>
<td>Area provided</td>
<td>2412.7 mm$^2$</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.26 %</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>EDGE ZONES</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars required</td>
<td>6</td>
</tr>
<tr>
<td>Max spacing of bars</td>
<td>175 mm</td>
</tr>
<tr>
<td>(Note: reduce spacing of outermost bars to obtain side cover.)</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>1120.8 mm$^2$</td>
</tr>
<tr>
<td>Area provided</td>
<td>1206.4 mm$^2$</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.25 %</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ACROSS ENTIRE BASE</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Total number of bars</td>
<td>18</td>
</tr>
<tr>
<td>Effective depth</td>
<td>457 mm</td>
</tr>
<tr>
<td>Total steel area reqd.</td>
<td>3362.5 mm$^2$</td>
</tr>
<tr>
<td>Total steel area prov.</td>
<td>3619.1 mm$^2$</td>
</tr>
<tr>
<td>Av. percentage provided</td>
<td>0.26 %</td>
</tr>
<tr>
<td>Av. weight of steel prov.</td>
<td>10.146 kg/m$^2$</td>
</tr>
</tbody>
</table>
### SUMMARY OF REINFORCEMENT REQUIRED IN Y-DIRECTION

Since base width (3.6 m) exceeds (3cx/2+9dy/2=2.6685 m), according to Cl.3.11.3.2, two-thirds of steel must be concentrated within central zone extending 1½d from each column face (i.e. total width =1.779 m).

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td><strong>CENTRAL ZONE</strong></td>
<td></td>
</tr>
<tr>
<td>Number of bars required</td>
<td>14</td>
</tr>
<tr>
<td>Max spacing of bars</td>
<td>130 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>1442.6 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>1583.4 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.18 %</td>
</tr>
<tr>
<td><strong>EDGE ZONES</strong></td>
<td></td>
</tr>
<tr>
<td>Number of bars required</td>
<td>12</td>
</tr>
<tr>
<td>Max spacing of bars</td>
<td>170 mm</td>
</tr>
<tr>
<td>(Note: reduce spacing of outermost bars to obtain side cover.)</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>1183.7 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>1357.2 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.15 %</td>
</tr>
<tr>
<td>(Note: steel area required is minimum permitted by Table 3.25.)</td>
<td></td>
</tr>
<tr>
<td><strong>ACROSS ENTIRE BASE</strong></td>
<td></td>
</tr>
<tr>
<td>Total number of bars</td>
<td>26</td>
</tr>
<tr>
<td>Effective depth</td>
<td>443 mm</td>
</tr>
<tr>
<td>Total steel area reqd.</td>
<td>2626.2 mm²</td>
</tr>
<tr>
<td>Total steel area prov.</td>
<td>2940.5 mm²</td>
</tr>
<tr>
<td>Av.percentage provided</td>
<td>0.16 %</td>
</tr>
<tr>
<td>Av.weight of steel prov.</td>
<td>6.412 kg/m²</td>
</tr>
</tbody>
</table>

### QUANTITIES

- Total volume of concrete required: 5.04 m³
- Total area of formwork (base sides only): 6.4 m²
- Total volume of reinforcement: 18486 cm³
- Total weight of reinforcement: 145.11 kg

No shear reinforcement is required.
**Location:** Large square base. High moment. Mild steel reinforcement.

**Design of r.c.pad footing with shear checks**

Design to BS8110(1997), partial safety factor for steel $\gamma_S = 1.15$

All calculations to requirements in Clauses 3.4.4.4 and 3.11 of BS8110: Part 1.

Moment & shear are positive in directions shown.

**Ultimate axial force in column** $N = 1000 \text{ kN}$

**Ultimate moment in column** $M = 5000 \text{ kNm}$

**Ultimate shear force at base top** $V = 0 \text{ kN}$

**Column dimension in X-direction** $c_x = 500 \text{ mm}$

**Column dimension in Y-direction** $c_y = 500 \text{ mm}$

**Base length [in bending (X) dir.]** $l_x = 10 \text{ m}$

**Width of base (i.e. in Y-dir.)** $l_y = 10 \text{ m}$

**Characteristic steel strength** $f_y = 250 \text{ N/mm}^2$

**Characteristic concrete strength** $f_{cu} = 50 \text{ N/mm}^2$

**Overall thickness of base** $h = 800 \text{ mm}$

**Designated exposure class is XC3**

**Specified fixing tolerance** $\text{tol} = 10 \text{ mm}$

**Cover to reinforcement** $\text{cover} = 30 \text{ mm}$
Diameter of X-direction bars \( \text{dia}_{x} = 32 \text{ mm} \)

As actual shear stress \( (v = 0.15041 \text{ N/mm}^{2}) \) does not exceed design stress \( (v_{c} = 0.76039 \text{ N/mm}^{2}) \), no shear steel is needed across Y-direction faces.

As actual shear stress \( (v = 0.056528 \text{ N/mm}^{2}) \) does not exceed design stress \( (v_{c} = 0.56049 \text{ N/mm}^{2}) \), no shear steel is needed across Y-direction faces.

Diameter of Y-direction bars \( \text{dia}_{y} = 32 \text{ mm} \)

As actual shear stress \( (v = 0.056528 \text{ N/mm}^{2}) \) does not exceed design stress \( (v_{c} = 0.56576 \text{ N/mm}^{2}) \), no shear steel is needed across X-direction faces.

As actual shear stress \( (v = 0.056528 \text{ N/mm}^{2}) \) does not exceed design stress \( (v_{c} = 0.57237 \text{ N/mm}^{2}) \), no shear steel is needed across X-direction faces.

**SUMMARY OF REINFORCEMENT REQUIRED IN X-DIRECTION**

Since base width \( (10 \text{ m}) \) exceeds \( (3cy/2+9dx/2)=4.134 \text{ m} \), according to Cl.3.11.3.2, two-thirds of steel must be concentrated within central zone extending \( 1\frac{1}{4}d \) from each column face \( (\text{i.e. total width =2.756 m}) \).

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>( 250 \text{ N/mm}^{2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>32 mm</td>
</tr>
<tr>
<td>Number of bars required</td>
<td>19</td>
</tr>
<tr>
<td>Alternate bars curtailed at 2435</td>
<td></td>
</tr>
<tr>
<td>mm from either side of column face.</td>
<td></td>
</tr>
<tr>
<td>Max spacing of bars</td>
<td>150 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>14770 mm(^2)</td>
</tr>
<tr>
<td>Area provided</td>
<td>15281 mm(^2)</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.69 %</td>
</tr>
</tbody>
</table>

**CENTRAL ZONE**

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>( 13908 \text{ mm}^{2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars required</td>
<td>20</td>
</tr>
<tr>
<td>Max spacing of bars</td>
<td>400 mm</td>
</tr>
<tr>
<td>(Note: reduce spacing of outermost bars to obtain side cover.)</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>16085 mm(^2)</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.28 %</td>
</tr>
<tr>
<td>(Note: steel area required is minimum permitted by Table 3.25.)</td>
<td></td>
</tr>
</tbody>
</table>

**EDGE ZONES**

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>( 31366 \text{ mm}^{2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total number of bars</td>
<td>39</td>
</tr>
<tr>
<td>Effective depth</td>
<td>752 mm</td>
</tr>
<tr>
<td>Total steel area reqd.</td>
<td>28679 mm(^2)</td>
</tr>
<tr>
<td>Total steel area prov.</td>
<td>31366 mm(^2)</td>
</tr>
<tr>
<td>Av.percentage provided</td>
<td>0.39 %</td>
</tr>
<tr>
<td>Av.weight of steel prov.</td>
<td>24.622 kg/m(^2)</td>
</tr>
</tbody>
</table>

---

Sample output for SCALE Proforma 110. (ans=2)  
Reinforced concrete design to BS8110 & Eurocode 2  
Made by: IFB  
Pad footing, uniaxial bending, incl section design  
Date: 02/12/19  
Copyright 1986-2019 Fitzroy Systems Ltd.  
Ref No: SC110 BS
SUMMARY OF REINFORCEMENT REQUIRED IN Y-DIRECTION
Since base width (10 m) exceeds (3cx/2+9dy/2=3.99 m),
according to Cl.3.11.3.2, two-thirds of steel must be concentrated
within central zone extending 1½d from each column face
(i.e. total width =2.66 m).

Characteristic strength 250 N/mm²
Diameter of bars 32 mm

CENTRAL ZONE
Number of bars required 7
Max spacing of bars 400 mm
Area required 5107.2 mm²
Area provided 5629.7 mm²
Percentage provided 0.26 %
(Note: steel area required is minimum
permitted by Table 3.25.)

EDGE ZONES
Number of bars required 20
Max spacing of bars 400 mm
(Note: reduce spacing of outermost
bars to obtain side cover.)
Area required 14093 mm²
Area provided 16085 mm²
Percentage provided 0.27 %
(Note: steel area required is minimum
permitted by Table 3.25.)

ACROSS ENTIRE BASE
Total number of bars 27
Effective depth 720 mm
Total steel area reqd. 19200 mm²
Total steel area prov. 21715 mm²
Av.percentage provided 0.27 %
Av.weight of steel prov. 17.046 kg/m²

QUANTITIES
Total volume of concrete required 80 m³
Total area of formwork (base sides only) 32 m²
Total volume of reinforcement 457758 cm³
Total weight of reinforcement 3593.4 kg

No shear reinforcement is required.
Location: Small base, Non-standard bars, no bar curtail./re-spacing

Design of r.c.pad footing with shear checks

Design to BS8110(1997), partial safety factor for steel gammaS=1.15

All calculations to requirements in Clauses 3.4.4.4 and 3.11 of BS8110: Part 1.  
Moment & shear are positive in directions shown.

Ultimate axial force in column  \( N=2500 \text{ kN} \)
Ultimate moment in column \( M=500 \text{ kNm} \)
Ultimate shear force at base top \( V=0 \text{ kN} \)
Column dimension in X-direction \( cx=350 \text{ mm} \)
Column dimension in Y-direction \( cy=350 \text{ mm} \)
Base length [in bending (X) dir.] \( lx=3 \text{ m} \)
Width of base (i.e. in Y-dir.) \( ly=2 \text{ m} \)
Characteristic steel strength \( f_y=350 \text{ N/mm}^2 \)
Characteristic concrete strength \( f_{cu}=40 \text{ N/mm}^2 \)
Min.permiss.proportion of steel \( P_{min1}=0.00196 \)
Critical depth for bar spacing \( h_{crit}=230 \text{ mm} \)
Ultimate bond stress \( f_{bu}=2.3274 \text{ N/mm}^2 \)
Overall thickness of base \( h=750 \text{ mm} \)

WARNING:
Estimated eff.depth of main steel is 0.6375 m.  As distance from base edge to column face in Y-direction (i.e. 0.825 m) is less than 1½ times this value, special consideration must be given to the location and configuration of planes controlling both direct and punching shear.  
This is beyond the scope of the current proforma.
Location: Typical problem

Design of reinforced concrete pad footing with shear checks

All calculations to EC2 Part 1-1 and IStructE Manual to EC2. Moment & shear are positive in directions shown.

Ultimate axial force in column \( N_{Ed} = 1606 \) kN
Ultimate moment in column \( M_{Ed} = 163.6 \) kNm
Ultimate shear force at base top \( V_{Ed}' = 73 \) kN
Column dimension in X-direction \( c_x = 450 \) mm
Column dimension in Y-direction \( c_y = 450 \) mm
Base length (bending in X-dir.) \( l_x = 3.6 \) m
Width of base (i.e. in Y-dir.) \( l_y = 2.8 \) m
Partial safety factor for steel \( \gamma_{a,m} = 1.15 \)
Partial safety factor for conc. \( \gamma_{c} = 1.5 \)
Note: A separate check should be made to ensure the resistance of the base against sliding due to this shear force.
Char yield strength of reinforc'ment \( f_{yk} = 500 \) N/mm²
Overall thickness of base \( h = 550 \) mm
Diameter of X-direction bars \( d_{ia} = 16 \) mm
Diameter of Y-direction bars \( d_{ia} = 12 \) mm

Ground pressures

Pressure at end A \( \frac{N_{Ed} + S_{w}}{l_x \times l_y} - \frac{N_{Ed} + S_{w}}{e/z} = 144.2 \) kN/m²
Pressure at end B

\[ pb = \frac{(NEd+Swt)}{(lx*ly)} + \frac{(NEd+Swt)*e}{z} = 211.58 \text{ kN/m}^2 \]

Ultimate moments and shears

Design shear force at 1.0dx

\[ Vdx = 566.4 \text{ kN} \]

Reinforcement to resist bending in X-direction

Diameter of X-direction bars

\[ diaX = 16 \text{ mm} \]

Area of tension steel required

\[ Asx = Mdx*10^6/(fyk/gams*z) = 3038 \text{ mm}^2 \]

Area of tension steel provided

\[ Asprx = Nox*\pi*diaX^2/4 = 3217 \text{ mm}^2 \]

Check spacing and proportion of steel

Percentage of steel provided

\[ per = \frac{Asprx}{10*widY*h} = 0.2089 \% \text{ of gross section.} \]

Shear across base in X-direction

Shear resistance

\[ VRdc = \beta' * CRdc * ks * VRdc * bw * d / 1000 = 1027 \text{ kN} \]

Modified shear resistance

\[ VRdc = VRdm = 1092 \text{ kN} \]

SHEAR SUMMARY

<table>
<thead>
<tr>
<th>Shear force</th>
<th>566.4 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>1092 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Reinforcement to resist bending in Y-direction

Area of tension steel required

\[ Asy = Mdy*10^6/(fyk/gams*z) = 1944 \text{ mm}^2 \]

Reinforcement: central strip of width 1.929 m

Area of tension steel provided

\[ Aspry = Noy*\pi*diay^2/4 = 1696 \text{ mm}^2 \]

Check spacing and proportion of steel

Percentage of steel provided

\[ per = \frac{Aspry}{10*widX*h} = 0.1599 \% \text{ of gross section.} \]

Reinforcement: edge strips, each of width 0.8355 m

Area of tension steel provided

\[ Aspry = Noy*\pi*diay^2/4 = 1583 \text{ mm}^2 \]

Check spacing and proportion of steel

Percentage of steel provided

\[ per = \frac{Aspry}{10*widX*h} = 0.1723 \% \text{ of gross section.} \]

Shear across base in Y-direction
Shear: central strip of width 1.929 m

Shear resistance
VRdc=beta'*CRdc*ks*vRdc*bw*d/1000
=638.6 kN

Modified shear resistance
VRdc=VRdm=737.7 kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>405.8 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>737.7 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Shear: edge strips, each of width 0.8355 m

Shear resistance
VRdc=beta'*CRdc*ks*vRdc*bw*d/1000
=567.2 kN

Modified shear resistance
VRdc=VRdm=639.1 kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>405.8 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>639.1 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face

Punching shear at column face
VEd=beta*NEd=1847 kN

Maximum shear resistance
VRdmax=0.5*v*fcd*uo*dav/1000=4476 kN

The punching shear at column face (VEd=1847 kN) is less than the maximum shear resistance (VRdmax=4476 kN), hence OK.

Check punching shear at critical plane

Punching-shear resistance reqd.
VEd=pf*lx*ly-Vup=878 kN

Shear stress at control perimeter
vEd=beta*VEd*1000/(u1*dav)
=0.2498 N/mm²

Design punching shear resistance
VRdc=(0.18/gamc)*k*(100*p1 ^*fck)^0.333=0.3534 N/mm²

Enhanced punching shear resist.
VRdc=senh*VRdc=0.3534 N/mm²

Design punching shear resistance
VRdc=senh*0.035*k^1.5*fck^0.5
=0.3863 N/mm²
SUMMARY OF REINFORCEMENT REQUIRED IN X-DIRECTION

As base width (2.8 m) does not exceed \((3cy/2+9dx/2=4.2 \text{ m})\), bars should be spaced uniformly across base.

- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm
- Total number of bars: 16
- Max spacing of bars: 180 mm
- Effective depth: 507 mm
- Total steel area req'd: 3038 mm²
- Total steel area prov'd: 3217 mm²

SUMMARY OF REINFORCEMENT REQUIRED IN Y-DIRECTION

Since base width (3.6 m) exceeds \((3cx/2+9dy/2=2.894 \text{ m})\), two-thirds of steel must be concentrated within central zone extending \(1/2d\) from each column face (i.e. total width =1.929 m).

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Central zone: Number of bars required: 15
  - Max spacing of bars: 130 mm
  - Area required: 1591 mm²
  - Area provided: 1696 mm²
  - Percentage provided: 0.16%
- Edge zones: Number of bars required: 14
  - Max spacing of bars: 130 mm
  - Area required: 1379 mm²
  - Area provided: 1583 mm²
  - Percentage provided: 0.17%
- Across entire base: Total number of bars: 29
  - Effective depth: 493 mm
  - Total steel area req'd: 2970 mm²
  - Total steel area prov'd: 3280 mm²

QUANTITIES

- Total concrete volume: 5.544 m³
- Total area of formwork: 7.04 m² (base sides only).
Location: Large square base with high moment

Design of reinforced concrete pad footing with shear checks

All calculations to EC2 Part 1-1 and IStructE Manual to EC2. Moment & shear are positive in directions shown.

Ultimate axial force in column \( N_{Ed} = 1000 \text{ kN} \)
Ultimate moment in column \( M_{Ed} = 5000 \text{ kNm} \)
Ultimate shear force at base top \( V_{Ed'} = 0 \text{ kN} \)
Column dimension in X-direction \( c_x = 500 \text{ mm} \)
Column dimension in Y-direction \( c_y = 500 \text{ mm} \)
Base length (bending in X-dir.) \( l_x = 10 \text{ m} \)
Width of base (i.e. in Y-dir.) \( l_y = 10 \text{ m} \)
Partial safety factor for steel \( \gamma_{ams} = 1.15 \)
Partial safety factor for conc. \( \gamma_{amc} = 1.5 \)
Char yield strength of reinf'ment \( f_{yk} = 500 \text{ N/mm}^2 \)
Overall thickness of base \( h = 900 \text{ mm} \)
Diameter of X-direction bars \( d_{iax} = 32 \text{ mm} \)
Diameter of Y-direction bars \( d_{iay} = 32 \text{ mm} \)

Ground pressures

Pressure at end A

\[
p_a = \frac{(N_{Ed} + S_{Wt})}{(l_x \times l_y)} - \frac{(N_{Ed} + S_{Wt}) \times e}{z} \]

\[
= 10.375 \text{ kN/m}^2
\]

Pressure at end B

\[
p_b = \frac{(N_{Ed} + S_{Wt})}{(l_x \times l_y)} + \frac{(N_{Ed} + S_{Wt}) \times e}{z} \]

\[
= 70.375 \text{ kN/m}^2
\]
Ultimate moments and shears

Design shear force at 1.0dx \( V_{dx} = 1117.6 \text{ kN} \)

Reinforcement to resist bending in X-direction

Diameter of X-direction bars \( \text{dia}_x = 32 \text{ mm} \)
Area of tension steel required \( A_{sx} = \frac{M_{dx} \times 10^6}{f_{yk} / g_{ams} \times z} = 9894 \text{ mm}^2 \)

Reinforcement: central strip of width 3.026 m

Area of tension steel provided \( A_{sprx} = \text{No}_x \times \pi \times \text{dia}_x^2 / 4 = 8847 \text{ mm}^2 \)

Check spacing and proportion of steel

Percentage of steel provided \( \text{per} = \frac{A_{sprx}}{10 \times \text{wid}_Y \times h} \)
\( = 0.3248 \% \text{ of gross section.} \)

Reinforcement: edge strips, each of width 3.487 m

Min area of main steel required \( A_{sx} = P_{min1} \times 1000 \times \text{wid}_Y \times h = 9415 \text{ mm}^2 \)
Area of tension steel provided \( A_{sprx} = \text{No}_x \times \pi \times \text{dia}_x^2 / 4 = 19302 \text{ mm}^2 \)

Check spacing and proportion of steel

Percentage of steel provided \( \text{per} = \frac{A_{sprx}}{10 \times \text{wid}_Y \times h} \)
\( = 0.3075 \% \text{ of gross section.} \)

Shear across base in X-direction

Shear: central strip of width 3.026 m

Shear resistance
\( VR_{dc} = \beta' \times CR_{dc} \times ks \times vR_{dc} \times bw \times d / 1000 \)
\( = 2186 \text{ kN} \)

SHEAR SUMMARY

<table>
<thead>
<tr>
<th>Shear force</th>
<th>1118 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>2186 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Shear: edge strips, each of width 3.487 m

Shear resistance
\( VR_{dc} = \beta' \times CR_{dc} \times ks \times vR_{dc} \times bw \times d / 1000 \)
\( = 4947 \text{ kN} \)

SHEAR SUMMARY

<table>
<thead>
<tr>
<th>Shear force</th>
<th>1118 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>4947 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.
Reinforcement to resist bending in Y-direction

Area of tension steel required \[ \text{Asy}=\text{Mdy}*10^6/(\text{fyk}/\text{gams}*\text{z})=3372 \text{ mm}^2 \]

Reinforcement: central strip of width 2.93 m

Area of tension steel provided \[ \text{Aspry}=\text{Noy}*\pi*\text{diay}^2/4=8042 \text{ mm}^2 \]

Check spacing and proportion of steel

Percentage of steel provided \[ \text{per}=\text{Aspry}/(10*\text{widX}*\text{h}) \]
\[ =0.305 \% \text{ of gross section.} \]

Reinforcement: edge strips, each of width 3.535 m

Area of tension steel provided \[ \text{Aspry}=\text{Noy}*\pi*\text{diay}^2/4=20910 \text{ mm}^2 \]

Check spacing and proportion of steel

Percentage of steel provided \[ \text{per}=\text{Aspry}/(10*\text{widX}*\text{h}) \]
\[ =0.3286 \% \text{ of gross section.} \]

Shear across base in Y-direction

Shear: central strip of width 2.93 m

Shear resistance \[ \text{VRdc}=\beta'^*\text{CRdc}^*\text{ks}^*\text{vRdc}^*\text{bw}^*\text{d}/1000 \]
\[ =2033 \text{ kN} \]

SHEAR SUMMARY

<table>
<thead>
<tr>
<th></th>
<th>Shear force</th>
<th>Shear resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>398.5 kN</td>
<td>2033 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Shear: edge strips, each of width 3.535 m

Shear resistance \[ \text{VRdc}=\beta'^*\text{CRdc}^*\text{ks}^*\text{vRdc}^*\text{bw}^*\text{d}/1000 \]
\[ =5029 \text{ kN} \]

SHEAR SUMMARY

<table>
<thead>
<tr>
<th></th>
<th>Shear force</th>
<th>Shear resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>398.5 kN</td>
<td>5029 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face

Punching shear at column face \[ \text{VEd}=\beta^*\text{NEd}=1150 \text{ kN} \]

Maximum shear resistance \[ \text{VRdmax}=0.5^*\text{v}^*\text{fcd}^*\text{uo}^*\text{dav}/1000=11101 \text{ kN} \]

The punching shear at column face ( \( \text{VEd}=1150 \text{ kN} \) ) is less than the maximum shear resistance ( \( \text{VRdmax}=11101 \text{ kN} \) ), hence OK.

Check punching shear at critical plane

Punching-shear resistance reqd. \[ \text{VEd}=\text{pf}^*\text{lx}^*\text{ly}^*\text{Vup}=3548 \text{ kN} \]
Shear stress at control perimeter $v_{Ed} = \beta \cdot V_{Ed} \cdot 1000 / (u_1 \cdot d_{av}) = 0.399 \text{ N/mm}^2$

Design punching shear resistance $v_{Rdc} = (0.18 / \gamma_{cm}) \cdot k \cdot (100 \cdot p_1 \cdot f_{ck})^{0.333} = 0.427 \text{ N/mm}^2$

Enhanced punching shear resistance $v_{Rdc} = \text{senh} \cdot v_{Rdc} = 0.427 \text{ N/mm}^2$

**SUMMARY OF REINFORCEMENT REQUIRED IN X-DIRECTION**

Since base width (10 m) exceeds (3cy/2+9dx/2=4.539 m)
two-thirds of steel must be concentrated within central zone extending $1/2d$ from each column face
(i.e. total width = 3.026 m).

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>32 mm</td>
</tr>
<tr>
<td>Number of bars required</td>
<td>11</td>
</tr>
<tr>
<td>Max spacing of bars</td>
<td>300 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>6596 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>8847 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.32 %</td>
</tr>
</tbody>
</table>

**CENTRAL ZONE**

| Number of bars required | 24        |
| Max spacing of bars     | 300 mm    |

(Note: reduce spacing as necessary to accommodate bars required)

| Area required           | 9415 mm²  |
| Area provided           | 19302 mm² |
| Percentage provided     | 0.31 %    |

**EDGE ZONES**

| Total number of bars    | 35        |
| Effective depth         | 842 mm    |
| Total steel area reqd.  | 16011 mm² |
| Total steel area prov.  | 28149 mm² |

**ACROSS ENTIRE BASE**
### SUMMARY OF REINFORCEMENT REQUIRED IN Y-DIRECTION

Since base width (10 m) exceeds \( 3cx/2+9dy/2=4.395 \text{ m} \)
two-thirds of steel must be concentrated within central zone extending \( 1\times d \) from each column face
(\( \text{i.e. } \) total width =2.93 m).

<table>
<thead>
<tr>
<th></th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>32 mm</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td></td>
</tr>
</tbody>
</table>

#### CENTRAL ZONE

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars required</td>
<td>10</td>
</tr>
<tr>
<td>Max spacing of bars</td>
<td>300 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>3956 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>8042 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.3 %</td>
</tr>
</tbody>
</table>

#### EDGE ZONES

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars required</td>
<td>26</td>
</tr>
<tr>
<td>Max spacing of bars</td>
<td>300 mm</td>
</tr>
<tr>
<td>(Note: reduce spacing as necessary to accommodate bars required)</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>9545 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>20910 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.33 %</td>
</tr>
</tbody>
</table>

#### ACROSS ENTIRE BASE

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Total number of bars</td>
<td>36</td>
</tr>
<tr>
<td>Effective depth</td>
<td>810 mm</td>
</tr>
<tr>
<td>Total steel area reqd.</td>
<td>13500 mm²</td>
</tr>
<tr>
<td>Total steel area prov.</td>
<td>28953 mm²</td>
</tr>
</tbody>
</table>

#### QUANTITIES

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Total concrete volume</td>
<td>90 m³</td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>36 m² (base sides only)</td>
</tr>
</tbody>
</table>
Location: Small base, non-standard bars, no bar curtail./re-spacing

Design of reinforced concrete pad footing with shear checks

All calculations to EC2 Part 1-1 and IStructE Manual to EC2.
Moment & shear are positive in directions shown.

Ultimate axial force in column \( N_{Ed} = 2500 \text{ kN} \)
Ultimate moment in column \( M_{Ed} = 500 \text{ kNm} \)
Ultimate shear force at base top \( V_{Ed}' = 0 \text{ kN} \)
Column dimension in X-direction \( c_x = 350 \text{ mm} \)
Column dimension in Y-direction \( c_y = 350 \text{ mm} \)
Base length (bending in X-dir.) \( l_x = 3 \text{ m} \)
Width of base (i.e. in Y-dir.) \( l_y = 2 \text{ m} \)
Partial safety factor for steel \( \gamma_{ams} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mac} = 1.5 \)
Char yield strength of reinf'ment \( f_yk = 500 \text{ N/mm}^2 \)
Overall thickness of base \( h = 750 \text{ mm} \)

WARNING:
Estimated eff. depth of main steel is 0.6375 m. As distance from base edge to column face in Y-direction (i.e. 0.825 m) is less than 1½ times this value, special consideration must be given to the location and configuration of planes controlling both direct and punching shear.
This is beyond the scope of the current proforma.
Location: Typical uniformly-loaded deep beam with high-yield steel.

Design of deep beam using Kong simplified method

See Table 148 of Reinforced Concrete Designer's Handbook 10th edition.

Span between centres of supports \( l = 3 \) m
Overall depth of beam \( h = 2 \) m
Breadth of beam \( b = 0.5 \) m
Load per unit length (factored) \( w = 500 \) kN/m
Grade of concrete \( f_{cu} = 30 \) N/mm\(^2\)
Yield strength of reinforcement \( f_y = 500 \) N/mm\(^2\)
From table of data given by Kong, Robins and Sharp, concrete cylinder splitting tensile strength \( F_t = 2.74 \) N/mm\(^2\)
Designated exposure class is XC1
Specified fixing tolerance \( \text{tol} = 10 \) mm
Cover to all bars \( \text{cover} = 35 \) mm

Main reinforcement

shear bars above opening (if required)

typical steel arrangement
provide nominal vertical links throughout span (max. diameter assumed to be 12 mm in this proforma)

Diameter of main tension bars \( \text{dia} = 12 \) mm
Number of main bars \( \text{NumBar} = 12 \)
Effective depth of main bars \( d = 1925 \) mm
### MAIN TENSION

| Characteristic strength | 500 N/mm² |

### REINFORCEMENT

| Diameter of bars | 12 mm |

### SUMMARY

| Number of bars | 12 |
| Area required  | 1300 mm² |
| Area provided  | 1357.2 mm² |
| Percentage provided | 0.13572 % |

### Anti-Cracking

Diameter of anti-cracking bars  Ldia=12 mm

### ANTI-CRACKING

| Characteristic strength | 500 N/mm² |

### REINFORCEMENT

| Diameter of bars | 12 mm |

### SUMMARY

| Number of bars (each face) | 11 |
| Spacing                  | 130 mm |
Location: Lightweight concrete beam with point loads and mild steel.

Design of deep beam using Kong simplified method

See Table 148 of Reinforced Concrete Designer's Handbook 10th edition.

Span between centres of supports $l=5\text{ m}$
Overall depth of beam $h=4\text{ m}$
Breadth of beam $b=0.6\text{ m}$

Value of each point load $F_{sep}=750\text{ kN}$
Distance to point load $a_1=1.5\text{ m}$

Support face to edge of opening $Na_1=0.3\text{ m}$
Width of opening                      \( B_{1} = 0.3 \) m
Beam soffit to bottom of opening     \( X_{\text{H}} = 1.6 \) m
Depth of opening                     \( Z_{\text{H}} = 0.3 \) m
Grade of concrete                    \( f_{\text{cu}} = 35 \) N/mm\(^2\)
Yield strength of reinforcement      \( f_{y} = 250 \) N/mm\(^2\)
Cylinder splitting tensile strength  \( f_{\text{t}} = 2.95 \) N/mm\(^2\)
Designated exposure class is XC2

Main reinforcement

Shear bars above opening (if required)

Provide nominal vertical links throughout span (max. diameter assumed to be 12 mm in this proforma)

Horizontal shear bars (or links) below opening (if required)

Main reinforcement

Note: If shear steel is needed, bars must be provided both above and below opening.

Diameter of main tension bars        \( \text{dia} = 25 \) mm
Number of main bars                  \( \text{NumBar} = 12 \)
Effective depth of main bars         \( d = 3890 \) mm

**Main Tension**

Characteristic strength             250 N/mm\(^2\)
Diameter of bars                     25 mm
Number of bars                       12
Area required                        5760 mm\(^2\)
Area provided                        5890.5 mm\(^2\)
Percentage provided                  0.24544%  

Diameter of anti-cracking bars       \( L_{\text{dia}} = 12 \) mm

**Anti-Cracking**

Characteristic strength             250 N/mm\(^2\)
Diameter of bars                     12 mm
Number of bars (each face)           39
Spacing                               70 mm
**Location:** Uniformly-loaded beam with square-twist reinforcement

**Design of deep beam using Kong simplified method**

See Table 148 of Reinforced Concrete Designer's Handbook 10th edition.

Span between centres of supports \( l = 4.5 \) m
Overall depth of beam \( h = 2.5 \) m
Breadth of beam \( b = 0.6 \) m
Load per unit length (factored) \( w = 500 \) kN/m

Support face to edge of opening \( Na1 = 0.2 \) m
Width of opening \( Ba1 = 0.5 \) m
Beam soffit to bottom of opening \( XiH = 0.8 \) m
Depth of opening \( ZetaH = 0.5 \) m
Grade of concrete \( fcu = 40 \) N/mm\(^2\)
Yield strength of reinforcement \( fy = 425 \) N/mm\(^2\)
Cylinder splitting tensile strength of concrete \( Ft = Ft = 3.16 \) N/mm\(^2\)
Min.permiss.percentage of steel \( k0 = 0.16 \)
Value of reinforcement constant \( k2 = 187.5 \)
Designated exposure class is XC3
Specified fixing tolerance \( tol = 10 \) mm
Cover to all bars \( cover = 35 \) mm
Main reinforcement

shear bars above opening (if required)

TYPICAL STEEL ARRANGEMENT

main reinforcement Note: If shear steel is needed, bars must be provided both above and below opening.

Diameter of main tension bars \( \text{dia} = 25 \text{ mm} \)
Number of main bars \( \text{NumBar} = 13 \)
Effective depth of main bars \( d = 2425 \text{ mm} \)

<table>
<thead>
<tr>
<th>MAIN TENSION</th>
<th>Characteristic strength</th>
<th>425 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>Area required</td>
<td>6351.4 mm²</td>
</tr>
<tr>
<td></td>
<td>Area provided</td>
<td>6381.4 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided</td>
<td>0.42542%</td>
</tr>
</tbody>
</table>

Diameter of anti-cracking bars \( \text{Ldia} = 16 \text{ mm} \)

<table>
<thead>
<tr>
<th>ANTI-CRACKING</th>
<th>Characteristic strength</th>
<th>425 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars (each face)</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>205 mm</td>
</tr>
</tbody>
</table>
Location: Ex1 - Typical uniformly-loaded deep beam (NWC)

Design of deep beam using Kong simplified method

Span between centres of supports \( L = 3 \text{ m} \)
Overall depth of beam \( h = 2 \text{ m} \)
Breadth of beam \( b = 0.5 \text{ m} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Design load per unit length \( w = 500 \text{ kN/m} \)
Char yield strength of reinforc'ent \( f_{yk} = 500 \text{ N/mm}^2 \)
From table of data given by Kong, Robins and Sharp, concrete cylinder splitting tensile strength \( f_t = 2.74 \text{ N/mm}^2 \)
Diameter of tension bars \( d_{iat} = 20 \text{ mm} \)
Diameter of link legs \( d_{ial} = 12 \text{ mm} \)

Main reinforcement

shear bars above opening (if required)

TYPICAL STEEL ARRANGEMENT

provide nominal vertical links throughout span (max. diameter assumed to be 12 mm in this proforma).

Diameter of main tension bars \( d_{ia} = 20 \text{ mm} \)
Number of main bars \( \text{NumBar} = 5 \)

NOTE: If shear steel is needed, bars must be provided both above and below the openings as indicated in the above diagram.
Effective depth of main bars \( d = 1925 \text{ mm} \)

<table>
<thead>
<tr>
<th>MAIN TENSION</th>
<th>Characteristic strength ( 500 \text{ N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars ( 20 \text{ mm} )</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars ( 5 )</td>
</tr>
<tr>
<td></td>
<td>Area required ( 1500 \text{ mm}^2 )</td>
</tr>
<tr>
<td></td>
<td>Area provided ( 1570.8 \text{ mm}^2 )</td>
</tr>
<tr>
<td></td>
<td>Percentage provided ( 0.15708 % )</td>
</tr>
</tbody>
</table>

Shear reinforcement

From Table 148 in 'Reinforced Concrete Designer's Handbook' using formulae C and D, we get the following:

Shear resistance \( \text{VRd} = V_x1 + V_x2 = 1964.5 \text{ kN} \)

As \( \text{VRd} \geq \text{VED} \) \( (1964.5 \text{ kN} \geq 798.6 \text{ kN}) \), the applied shear force is within the shear resistance from the main bars. Hence provide nominal shear reinforcement only.

Diameter of anti-cracking bars \( L_{\text{dia}} = 12 \text{ mm} \)

<table>
<thead>
<tr>
<th>ANTI-CRACKING</th>
<th>Characteristic strength ( 500 \text{ N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars ( 12 \text{ mm} )</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>No. of bars (each face) ( 11 )</td>
</tr>
<tr>
<td></td>
<td>Spacing ( 130 \text{ mm} )</td>
</tr>
</tbody>
</table>
Design of deep beam using Kong simplified method

Span between centres of supports  L=5 m
Overall depth of beam  h=4 m
Breadth of beam  b=0.6 m
Partial safety factor for steel  \( g_{ams} = 1.15 \)
Partial safety factor for conc.  \( g_{amc} = 1.5 \)

Factored point load  \( F_{sep} = 750 \text{ kN} \)
Distance to point load  \( a_1 = 1.5 \text{ m} \)
Support face to edge of opening \( Na_1 = 0.3 \text{ m} \)
Width of opening \( Ba_1 = 0.3 \text{ m} \)
Beam soffit to bottom of opening \( Xi_H = 1.6 \text{ m} \)
Depth of opening \( Zeta_H = 0.3 \text{ m} \)
Char yield strength of reinforcement \( f_yk = 500 \text{ N/mm}^2 \)
Cylinder splitting tensile strength of concrete \( F_{ft} = 2.95 \text{ N/mm}^2 \)
Diameter of tension bars \( diat = 20 \text{ mm} \)
Diameter of link legs \( dial = 12 \text{ mm} \)

**Main reinforcement**

shear bars above opening (if required)

**TYPICAL STEEL ARRANGEMENT**

provide nominal vertical links throughout span (max. diameter assumed to be 12 mm in this proforma).

NOTE: If shear steel is needed, bars must be provided both above and below the openings as indicated in the above diagram.

Diameter of main tension bars \( dia = 16 \text{ mm} \)
Number of main bars \( NumBar = 18 \)
Effective depth of main bars \( d = 3890 \text{ mm} \)

**MAIN TENSION**

**REINFORCEMENT**

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>18</td>
</tr>
<tr>
<td>Area required</td>
<td>3600 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>3619.1 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.1508 %</td>
</tr>
</tbody>
</table>

**Shear reinforcement**

From Table 148 in 'Reinforced Concrete Designer's Handbook' using formulae C and D, we get the following:
Shear resistance \( VR_d = V_x1 + V_x2 = 1924.4 \text{ kN} \)
As \( VR_d \geq V_{Ed} \) (1924.4 kN \geq 750 kN), the applied shear force is within the shear resistance from the main bars. Hence provide nominal shear reinforcement only.
Diameter of anti-cracking bars \( L_{dia} = 12 \text{ mm} \)

**ANTI-CRACKING**

**REINFORCEMENT**

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>No. of bars (each face)</td>
<td>20</td>
</tr>
<tr>
<td>Spacing</td>
<td>140 mm</td>
</tr>
</tbody>
</table>
WARNING:
Two different tension steel diameters have been used. Please rerun calculation using the same tension steel diameter.
Location: Ex3 - Deep beam with UDL and shear bars (NWC)

Design of deep beam using Kong simplified method

Span between centres of supports  $L=3$ m
Overall depth of beam  $h=2$ m
Breadth of beam  $b=0.55$ m
Partial safety factor for steel  $\gamma_{ams}=1.15$
Partial safety factor for conc.  $\gamma_{m}=1.5$
Design load per unit length  $w=1800$ kN/m
Char yield strength of reinf'ment  $f_{yk}=500$ N/mm²
From table of data given by Kong, Robins and Sharp, concrete cylinder splitting tensile strength  $f_{t}=2.74$ N/mm²
Diameter of tension bars  $d_{iat}=25$ mm
Diameter of link legs  $d_{ial}=12$ mm

Main reinforcement

shear bars above opening (if required)

provide nominal vertical links throughout span (max. diameter assumed to be 12 mm in this proforma).

NOTE: If shear steel is needed, bars must be provided both above and below the openings as indicated in the above diagram.

Diameter of main tension bars  $d_{ia}=25$ mm
Number of main bars  NumBar=8
Effective depth of main bars  \( d = 1925 \text{ mm} \)

<table>
<thead>
<tr>
<th>MAIN TENSION</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Area required</td>
<td>3923.7 mm²</td>
</tr>
<tr>
<td></td>
<td>Area provided</td>
<td>3927 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided</td>
<td>0.357 %</td>
</tr>
</tbody>
</table>

Shear reinforcement

From Table 148 in 'Reinforced Concrete Designer's Handbook' using formulae C and D, we get the following:

Shear resistance \( V_{Rd} = V_x1 + V_x2 = 2578.5 \text{ kN} \)

As the applied shear force of 2753.5 kN exceeds the resistance provided by the main steel of 2578.5 kN extra web steel must be employed to resist the excess force of 174.98 kN.

Diameter of shear bars \( \ell_{dia} = 12 \text{ mm} \)

<table>
<thead>
<tr>
<th>HORIZONTAL SHEAR</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars (each face)</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>230 mm</td>
</tr>
<tr>
<td></td>
<td>Resistance required</td>
<td>174.98 kN</td>
</tr>
<tr>
<td></td>
<td>Resistance provided</td>
<td>186.51 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ANTI-CRACKING</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>No. of bars (each face)</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>130 mm</td>
</tr>
</tbody>
</table>
Location: Typical circular column, tensile and compressive stress

Modular-ratio theory: design of reinforcement for circular column

Calculations for reinforcement are in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of (mr-1) is considered where the steel is in compression). Min.steel = six bars arranged symmetrically.

Modular ratio \( mr = 15 \)
Diameter of column \( D = 600 \text{ mm} \)
Depth to centre of steel \( d' = 56 \text{ mm} \)
Axial load \( N = 1000 \text{ kN} \)
Applied bending moment \( M = 250 \text{ kNm} \)
Limiting concrete stress \( f_c = 15 \text{ N/mm}^2 \)
Limiting steel stress \( f_s = 250 \text{ N/mm}^2 \)
Main steel diameter \( \text{dia} = 12 \text{ mm} \)
Number of bars to be provided \( \text{NumBar} = 37 \)
Depth to neutral axis \( 316.65 \text{ mm} \)
Maximum stress in concrete \( 14.99 \text{ N/mm}^2 \) compression
Theoretical min.concrete stress \( 13.414 \text{ N/mm}^2 \) tension
Max.stress in steel near top \( 185.09 \text{ N/mm}^2 \) compression
Max.stress in steel near bottom \( 161.44 \text{ N/mm}^2 \) tension

Axial-load and moment resistance of section as designed

Axial resistance due to concrete:
\[
N_c = \frac{f_1}{2} \times D \times D / (1 - \cos(BE)) \times (\sin(BE)^3 / 3 - BE/2 \times \cos(BE) + \cos(BE) \times \sin(2 \times BE) / 4) = 968131 \text{ N}
\]

Axial resistance due to compression reinforcement:
\[
N_s1 = f_1 \times (mr-1) \times R_0 / \pi \times A_{prov} / (R - R_0 \times \cos(AL)) \times (\sin(AL) - AL \times \cos(AL)) = 239151 \text{ N}
\]

Axial resistance due to tension reinforcement:
\[
N_s2 = f_1 \times mr \times R_0 / \pi \times A_{prov} / (R - R_0 \times \cos(AL)) \times (\pi/2 - AL/2 + \sin(2 \times AL) / 4) = 206717 \text{ N}
\]

Total axial resistance provided \( N_{res} = (N_c + N_s1 - N_s2) / 1E3 = 1000.6 \text{ kN} \)

Resistance moment due to concrete:
\[
M_c = \frac{f_1}{4} \times D^3 / (1 - \cos(BE)) \times (BE/8 - \sin(4 \times BE) / 32 - \cos(BE) \times \sin(BE)^3 / 3) / 3 = 164.86E6 \text{ Nmm}
\]

Resistance moment due to compression reinforcement:
\[
M_s1 = f_1 \times (mr-1) \times R_0 / \pi \times A_{prov} / (R - R_0 \times \cos(AL)) \times (AL/2 - \sin(2 \times AL) / 4) = 44.892E6 \text{ Nmm}
\]

Resistance moment due to tension reinforcement:
\[
M_s2 = f_1 \times mr \times R_0 / \pi \times A_{prov} / (R - R_0 \times \cos(AL)) \times (\pi/2 - AL/2 + \sin(2 \times AL) / 4) = 40.413E6 \text{ Nmm}
\]

Total resistance moment provided \( M_{res} = (M_c + M_s1 + M_s2) / 1E6 = 250.16 \text{ kNm} \)
Location: Small-diameter circular column subjected to heavy load

Modular-ratio theory: design of reinforcement for circular column

Calculations for reinforcement are in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of \( \text{mr}-1 \)) is considered where the steel is in compression.
Min.steel = six bars arranged symmetrically.

Modular ratio \( \text{mr}=15 \)
Diameter of column \( D=300 \) mm
Depth to centre of steel \( d'=48 \) mm
Axial load \( N=375 \) kN
Applied bending moment \( M=45 \) kNm
Limiting concrete stress \( f_c=15 \) N/mm²
Limiting steel stress \( f_s=250 \) N/mm²
Main steel diameter \( \text{dia}=20 \) mm
Number of bars to be provided \( \text{NumBar}=12 \)
Depth to neutral axis \( 174.94 \) mm
Maximum stress in concrete \( 14.701 \) N/mm² compression
Theoretical min.concrete stress \( 10.51 \) N/mm² tension
Max.stress in steel near top \( 160.01 \) N/mm² compression
Max.stress in steel near bottom \( 97.141 \) N/mm² tension

Axial-load and moment resistance of section as designed

Axial resistance due to concrete:
\[
N_c = f_1/2*D*D/(1-COS(BE))*(SIN(BE)^3/3-BE/2*COS(BE)+COS(BE)*SIN(2*BE))/4 = 276479 \text{ N}
\]

Axial resistance due to compression reinforcement:
\[
N_{s1} = f_1*(\text{mr}-1)*R_0/\pi*A_{Prov}/(R-R_0*COS(AL))*(SIN(AL)-AL*COS(AL)) = 207774 \text{ N}
\]

Axial resistance due to tension reinforcement:
\[
N_{s2} = f_1*mr*R_0/\pi*A_{Prov}/(R-R_0*COS(AL))*(\pi/2-AL/2+SIN(2*AL)/4) = 101698 \text{ N}
\]

Total axial resistance provided \( N_{res} = (N_c+N_{s1}-N_{s2})/1E3 = 382.56 \) kN

Resistance moment due to concrete:
\[
M_c = f_1/4*D^3/(1-COS(BE))*(BE/8-SIN(4*BE)/32-COS(BE)*SIN(2*BE)) = 21.791E6 \text{ Nmm}
\]

Resistance moment due to compression reinforcement:
\[
M_{s1} = f_1*(\text{mr}-1)*R_0/\pi*A_{Prov}/(R-R_0*COS(AL))*(AL/2-SIN(2*AL)/4) = 15.398E6 \text{ Nmm}
\]

Resistance moment due to tension reinforcement:
\[
M_{s2} = f_1*mr*R_0/\pi*A_{Prov}/(R-R_0*COS(AL))*(\pi/2-AL/2+SIN(2*AL)/4) = 8.725E6 \text{ Nmm}
\]

Total resistance moment provided \( M_{res} = (M_c+M_{s1}+M_{s2})/1E6 = 45.915 \) kNm
Location: Massive lightly-loaded column reinforced with mild steel

Modular-ratio theory: design of reinforcement for circular column

Calculations for reinforcement are in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of $(mr-1)$ is considered where the steel is in compression). Min.steel = six bars arranged symmetrically.

Modular ratio  $mr=15$
Diameter of column  $D=1500$ mm
Depth to centre of steel  $d'=60$ mm
Axial load  $N=24000$ kN
Applied bending moment  $M=1000$ kNm
Limiting concrete stress  $fc=15$ N/mm$^2$
Limiting steel stress  $fs=180$ N/mm$^2$
Main steel diameter  $dia=16$ mm
Number of bars to be provided  $NumBar=60$
Maximum stress in concrete  $14.994$ N/mm$^2$ compression
Minimum stress in concrete  $9.7987$ N/mm$^2$ compression
Max.stress in steel near top  $221.8$ N/mm$^2$ compression
Max.stress in steel near bottom  $150.1$ N/mm$^2$ compression
Location: Typical circular column, tensile and compressive stress

Modular-ratio theory: design of reinforcement for circular column

Calculations for reinforcement are in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of (mr-1) is considered where the steel is in compression). Min. steel = six bars arranged symmetrically.

Diameter of column  D=600 mm
Depth to centre of steel  d'=56 mm
Design axial load  N=1000 kN
Design moment  M=250 kNm
Modular ratio  mr=15
Limiting concrete stress  fc=15 N/mm²
Limiting steel stress  fs=250 N/mm²
Characteristic steel strength  fyk=500 N/mm²
Main steel diameter  dia=12 mm
Number of bars to be provided  NumBar=37

Tension and compression developed across section

Depth to neutral axis  316.65 mm
Maximum stress in concrete  14.99 N/mm² compression
Theoretical min.concrete stress  13.414 N/mm² tension
Max.stress in steel near top  185.09 N/mm² compression
Max.stress in steel near bottom  161.44 N/mm² tension

Axial-load and moment resistance of section as designed

Axial resistance due to concrete:
\[ N_c = f_1/2*D*D/(1-COS(BE))*(SIN(BE)^3/3-BE/2*COS(BE)+COS(BE)*SIN(2*BE)/4) = 968131 \text{ N} \]

Axial resistance due to compression reinforcement:
\[ N_{s1} = f_1*(mr-1)*R_0/PI*A_{prov}/(R-R_0*COS(AL))*(AL/2-SIN(2*AL)/4) = 239151 \text{ N} \]

Axial resistance due to tension reinforcement:
\[ N_{s2} = f_1*mr*R_0/PI*A_{prov}/(R-R_0*COS(AL))*(SIN(AL)+(PI-AL)*COS(AL)) = 206717 \text{ N} \]

Total axial resistance provided  \( N_{res} = (N_c+N_{s1}-N_{s2})/1E3 = 1000.6 \text{ kN} \)

Resistance moment due to concrete:
\[ M_c = f_1/4*D^3/(1-COS(BE))*(BE/8-SIN(4*BE)/32-COS(BE)*SIN(BE)^3/3) = 164.86E6 \text{ Nmm} \]

Resistance moment due to compression reinforcement:
\[ M_{s1} = f_1*(mr-1)*R_0*R_0/PI*A_{prov}/(R-R_0*COS(AL))*(AL/2-SIN(2*AL)/4) = 44.892E6 \text{ Nmm} \]
Resistance moment due to tension reinforcement:

\[ Ms_2 = f_1 \times m_r \times R_0 \times R_0 / \pi \times A_{P_{prov}} / (R - R_0 \times \cos(\alpha)) \times (\pi / 2 - \alpha / 2 + \sin(2 \times \alpha) / 4) \]

\[ = 40.413 \times 10^6 \text{ Nmm} \]

Total resistance moment provided \( M_{res} = (M_c + M_s_1 + M_s_2) / 10^6 = 250.16 \text{ kNm} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load</td>
<td>Axial resistance provided</td>
</tr>
<tr>
<td>1000 kN</td>
<td>1000.6 kN</td>
</tr>
<tr>
<td>Design moment</td>
<td>Resistance moment provided</td>
</tr>
<tr>
<td>250 kNm</td>
<td>250.16 kNm</td>
</tr>
</tbody>
</table>
Location: Small-diameter circular column subjected to heavy load

Modular-ratio theory: design of reinforcement for circular column

Calculations for reinforcement are in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of \( mr-1 \) is considered where the steel is in compression). Min. steel = six bars arranged symmetrically.

Diameter of column: \( D=300 \text{ mm} \)
Depth to centre of steel: \( d'=48 \text{ mm} \)
Design axial load: \( N=375 \text{ kN} \)
Design moment: \( M=45 \text{ kNm} \)
Modular ratio: \( mr=15 \)
Limiting concrete stress: \( f_{c}=15 \text{ N/mm}^2 \)
Limiting steel stress: \( f_{s}=250 \text{ N/mm}^2 \)
Characteristic steel strength: \( f_{yk}=500 \text{ N/mm}^2 \)
Main steel diameter: \( dia=20 \text{ mm} \)
Number of bars to be provided: \( num\text{Bar}=12 \)

Depth to neutral axis: \( 174.94 \text{ mm} \)
Maximum stress in concrete: \( 14.701 \text{ N/mm}^2 \) compression
Theoretical min. concrete stress: \( 10.51 \text{ N/mm}^2 \) tension
Max. stress in steel near top: \( 160.01 \text{ N/mm}^2 \) compression
Max. stress in steel near bottom: \( 97.141 \text{ N/mm}^2 \) tension

Axial-load and moment resistance of section as designed

Axial resistance due to concrete:
\[
N_c = f_{1} \times \frac{D^2}{4} \times \frac{1}{1 - \cos (BE)} \times (\sin (BE))^3 - \frac{BE}{3} - \frac{2 \cos (BE) + \cos (BE) \times \sin (2 \times BE)}{3} = 276479 \text{ N}
\]
Axial resistance due to compression reinforcement:
\[
N_{s1} = f_{1} \times (mr-1) \times \frac{R_0 \times \text{AProv}}{(R + R_0 \times \cos (AL))} \times \left( \sin (AL) - AL \times \cos (AL) \right) = 207774 \text{ N}
\]
Axial resistance due to tension reinforcement:
\[
N_{s2} = f_{1} \times mr \times R_0 \times \text{AProv} \times \left( R - R_0 \times \cos (AL) \right) \times \left( \sin (AL) + (\pi - AL) \times \cos (AL) \right) = 101698 \text{ N}
\]
Total axial resistance provided:
\[
N_{\text{res}} = (N_c + N_{s1} - N_{s2}) / 10^3 = 382.56 \text{ kN}
\]

Resistance moment due to concrete:
\[
M_c = f_{1} \times \frac{D^3}{8} \times \left( 1 - \cos (BE) \right) \times \left( \sin (BE) / 8 - \sin (4 \times BE) / 32 - \cos (BE) \times \sin (BE) / 3 \right) = 21.791 \text{ kNm}
\]
Resistance moment due to compression reinforcement:
\[
M_{s1} = f_{1} \times (mr-1) \times R_0 \times \text{AProv} \times \left( R - R_0 \times \cos (AL) \right) \times \left( AL / 2 - \sin (2 \times AL) / 4 \right) = 15.398 \text{ kNm}
\]
Resistance moment due to tension reinforcement:
\[ Ms2 = f1 \times mr \times R0^2 \times \frac{R0}{\pi} \times AProv \times (R - R0 \times \cos(AL)) \times (\pi/2 - AL/2 + \sin(2\times AL)/4) \]
\[ = 8.7252E6 \text{ Nmm} \]

Total resistance moment provided: \[ Mres = (M_r + M_s1 + M_s2) / 1E6 = 45.915 \text{ kNm} \]

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load</td>
<td>375 kN</td>
</tr>
<tr>
<td>Axial resistance provided</td>
<td>382.56 kN</td>
</tr>
<tr>
<td>Design moment</td>
<td>45 kNm</td>
</tr>
<tr>
<td>Resistance moment provided</td>
<td>45.915 kNm</td>
</tr>
</tbody>
</table>

NOTE: The percentage of steel provided is 5.33\% which exceeds the 4\% maximum limit. However this is acceptable provided that the concrete can be placed and compacted sufficiently.
Location: Massive lightly-loaded column reinforced with mild steel

Modular-ratio theory: design of reinforcement for circular column

Calculations for reinforcement are in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of (mr-1) is considered where the steel is in compression). Min.steel = six bars arranged symmetrically.

Diameter of column \( D = 1500 \text{ mm} \)
Depth to centre of steel \( d' = 60 \text{ mm} \)
Design axial load \( N = 24000 \text{ kN} \)
Design moment \( M = 1000 \text{ kNm} \)
Modular ratio \( m_r = 15 \)
Limiting concrete stress \( f_c = 15 \text{ N/mm}^2 \)
Limiting steel stress \( f_s = 180 \text{ N/mm}^2 \)
Characteristic steel strength \( f_y = 500 \text{ N/mm}^2 \)
Main steel diameter \( d_{ia} = 16 \text{ mm} \)
Number of bars to be provided \( \text{NumBar} = 60 \)

**Compression developed throughout section**

Maximum stress in concrete \( 14.994 \text{ N/mm}^2 \) compression
Minimum stress in concrete \( 9.7987 \text{ N/mm}^2 \) compression
Max.stress in steel near top \( 221.8 \text{ N/mm}^2 \) compression
Max.stress in steel near bottom \( 150.1 \text{ N/mm}^2 \) compression
Modular-ratio theory: calculation of stresses in circular column

Location: Typical circular column

Calculations of stresses is in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of \( \text{mr-1} \) is considered where the steel is in compression). Min.steel = six bars arranged symmetrically.

Modular ratio \( \text{mr}=15 \)
Diameter of column \( D=600 \text{ mm} \)
Depth to centre of steel \( d'=56 \text{ mm} \)
Axial load \( N=1000 \text{ kN} \)
Applied bending moment \( M=200 \text{ kNm} \)

Tension and compression developed across section

Depth to neutral axis \( 324.35 \text{ mm} \)
Maximum stress in concrete \( 14.651 \text{ N/mm}^2 \) compression
Theoretical min.concrete stress \( 12.451 \text{ N/mm}^2 \) tension
Max.stress in steel near bottom \( 148.82 \text{ N/mm}^2 \) tension
Max.stress in steel near top \( 181.82 \text{ N/mm}^2 \) compression
Modular-ratio theory: calculation of stresses in circular column

Location: Circular column subjected to compression throughout

Calculations of stresses is in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of \((mr-1)\) is considered where the steel is in compression). Min.steel = six bars arranged symmetrically.

Modular ratio \(mr=15\)
Diameter of column \(D=600\) mm
Depth to centre of steel \(d'=56\) mm
Axial load \(N=1000\) kN
Applied bending moment \(M=50\) kNm

Compression developed throughout section

- Maximum stress in concrete \(5.3336\) N/mm\(^2\) compression
- Minimum stress in concrete \(1.1362\) N/mm\(^2\) compression
- Maximum stress in reinforcement \(74.127\) N/mm\(^2\) compression
- Minimum stress in reinforcement \(22.919\) N/mm\(^2\) compression
Modular-ratio theory: calculation of stresses in circular column

Location: Circular column subjected to tension throughout

Calculations of stresses is in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of \(mr-1\) is considered where the steel is in compression). Min.steel = six bars arranged symmetrically.

Modular ratio \(mr=15\)
Diameter of column \(D=600\) mm
Depth to centre of steel \(d'=56\) mm
Axial load \(N=-1000\) kN
Applied bending moment \(M=50\) kNm

Tension developed throughout section

- Theoretical max.concrete stress \(13.456\) N/mm\(^2\) tension
- Theoretical min.concrete stress \(57.278\) N/mm\(^2\) tension
- Max.stress in steel near top \(263.18\) N/mm\(^2\) tension
- Max.stress in steel near bottom \(797.82\) N/mm\(^2\) tension
Location: Typical circular column

Modular-ratio theory: calculation of stresses in circular column

Calculations of stresses is in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of (mr-1) is considered where the steel is in compression). Min.steel = six bars arranged symmetrically.

Diameter of column       D=600 mm
Depth to centre of steel  d'=56 mm
Design axial load        N=1000 kN
Design moment (positive) M=200 kNm
Modular ratio            mr=15
Characteristic steel strength f_yk=500 N/mm²

Tension and compression developed across section

Depth to neutral axis     324.35 mm
Maximum stress in concrete 14.651 N/mm² compression
Theoretical min.concrete stress 12.451 N/mm² tension
Max.stress in steel near bottom 148.82 N/mm² tension
Max.stress in steel near top 181.82 N/mm² compression
Location: Circular column subjected to compression throughout

Modular-ratio theory: calculation of stresses in circular column

Calculations of stresses is in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of (mr-1) is considered where the steel is in compression). Min. steel = six bars arranged symmetrically.

Diameter of column D=600 mm
Depth to centre of steel d'=56 mm
Design axial load N=1000 kN
Design moment (positive) M=50 kNm
Modular ratio mr=15
Characteristic steel strength fyk=500 N/mm²

Compression developed throughout section

Maximum stress in concrete 5.3336 N/mm² compression
Minimum stress in concrete 1.1362 N/mm² compression
Maximum stress in reinforcement 74.127 N/mm² compression
Minimum stress in reinforcement 22.919 N/mm² compression
**Location:** Circular column subjected to tension throughout

**Modular-ratio theory: calculation of stresses in circular column**

![Diagram](image)

Calculations of stresses is in accordance with normal modular-ratio theory (i.e. an 'effective' modular ratio of (mr-1) is considered where the steel is in compression). Min.steel = six bars arranged symmetrically.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of column</td>
<td>D=600 mm</td>
</tr>
<tr>
<td>Depth to centre of steel</td>
<td>d'=56 mm</td>
</tr>
<tr>
<td>Design axial load</td>
<td>N=-1000 kN</td>
</tr>
<tr>
<td>Design moment (positive)</td>
<td>M=50 kNm</td>
</tr>
<tr>
<td>Modular ratio</td>
<td>mr=15</td>
</tr>
<tr>
<td>Characteristic steel strength</td>
<td>f_yk=500 N/mm²</td>
</tr>
</tbody>
</table>

**Tension developed throughout section**

- Theoretical max.concrete stress: 13.456 N/mm² tension
- Theoretical min.concrete stress: 57.278 N/mm² tension
- Max.stress in steel near top: 263.18 N/mm² tension
- Max.stress in steel near bottom: 797.82 N/mm² tension
Location: Ex1 - Staircase between Beams C1-C2 and D1-D2

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

**Reinforced concrete staircase spanning between supports at the end of the flight. No landings.**

Design in accordance with BS 8110:Part 1 "Structural use of concrete".

**Number of treads** \( \text{TrdNo}=10 \)

**Total length of treads on flight** \( \text{Trdlen}=2500 \text{ mm} \)

**Finished riser height** \( \text{frish}=165 \text{ mm} \)

**Staircase waist thickness** \( h=200 \text{ mm} \)

**Staircase flight width** \( b=1100 \text{ mm} \)

**Depth of finish on tread** \( f_{\text{trd}}=25 \text{ mm} \)

**Depth of finish on riser** \( f_{\text{ris}}=15 \text{ mm} \)

**Depth of finish on flight soffit** \( f_{\text{soft}}=0 \text{ mm} \)

**Density of concrete** \( \text{Gamcon}=24 \text{ kN/m}^3 \)

**Density of finishes** \( \text{Gamfin}=24 \text{ kN/m}^3 \)

**Imposed load** \( q_k=5 \text{ kN/m} \)

**Simply supported rectangular reinforced concrete staircase**

**W1**

<table>
<thead>
<tr>
<th>Load Gk</th>
<th>Qk kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1 9.4257</td>
<td>5.5</td>
</tr>
</tbody>
</table>

**Maximum span bending moment** \( 17.184 \text{ kNm} \)

**End shears**

- Shear force at left hand end \( 27.495 \text{ kN} \)
- Shear force at right hand end \( 27.495 \text{ kN} \)
- Unfactored dead shear at LHE \( 11.782 \text{ kN} \)
- Unfactored imposed shear at LHE \( 6.875 \text{ kN} \)
- Unfactored dead shear at RHE \( 11.782 \text{ kN} \)
- Unfactored imposed shear at RHE \( 6.875 \text{ kN} \)
Characteristic concrete strength \( f_{cu} = 25 \text{ N/mm}^2 \)
Characteristic steel strength \( f_y = 250 \text{ N/mm}^2 \)
Mild steel reinforcement.
Diameter of tension bars \( \text{dia} = 20 \text{ mm} \)
Designated exposure class is \( \text{XC1} \)
Specified fixing tolerance \( \text{tol} = 10 \text{ mm} \)
Nominal concrete cover \( \text{cover} = 25 \text{ mm} \)
Effective depth of section \( d = 155 \text{ mm} \)
Chosen spacing of tension bars \( \text{pchTA} = 320 \text{ mm} \)
Diameter of distribution bars \( \text{diamn} = 12 \text{ mm} \)
Spacing of distribution bars \( \text{pchDA} = 220 \text{ mm} \)

**TENSION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>Characteristic steel strength</th>
<th>Diameter of bars</th>
<th>Number of bars</th>
<th>Spacing of bars</th>
<th>Effective depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>250 \text{ N/mm}^2</td>
<td>( f_y = 250 \text{ N/mm}^2 )</td>
<td>20 mm</td>
<td>4</td>
<td>320 mm</td>
<td>155 mm</td>
</tr>
</tbody>
</table>

**REINFORCEMENT**

<table>
<thead>
<tr>
<th>Area of steel required</th>
<th>Area of steel provided</th>
<th>Percentage provided</th>
<th>Weight of steel provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>536.83 \text{ mm}^2</td>
<td>1256 \text{ mm}^2</td>
<td>0.57091 %</td>
<td>9.8596 kg/m</td>
</tr>
</tbody>
</table>

**SUMMARY**

Check on span/effective-depth ratio

The staircase is assumed to be a simply-supported slab.
Basic ratio for simp.-sup.slab \( \text{bs}'d = 20 \) (see Table 3.9)
From Table 3.10 of Code, with \( f_s = 71.235 \text{ N/mm}^2 \) and \( M/bd^2 = 0.65025 \)
Mod.factor for tension steel \( \text{modf}1 = 2 \)
Maximum permissible span/effective-depth ratio \( ps'd = \text{bs}'d \times \text{modf1} = 40 \)
Actual span/effective-depth ratio as \( d = 1000 \times \text{span}/d = 16.129 \)
Since this does not exceed 46, this is Acceptable.

Shear in longitudinal zone of solid slab of given width

Since distance from support varies throughout zone considered, enhancement of shear resistance due to proximity to support is not applicable.

The calculations for shear in the zone of solid slab are in accordance with Clauses 3.5.5 of BS8110 throughout.
In the case of a uniform load, the design section to be considered need not be less than a minimum distance from support equal to effective depth.
Dia of tension bars @ shear sect  dias=16
No of tension bars @ shear sect  nbars=4

Effective depth of section  ds=157 mm

Since design shear stress of 0.15921 N/mm² is less than critical shear stress of 0.62089 N/mm², no special shear reinforcement is required.
Location: Ex2 - Type 1 Stair

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

Reinforced concrete staircase spanning between supports at the ends of the upper and lower landings. Design in accordance with BS 8110:Part 1 "Structural use of concrete".

Number of treads: \( \text{TrdNo}=12 \)
Total length of treads on flight: \( \text{Trdlen}=2500 \text{ mm} \)
Finished riser height: \( \text{frish}=175 \text{ mm} \)
Staircase waist thickness: \( h=250 \text{ mm} \)
Staircase flight width: \( b=1100 \text{ mm} \)
Depth of finish on tread: \( \text{ftrd}=25 \text{ mm} \)
Depth of finish on riser: \( \text{fris}=15 \text{ mm} \)
Depth of finish on flight soffit: \( fsoft'=10 \text{ mm} \)
Upper landing length: \( \text{Ulan}=1200 \text{ mm} \)
Upper landing thickness: \( \text{Ulthk}=200 \text{ mm} \)
Depth of finish on upper landing: \( \text{fupp}=25 \text{ mm} \)
Depth upper landing soffit finish: \( \text{supp}=10 \text{ mm} \)
Lower landing length: \( \text{Llan}=1200 \text{ mm} \)
Lower landing thickness: \( \text{Llthk}=200 \text{ mm} \)
Depth of finish on lower landing: \( \text{flow}=25 \text{ mm} \)
Depth lower landing soffit finish: \( \text{slow}=10 \text{ mm} \)
Density of concrete: \( \text{Gamcon}=24 \text{ kN/m}^3 \)
Density of finishes: \( \text{Gamfin}=24 \text{ kN/m}^3 \)
Density of soffit finishes: \( \text{Gamsfin}=20 \text{ kN/m}^3 \)
Imposed load: \( qk=5 \text{ kN/m}^2 \)

Simply supported rectangular reinforced concrete staircase

<table>
<thead>
<tr>
<th>W1</th>
<th>W2</th>
<th>W3</th>
<th>Load</th>
<th>Gk kN/m</th>
<th>Qk kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>upper</td>
<td>6.204</td>
<td>6.204</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>lower</td>
<td>12.209</td>
<td>6.204</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Maximum span bending moment 71.657 kNm

### End shears

- Shear force at left hand end 53.348 kN
- Shear force at right hand end 53.348 kN
- Unfactored dead shear at LHE 22.706 kN
- Unfactored imposed shear at LHE 13.475 kN
- Unfactored dead shear at RHE 22.706 kN
- Unfactored imposed shear at RHE 13.475 kN

Characteristic concrete strength $f_{cu}=30$ N/mm$^2$

Characteristic steel strength $f_y=500$ N/mm$^2$

High-yield steel reinforcement.

- Diameter of tension bars $d_{ia}=20$ mm
- Designated exposure class is XC1
- Specified fixing tolerance $t_{ol}=10$ mm
- Nominal concrete cover $c_{over}=35$ mm
- Effective depth of section $d=205$ mm
- Chosen spacing of tension bars $p_{chTA}=320$ mm

- Diameter of distribution bars $d_{iamn}=12$ mm
- Spacing of distribution bars $p_{chDA}=340$ mm

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Characteristic strength 500 N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 20 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Number of bars 4</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars 320 mm</td>
</tr>
<tr>
<td></td>
<td>Effective depth 205 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required 856.32 mm$^2$</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided 1256 mm$^2$</td>
</tr>
<tr>
<td></td>
<td>Percentage provided 0.45673%</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided 9.8596 kg/m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DISTRIBUTION</th>
<th>Characteristic strength 500 N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Diameter of bars 12 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing of bars 340 mm</td>
</tr>
<tr>
<td></td>
<td>Depth to bar centres 189 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required 325 mm$^2$/m</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided 332 mm$^2$/m</td>
</tr>
<tr>
<td></td>
<td>Percentage provided 0.1328%</td>
</tr>
<tr>
<td></td>
<td>Weight of steel provided 2.6062 kg/m</td>
</tr>
</tbody>
</table>

**Check on span/effective-depth ratio**

The staircase is assumed to be a simply-supported slab.

- Basic ratio for simp.-sup.slab $bs'd=20$ (see Table 3.9)
- From Table 3.10 of Code, with $fs=227.26$ N/mm$^2$ and $M/bd^2=1.5501$
- Mod.factor for tension steel $modf_1=1.4024$
- Maximum permissible $span/effective-depth$ ratio $ps'd=bs'd*modf_1=28.047$
- Actual span/effective-depth ratio $as'd=1000*span/d=23.902$
- Since this does not exceed 28.047, this is Acceptable.
Shear in longitudinal zone of solid slab of given width

Since distance from support varies throughout zone considered, enhancement of shear resistance due to proximity to support is not applicable.

The calculations for
shear in the zone of solid slab are in accordance with Clauses 3.5.5 of BS8110 throughout.

In the case of a uniform load, the design section to be considered need not be less than a minimum distance from support equal to effective depth.

Dia of tension bars @ shear sect dias=16
No of tension bars @ shear sect nbars=4

Effective depth of section ds=157 mm

Since design shear stress of 0.30891 N/mm² is less than critical shear stress of 0.61525 N/mm², no special shear reinforcement is required.
Location: Ex3 - Type 6 Stair

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Open well staircase supported on structure at floor level. Each half landing supported by walls on two sides.

Stair flight designed in accordance with BS8110:Part 1 with 1993 amendments. "Structural use of concrete"

Number of treads: TrdNo=8
Total length of treads on flight: Trdlen=2000 mm
Finished riser height: frish=170 mm
Staircase waist thickness: h=200 mm
Staircase flight width: b=1200 mm
Depth of finish on tread: ftrd=20 mm
Depth of finish on riser: fris=20 mm
Depth of finish on flight soffit: fsoft'=10 mm
Lower landing length: Llan=1250 mm
Lower landing thickness: Llthk=250 mm
Depth of finish on lower landing: flow=20 mm
Depth lower landing soffit finish: slow=10 mm
Upper landing length: Ulan=1250 mm
Upper landing thickness: Ulthk=250 mm
Depth of finish on upper landing: fupp=20 mm
Depth upper landing soffit finish: supp=10 mm
Density of concrete: Gamcon=24 kN/m³
Density of finishes: Gamfin=24 kN/m³
Density of soffit finishes: Gamsfin=20 kN/m³
Imposed load: qk=4 kN/m²

Simply supported rectangular reinforced concrete staircase

<table>
<thead>
<tr>
<th></th>
<th>Load</th>
<th>Gk</th>
<th>Qk</th>
</tr>
</thead>
<tbody>
<tr>
<td>upper</td>
<td>W1</td>
<td>4.032</td>
<td>2.4</td>
</tr>
<tr>
<td>half</td>
<td>W2</td>
<td>10.671</td>
<td>4.8</td>
</tr>
<tr>
<td>landing</td>
<td>W3</td>
<td>8.064</td>
<td>4.8</td>
</tr>
<tr>
<td>lower</td>
<td>W1</td>
<td>4.032</td>
<td>2.4</td>
</tr>
<tr>
<td>floor</td>
<td>W2</td>
<td>10.671</td>
<td>4.8</td>
</tr>
<tr>
<td>landing</td>
<td>W3</td>
<td>8.064</td>
<td>4.8</td>
</tr>
</tbody>
</table>
Maximum span bending moment       50.699 kNm

End shears

Shear force at left hand end      36.122 kN
Shear force at right hand end     44.685 kN
Unfactored dead shear at LHE       16.411 kN
Unfactored imposed shear at LHE    8.2167 kN
Unfactored dead shear at RHE       20.051 kN
Unfactored imposed shear at RHE    10.383 kN

Characteristic concrete strength  fcu=30 N/mm²
Characteristic steel strength     fy=500 N/mm²
High-yield steel reinforcement.
Diameter of tension bars           dia=20 mm
Nominal concrete cover             cover=25 mm
Effective depth of section         d=165 mm
Chosen spacing of tension bars     pchTA=375 mm

Diameter of distribution bars     diamn=12 mm
Spacing of distribution bars       pchDA=400 mm

TENSION                       Characteristic strength  500 N/mm²
REINFORCEMENT                 Diameter of bars         20 mm
SUMMARY                       Number of bars           4
                               Spacing of bars         375 mm
                               Effective depth         165 mm
                               Area of steel required  752.8 mm²
                               Area of steel provided 1256 mm²
                               Percentage provided    0.52333 %
                               Weight of steel provided 9.8596 kg/m

DISTRIBUTION                  Characteristic strength  500 N/mm²
REINFORCEMENT                 Diameter of bars         12 mm
SUMMARY                       Spacing of bars         400 mm
                               Depth to bar centres  149 mm
                               Area of steel required 260 mm²/m
                               Area of steel provided 282 mm²/m
                               Percentage provided    0.141 %
                               Weight of steel provided 2.2137 kg/m

Check on span/effective-depth ratio

The staircase is assumed to be a simply-supported slab.
Basic ratio for simp.-sup.slab       bs'd=20 (see Table 3.9)
From Table 3.10 of Code, with fs=199.79 N/mm² and M/bd²=1.5519
Mod.factor for tension steel       modf1=1.4941
Maximum permissible
span/effective-depth ratio          ps'd=bs'd*modf1=29.882
Actual span/effective-depth ratio  as'd=1000*span/d=27.273
Since this does not exceed 29.882, this is Acceptable.

Shear in longitudinal zone of solid slab of given width

Since distance from support varies throughout zone considered, enhancement of shear resistance due to proximity to support is not applicable.
The calculations for shear in the zone of solid slab are in accordance with Clauses 3.5.5 of BS8110 throughout. In the case of a uniform load, the design section to be considered need not be less than a minimum distance from support equal to effective depth. 

Dia of tension bars @ shear sect dias=16
No of tension bars @ shear sect nbars=4
Effective depth of section ds=217 mm

Since design shear stress of 0.13872 N/mm² is less than critical shear stress of 0.56646 N/mm², no special shear reinforcement is required.
Location: Exl - Staircase between Beams C1-C2 and D1-D2

Reinforced concrete staircase spanning between supports at the end of the flight. No landings. Design in accordance with EC2:Part 1.

Number of treads TrdNo=10
Total length of treads on flight Trdlen=2500 mm
Finished riser height frish=165 mm
Staircase waist thickness h=200 mm
Staircase flight width b=1100 mm
Depth of finish on tread ftrd=25 mm
Depth of finish on riser fris=15 mm
Depth of finish on flight soffit fsoft'=0 mm
Density of concrete Gamcon=25 kN/m³
Density of finishes Gamfin=24 kN/m³
Variable load qk=5 kN/m²

Simply supported rectangular reinforced concrete staircase

<table>
<thead>
<tr>
<th>W1</th>
<th>Load</th>
<th>Gk kN/m</th>
<th>Qk kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>9.78</td>
<td>5.5</td>
<td></td>
</tr>
</tbody>
</table>

2500 mm

Permanent load factor gamG=1.25
Variable load factor gamQ=1.5

Maximum span bending moment M=15.996 kNm (ultimate)

End shears

Shear force at left hand end 25.594 kN (ultimate)
Shear force at right hand end 25.594 kN (ultimate)
Unfactored permanent shear at LHE 12.225 kN
Unfactored variable shear at LHE 6.875 kN
Unfactored permanent shear at RHE 12.225 kN
Unfactored variable shear at RHE  6.875 kN

High-yield steel reinforcement.
Diameter of tension bars dia=12 mm
Effective depth of section d=159 mm

| DESIGN | Overall depth | 200 mm |
| SUMMARY | Effective depth | 159 mm |
| FLEXURE | Parameter K | 0.023 |
| | Parameter K' | 0.1684 |
| | Lever arm ratio z/d | 0.95 |
| TENSION REINFORCEMENT | Steel area required | 243.6 mm² |
| | Steel area provided | 565.5 mm² |
| | Diameter of bars | 12 mm |
| | Number of bars | 5 |
| | Steel percentage | 0.1393 % |
| | Minimum area of steel | 233.3 mm² |
| | Maximum area of steel | 8800 mm² |
| | Distribution steel | 233.3 mm² |

| TENSION REINFORCEMENT | Characteristic strength | 500 N/mm² |
| SUMMARY | Diameter of bars | 12 mm |
| | Number of bars | 5 |
| | Spacing of bars | 255 mm |
| | Effective depth | 159 mm |
| | Area of steel required | 243.6 mm² |
| | Area of steel provided | 565.5 mm² |
| | Percentage provided | 0.257 % |

| DISTRIBUTION | Characteristic strength | 500 N/mm² |
| REINFORCEMENT | Max spacing of bars | 400 mm |
| SUMMARY | Min area required | 233 mm² |

Deflection check
Actual span to depth ratio 1'd=L*1000/d=15.72
Allowable span/depth ratio 1'da=k*N*F1*F2*F3=157
Absolute value of span/depth 1'da=40*k=40

| DESIGN | Actual 1/d ratio | 15.72 |
| SUMMARY | Basic 1/d ratio | 104.6 |
| DEFLECTION | Span factor | 1 |
| | Flange beam factor F1 | 1 |
| | Long spans factor F2 | 1 |
| | Steel stress factor F3 | 1.5 |
| | Allowable 1/d ratio | 46 |

NOTE: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for lightly stressed concrete (C30/37 & ρ=0.5%) as follows:
Simply supported spans 23
Continuous spans 30
Shear in longitudinal zone of solid slab of given width

Location for shear calculation: Stair 22
Shear force due to ultimate load \( V_{Ed} = 25.59 \text{ kN} \)
Term for shear resistance \( v_{Rdc} = (\rho_1 f_{ck})^{1/3} = 1.593 \)
Shear resistance \( V_{Rdc} = C_{Rdc} ks v_{Rdc} bw d / 1000 = 66.86 \text{ kN} \)
Minimum stress \( v_{min} = 0.035 ks^{1.5} f_{ck}^{0.5} = 0.495 \text{ N/mm}^2 \)
Minimum shear resistance \( V_{Rdm} = v_{min} bw d / 1000 = 86.57 \text{ kN} \)
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 86.57 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Design shear force</th>
<th>25.59 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design shear resistance</td>
<td>86.57 kN</td>
</tr>
</tbody>
</table>
Location: Ex2 - Type 1 Stair

Reinforced concrete staircase spanning between supports at the ends of the upper and lower landings. Design in accordance with EC2:Part 1.

Number of treads  TrdNo=12
Total length of treads on flight  Trdlen=2500 mm
Finished riser height  frish=175 mm
Staircase waist thickness  h=250 mm
Staircase flight width  b=1100 mm
Depth of finish on tread  ftrd=25 mm
Depth of finish on riser  fris=15 mm
Depth of finish on flight soffit  fsoft'=10 mm
Upper landing length  Ulan=1200 mm
Upper landing thickness  Ulthk=200 mm
Depth of finish on upper landing  fupp=25 mm
Depth upper landing soffit finish  supp=10 mm
Lower landing length  Llan=1200 mm
Lower landing thickness  Llthk=200 mm
Depth of finish on lower landing  flow=25 mm
Depth lower landing soffit finish  slow=10 mm
Density of concrete  Gamcon=25 kN/m³
Density of finishes  Gamfin=24 kN/m³
Density of soffit finishes  Gamsfin=20 kN/m³
Variable load  qk=5 kN/m²

Simply supported rectangular reinforced concrete staircase

<table>
<thead>
<tr>
<th>W1</th>
<th>W2</th>
<th>W3</th>
<th>Load</th>
<th>Gk kN/m</th>
<th>Qk kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>upper landing</td>
<td>lower</td>
<td>W1</td>
<td>6.424</td>
<td>5.5</td>
<td></td>
</tr>
<tr>
<td>W2</td>
<td>12.665</td>
<td>5.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W3</td>
<td>6.424</td>
<td>5.5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| Permanent load factor  gamG=1.25
Variable load factor  gamQ=1.5
**Maximum span bending moment**  
\[ M = 66.657 \text{ kNm (ultimate)} \]

**End shears**

- Shear force at left hand end  \( 49.638 \text{ kN (ultimate)} \)
- Shear force at right hand end  \( 49.638 \text{ kN (ultimate)} \)
- Unfactored permanent shear at LHE  \( 23.54 \text{ kN} \)
- Unfactored variable shear at LHE  \( 13.475 \text{ kN} \)
- Unfactored permanent shear at RHE  \( 23.54 \text{ kN} \)
- Unfactored variable shear at RHE  \( 13.475 \text{ kN} \)

**High-yield steel reinforcement.**

- Diameter of tension bars  \( \text{dia}=16 \text{ mm} \)
- Effective depth of section  \( d=207 \text{ mm} \)

*DESIGN*

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall depth</td>
<td>250 mm</td>
</tr>
<tr>
<td>Effective depth</td>
<td>207 mm</td>
</tr>
</tbody>
</table>

*SUMMARY*

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective depth</td>
<td>207 mm</td>
</tr>
</tbody>
</table>

*FLEXURE*

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter K</td>
<td>0.0471</td>
</tr>
<tr>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td>Lever arm ratio z/d</td>
<td>0.95</td>
</tr>
</tbody>
</table>

*TENSION REINFORCEMENT*

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel area required</td>
<td>779.6 mm²</td>
</tr>
<tr>
<td>Steel area provided</td>
<td>804.2 mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>4</td>
</tr>
<tr>
<td>Steel percentage</td>
<td>0.3424 %</td>
</tr>
<tr>
<td>Minimum area of steel</td>
<td>343 mm²</td>
</tr>
<tr>
<td>Maximum area of steel</td>
<td>11000 mm²</td>
</tr>
<tr>
<td>Distribution steel</td>
<td>343 mm²</td>
</tr>
</tbody>
</table>

**Factor \( \psi_2 \) for variable load**  \( \psi_2=0.3 \)

**SLS stress in reinforcement**  \( \sigma_{SLS} = \frac{500}{1.15} \times \frac{(As/Aspr) \times \text{ratio1}}{\delta} = 234.2 \text{ N/mm}^2 \)

**Design crack width**  \( w_k = 0.3 \text{ mm} \)

**Max allowable SLS stress**  \( \sigma_{SLS_{max}} = \text{TABLE 7.2 for dia=16} = 240 \text{ N/mm}^2 \)

The bar size 16 mm used complies with Table 7.2N of EC2 Part 1:2004 hence satisfactory.

**TENSION REINFORCEMENT**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>4</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>340 mm</td>
</tr>
<tr>
<td>Effective depth</td>
<td>207 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>779.6 mm²</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>804.2 mm²</td>
</tr>
<tr>
<td>Percentage provided</td>
<td>0.2925 %</td>
</tr>
</tbody>
</table>

**DISTRIBUTION REINFORCEMENT**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Max spacing of bars</td>
<td>400 mm</td>
</tr>
<tr>
<td>Min area required</td>
<td>342 mm²</td>
</tr>
</tbody>
</table>

**Deflection check**

- Actual span to depth ratio  \( l'd/L\times1000/d=23.67 \)
- Allowable span/depth ratio  \( l'da=k\timesN\timesF1\timesF2\timesF3=33.3 \)
### DESIGN
- Actual l/d ratio: 23.67

### SUMMARY
- Basic l/d ratio: 32.28

### DEFLECTION
- Span factor: 1
- Flange beam factor F1: 1
- Long spans factor F2: 1
- Steel stress factor F3: 1.032
- Allowable l/d ratio: 33.3

**NOTE:** Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for lightly stressed concrete (C30/37 & ρ=0.5%) as follows:

- Simply supported spans: 20
- End spans: 26
- Interior spans: 30

#### Shear in longitudinal zone of solid slab of given width

**Location for shear calculation:**
- Shear force due to ultimate load: $V_{Ed} = 49.64$ kN
- Term for shear resistance: $v_{Rdc} = (\rho_1 \times f_{ck})^{(1/3)} = 1.743$
- Shear resistance: $V_{Rdc} = C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d / 1000 = 94.46$ kN
- Minimum stress: $v_{min} = 0.035 \times k_s \times (1.5 \times f_{ck})^{0.5} = 0.5353$ N/mm²
- Minimum shear resistance: $V_{Rdm} = v_{min} \times b_w \times d / 1000 = 121.9$ kN
- Modified shear resistance: $V_{Rdc} = V_{Rdm} = 121.9$ kN

#### SHEAR SUMMARY
- Design shear force: 49.64 kN
- Design shear resistance: 121.9 kN
Location: Ex3 - Type 6 Stair

Stair flight design
Open well staircase supported on structure at floor level.
Each half landing is supported by walls on two sides.
Design in accordance with EC2:Part 1.

Number of treads: TrdNo=8
Total length of treads on flight: Trdlen=2000 mm
Finished riser height: frish=170 mm
Staircase waist thickness: h=200 mm
Staircase flight width: b=1200 mm
Depth of finish on tread: ftrd=20 mm
Depth of finish on riser: fris=20 mm
Depth of finish on flight soffit: fsoft'=10 mm
Lower landing length: Llan=1250 mm
Lower landing thickness: Llthk=250 mm
Depth of finish on lower landing: flow=20 mm
Depth lower landing soffit finish: slow=10 mm
Upper landing length: Ulan=1250 mm
Upper landing thickness: Ulthk=250 mm
Depth of finish on upper landing: fupp=20 mm
Depth upper landing soffit finish: supp=10 mm
Density of concrete: Gamcon=25 kN/m³
Density of finishes: Gamfin=24 kN/m³
Density of soffit finishes: Gamsfin=20 kN/m³
Variable load: qk=4 kN/m²

Simply supported rectangular reinforced concrete staircase

<table>
<thead>
<tr>
<th>W1</th>
<th>W2</th>
<th>W3</th>
<th>Load</th>
<th>Gk kN/m</th>
<th>Qk kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>upper</td>
<td></td>
<td>lower</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>half</td>
<td>floor</td>
<td></td>
<td></td>
<td>4.182</td>
<td>2.4</td>
</tr>
<tr>
<td>landing</td>
<td>landing</td>
<td></td>
<td></td>
<td>11.064</td>
<td>4.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>8.364</td>
<td>4.8</td>
</tr>
</tbody>
</table>

Permanent load factor: gamG=1.25
Variable load factor: gamQ=1.5
Maximum span bending moment  \( M = 47.147 \text{ kNm (ultimate)} \)

### End shears

- Shear force at left hand end  \( 33.597 \text{ kN (ultimate)} \)
- Shear force at right hand end  \( 41.566 \text{ kN (ultimate)} \)
- Unfactored permanent shear at LHE  \( 17.018 \text{ kN} \)
- Unfactored variable shear at LHE  \( 8.2167 \text{ kN} \)
- Unfactored permanent shear at RHE  \( 20.793 \text{ kN} \)
- Unfactored variable shear at RHE  \( 10.383 \text{ kN} \)

### High-yield steel reinforcement.
- Diameter of tension bars  \( \text{dia}=16 \text{ mm} \)
- Nominal concrete cover  \( \text{cover}=25 \text{ mm} \)
- Effective depth of section  \( d=167 \text{ mm} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth</th>
<th>200 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth</td>
<td>167 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K</td>
<td>0.047</td>
</tr>
<tr>
<td></td>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio z/d</td>
<td>0.95</td>
</tr>
<tr>
<td>TENSION REINFORCEMENT</td>
<td>Steel area required</td>
<td>683.5 mm²</td>
</tr>
<tr>
<td></td>
<td>Steel area provided</td>
<td>1206 mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Steel percentage</td>
<td>0.3411 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel</td>
<td>301.8 mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel</td>
<td>9600 mm²</td>
</tr>
<tr>
<td></td>
<td>Distribution steel</td>
<td>301.8 mm²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TENSION REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
<td>220 mm</td>
</tr>
<tr>
<td></td>
<td>Effective depth</td>
<td>167 mm</td>
</tr>
<tr>
<td></td>
<td>Area of steel required</td>
<td>683.5 mm²</td>
</tr>
<tr>
<td></td>
<td>Area of steel provided</td>
<td>1206 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentage provided</td>
<td>0.5027 %</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DISTRIBUTION</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCEMENT</td>
<td>Max spacing of bars</td>
<td>400 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Min area required</td>
<td>301 mm²</td>
</tr>
</tbody>
</table>

### Deflection check

- Actual span to depth ratio  \( l'd=\frac{L*1000}{d}=26.95 \)
- Allowable span/depth ratio  \( l'da=k*N*F1*F2*F3=48.69 \)
- Absolute value of span/depth  \( l'da=40*k=40 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Actual 1/d ratio</th>
<th>26.95</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Basic 1/d ratio</td>
<td>32.46</td>
</tr>
<tr>
<td>DEFLECTION</td>
<td>Span factor</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Flange beam factor F1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Long spans factor F2</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Steel stress factor F3</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Allowable 1/d ratio</td>
<td>40</td>
</tr>
</tbody>
</table>

**NOTE:** Care should be taken with the EC2 values as the deflection
equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for lightly stressed concrete (C30/37 & ρ=0.5%) as follows:

- Simply supported spans: 20
- End spans: 26
- Interior spans: 30

Shear in longitudinal zone of solid slab of given width

Location for shear calculation: Stair 2
Shear force due to ultimate load: $V_{Ed} = 33.6$ kN
Term for shear resistance: $v_{Rdc} = (\rho_1 \cdot f_{ck})^{1/3} = 2.082$
Shear resistance: $V_{Rdc} = C_{Rdc} \cdot k_s \cdot v_{Rdc} \cdot b \cdot w \cdot d / 1000 = 100.2$ kN
Minimum stress: $v_{min} = 0.035 \cdot k_s \cdot 1.5 \cdot f_{ck} \cdot 0.5 = 0.5422$ N/mm²
Minimum shear resistance: $V_{Rdm} = v_{min} \cdot b \cdot w \cdot d / 1000 = 108.7$ kN
Modified shear resistance: $V_{Rdc} = V_{Rdm} = 108.7$ kN

**SHEAR SUMMARY**

- Design shear force: 33.6 kN
- Design shear resistance: 108.7 kN
**Location:** Ex4 - Staircase between Beams C1-C2 and D1-D2

Reinforced concrete staircase spanning between supports at the end of the flight. No landings. Design in accordance with EC2:Part 1.

---

**Number of treads**  
TrdNo=10

**Total length of treads on flight**  
Trdlen=2500 mm

**Finished riser height**  
frish=165 mm

**Staircase waist thickness**  
h=200 mm

**Staircase flight width**  
b=1100 mm

**Depth of finish on tread**  
ftrd=25 mm

**Depth of finish on riser**  
fris=15 mm

**Depth of finish on flight soffit**  
fsoft'=0 mm

**Density of concrete**  
Gamcon=25 kN/m³

**Density of finishes**  
Gamfin=24 kN/m³

**Variable load**  
qk=5 kN/m²

---

**Simply supported rectangular reinforced concrete staircase**

<table>
<thead>
<tr>
<th>W1</th>
<th>Load</th>
<th>Gk kN/m</th>
<th>Qk kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>W1</td>
<td>9.78</td>
<td>5.5</td>
</tr>
</tbody>
</table>

2500 mm

---

**Permanent load factor**  
gamG=1.25

**Variable load factor**  
gamQ=1.5

**Maximum span bending moment**  
M=15.996 kNm (ultimate)

---

**End shears**

Shear force at left hand end  
25.594 kN (ultimate)

Shear force at right hand end  
25.594 kN (ultimate)

Unfactored permanent shear at LHE  
12.225 kN

Unfactored variable shear at LHE  
6.875 kN

Unfactored permanent shear at RHE  
12.225 kN
Unfactored variable shear at RHE 6.875 kN

High-yield steel reinforcement.
Diameter of tension bars \( \text{dia}=12 \text{ mm} \)
Effective depth of section \( d=159 \text{ mm} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth</th>
<th>200 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth</td>
<td>159 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K</td>
<td>0.023</td>
</tr>
<tr>
<td></td>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio ( z/d )</td>
<td>0.95</td>
</tr>
<tr>
<td>TENSION REINFORCEMENT</td>
<td>Steel area required</td>
<td>243.6 mm²</td>
</tr>
<tr>
<td></td>
<td>Steel area provided</td>
<td>565.5 mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Steel percentage</td>
<td>0.1393 %</td>
</tr>
<tr>
<td></td>
<td>Minimum area of steel</td>
<td>233.3 mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum area of steel</td>
<td>8800 mm²</td>
</tr>
<tr>
<td></td>
<td>Distribution steel</td>
<td>233.3 mm²</td>
</tr>
</tbody>
</table>

| TENSION REINFORCEMENT | Characteristic strength | 500 N/mm² |
|                       | Diameter of bars | 12 mm |
| SUMMARY               | Number of bars  | 5      |
|                       | Spacing of bars | 255 mm |
|                       | Effective depth | 159 mm |
|                       | Area of steel required | 243.6 mm² |
|                       | Area of steel provided | 565.5 mm² |
|                       | Percentage provided | 0.257 % |

| DISTRIBUTION          | Characteristic strength | 500 N/mm² |
|                       | Max spacing of bars | 400 mm |
| SUMMARY               | Min area required | 233 mm² |

Deflection check

Actual span to depth ratio \( 1'd=L*1000/d=15.72 \)
Factor \( \psi_2 \) for variable load \( \psi_2=0.3 \)
SLS stress in reinforcement \( \sigma_{SLS}=500/1.15*(As/Aspr)*ratio*1/delta=104.5 \text{ N/mm²} \)
Allowable span/depth ratio \( 1'da=k*N*F1*F2*F3=157 \)
Absolute value of span/depth \( 1'da=40^k=40 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Actual 1/d ratio</th>
<th>15.72</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Basic 1/d ratio</td>
<td>104.6</td>
</tr>
<tr>
<td>DEFLECTION</td>
<td>Span factor</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Flange beam factor F1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Long spans factor F2</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Steel stress factor F3</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Allowable 1/d ratio</td>
<td>46</td>
</tr>
</tbody>
</table>

NOTE: Care should be taken with the EC2 values as the deflection equations used give unrealistically high values. National Annex to EC2 Part 1:2004, Table NA.5 gives span/effective depth ratios for lightly stressed concrete \( \text{C30/37} \ & \ p=0.5\% \) as follows:
Simply supported spans 23
Continuous spans 30
Shear in longitudinal zone of solid slab of given width

Location for shear calculation: Stair 22
Shear force due to ultimate load \( V_{Ed}=25.59 \text{ kN} \)
Term for shear resistance \( v_{Rdc}=(\rho_1*f_{ck})^{(1/3)}=1.593 \)
Shear resistance \( v_{Rdc}=C_{Rdc}*k_s*v_{Rdc}*b_w*d/1000=66.86 \text{ kN} \)
Minimum stress \( v_{min}=0.035*k_s^{1.5}*f_{ck}^{0.5}=0.495 \text{ N/mm}^2 \)
Minimum shear resistance \( v_{Rdm}=v_{min}*b_w*d/1000=86.57 \text{ kN} \)
Modified shear resistance \( v_{Rdc}=v_{Rdm}=86.57 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Design shear force</th>
<th>25.59 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design shear resistance</td>
<td>86.57 kN</td>
</tr>
</tbody>
</table>
Location: Typical problem

Design of concrete nibs to BS8110

Design to BS8110(1997), partial safety factor for steel gams=1.15

In addition to providing shear resistance for beam, these links must be sufficiently strong to transfer load on nib to compression zone of beam.

Load acting on nib: load=26.3 kN/m
Dist. from load to CL beam links: av=120 mm
Concrete grade: fcu=40 N/mm²
Steel strength: fy=500 N/mm²
Designated exposure class is XC1
Specified fixing tolerance: tol=10 mm
Concrete cover to nib steel: cover=25 mm
Size of nib bars: diam=8 mm
Chosen nib thickness: h=115 mm
Chosen bar spacing: Sprov=265 mm

SUMMARY

Characteristic strength 500 N/mm²
Diameter of bars 8 mm
Spacing of bars 265 mm
Area required 188.97 mm²
Area provided 189.68 mm²

Check shear resistance

Permissible shear stress: vcp=vc*sdfact=0.94009 N/mm²
Actual shear stress: vca=Nload/(b*d)=0.30581 N/mm²
Shear resistance is satisfactory.
Location: Heavy loading on large nib with mild steel reinforcement

Design of concrete nibs to BS8110

Design to BS8110(1997), partial safety factor for steel gams=1.15

In addition to providing shear resistance for beam, these links must be sufficiently strong to transfer load on nib to compression zone of beam.

Load acting on nib load=100 kN/m
Dist. from load to CL beam links av=500 mm
Concrete grade fcu=35 N/mm²
Steel strength fy=250 N/mm²
Designated exposure class is XC3
Specified fixing tolerance tol=10 mm
Concrete cover to nib steel cover=70 mm
Size of nib bars diam=12 mm
Chosen nib thickness h=300 mm
Chosen bar spacing Sprov=110 mm

SUMMARY

Characteristic strength 250 N/mm²
Diameter of bars 12 mm
Spacing of bars 110 mm
Area required 1026.8 mm²
Area provided 1028.2 mm²

Check shear resistance

Permissible shear stress vcp=vc*sdfact=0.63045 N/mm²
Actual shear stress vca=Nload/(b*d)=0.44643 N/mm²
Shear resistance is satisfactory.
Location: Light loading on small nib with non-standard reinforcement

Design of concrete nibs to BS8110

Design to BS8110(1997), partial safety factor for steel gams=1.15

In addition to providing shear resistance for beam, these links must be sufficiently strong to transfer load on nib to compression zone of beam.

Load acting on nib
- load=5 kN/m

Dist. from load to CL beam links
- av=350 mm

Concrete grade
- fcu=25 N/mm²

Steel strength
- fy=300 N/mm²

Min. permiss. proportion of steel
- Pmin1=0.00218

Bond coefficient β
- beta=0.5

Designated exposure class is XC1

Specified fixing tolerance
- tol=10 mm

Concrete cover to nib steel
- cover=40 mm

Size of nib bars
- diam=16 mm

Chosen nib thickness
- h=100 mm

Chosen bar spacing
- Sprov=170 mm

SUMMARY
- Characteristic strength 300 N/mm²
- Diameter of bars 16 mm
- Spacing of bars 170 mm
- Area required 1169 mm²
- Area provided 1182.7 mm²

Check shear resistance

Permissible shear stress
- vcp=vc*sdfact=1.3842 N/mm²

Actual shear stress
- vca=Nload/(b*d)=0.096154 N/mm²

Shear resistance is satisfactory.
Location: Ex1 - Typical problem

Design of concrete nibs to EC2 Part 1:2004

In addition to providing shear resistance to beam, these links must be sufficiently strong to transfer load on nib to compression zone of beam.

FEd = load on nib

Nib reinforcement in form of vertical or horizontal loops with internal radius of 2Ø

Design load acting on nib

FEd = 26.3 kN/m

Dist. from load to CL beam links

av = 120 mm

Char yield strength (reinf'ment)

fyk = 500 N/mm²

Size of nib bars

dia = 8 mm

Chosen nib thickness

h = 115 mm

Chosen bar spacing

Sprov = 230 mm

SUMMARY

OF MAIN

Characteristic strength 500 N/mm²

Diameter of bars 8 mm

REINFORCEMENT

Spacing of bars 230 mm

Area of steel required 218 mm²/m

Area of steel provided 218 mm²/m

Check shear resistance

Term for shear resistance

VRdc=(rho1*fck)^(1/3)=2.0093

Shear resistance

VRdc=CRdc*ks*Vrdc*bw*d/1000=41.471 kN

Minimum stress

vmin=0.035*ks^1.5*fck^0.5=0.56 N/mm²

Minimum shear resistance

VRdm=vmin*bw*d/1000=48.16 kN

Modified shear resistance

VRdc=VRdm=48.16 kN

SHEAR SUMMARY

Design shear force 26.3 kN

Design shear resistance 48.16 kN

Hence nib shear resistance OK.
Location: Ex2 - Heavy loading on large nib with mild steel r'ment

Design of concrete nibs to EC2 Part 1:2004

In addition to providing shear resistance to beam, these links must be sufficiently strong to transfer load on nib to compression zone of beam.

FEd = load on nib

Nib reinforcement in form of vertical or horizontal loops with internal radius of 2Ø

Design load acting on nib

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>FEd</td>
<td>100 kN/m</td>
</tr>
<tr>
<td>Dist. from load to CL beam links</td>
<td>500 mm</td>
</tr>
<tr>
<td>Char yield strength (reinf'ment)</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Size of nib bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Chosen nib thickness</td>
<td>300 mm</td>
</tr>
<tr>
<td>Chosen bar spacing</td>
<td>230 mm</td>
</tr>
</tbody>
</table>

SUMMARY

<table>
<thead>
<tr>
<th>Component</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>230 mm</td>
</tr>
<tr>
<td>Area of steel required</td>
<td>489 mm²/m</td>
</tr>
<tr>
<td>Area of steel provided</td>
<td>491 mm²/m</td>
</tr>
</tbody>
</table>

Check shear resistance

Term for shear resistance

vr = (rho1*fck)^(1/3) = 1.8019

Shear resistance

VR = CR * k * vRdc * bw * d / 1000 = 97.689 kN

Minimum stress

vmin = 0.035 * k * fck * 0.5 = 0.49369 N/mm²

Minimum shear resistance

VRdm = vmin * bw * d / 1000 = 116.02 kN

Modified shear resistance

VRdc = VRdm = 116.02 kN

SHEAR SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force</td>
<td>100 kN</td>
</tr>
<tr>
<td>Design shear resistance</td>
<td>116.02 kN</td>
</tr>
</tbody>
</table>

Hence nib shear resistance OK.
Location: Ex3 - Light loading on small nib with non-standard r'ment

Design of concrete nibs to EC2 Part 1:2004

In addition to providing shear resistance to beam, these links must be sufficiently strong to transfer load on nib to compression zone of beam.

FEd = load on nib

Nib reinforcement in form of vertical or horizontal loops with internal radius of 2Ø

Design load acting on nib FEd=5 kN/m
Dist. from load to CL beam links av=350 mm
Char yield strength (reinf'ment) fyk=500 N/mm²
Size of nib bars dia=16 mm
Chosen nib thickness h=125 mm
Chosen bar spacing Sprov=250 mm

SUMMARY
Characteristic strength 500 N/mm²
Diameter of bars 16 mm
Spacing of bars 250 mm
Area of steel required 804 mm²/m
Area of steel provided 804 mm²/m

Check shear resistance

Term for shear resistance vRdc=(rho1*fck)^(1/3)=2.9049
Shear resistance VRdc=CRdc*ks*vRdc*bw*d/1000=57.168 kN
Minimum stress vmin=0.035*ks^1.5*fck^0.5 =0.49497 N/mm²
Minimum shear resistance VRdm=vmin*bw*d/1000=40.588 kN

SHEAR SUMMARY
Design shear force 5 kN
Design shear resistance 57.168 kN
Hence nib shear resistance OK.
Location: All options using normal-weight concrete

Fire resistance requirements to BS8110: Part 2:1985 Tables 4.2-4.6

Chosen fire resistance period \( \text{fire}=2 \text{ hours} \)
Thickness of wall \( w_{\text{Thick}}=350 \text{ mm} \)
Size of bars \( d_i=20 \text{ mm} \)
Spacing of bars \( B_{\text{Space}}=200 \text{ mm} \)
Bars near both faces \( (1=\text{Yes}/0=\text{No}) \) \( \text{faces}=1 \)
Proportion of steel provided \( \text{prop}=(1+\text{faces})*\pi/4*d_i^2/(B_{\text{Space}}*w_{\text{Thick}})=0.008976 \)

The values illustrated below correspond to data from Table 4.6

---

COLUMNS (Plan view)

The values illustrated below correspond to data from Table 4.3
The values illustrated below correspond to data from Tables 4.4 and 4.5.

Plain soffit floor

25 mm

Plain soffit floor

25 mm

Ribbed soffit floor

35

115 mm

110 mm

CONTINUOUS FLOOR CONSTRUCTION
Location: Typical lightweight concrete calculations

Fire resistance requirements to BS8110:Part 2:1985 Tables 4.2-4.6

Chosen fire resistance period fire=4 hours
Proportion of steel provided prop=0.008

The values illustrated below correspond to data from Table 4.6

![Reinforced Wall Diagram]

REINFORCED WALL

The values illustrated below correspond to data from Table 4.2.

![Columns Diagram]

COLUMN(S) (Plan view)

The values illustrated below correspond to data from Table 4.3

![Continuous Rectangular Beam Diagram]  ![Continuous Flanged Beam Diagram]
The values illustrated below correspond to data from Tables 4.4 and 4.5.

Plain soffit floor

Plain soffit floor

Ribbed soffit floor

CONTINUOUS FLOOR CONSTRUCTION
Location: Example of automatic calculation of steel proportion

Fire resistance requirements to BS8110: Part 2:1985 Tables 4.2-4.6

Chosen fire resistance period  fire=2.75 hours  
Thickness of wall  wThick=350 mm  
Size of bars  di=12 mm  
Spacing of bars  BSpace=300 mm  
Bars near both faces (1=Yes/0=No)  faces=0  
Proportion of steel provided  prop=(1+faces)*π/4*di^2/(BSpace*wThick)=0.0010771

As steel proportion is less than 0.004, the steel provided is insufficient for the wall to be considered as reinforced, and wall must be considered as unreinforced for design purposes.

The values illustrated below correspond to data from Table 4.2.

![Diagram](image-url)

100% exposure  50% exposure  One face only exposed

C O L U M N S (P l a n v i e w)

The values illustrated below correspond to data from Table 4.3

![Diagram](image-url)

The values illustrated below correspond to data from Tables 4.4 and 4.5.
Location: All options using normal-weight concrete

**Flat slab construction fire resistance to EC2 Part 1-2:2004**

Fire resistance period required  fire=2 hours

---

**NORMAL - WEIGHT CONCRETE**

**FLAT SLAB CONSTRUCTION**

**Reinforced wall fire resistance to EC2 Part 1-2:2004**

Fire resistance period required  fire=2 hours

---

**NORMAL - WEIGHT CONCRETE**

**REINFORCED WALL**

**Column fire resistance to EC2 Part 1-2:2004**

Fire resistance period required  fire=2 hours

---

**NORMAL - WEIGHT CONCRETE**

**COLUMNS (Plan view)**
Floor beam fire resistance to EC2 Part 1-2:2004

Fire resistance period required  fire=2 hours

Floor slab fire resistance to EC2 Part 1-2:2004

Fire resistance period required  fire=2 hours
Location: Typical lightweight concrete calculations

Flat slab construction fire resistance to EC2 Part 1-2:2004

Fire resistance period required  fire=4 hours

---

NORMAL - WEIGHT CONCRETE
FLAT SLAB CONSTRUCTION

Reinforced wall fire resistance to EC2 Part 1-2:2004

Fire resistance period required  fire=4 hours

---

NORMAL - WEIGHT CONCRETE
REINFORCED WALL

Column fire resistance to EC2 Part 1-2:2004

Fire resistance period required  fire=4 hours

---

NORMAL - WEIGHT CONCRETE
COLUMN S (Plan view)
**Floor beam fire resistance to EC2 Part 1-2:2004**

Fire resistance period required  fire=4 hours

![Diagram of floor beam fire resistance]

**Normal - Weight Concrete**

Continuous Rectangular Beam  Flanged Beam

**Floor slab fire resistance to EC2 Part 1-2:2004**

Fire resistance period required  fire=4 hours

![Diagram of floor slab fire resistance]

**Normal - Weight Concrete**

Continuous Floor Construction

Plain soffit floor

Ribbed soffit floor
Location: Example of automatic calculation of steel proportion

Flat slab construction fire resistance to EC2 Part 1-2:2004

Fire resistance period required fire=3 hours

---

NORMAL - WEIGHT CONCRETE
FLAT SLAB CONSTRUCTION

Reinforced wall fire resistance to EC2 Part 1-2:2004

Fire resistance period required fire=3 hours

---

NORMAL - WEIGHT CONCRETE
REINFORCED WALL

Column fire resistance to EC2 Part 1-2:2004

Fire resistance period required fire=3 hours

---

NORMAL - WEIGHT CONCRETE
COLUMNS (Plan view)
**Floor beam fire resistance to EC2 Part 1-2:2004**

Fire resistance period required    fire=3 hours

---

**Floor slab fire resistance to EC2 Part 1-2:2004**

Fire resistance period required    fire=3 hours

---

**Flange beam**

- **Axis distance** = 80 mm
- **Slab depth** = 240 mm

---

**Plain soffit floor**

- **Axis distance** = 80 mm
- **Slab depth** = 150 mm

---

**Ribbed soffit floor**

- **Axis distance** = 70 mm
- **Rib width** = 260 mm

---

**NORMAL - WEIGHT CONCRETE**

**FREELY-SUPPORTED**

**RECTANGULAR BEAM**

**FLANGED BEAM**

---

**NORMAL - WEIGHT CONCRETE**

**FREELY-SUPPORTED FLOOR CONSTRUCTION**
Location: Ex1 - Metre width of 500mm deep slab

General reinforced concrete section to BS5400:Part 4:1990

Char. strength of concrete \( f_{cm} = 40 \, \text{N/mm}^2 \)
Number of cross-sections \( n_{cs} = 1 \)
Cross-section 1
Number of layers of reinforcement \( n_{ols} = 2 \)
Char. strength of reinforcement \( f_{ym} = 460 \, \text{N/mm}^2 \)
Young's modulus of reinforcement \( E_m = 200000 \, \text{N/mm}^2 \)
Layer 1
Area of reinforcement \( A_s(1) = 402 \, \text{mm}^2 \)
Depth from compression face \( d_s(1) = 50 \, \text{mm} \)
Layer 2
Area of reinforcement \( A_s(2) = 5629 \, \text{mm}^2 \)
Depth from compression face \( d_s(2) = 444 \, \text{mm} \)

Ultimate limit state : flexure of general reinforced concrete section

RESULTS FROM ULTIMATE MOMENT ANALYSIS

Section moment resistance \( 866.36 \, \text{kNm} \)
Depth to Neutral Axis \( 134.66 \, \text{mm} \)

Strain at centroid of tension reinforcement 0.0080403 is greater than limiting strain 0.004 from Clause 5.3.2.1. Ductility requirements are satisfied.

Ultimate limit state : Shear calculations to Clauses 5.3.3 & 5.4.4

Ultimate shear force \( V = 500 \, \text{kN} \)
Number of layers \( s_l = 1 \)
Layer number \( l_n(1) = 2 \)
Area to be included in calcs. \( A_{sc}(1) = 5629 \, \text{mm}^2 \)
Breadth of section for shear \( b = 1000 \, \text{mm} \)
Spacing of links along member \( s_v(1) = 200 \, \text{mm} \)
Number of legs \( s_v(2) = 4 \)
Diameter of link reinforcement \( b_{dl} = 12 \, \text{mm} \)
Characteristic strength of links \( f_{yl} = 460 \, \text{N/mm}^2 \)
Characteristic strength \( f_{yl,sa} = 460 \, \text{N/mm}^2 \)

RESULTS FROM SHEAR ANALYSIS

Slab section
Shear reinforcement adequate: \( A_s(1) = 5629 \, \text{mm}^2 \)
\( A_s(2) = 452.39 \, \text{mm}^2 \)

An area of longitudinal reinforcement 624.69 \( \text{mm}^2 \) is required in the tensile zone in excess of that required for bending.
Serviceability limit state calculations

For stresses to Clause 4.1.1.3 & crack widths to Clause 5.8.8.2

Factored dead load moment \( d_{lm}=200 \text{ kNm} \)
Factored live load moment \( l_{lm}=300 \text{ kNm} \)
Cover from notional surface at which the crack width is calculated to outermost reinforcement \( c_{nom}=35 \text{ mm} \)
Diameter \( b_{d1}=32 \text{ mm} \)
Spacing \( b_{spac}=150 \text{ mm} \)

RESULTS FROM SERVICEABILITY LIMIT STATE ANALYSIS

Depth to Neutral Axis \( 158.73 \text{ mm} \)

Concrete stresses

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Stress at top ( \text{N/mm}^2 )</th>
<th>Permissible ( \text{N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15.667</td>
<td>20</td>
</tr>
</tbody>
</table>

Reinforcement stresses ( tension-negative, compression-positive )

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth ( \text{mm} )</th>
<th>Stress ( \text{N/mm}^2 )</th>
<th>Permissible ( \text{N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50</td>
<td>86.549</td>
<td>345</td>
</tr>
<tr>
<td>2</td>
<td>444</td>
<td>-227.08</td>
<td>-345</td>
</tr>
</tbody>
</table>

Design crack width \( 0.24331 \text{ mm} \)
Design crack width should be less than the values in Table 1.

DESIGN SUMMARY

| Char.comp.strength of concrete \( \text{N/mm}^2 \) | 40 |
| Char.strength of reinforcement \( \text{N/mm}^2 \) | 460 |
| Reinforcement material factor | 1.15 |
| Section moment resistance \( \text{kNm} \) | 866.36 |
| Ultimate shear force \( \text{kN} \) | 500 |
| Area of shear re'ment required \( \text{mm}^2 \) | 351.62 |
| Area of shear re'ment provided \( \text{mm}^2 \) | 452.39 |
| Diameter of tensile re'ment \( \text{mm} \) | 32 |
| Spacing of tensile re'ment \( \text{mm} \) | 150 |
| Crack width \( \text{mm} \) | 0.24331 |
Location: I-beam section

General reinforced concrete section to BS5400:Part 4:1990

Number of cross-sections           ncs=5
Cross-section 1
Char. strength of concrete        fcu(1)=40 N/mm²
Cross-section 2
Char. strength of concrete        fcu(2)=30 N/mm²
Cross-section 3
Char. strength of concrete        fcu(3)=30 N/mm²
Cross-section 4
Char. strength of concrete        fcu(4)=30 N/mm²
Cross-section 5
Char. strength of concrete        fcu(5)=30 N/mm²
Number of layers of reinforcement nols=4
Layer 1
Area of reinforcement           As(1)=678 mm²
Depth from compression face       ds(1)=50 mm
Characteristic strength           fy(1)=250 N/mm²
Young's modulus for reinforcement E(1)=190000 N/mm²
Layer 2
Area of reinforcement           As(2)=678 mm²
Depth from compression face       ds(2)=150 mm
Characteristic strength           fy(2)=250 N/mm²
Young's modulus for reinforcement E(2)=190000 N/mm²
Layer 3
Area of reinforcement           As(3)=3927 mm²
Depth from compression face       ds(3)=850 mm
Characteristic strength           fy(3)=460 N/mm²
Young's modulus for reinforcement E(3)=200000 N/mm²
Layer 4
Area of reinforcement           As(4)=6434 mm²
Depth from compression face       ds(4)=950 mm
Characteristic strength           fy(4)=460 N/mm²
Young's modulus for reinforcement E(4)=200000 N/mm²

Ultimate limit state : flexure of general reinforced concrete section
RESULTS FROM ULTIMATE MOMENT ANALYSIS

| Section moment resistance | 3321.2 kNm |
| Depth to Neutral Axis | 324.55 mm |

Strain at centroid of tension reinforcement 0.0063361 is greater than limiting strain 0.004 from Clause 5.3.2.1. Ductility requirements are satisfied.

Ultimate limit state : Shear calculations to Clauses 5.3.3 & 5.4.4

Ultimate shear force V=200 kN
Number of layers sl=2
Layer number ln(1)=3
Area to be included in calcs. Asc(1)=1200 mm²
Layer number ln(2)=4
Area to be included in calcs. Asc(2)=2000 mm²
Breadth of section for shear b=200 mm
Char. strength for analysis fcu=30 N/mm²
Distance of section from support av=1200 mm
Spacing of links along member sv1=650 mm
Number of legs sv2=2
Diameter of link reinforcement bdl=16 mm
Characteristic strength of links fyv=460 N/mm²
Characteristic strength fyasa=460 N/mm²

RESULTS FROM SHEAR ANALYSIS

Beam section
Shear reinforcement adequate: Asv required 142.38 mm²
Asv provided 402.12 mm²

An area of longitudinal reinforcement 249.88 mm² is required in the tensile zone in excess of that required for bending.

Serviceability limit state calculations

For stresses to Clause 4.1.1.3 & crack widths to Clause 5.8.8.2

Factored dead load moment dlm=400 kNm
Factored live load moment llm=450 kNm
Cover from notional surface at which the crack width is calculated
To outermost reinforcement cnom=20 mm
To Layer 4 cre=30 mm
Diameter bd1=32 mm
Spacing bspac=125 mm
RESULTS FROM SERVICEABILITY LIMIT STATE ANALYSIS

Depth to Neutral Axis 331.29 mm

Concrete stresses

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Stress at top N/mm²</th>
<th>Permissible N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.7287</td>
<td>20</td>
</tr>
<tr>
<td>2</td>
<td>2.4085</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>0.57396</td>
<td>15</td>
</tr>
</tbody>
</table>

Reinforcement stresses (tension-negative, compression-positive)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth mm</th>
<th>Stress N/mm²</th>
<th>Permissible N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50</td>
<td>45.79</td>
<td>187.5</td>
</tr>
<tr>
<td>2</td>
<td>150</td>
<td>29.511</td>
<td>187.5</td>
</tr>
<tr>
<td>3</td>
<td>850</td>
<td>-88.884</td>
<td>-345</td>
</tr>
<tr>
<td>4</td>
<td>950</td>
<td>-106.02</td>
<td>-345</td>
</tr>
</tbody>
</table>

Design crack width 0.10525 mm

Design crack width should be less than the values in Table 1.

DESIGN SUMMARY

- Reinforcement material factor: 1.15
- Section moment resistance: 3321.2 kNm
- Ultimate shear force: 200 kN
- Area of shear re'ment required: 142.38 mm²
- Area of shear re'ment provided: 402.12 mm²
Location: 500mm wide beam section

General reinforced concrete section to BS5400:Part 4:1990

Char. strength of concrete \( f_{cum} = 18 \text{ N/mm}^2 \)
Number of cross-sections \( n_{cs} = 1 \)
Cross-section 1
Number of layers of reinforcement \( n_{ols} = 2 \)
Young's modulus of reinforcement \( E_m = 200000 \text{ N/mm}^2 \)
Layer 1
Area of reinforcement \( A_s(1) = 678 \text{ mm}^2 \)
Depth from compression face \( d_s(1) = 50 \text{ mm} \)
Characteristic strength \( f_y(1) = 250 \text{ N/mm}^2 \)
Layer 2
Area of reinforcement \( A_s(2) = 3927 \text{ mm}^2 \)
Depth from compression face \( d_s(2) = 950 \text{ mm} \)
Characteristic strength \( f_y(2) = 460 \text{ N/mm}^2 \)

Ultimate limit state: flexure of general reinforced concrete section

RESULTS FROM ULTIMATE MOMENT ANALYSIS

<table>
<thead>
<tr>
<th>Section moment resistance</th>
<th>1227.5 kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth to Neutral Axis</td>
<td>388.95 mm</td>
</tr>
</tbody>
</table>

Strain at centroid of tension reinforcement 0.0050486 is greater than limiting strain 0.004 from Clause 5.3.2.1. Ductility requirements are satisfied.

Ultimate limit state: Shear calculations to Clauses 5.3.3 & 5.4.4

Ultimate shear force \( V = 2000 \text{ kN} \)
Number of layers \( s_l = 1 \)
Layer number \( l_n(1) = 2 \)
Area to be included in calcs. \( A_{sc}(1) = 1000 \text{ mm}^2 \)
Breadth of section for shear \( b = 500 \text{ mm} \)

***WARNING***
Shear force for calculations restricted to \( V = V_{MAX1} = 1511.4 \text{ kN} \)

Revised applied shear stress \( v = V \times 1000/(b \times d) = 3.182 \text{ N/mm}^2 \)

***WARNING***
Concrete grade assumed for calculations \( c_u = 20 \)
Spacing of links along member \( s_{vl} = 750 \text{ mm} \)

***WARNING***
Spacing of links along member 750 mm exceeds 712.5 mm
Number of legs \( s_{vl} = 4 \)
Diameter of link reinforcement \( b_{dl} = 12 \text{ mm} \)
Characteristic strength of links \( f_{yv} = 250 \text{ N/mm}^2 \)

***WARNING***
Shear reinforcement inadequate:
Characteristic strength \( f_{yasa} = 460 \text{ N/mm}^2 \)
RESULTS FROM SHEAR ANALYSIS

Beam section

***WARNING*** Shear force for calculations restricted to 1511.4 kN to comply with maximum shear stress limitations. Clause 5.3.5.1.

***WARNING*** Shear reinforcement inadequate
increase area or reduce spacing    Asv required 5663.9 mm²
Asv provided 452.39 mm²

***WARNING*** Spacing of links along member 750 mm exceeds 712.5 mm
An area of longitudinal reinforcement 1888.4 mm² is required in the tensile zone in excess of that required for bending.

Serviceability limit state calculations

For stresses to Clause 4.1.1.3 & crack widths to Clause 5.8.8.2

Factored dead load moment    dlm=200 kNm
Factored live load moment    llm=300 kNm
Cover from notional surface at which the crack width is calculated to outermost reinforcement    cnom=30 mm
Diameter    bd1=32 mm
Spacing    bspac=150 mm

RESULTS FROM SERVICEABILITY LIMIT STATE ANALYSIS

Depth to Neutral Axis    306.73 mm

Concrete stresses

Cross-section

Stress at top Permissible
N/mm² N/mm²
1 7.1312 9

Reinforcement stresses (tension-negative, compression-positive)

Layer Depth Stress Permissible
mm N/mm² N/mm²
1 50 59.687 187.5
2 950 -149.55 -345

Design crack width    0.15433 mm
Design crack width should be less than the values in Table 1.

DESIGN SUMMARY

Char.comp.strength of concrete    18 N/mm²
Reinforcement material factor    1.15
Section moment resistance    1227.5 kNm
Ultimate shear force    2000 kN
Area of shear re'ment required    5663.9 mm²
Area of shear re'ment provided    452.39 mm²

WARNING:
Shear reinforcement inadequate
<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of tensile re'ment</td>
<td>32 mm</td>
</tr>
<tr>
<td>Spacing of tensile re'ment</td>
<td>150 mm</td>
</tr>
<tr>
<td>Crack width</td>
<td>0.15433 mm</td>
</tr>
</tbody>
</table>
Location: 190mm deep slab section

General reinforced concrete section to BS5400:Part 4:1990

Char. strength of concrete       \( f_{\text{cum}} = 30 \text{ N/mm}^2 \)
Number of cross-sections         \( n_{\text{cs}} = 1 \)
Cross-section 1
Number of layers of reinforcement \( n_{\text{ols}} = 1 \)
Char. strength of reinforcement   \( f_{\text{ym}} = 460 \text{ N/mm}^2 \)
Young's modulus of reinforcement \( E_m = 200000 \text{ N/mm}^2 \)
Layer 1
Area of reinforcement            \( A_s(1) = 452 \text{ mm}^2 \)
Depth from compression face      \( d_s(1) = 140 \text{ mm} \)

Ultimate limit state : flexure of general reinforced concrete section

RESULTS FROM ULTIMATE MOMENT ANALYSIS

Section moment resistance       \( 22.856 \text{ kNm} \)
Depth to Neutral Axis           \( 30.111 \text{ mm} \)

Strain at centroid of tension reinforcement 0.012773 is greater than limiting strain 0.004 from Clause 5.3.2.1. Ductility requirements are satisfied.

Ultimate limit state : Shear calculations to Clauses 5.3.3 & 5.4.4

Ultimate shear force            \( V = 300 \text{ kN} \)
Number of layers                \( s_l = 1 \)
Layer number                    \( l_n(1) = 1 \)
Area to be included in calcs.    \( A_{s}(1) = 452 \text{ mm}^2 \)
Breadth of section for shear    \( b = 500 \text{ mm} \)

***WARNING***
Shear force for calculations restricted to \( V = V_{\text{MAX}1} = 287.55 \text{ kN} \)
Revised applied shear stress    \( v = V * 1000 / (b * d) = 4.1079 \text{ N/mm}^2 \)

***WARNING***
Shear force for calculations further restricted to \( V = V_{\text{MAX}2} = 55.768 \text{ kN} \)
Revised applied shear stress    \( v = V * 1000 / (b * d) = 0.79669 \text{ N/mm}^2 \)

RESULTS FROM SHEAR ANALYSIS

Slab section

***WARNING*** Shear force for calculations restricted to 287.55 kN to comply with maximum shear stress limitations. Clause 5.3.5.1.

***WARNING*** Shear force for calculations further restricted to 55.768 kN to comply with stress limitations for slabs with depth less than 200mm. Clause 5.4.4.1.

Applied shear stress is less than the ultimate shear stress
Shear reinforcement not allowed in slabs less than 200mm thick
### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char. comp. strength of concrete</td>
<td>30 N/mm²</td>
</tr>
<tr>
<td>Char. strength of reinforcement</td>
<td>460 N/mm²</td>
</tr>
<tr>
<td>Reinforcement material factor</td>
<td>1.15</td>
</tr>
<tr>
<td>Section moment resistance</td>
<td>22.856 kNm</td>
</tr>
</tbody>
</table>

---

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Ref No: SC122 BS
Location: 300mm deep slab section

General reinforced concrete section to BS5400:Part 4:1990

Char. strength of concrete \( f_{cum} = 30 \text{ N/mm}^2 \)
Number of cross-sections \( n_{cs} = 1 \)
Cross-section 1
Number of layers of reinforcement \( n_{ols} = 1 \)
Char. strength of reinforcement \( f_{ym} = 460 \text{ N/mm}^2 \)
Young's modulus of reinforcement \( E_m = 200000 \text{ N/mm}^2 \)
Layer 1
Area of reinforcement \( A_s(1) = 1000 \text{ mm}^2 \)
Depth from compression face \( d_s(1) = 250 \text{ mm} \)

Ultimate limit state: flexure of general reinforced concrete section

RESULTS FROM ULTIMATE MOMENT ANALYSIS

Section moment resistance \( 87.982 \text{ kNm} \)
Depth to Neutral Axis \( 66.613 \text{ mm} \)

Strain at centroid of tension reinforcement 0.0096356 is greater than limiting strain 0.004 from Clause 5.3.2.1. Ductility requirements are satisfied.

Ultimate limit state: Shear calculations to Clauses 5.3.3 & 5.4.4

Ultimate shear force \( V = 50 \text{ kN} \)
Number of layers \( s_l = 1 \)
Layer number \( l_n(1) = 1 \)
Area to be included in calcs. \( A_s(1) = 1000 \text{ mm}^2 \)
Breadth of section for shear \( b = 500 \text{ mm} \)

RESULTS FROM SHEAR ANALYSIS

Slab section
Applied shear stress is less than the ultimate shear stress
Shear reinforcement not required for a slab. Clause 5.4.4.1.

DESIGN SUMMARY

Char. comp. strength of concrete \( 30 \text{ N/mm}^2 \)
Char. strength of reinforcement \( 460 \text{ N/mm}^2 \)
Reinforcement material factor \( 1.15 \)
Section moment resistance \( 87.982 \text{ kNm} \)
Location: Ex1 - Reinforced concrete deck slab Ex 6.1.1

General reinforced concrete section

Concrete compressive strength \( f_{ckm} = 35 \text{ N/mm}^2 \)
Material factor \( \gamma_{mc} = 1.5 \)
Number of cross-sections \( n_{cs} = 1 \)
Cross-section 1
Number of layers of reinforcement \( n_{ols} = 1 \)
Characteristic yield strength \( f_{ykm} = 500 \text{ N/mm}^2 \)
Young's modulus of reinforcement \( E_s = 200000 \text{ N/mm}^2 \)
Material factor for reinforcement \( \gamma_{ms} = 1.15 \)
Reinforcement stress-strain relationship
Horizontal top branch with no strain limits
Layer 1
Area of reinforcement \( A_s(1) = 2094 \text{ mm}^2 \)
Depth from compression face \( d_s(1) = 200 \text{ mm} \)

Ultimate limit state: flexure of general reinforced concrete section

The calculation is based on BS EN 1992-1-1 Clause 6.1(2)P.

RESULTS FROM ULTIMATE MOMENT ANALYSIS

Section moment resistance 164.08 kNm
Depth to Neutral Axis 56.707 mm

Ultimate limit state: Shear calculations to EN 1992-1-1 Cl.6

Slab section:
Longitudinal Reinforcement
Area to be included in calcs. \( A_{sc}(1) = 2094 \text{ mm}^2 \)
Concrete Section
Effective width for shear \( b_w = 1000 \text{ mm} \)

Distance from edge of support or centre of flexible bearings
Distance \( a_v = 1000 \text{ mm} \)
Design shear resistance without shear reinforcement \( V_{Rdc} \), as per EN 1992-2 Clause 6.2.2 (6.2a)
Shear resistance \( V_{Rdc} = C R_{dc} k^* (100*rho_1*f_{ckl})^{(1/3)}*b_w*d/1000 \)
\( = 159.43 \text{ kN} \)

The minimum shear resistance below is to EN 1992-2 Cl.6.2.2 (6.2b)
Minimum shear resistance \( V_{Rdmin} = v_{min}*b_w*d/1000 = 117.13 \text{ kN} \)
Shear resistance 159.43 kN is greater than minimum 117.13 kN
Shear reinforcement not provided.
Serviceability limit state calculations

Stresses to EN 1992-1-1 Cl.7.2 and crack widths to EN 1992-1-1 Cl.7.3

For stress calculations:
- Moment from permanent effects: \( M_{ps} = 12.75 \text{ kN}\cdot\text{m} \)
- Moment from transient actions: \( M_{vs} = 72.75 \text{ kN}\cdot\text{m} \)

For crack width calculations:
- Moment from quasi-permanent effects: \( M_{pc} = 12.75 \text{ kN}\cdot\text{m} \)
- Moment from associated transient actions: \( M_{vc} = 5 \text{ kN}\cdot\text{m} \)

RESULTS FROM SERVICEABILITY LIMIT STATE ANALYSIS

**Stresses to EN 1992-1-1 Cl.7.2**

Stresses after minimal creep effects
- Depth to Neutral Axis: 58.893 mm

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Stress at top (N/mm²)</th>
<th>Permissible Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>16.098</td>
<td>21</td>
</tr>
</tbody>
</table>

Reinforcement stresses (tension-negative, compression-positive)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (mm)</th>
<th>Stress (N/mm²)</th>
<th>Permissible Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>200</td>
<td>-226.37</td>
<td>-400</td>
</tr>
</tbody>
</table>

Stresses under long term loading
- Concrete age at time of loading: \( \text{agec} = 30 \text{ days} \)
- Relative humidity: RH = 80 %
- Effective thickness: ho = 250 mm

Creep Coefficient (EN 1992-1-1 Cl.3.1.4(2)) = 1.5108

Long term modulus:
\[
E_{cl(1)} = \frac{E_{cs(i)} \cdot (M_{ps} + M_{vs})}{M_{vs} + (1 + cf(i)) \cdot M_{ps}}
\]
\[= 27811 \text{ N/mm}^2\]

- Depth to Neutral Axis: 64 mm

Concrete stresses

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Stress at top (N/mm²)</th>
<th>Permissible Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>14.955</td>
<td>21</td>
</tr>
</tbody>
</table>

Reinforcement stresses (tension-negative, compression-positive)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (mm)</th>
<th>Stress (N/mm²)</th>
<th>Permissible Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>200</td>
<td>-228.53</td>
<td>-400</td>
</tr>
</tbody>
</table>

Crack widths EN 1992-1-1 Clause 7.3

Long term modulus:
\[
E_{cl(1)} = \frac{E_{cs(i)} \cdot (M_{pc} + M_{vc})}{M_{vc} + (1 + cf(i)) \cdot M_{pc}}
\]
\[= 16342 \text{ N/mm}^2\]
Depth to Neutral Axis             78.812 mm

Concrete stresses
Cross-section Stress at top Permissible Stress
N/mm² N/mm²
1 2.5928 21

Reinforcement stresses ( tension-negative, compression-positive )

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (mm)</th>
<th>Stress (N/mm²)</th>
<th>Permissible Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>200</td>
<td>-48.792</td>
<td>-400</td>
</tr>
</tbody>
</table>

(a) Crack widths without direct calculation (EN 1992-2 Cl.7.3.3)
Diameter of tensile reinforcement bdia=16 mm
Spacing of tensile reinforcement bspac=100 mm
Limiting crack width (EN 1992-2 Table 7.101N) limcw=0.3 mm
Bar diameter 16 is less than maximum 32
Bar spacing 100 is less than maximum 300
Crack width less than limiting value 0.3 mm

Minimum reinforcement area (EN 1992-2 Clause 7.3.2)
Before first crack
Area of concrete in tensile zone 125000 mm²
Tensile fibre stress (BS EN 1992-2 Clause 7.3.2(105)) 3.21 N/mm²
Bar diameter 16 mm
Max. allowable stress (EN 1992-1-1 7.2N) sigs=TABLE 4 for bdia=16
=240 N/mm²
Min.r'ment area (EN 1992-2 Exp.7.1) smin=kc*k*fct(ncs)*Act/sigs
=668.74 mm²

(b) Crack widths with direct calculation (EN 1992-2 Cl.7.3.4)

Depth to Neutral Axis             78.812 mm
Depth of section                  250 mm
Effective tension reinforcement
Area 2094 mm²
Centroid 200 mm
Effective tension depth (EN 1992-1-1 Cl.7.3.2(3)) 57.063 mm
Effective tension area 57063 mm²
Number of bar diameters nbdia=1
Bar diameter ph=16 mm
Nominal cover c=40 mm
Crack width cw=Srmax*mult=0.030757 mm
# DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char. cylinder compr. strength</td>
<td>35 N/mm² (concrete)</td>
</tr>
<tr>
<td>Char. yield strength of re'ment</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Concrete material partial factor</td>
<td>1.5</td>
</tr>
<tr>
<td>Re'ment material partial factor</td>
<td>1.15</td>
</tr>
<tr>
<td>Section moment resistance</td>
<td>164.08 kNm</td>
</tr>
<tr>
<td>Shear resistance</td>
<td>159.43 kN</td>
</tr>
<tr>
<td>Diameter of tensile re'ment</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing of tensile re'ment</td>
<td>100 mm</td>
</tr>
<tr>
<td>Maximum permitted crack spacing</td>
<td>210.12 mm</td>
</tr>
<tr>
<td>Crack width</td>
<td>0.030757 mm</td>
</tr>
<tr>
<td>Limiting crack width</td>
<td>0.3 mm</td>
</tr>
</tbody>
</table>
Location: Ex2 - Voided reinforced concrete beam Ex 6.1.2

General reinforced concrete section

Concrete compressive strength \( f_{ckm} = 35 \text{ N/mm}^2 \)
Material factor \( \gamma_{cm} = 1.5 \)
Number of cross-sections \( n_{cs} = 3 \)

Cross-section 1
Cross-section 2
Cross-section 3

Number of layers of reinforcement \( n_{ols} = 1 \)
Characteristic yield strength \( f_{ykm} = 500 \text{ N/mm}^2 \)
Young's modulus of reinforcement \( E_s = 200000 \text{ N/mm}^2 \)
Material factor for reinforcement \( \gamma_{ms} = 1.15 \)

Reinforcement stress-strain relationship
Inclined top branch with strain limits
Ductility class is constant for whole section

Ductility Class (1=A, 2=B, 3=C) \( d_{cm} = 2 \)
Strain at maximum force \( \varepsilon_{ukm} = 0.05 \)
Multiplication factor \( k_m = 1.08 \)

Layer 1
Area of reinforcement \( A_s(1) = 4581 \text{ mm}^2 \)
Depth from compression face \( d_s(1) = 1425 \text{ mm} \)

Ultimate limit state : flexure of general reinforced concrete section

The calculation is based on BS EN 1992-1-1 Clause 6.1(2)P.

RESULTS FROM ULTIMATE MOMENT ANALYSIS

<table>
<thead>
<tr>
<th>Section moment resistance</th>
<th>2976.9 kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth to Neutral Axis</td>
<td>102.84 mm</td>
</tr>
</tbody>
</table>

Ultimate limit state : Shear calculations to EN 1992-1-1 Cl.6

Slab section:
Longitudinal Reinforcement
Area to be included in calcs. \( A_{sc}(1) = 4581 \text{ mm}^2 \)
Concrete Section
Effective width for shear \( b_w = 400 \text{ mm} \)

Distance from edge of support or centre of flexible bearings
Distance \( a_v = 5000 \text{ mm} \)
Design shear resistance without shear reinforcement \( V_{Rdc} \), as per
EN 1992-2 Clause 6.2.2 (6.2a)
Shear resistance
\[
V_{Rdc} = C_{Rdc} * k^* (100 * \rho_1 * f_{ckl})^{(1/3)} * b_w * d / 1000
\]
\[
= 285.95 \text{ kN}
\]
The minimum shear resistance below is to EN 1992-2 Cl.6.2.2 (6.2b)
Minimum shear resistance \( V_{Rdmin} = v_{min} * b_w * d / 1000 = 190.22 \text{ kN} \)
Shear resistance 285.95 kN is greater than minimum 190.22 kN
Shear reinforcement is links
Area at a cross-section           Asw=402 mm²
Angle alpha                       alpha=90 deg
Spacing along member              s=200 mm
Limiting transverse spacing legs  1.50*d=2137.5 mm
(EN 1992-1-1 Cl.9.3.2(4))
Lever arm z                       z=1282.5 mm
Characteristic strength           fywk=500 N/mm²
Design value of shear resistance sustained by shear reinforcement VRds
Angle of inclination of concrete strut theta=45 deg
Design shear resistance sustained by yielding shear reinforcement
VRds=(Asw/s)*z*((1/TAN(thetr)+1/TAN(alpr)))*fywd*SIN(alpr)/1000
=1120.8 kN
Maximum shear force  VRdmax=alphcw*bw*z*v1*fcd*(CT+CA)/(1+CT^2)/1000
=3088.3 kN
Applied moment at Section 'B'     AM=2000 kNm
or bent up bars                   AV=800 kN
Area of long.reinf'ment provided  Aprov=4581 mm²
Tensile capacity of longitudinal reinforcement adequate.
Capacity provided 1991.7 kN exceeds required capacity of 1959.5 kN

DESIGN SUMMARY

Char.cylinder compr.strength      35 N/mm² (concrete)
Char.yield strength of re'ment    500 N/mm²
Concrete material partial factor  1.5
Re'ment material partial factor   1.15
Section moment resistance        2976.9 kNm
Shear resistance                 285.95 kN
Location: Ex3 - Doubly reinforced concrete slab Ex 6.1.3

General reinforced concrete section

Concrete compressive strength \( f_{ckm} = 35 \text{ N/mm}^2 \)
Material factor \( \gamma_{cm} = 1.5 \)
Number of cross-sections \( n_{cs} = 1 \)
Cross-section 1
Number of layers of reinforcement \( n_{ols} = 2 \)
Characteristic yield strength \( f_{ykm} = 500 \text{ N/mm}^2 \)
Young's modulus of reinforcement \( E_s = 200000 \text{ N/mm}^2 \)
Material factor for reinforcement \( \gamma_{ms} = 1.15 \)
Reinforcement stress-strain relationship
Inclined top branch with strain limits
Ductility class is constant for whole section
Ductility Class (1=A, 2=B, 3=C) \( d_{cm} = 2 \)
Strain at maximum force \( e_{ukm} = 0.05 \)
Multiplication factor \( k_m = 1.08 \)
Layer 1
Area of reinforcement \( A_s(1) = 2693 \text{ mm}^2 \)
Depth from compression face \( d_s(1) = 50 \text{ mm} \)
Layer 2
Area of reinforcement \( A_s(2) = 8377 \text{ mm}^2 \)
Depth from compression face \( d_s(2) = 275 \text{ mm} \)

Ultimate limit state: flexure of general reinforced concrete section

The calculation is based on BS EN 1992-1-1 Clause 6.1(2)P.

RESULTS FROM ULTIMATE MOMENT ANALYSIS

Section moment resistance \( 810.98 \text{ kNm} \)
Depth to Neutral Axis \( 154.11 \text{ mm} \)

Strain 0.0027454 at centroid of layer 2 is less than limiting strain 0.045

DESIGN SUMMARY

Char. cylinder compr. strength \( 35 \text{ N/mm}^2 \) (concrete)
Char. yield strength of re'ment \( 500 \text{ N/mm}^2 \)
Concrete material partial factor \( 1.5 \)
Re'ment material partial factor \( 1.15 \)
Section moment resistance \( 810.98 \text{ kNm} \)
Location: Ex4 - Flanged reinforced concrete beam Ex 6.1.4

General reinforced concrete section

Concrete compressive strength \( f_{ckm} = 35 \text{ N/mm}^2 \)
Material factor \( \gamma_{cm} = 1.5 \)
Number of cross-sections \( n_{cs} = 2 \)
Cross-section 1
Cross-section 2
Number of layers of reinforcement \( n_{ols} = 2 \)
Characteristic yield strength \( f_{ykm} = 500 \text{ N/mm}^2 \)
Young's modulus of reinforcement \( E_s = 200000 \text{ N/mm}^2 \)
Material factor for reinforcement \( \gamma_{ms} = 1.15 \)
Reinforcement stress-strain relationship
Inclined top branch with strain limits
Ductility class is constant for whole section
Ductility Class (1=A, 2=B, 3=C) \( \beta = 2 \)
Strain at maximum force \( \varepsilon_{ukm} = 0.05 \)
Multiplication factor \( k_m = 1.08 \)
Layer 1
Area of reinforcement \( A_s(1) = 7540 \text{ mm}^2 \)
Depth from compression face \( d_s(1) = 1140 \text{ mm} \)
Layer 2
Area of reinforcement \( A_s(2) = 7540 \text{ mm}^2 \)
Depth from compression face \( d_s(2) = 1220 \text{ mm} \)

Ultimate limit state: flexure of general reinforced concrete section

The calculation is based on BS EN 1992-1-1 Clause 6.1(2)P.

RESULTS FROM ULTIMATE MOMENT ANALYSIS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section moment resistance</td>
<td>7081.2 kNm</td>
</tr>
<tr>
<td>Depth to Neutral Axis</td>
<td>336.2 mm</td>
</tr>
</tbody>
</table>

Strain 0.0092007 at centroid of layer 2 is less than limiting strain 0.045
Strain 0.0083679 at centroid of layer 1 is less than limiting strain 0.045

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char. cylinder compr. strength</td>
<td>35 N/mm²</td>
</tr>
<tr>
<td>Char. yield strength of re'ment</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Concrete material partial factor</td>
<td>1.5</td>
</tr>
<tr>
<td>Re'ment material partial factor</td>
<td>1.15</td>
</tr>
<tr>
<td>Section moment resistance</td>
<td>7081.2 kNm</td>
</tr>
</tbody>
</table>
Location: Ex5 - Reinforced concrete deck slab Ex 7.1

General reinforced concrete section

Concrete compressive strength \( f_{ckm} = 35 \, \text{N/mm}^2 \)
Material factor \( \gamma_{cm} = 1.5 \)
Number of cross-sections \( n_{cs} = 1 \)
Cross-section 1
Number of layers of reinforcement \( n_{ols} = 1 \)
Characteristic yield strength \( f_{ykm} = 500 \, \text{N/mm}^2 \)
Young's modulus of reinforcement \( E_s = 200000 \, \text{N/mm}^2 \)
Material factor for reinforcement \( \gamma_{sm} = 1.15 \)
Reinforcement stress-strain relationship
Horizontal top branch with no strain limits
Layer 1
Area of reinforcement \( A_s(1) = 2010 \, \text{mm}^2 \)
Depth from compression face \( d_s(1) = 192 \, \text{mm} \)

Ultimate limit state: flexure of general reinforced concrete section

The calculation is based on BS EN 1992-1-1 Clause 6.1(2)P.

RESULTS FROM ULTIMATE MOMENT ANALYSIS

Section moment resistance \( 151.2 \, \text{kNm} \)
Depth to Neutral Axis \( 54.432 \, \text{mm} \)

Ultimate limit state: Shear calculations to EN 1992-1-1 Cl.6

Slab section:
Longitudinal Reinforcement
Area to be included in calcs. \( A_{sc}(1) = 2010 \, \text{mm}^2 \)
Concrete Section
Effective width for shear \( b_w = 1000 \, \text{mm} \)

Distance from edge of support or centre of flexible bearings
Distance \( a_v = 1000 \, \text{mm} \)
Design shear resistance without shear reinforcement \( V_{RdC} \), as per EN 1992-2 Clause 6.2.2 (6.2a)
Shear resistance \( V_{RdC} = C_{RdC} \cdot k \cdot (100 \cdot r_{ho1} \cdot f_{ck1})^{(1/3)} \cdot b_w \cdot d/1000 = 153.05 \, \text{kN} \)

The minimum shear resistance below is to EN 1992-2 Cl.6.2.2 (6.2b)
Minimum shear resistance \( V_{Rdmin} = v_{min} \cdot b_w \cdot d/1000 = 112.45 \, \text{kN} \)
Shear resistance 153.05 kN is greater than minimum 112.45 kN
Shear reinforcement not provided.
Serviceability limit state calculations

Stresses to EN 1992-1-1 Cl.7.2 and crack widths to EN 1992-1-1 Cl.7.3

For stress calculations:
Moment from permanent effects \( M_{ps} = 12.75 \) kNm
Moment from transient actions \( M_{vs} = 72.75 \) kNm

For crack width calculations:
Moment from quasi-permanent effects \( M_{pc} = 12.75 \) kNm
Moment from associated transient actions \( M_{vc} = 5 \) kNm

RESULTS FROM SERVICEABILITY LIMIT STATE ANALYSIS

Stresses to EN 1992-1-1 Cl.7.2
Stresses after minimal creep effects
Depth to Neutral Axis \( 56.533 \) mm

Concrete stresses

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Stress at top (N/mm²)</th>
<th>Permissible Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>17.468</td>
<td>21</td>
</tr>
</tbody>
</table>

Reinforcement stresses (tension-negative, compression-positive)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (mm)</th>
<th>Stress (N/mm²)</th>
<th>Permissible Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>192</td>
<td>-245.66</td>
<td>-400</td>
</tr>
</tbody>
</table>

Stresses under long term loading
Concrete age at time of loading \( \text{age}_c = 30 \) days
Relative humidity \( \text{RH} = 80 \% \)
Effective thickness \( h_0 = 250 \) mm
Creep Coefficient (EN 1992-1-1 Cl.3.1.4(2)) \( k_1 = 1.5108 \)
Long term modulus \( E_{cl} = E_{cs}(i) * (M_{ps} + M_{vs}) / (M_{vs} + (1 + cf(i)) * M_{ps}) \)
\( = 27811 \) N/mm²

Depth to Neutral Axis \( 61.437 \) mm

Concrete stresses

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Stress at top (N/mm²)</th>
<th>Permissible Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>16.227</td>
<td>21</td>
</tr>
</tbody>
</table>

Reinforcement stresses (tension-negative, compression-positive)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (mm)</th>
<th>Stress (N/mm²)</th>
<th>Permissible Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>192</td>
<td>-248</td>
<td>-400</td>
</tr>
</tbody>
</table>

Crack widths EN 1992-1-1 Clause 7.3

Long term modulus \( E_{cl} = E_{cs}(i) * (M_{pc} + M_{vc}) / (M_{vc} + (1 + cf(i)) * M_{pc}) \)
\( = 16342 \) N/mm²
Depth to Neutral Axis             75.658 mm

Concrete stresses

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Stress at top N/mm²</th>
<th>Permissible Stress N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.8136</td>
<td>21</td>
</tr>
</tbody>
</table>

Reinforcement stresses ( tension-negative, compression-positive )

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth mm</th>
<th>Stress N/mm²</th>
<th>Permissible Stress N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>192</td>
<td>-52.95</td>
<td>-400</td>
</tr>
</tbody>
</table>

(a) Crack widths without direct calculation (EN 1992-2 Cl.7.3.3)
Diameter of tensile reinforcement bdia=16 mm
Spacing of tensile reinforcement bspac=100 mm
Limiting crack width (EN 1992-2 Table 7.101N) limcw=0.3 mm
Bar diameter 16 is less than maximum 32
Bar spacing 100 is less than maximum 300
Crack width less than limiting value 0.3 mm

Minimum reinforcement area (EN 1992-2 Clause 7.3.2)
Before first crack
Area of concrete in tensile zone 125000 mm²
Tensile fibre stress (BS EN 1992-2 Clause 7.3.2(105)) 3.21 N/mm²
Bar diameter 16 mm
Max. allowable stress (EN 1992-1-1 7.2N) sigs=TABLE 4 for bdia=16
=240 N/mm²
Min.r'ment area (EN 1992-2 Exp.7.1) smin=kc*k*fct(ncs)*Act/sigs =668.74 mm²

(b) Crack widths with direct calculation (EN 1992-2 Cl.7.3.4)

Depth to Neutral Axis             75.658 mm
Depth of section                  250 mm
Effective tension reinforcement
Area                               2010 mm²
Centroid                           192 mm
Effective tension depth (EN 1992-1-1 Cl.7.3.2(3)) 58.114 mm
Effective tension area             58114 mm²
Number of bar diameters            nbdia=1
Bar diameter                       ph=16 mm
Nominal cover                      c=50 mm
Crack width                        cw=Srmax*mult=0.039497 mm
**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char. cylinder compr. strength</td>
<td>35 N/mm² (concrete)</td>
</tr>
<tr>
<td>Char. yield strength of re'ment</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Concrete material partial factor</td>
<td>1.5</td>
</tr>
<tr>
<td>Re'ment material partial factor</td>
<td>1.15</td>
</tr>
<tr>
<td>Section moment resistance</td>
<td>151.2 kNm</td>
</tr>
<tr>
<td>Shear resistance</td>
<td>153.05 kN</td>
</tr>
<tr>
<td>Diameter of tensile re'ment</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing of tensile re'ment</td>
<td>100 mm</td>
</tr>
<tr>
<td>Maximum permitted crack spacing</td>
<td>248.64 mm</td>
</tr>
<tr>
<td>Crack width</td>
<td>0.039497 mm</td>
</tr>
<tr>
<td>Limiting crack width</td>
<td>0.3 mm</td>
</tr>
</tbody>
</table>
Location: Metre width strip of 500mm deep slab

General reinforced concrete section to Departmental Standard

BD 44/15

Number of cross-sections \( ncs = 1 \)
Concrete strength \( f_{cum} = 40 \text{ N/mm}^2 \)

Cross-section 1

Number of layers of reinforcement \( nols = 2 \)
Reinforcement strength \( f_{ym} = 460 \text{ N/mm}^2 \)
Young's modulus of reinforcement \( E_{m} = 200000 \text{ N/mm}^2 \)

Layer 1:
Area of reinforcement \( A_{s(1)} = 4021 \text{ mm}^2 \)
Depth from compression face \( d_{s(1)} = 444 \text{ mm} \)

Layer 2:
Area of reinforcement \( A_{s(2)} = 402 \text{ mm}^2 \)
Depth from compression face \( d_{s(2)} = 50 \text{ mm} \)

Ultimate limit state: flexure of general reinforced concrete section

Factor for reinforcement \( g_{ms} = 1.15 \)
Factor for concrete \( g_{mc} = 1.5 \)

RESULTS FROM ULTIMATE MOMENT ANALYSIS

<table>
<thead>
<tr>
<th>Moment of Resistance</th>
<th>645.86 kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth to Neutral Axis</td>
<td>94.067 mm</td>
</tr>
</tbody>
</table>

Shear calculations

Factor for reinforcement \( g_{ms} = 1.15 \)
Factor for concrete \( g_{mc} = 1.5 \)
Factor for shear in concrete \( g_{mv} = 1.25 \)
Breadth of section for shear \( b_s = 1000 \text{ mm} \)
Effective depth for shear \( d_s = 444 \text{ mm} \)
Distance of section from support \( a_v = 1000 \text{ mm} \)

Angle of inclination of shear \( r \)'ment \( \alpha = 75^\circ \)
Area of shear reinforcement \( A_{sv} = 400 \text{ mm}^2 \)
Strength of shear reinforcement \( f_{yv} = 460 \text{ N/mm}^2 \)
In direction of span \( s_{v1} = 300 \text{ mm} \)
At right angles to span \( s_{v2} = 400 \text{ mm} \)
Value of \( k \) for calculations \( k = 1.2679 \)
Area of tension reinforcement \( A_s = 1000 \text{ mm}^2 \)
Strength of concrete \( f_{cuv} = 40 \text{ N/mm}^2 \)
Constant KVC \( KVC = 0.24 \)
Applied moment at Section 'S' AM=633 kNm
Area of long. r'ment provided Aprov=4021 mm²
Longitudinal r'ment strength fyl=460 N/mm²

RESULTS FROM SHEAR ANALYSIS

Concrete shear capacity 243.22 kN
Shear reinforcement capacity 290.02 kN
Total shear capacity 533.24 kN

Serviceability limit state calculations

for stresses to Cl. 4.1.1.3. and crack widths
to BD 44/15 Appendix A Clause 5.8.8.2

Factored dead load moment dlm=200 kNm
Factored live load moment llm=100 kNm
Diameter bd1=32 mm
Spacing bspac=150 mm
Breadth bc=1000 mm

RESULTS FROM SERVICEABILITY LIMIT STATE ANALYSIS

Depth to Neutral Axis 149.34 mm

Concrete stresses

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Stress at top N/mm²</th>
<th>Permissible N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.848</td>
<td>20</td>
</tr>
</tbody>
</table>

Reinforcement stresses ( tension-negative, compression-positive )

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth mm</th>
<th>Stress N/mm²</th>
<th>Permissible N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>444</td>
<td>-189.26</td>
<td>345</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>63.807</td>
<td>345</td>
</tr>
</tbody>
</table>

Design crack width 0.16801 mm
Design crack width should be less than the values in Table 1.
Location: I-beam section

General reinforced concrete section to Departmental Standard

BD 44/15

Number of cross-sections ncs=5

Cross-section 1
Concrete strength fcu(1)=40 N/mm²

Cross-section 2
Concrete strength fcu(2)=30 N/mm²

Cross-section 3
Concrete strength fcu(3)=30 N/mm²

Cross-section 4
Concrete strength fcu(4)=30 N/mm²

Cross-section 5
Concrete strength fcu(5)=30 N/mm²

Number of layers of reinforcement nols=4

Layer 1:
Area of reinforcement As(1)=678 mm²
Depth from compression face ds(1)=50 mm
Reinforcement strength fy(1)=250 N/mm²
Young's modulus for reinforcement E(1)=190000 N/mm²

Layer 2:
Area of reinforcement As(2)=678 mm²
Depth from compression face ds(2)=150 mm
Reinforcement strength fy(2)=250 N/mm²
Young's modulus for reinforcement E(2)=190000 N/mm²

Layer 3:
Area of reinforcement As(3)=3927 mm²
Depth from compression face ds(3)=850 mm
Reinforcement strength fy(3)=460 N/mm²
Young's modulus for reinforcement E(3)=200000 N/mm²

Layer 4:
Area of reinforcement As(4)=6434 mm²
Depth from compression face ds(4)=950 mm
Reinforcement strength fy(4)=460 N/mm²
Young's modulus for reinforcement E(4)=200000 N/mm²
Ultimate limit state: flexure of general reinforced concrete section

Factor for reinforcement: \( g_{ms} = 1.15 \)
Factor for concrete: \( g_{mc} = 1.5 \)

RESULTS FROM ULTIMATE MOMENT ANALYSIS

- Moment of Resistance: 3321.2 kNm
- Depth to Neutral Axis: 324.55 mm

Shear calculations

Factor for reinforcement: \( g_{ms} = 1.15 \)
Factor for concrete: \( g_{mc} = 1.5 \)
Factor for shear in concrete: \( g_{mv} = 1.25 \)
Breadth of section for shear: \( b_s = 200 \) mm
Effective depth for shear: \( d_s = 912.5 \) mm
Distance of section from support: \( a_v = 1500 \) mm

Angle of inclination of shear reinforcement: \( \alpha = 90^\circ \)
Area of shear reinforcement: \( A_{sv} = 402 \) mm\(^2\)
Strength of shear reinforcement: \( f_{yv} = 460 \) N/mm\(^2\)
In direction of span: \( s_{v1} = 650 \) mm
At right angles to span: \( s_{v2} = 250 \) mm
Area of tension reinforcement: \( A_s = 3200 \) mm\(^2\)
Strength of concrete: \( f_{cuv} = 30 \) N/mm\(^2\)
Constant KVC: \( K_{VC} = 0.24 \)
Applied moment at Section 'S': \( A_{M} = 1000 \) kNm
Area of long. reinforcement provided: \( A_{prov} = 3200 \) mm\(^2\)
Longitudinal reinforcement strength: \( f_{yl} = 460 \) N/mm\(^2\)

RESULTS FROM SHEAR ANALYSIS

Concrete shear capacity: 205.88 kN
Shear reinforcement capacity: 225.74 kN
Total shear capacity: 431.62 kN

Serviceability limit state calculations

for stresses to Cl. 4.1.1.3. and crack widths

Factored dead load moment: \( d_{lm} = 400 \) kNm
Factored live load moment: \( l_{lm} = 450 \) kNm
controlling the crack width cre = 30 mm
Diameter bd1 = 32 mm
Spacing bspac = 125 mm

RESULTS FROM SERVICEABILITY LIMIT STATE ANALYSIS

<table>
<thead>
<tr>
<th>Depth to Neutral Axis</th>
<th>332.18 mm</th>
</tr>
</thead>
</table>

Concrete stresses

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Stress at top</th>
<th>Permissible</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N/mm²</td>
<td>N/mm²</td>
</tr>
<tr>
<td>1</td>
<td>6.715</td>
<td>20</td>
</tr>
<tr>
<td>2</td>
<td>2.4378</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>0.59353</td>
<td>15</td>
</tr>
</tbody>
</table>

Reinforcement stresses (tension-negative, compression-positive)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth</th>
<th>Stress N/mm²</th>
<th>Permissible N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50</td>
<td>46.016</td>
<td>187.5</td>
</tr>
<tr>
<td>2</td>
<td>150</td>
<td>29.709</td>
<td>187.5</td>
</tr>
<tr>
<td>3</td>
<td>850</td>
<td>-88.886</td>
<td>345</td>
</tr>
<tr>
<td>4</td>
<td>950</td>
<td>-106.05</td>
<td>345</td>
</tr>
</tbody>
</table>

Design crack width 0.10529 mm
Design crack width should be less than the values in Table 1.
Location: 500mm wide beam section

General reinforced concrete section to Departmental Standard

BD 44/15

Number of cross-sections \( ncs = 1 \)
Concrete strength \( f_{cum} = 18 \) N/mm²
Cross-section 1
Number of layers of reinforcement \( nols = 2 \)
Young's modulus of reinforcement \( E_m = 200000 \) N/mm²
Layer 1:
Area of reinforcement \( A_s(1) = 678 \) mm²
Depth from compression face \( d_s(1) = 50 \) mm
Reinforcement strength \( f_y(1) = 250 \) N/mm²
Layer 2:
Area of reinforcement \( A_s(2) = 3927 \) mm²
Depth from compression face \( d_s(2) = 950 \) mm
Reinforcement strength \( f_y(2) = 460 \) N/mm²

Ultimate limit state: flexure of general reinforced concrete section

RESULTS FROM ULTIMATE MOMENT ANALYSIS

Moment of Resistance \( 1227.5 \) kNm
Depth to Neutral Axis \( 388.95 \) mm

Shear calculations

Factor for reinforcement \( g_{ms} = 1.15 \)
Factor for concrete \( g_{mc} = 1.5 \)
Factor for shear in concrete \( g_{mv} = 1.25 \)
Breadth of section for shear \( b_s = 500 \) mm
Effective depth for shear \( d_s = 950 \) mm
Distance of section from support \( a_v = 1500 \) mm

Angle of inclination of shear r'ment \( \alpha = 75° \)
Area of shear reinforcement \( A_{sv} = 804 \) mm²
Strength of shear reinforcement \( f_{yv} = 460 \) N/mm²
In direction of span \( s_{v1} = 750 \) mm
At right angles to span \( s_{v2} = 125 \) mm
Value of k for calculations \( k = 1.2679 \)
Area of tension reinforcement \( A_s = 3000 \) mm²
Strength of concrete \( f_{cuvi} = 18 \) N/mm²
Constant KVC \( KVC = 0.24 \)
Applied moment at Section 'S' \( AM = 1000 \text{ kNm} \)
Area of long. r'ment provided \( A_{prov} = 3200 \text{ mm}^2 \)
Longitudinal r'ment strength \( f_y = 460 \text{ N/mm}^2 \)

**RESULTS FROM SHEAR ANALYSIS**

Concrete shear capacity \( 331.6 \text{ kN} \)
Shear reinforcement capacity \( 498.91 \text{ kN} \)
Total shear capacity \( 830.51 \text{ kN} \)

Serviceability limit state calculations

for stresses to Cl. 4.1.1.3. and crack widths

to BD 44/15 Appendix A Clause 5.8.8.2

Factored dead load moment \( d_{lm} = 200 \text{ kNm} \)
Factored live load moment \( l_{lm} = 300 \text{ kNm} \)
Diameter \( b_{d1} = 32 \text{ mm} \)
Spacing \( b_{spac} = 150 \text{ mm} \)

**RESULTS FROM SERVICEABILITY LIMIT STATE ANALYSIS**

Depth to Neutral Axis \( 307.38 \text{ mm} \)

Concrete stresses

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Stress at top ( \text{N/mm}^2 )</th>
<th>Permissible ( \text{N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.1151</td>
<td>9</td>
</tr>
</tbody>
</table>

Reinforcement stresses (tension-negative, compression-positive)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth ( \text{mm} )</th>
<th>Stress ( \text{N/mm}^2 )</th>
<th>Permissible ( \text{N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50</td>
<td>59.913</td>
<td>187.5</td>
</tr>
<tr>
<td>2</td>
<td>950</td>
<td>-149.59</td>
<td>345</td>
</tr>
</tbody>
</table>

Design crack width \( 0.15437 \text{ mm} \)
Design crack width should be less than the values in Table 1.
Location: 190mm deep slab section

General reinforced concrete section to Departmental Standard

BD 44/15

Number of cross-sections  ncs=1
Concrete strength  fcum=30 N/mm²
Cross-section 1
Number of layers of reinforcement  nols=1
Reinforcement strength  fym=460 N/mm²
Young's modulus of reinforcement  Em=200000 N/mm²
Layer 1:
Area of reinforcement  As(1)=452 mm²
Depth from compression face  ds(1)=140 mm

Ultimate limit state: flexure of general reinforced concrete section

Factor for reinforcement  gms=1.15
Factor for concrete  gmc=1.5

RESULTS FROM ULTIMATE MOMENT ANALYSIS

Moment of Resistance  22.856 kNm
Depth to Neutral Axis  30.111 mm

Shear calculations

Factor for reinforcement  gms=1.15
Factor for concrete  gmc=1.5
Factor for shear in concrete  gmv=1.25
Breadth of section for shear  bs=500 mm
Effective depth for shear  ds=140 mm
Distance of section from support  av=2000 mm
Area of tension reinforcement  As=452 mm²
Strength of concrete  fcuv=30 N/mm²
Constant KVC  KVC=0.24

RESULTS FROM SHEAR ANALYSIS

Concrete shear capacity  49.572 kN
Shear reinforcement not assessed for a slab less than 200mm thick
Location: 300mm deep slab section

General reinforced concrete section to Departmental Standard

BD 44/15

Number of cross-sections  ncs=1
Concrete strength  fcum=30 N/mm²
Cross-section 1
Number of layers of reinforcement  nols=1
Reinforcement strength  fym=460 N/mm²
Young's modulus of reinforcement  Em=200000 N/mm²
Layer 1:
Area of reinforcement  As(1)=1000 mm²
Depth from compression face  ds(1)=250 mm

Ultimate limit state: flexure of general reinforced concrete section

Factor for reinforcement  gms=1.15
Factor for concrete  gmc=1.5

RESULTS FROM ULTIMATE MOMENT ANALYSIS

Moment of Resistance  87.982 kNm
Depth to Neutral Axis  66.613 mm

Shear calculations

Factor for reinforcement  gms=1.15
Factor for concrete  gmc=1.5
Factor for shear in concrete  gmv=1.25
Breadth of section for shear  bs=500 mm
Effective depth for shear  ds=250 mm
Distance of section from support  av=500 mm
Area of tension reinforcement  As=1000 mm²
Strength of concrete  fcuv=30 N/mm²
Constant KVC  KVC=0.24

RESULTS FROM SHEAR ANALYSIS

Concrete shear capacity  123.36 kN
Shear reinforcement not provided
Location: 500mm thick slab

Early Thermal Cracking in Concrete to BD 28/87 (Amd. No 1)

Calculations for distribution reinforcement for the control of early thermal cracking of reinforced and prestressed post-tensioned concrete members.

Area of effective concrete $A_c = 2.5E6 \text{ mm}^2$
Char tensile strength of r'ment $f_y = 460 \text{ N/mm}^2$
Char cube strength of concrete $f_{cu} = 40 \text{ N/mm}^2$
Ratio of $(f_{ct}/f_b)$ $b_r = 0.67$
Nominal diameter of r'ment $\phi = 16 \text{ mm}$
Permissible crack width $w = 0.25 \text{ mm}$

Short-term fall in temperature $T_1 = 35 \degree C$
Long-term fall in temperature $T_2 = 20 \degree C$
Restraint factor $R = 0.6$

Minimum r'ment area to ensure acceptable crack widths (Equation 3)

$$A_{sw} = b_r A_c \phi (R (e_{sh} 1E-6 + e_{th}) - 0.5 e_{ult} 1E-6) / (2 w) = 14836 \text{ mm}^2$$

Required reinforcement area, $A_s$, is the greater value obtained from Equation 2 and Equation 3.

Area from Equation 3 is greater than that from Equation 2

Required area from Equation 3 $A_s = A_{sw} = 14836 \text{ mm}^2$

Reinforcement that is present for other purposes may be included as part of the area of reinforcement $A_s = 14836 \text{ mm}^2$ (Section 5.10).

Arrangement of reinforcement - External edge restraint (Section 6.2)

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Reinforcement shall be provided parallel to the restraint. Where the length to height ratio exceeds 2:1 this reinforcement shall be provided over the full height of the section. When the length to height ratio is 2:1 or less, and there is no end restraint, reinforcement need only be placed in the half of the section adjacent to the restraint.

The area of reinforcement, $A_s = 14836 \text{ mm}^2$ shall be provided in the faces of the section.
In addition to the reinforcement parallel to the edge restraint, reinforcement of the same unit area shall also be provided to control the effects of warping restraint at the ends. This reinforcement shall be perpendicular to the edge restraint and be provided over a length at the end equivalent to the lesser of the height of the section, or 0.2 times the overall length.

**SUMMARY**

*Section is subject to external edge restraint*

*Reinforcement As=14836 mm² should be arranged in accordance with Section 6.2.*
Location: Ex1 - Consider a 500mm thick slab (edge restraint)

Early Thermal Cracking in Concrete - EN 1992-1-1 and EN 1992-3


Overall depth of concrete section h=500 mm
Char. tensile strength (reinf'ment) fyk=500 N/mm²

The minimum reinforcement area to control crack spacing will be evaluated in accordance with BS EN 1992-1-1:2004 Clause 7.3.2(2).

The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack. The section behaves elastically until the tensile fibre stress reaches fctm.

For a rectangular section, the area in tension is half the slab depth.

Area of conc. within tensile zone Act=b*h/2=250000 mm²/m
Stress distribution coefficient kc=1

The coefficient (k) below allows for the effect of non-uniform self equilibrating stresses, which lead to a reduction of restraint forces.
Coefficient ( Clause 7.3.2(2) ) k'=TABLE 6.13 for h=500
=0.86

Coefficient value to be adopted k=0.86
Minimum reinforcement area Asmin=kc*k*Act*fcteff/fyk=1300.2 mm²/m
Permissible crack width wk=0.25 mm
Temp.fall: hydrat. peak & ambient T1=35 øC
Long-term drop in temperature T2=20 °C
Bond coefficient k1 k1=0.8

Crack inducing strain

Coefficient of thermal expansion aTc=12 µε/°C
Restraint factor (short-term) R1=0.5 (thermal)
Restraint factor (long-term) R2=0.5 (thermal)
Restraint factor (long-term) R3=0.5 (drying)
Shrinkage strain (short-term) ecas=16.6 µε
Shrinkage strain (long-term) ecal=55 µε
Tensile strain capacity ectus=78 µε (short-term)
Tensile strain capacity ectul=111.2 µε (long-term)

The coefficient below deals with the effect of stress relaxation due to creep under sustained loading.
Creep coefficient K1=1
Short-term crack inducing strain ecrs=K1*(aTc*T1+ecas)*R1-0.5*ectus =179.3 µε
Long-term crack inducing strain ecrl=K1*(c1+c2+c3)-0.5*ectul=469.9 µε

Check crack width

Min steel area each face Asmin=1300.2 mm²/m
Diameter of distribution steel dia=16 mm
Spacing of distribution steel spacd=150 mm
Area of distribution steel \[ A_{\text{dis}} = \text{INT}(250 \times \pi \times \text{dia}^2 / \text{spacd}) \]  
= 1340 \text{ mm}^2 / \text{m}

Cover to reinforcement \[ c = 30 \text{ mm} \]

Design crack width (short-term) \[ w_{ks} = \text{srm}_{\text{max}} \times \text{ecrs} / 10^6 = 0.08744 \text{ mm} \]

Design crack width (long-term) \[ w_{kl} = \text{srm}_{\text{max}} \times \text{ecrl} / 10^6 = 0.22916 \text{ mm} \]

The short-term crack width 0.08744 mm is less than the permissible value of 0.25 mm hence satisfactory.

The long-term crack width 0.22916 mm is less than the permissible value of 0.25 mm hence satisfactory.

**Arrangement of reinforcement**

(External edge restraint (Fig L.1(a), EN 1992-3))

Reinforcement shall be provided parallel to the restraint. Where the length to height ratio exceeds 2:1 this reinforcement shall be provided over the full height of the section. When the length to height ratio is 2:1 or less, and there is no end restraint, reinforcement need only be placed in the half of the section adjacent to the restraint. The area of reinforcement, \( A_s = 1340 \text{ mm}^2 / \text{m} \) shall be provided in the faces of the section.

In addition to the reinforcement parallel to the edge restraint, reinforcement of the same unit area shall also be provided to control the effects of warping restraint at the ends. This reinforcement shall be perpendicular to the edge restraint and be provided over a length at the end equivalent to the lesser of the height of the section, or 0.2 times the overall length.

**SUMMARY**

Section is subject to (external) edge restraint. Reinforcement \( A_s = 1340 \text{ mm}^2 / \text{m} \) should be provided parallel to the edge restraint. Furthermore provide warping reinforcement.
Location: Ex2 - Consider a 1000mm thick slab (edge restraint)

Early Thermal Cracking in Concrete - EN 1992-1-1 and EN 1992-3


Overall depth of concrete section h=1000 mm
Char. tensile strength (reinf'ment) f_yk=500 N/mm²

The minimum reinforcement area to control crack spacing will be evaluated in accordance with BS EN 1992-1-1:2004 Clause 7.3.2(2).

The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack. The section behaves elastically until the tensile fibre stress reaches fctm.

For a rectangular section, the area in tension is half the slab depth.

Area of conc. within tensile zone Act=b*h/2=500000 mm²/m

Stress distribution coefficient k_c=1
The coefficient (k) below allows for the effect of non-uniform self equilibrating stresses, which lead to a reduction of restraint forces.

Coefficient (Clause 7.3.2(2)) k'=TABLE 6.13 for h=1000

=0.65

Coefficient value to be adopted k=0.65

Minimum reinforcement area Asmin=k_c*k*Act*fcteff/f_yk=1798.1 mm²/m

Permissible crack width wk=0.25 mm

Temp. fall: hydrat. peak & ambient T_1=20 °C

Long-term drop in temperature T_2=5 °C

Bond coefficient k_1 k_1=0.8

Crack inducing strain

Coefficient of thermal expansion a_Tc=10 με/°C
Restraint factor (short-term) R_1=0.5 (thermal)
Restraint factor (long-term) R_2=0.2 (thermal)
Restraint factor (long-term) R_3=0.2 (drying)
Shrinkage strain (short-term) e_cas=13.6 με
Shrinkage strain (long-term) e_cal=45 με
Tensile strain capacity e_cus=73.6 με (short-term)
Tensile strain capacity e_cul=104.8 με (long-term)

The coefficient below deals with the effect of stress relaxation due to creep under sustained loading.

Creep coefficient K_1=1

Short-term crack inducing strain e_crs=K_1*(a_Tc*T_1+e_cas)*R_1-0.5*e_cus=70 με

Long-term crack inducing strain e_crl=K_1*(c_1+c_2+c_3)-0.5*e_cul=147.3 με

Check crack width

Min steel area each face Asmin=1798.1 mm²/m
Diameter of distribution steel dia=16 mm
Spacing of distribution steel spacd=111 mm
Area of distribution steel 
\[ A_{d\text{is}} = \text{INT}(250 \times \pi \times \text{dia}^2 / \text{spacd}) = 1811 \text{ mm}^2 / \text{m} \]

Cover to reinforcement 
\[ c = 72 \text{ mm} \]

Design crack width (short-term) 
\[ w_{k\text{s}} = \text{srmax} \times \text{ecrs} / 10^6 = 0.05919 \text{ mm} \]

Design crack width (long-term) 
\[ w_{k\text{l}} = \text{srmax} \times \text{ecrl} / 10^6 = 0.12455 \text{ mm} \]

The short-term crack width 0.05919 mm is less than the permissible value of 0.25 mm hence satisfactory.

The long-term crack width 0.12455 mm is less than the permissible value of 0.25 mm hence satisfactory.

**Arrangement of reinforcement**

(External edge restraint (Fig L.1(a), EN 1992-3))

Reinforcement shall be provided parallel to the restraint. Where the length to height ratio exceeds 2:1 this reinforcement shall be provided over the full height of the section. When the length to height ratio is 2:1 or less, and there is no end restraint, reinforcement need only be placed in the half of the section adjacent to the restraint.

The area of reinforcement, \[ A_s = 1811 \text{ mm}^2 / \text{m} \] shall be provided in the faces of the section.

In addition to the reinforcement parallel to the edge restraint, reinforcement of the same unit area shall also be provided to control the effects of warping restraint at the ends. This reinforcement shall be perpendicular to the edge restraint and be provided over a length at the end equivalent to the lesser of the height of the section, or 0.2 times the overall length.

**SUMMARY**

Section is subject to (external) edge restraint. Reinforcement \[ A_s = 1811 \text{ mm}^2 / \text{m} \] should be provided parallel to the edge restraint. Furthermore provide warping reinforcement.
Location: Ex3 - Consider a 300mm thick slab (end restraint)

Early Thermal Cracking in Concrete – EN 1992-1-1 and EN 1992-3


Overall depth of concrete section h=300 mm
Char. tensile strength (reinf’ment) fyk=500 N/mm²

The minimum reinforcement area to control crack spacing will be evaluated in accordance with BS EN 1992-1-1:2004 Clause 7.3.2(2). The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack. The section behaves elastically until the tensile fibre stress reaches fctm. For a rectangular section, the area in tension is half the slab depth. Area of conc. within tensile zone Act=b*h/2=150000 mm²/m
Stress distribution coefficient kc=1
The coefficient (k) below allows for the effect of non-uniform self equilibrating stresses, which lead to a reduction of restraint forces.
Coefficient (Clause 7.3.2(2)) k’=TABLE 6.13 for h=300

Coefficient value to be adopted k=1
Minimum reinforcement area Asmin=kc*k*Act*fcteff/fyk=868.94 mm²/m
Permissible crack width wk=0.25 mm

Bond coefficient k₁ k₁=0.8

Crack inducing strain

Short-term crack inducing strain ecrs=0.5*ae*kc*k*fcteff*(1+1/(ae*p))/Es=0.23282E-3 με
Long-term crack inducing strain ecrl=0.5*ae*kc*k*fcteff*(1+1/(ae*p))/Es=0.39028E-3 με

Check crack width

Diameter of distribution steel dia=20 mm
Spacing of distribution steel spacd=INT(250*PI*dia^2/As1)=100 mm
Area of distribution steel Asdis=As1=3140 mm²/m
Cover to reinforcement c=70 mm
Design crack width (short-term) wks=srmax*ecrs=0.13104 mm
Design crack width (long-term) wk1=srmax*ecrl=0.21966 mm
The short-term crack width 0.13104 mm is less than the permissible value of 0.25 mm hence satisfactory.
The long-term crack width 0.21966 mm is less than the permissible value of 0.25 mm hence satisfactory.
Arrangement of reinforcement

(External end restraint (Fig L.1(b), EN 1992-3))

 SUMMARY Section is subject to (external) end restraint. Reinforcement As=3140 mm²/m should be provided throughout the member perpendicular to the restraining edge.
Location: Ex4 - Consider a 300mm thick slab (end restraint)

Early Thermal Cracking in Concrete - EN 1992-1-1 and EN 1992-3


Overall depth of concrete section \( h = 300 \) mm
Char. tensile strength (reinf'ment) \( f_yk = 500 \) N/mm\(^2\)

The minimum reinforcement area to control crack spacing will be evaluated in accordance with BS EN 1992-1-1:2004 Clause 7.3.2(2). The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack. The section behaves elastically until the tensile fibre stress reaches \( f_{ctm} \).

For a rectangular section, the area in tension is half the slab depth. Area of conc. within tensile zone \( A_{ct} = b \times h / 2 = 150000 \) mm\(^2\)/m
 Stress distribution coefficient \( k_c = 1 \)

The coefficient \( k \) below allows for the effect of non-uniform self equilibrating stresses, which lead to a reduction of restraint forces. Coefficient (Clause 7.3.2(2)) \( k' = \text{TABLE 6.13 for } h = 300 \) mm

Coefficient value to be adopted \( k = 1 \)
Minimum reinforcement area \( A_{s,\text{min}} = k_c \times k \times A_{ct} \times f_{ct,\text{eff}} / f_yk = 868.94 \) mm\(^2\)/m
Permissible crack width \( w_k = 0.3 \) mm

Bond coefficient \( k_1 = 1.14 \)

Crack inducing strain

Short-term crack inducing strain \( \varepsilon_{crs} = 0.5 \times a_e \times k_c \times k \times f_{ct,\text{eff}} \times (1 + 1/(a_e \times p)) / E_s = 0.33603 \times 10^{-3} \) με
Long-term crack inducing strain \( \varepsilon_{crl} = 0.5 \times a_e \times k_c \times k \times f_{ct,\text{eff}} \times (1 + 1/(a_e \times p)) / E_s = 0.56328 \times 10^{-3} \) με

Check crack width

Diameter of distribution steel \( \text{dia} = 16 \) mm
Spacing of distribution steel \( \text{spacd} = \text{INT}(250 \times \pi \times \text{dia}^2 / A_s) = 96 \) mm
Area of distribution steel \( A_{s,\text{dis}} = A_s = 2094 \) mm\(^2\)/m
Cover to reinforcement \( c = 30 \) mm
Design crack width (short-term) \( w_{k,s} = s_{rm} \times \varepsilon_{crs} = 0.15245 \) mm
Design crack width (long-term) \( w_{k,l} = s_{rm} \times \varepsilon_{crl} = 0.25555 \) mm
The short-term crack width 0.15245 mm is less than the permissible value of 0.3 mm hence satisfactory.
The long-term crack width 0.25555 mm is less than the permissible value of 0.3 mm hence satisfactory.
Arrangement of reinforcement

(External end restraint (Fig L.1(b), EN 1992-3))

Reinforcement for end restraint

SUMMARY  Section is subject to (external) end restraint. Reinforcement As=2094 mm²/m should be provided throughout the member perpendicular to the restraining edge.
**Location:** Ex5 - Consider a 2500mm thick slab (internal restraint)

**Early Thermal Cracking in Concrete - EN 1992-1-1 and EN 1992-3**


Overall depth of concrete section $h=2500$ mm  
Char.tensile strength (reinf'ment) $f_yk=500$ N/mm$^2$

The minimum reinforcement area to control crack spacing will be evaluated in accordance with BS EN 1992-1-1:2004 Clause 7.3.2(2).  
The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack. The section behaves elastically until the tensile fibre stress reaches $f_{ctm}$.  
The following is in accordance with CIRIA C660 Section 4.11.3.  
Area of conc. within tensile zone $A_{ct}=b*(0.2*h)=500000$ mm$^2$/m  
The coefficient ($k$) below allows for the effect of non-uniform self equilibrating stresses, which lead to a reduction of restraint forces.  
Coefficient value to be adopted $k=1$ (internal restraint dominant)  
Minimum reinforcement area $A_{smin}=kc*k*A_{ct}*f_{cteff}/f_yk=865$ mm$^2$/m

Permissible crack width $w_k=0.3$ mm  
Bond coefficient $k_1=1.14$

**Crack inducing strain**

When designing reinforcement to control cracking caused by solely by temperature differentials, autogenous strain may be omitted from the calculation as it will occur uniformly through the section and will not contribute to differential strain. Long-term strains are also omitted as the stress condition is transient and does not extend beyond the early thermal cycle.

- Tensile strain capacity $\epsilon_{ctu}=76$ $\mu$e  
- Coefficient of thermal expansion $a_{Tc}=12$ $\mu$e/$^\circ$C  
- Temperature differential ($\Delta T$) $\Delta lT=46$ $^\circ$C  
- Short-term crack inducing strain $\epsilon_{crs}=K_1*\Delta lT*a_{Tc*R}^{-0.5}*\epsilon_{ctu}=112.7$ $\mu$e

**Check crack width**

- Min steel area each face $A_{smin}=865$ mm$^2$/m  
- Diameter of distribution steel $d_{ia}=16$ mm  
- Spacing of distribution steel $s_{pcd}=225$ mm  
- Area of distribution steel $A_{sdis}=\text{INT}(250*\pi*d_{ia}^2/s_{pcd})=893$ mm$^2$/m  
- Cover to reinforcement $c=50$ mm  
- Design crack width (short-term) $w_k=s_{rmax}*\epsilon_{crs}/10^6=0.16101$ mm

The short-term crack width 0.16101 mm is less than the permissible value of 0.3 mm hence satisfactory.

**Arrangement of reinforcement - Internal restraint**

Where the least dimension of the member exceeds 1m, and the sum of the two smallest dimensions exceeds 4m, the section is considered to be subject to internal restraint due to the temperature gradient between the core and the surface. The area of reinforcement,
As=893 mm²/m shall be provided in each of two directions in all surfaces of the member.

**SUMMARY**

Section is subject to (internal) restraint due to temperature differentials within the member. Reinforcement As=893 mm²/m should be provided in each of the two directions in all surfaces of the member.
Location: Ex6 - Consider a 500mm thick slab (edge restraint)

Early Thermal Cracking in Concrete - EN 1992-1-1 and EN 1992-3


Overall depth of concrete section \( h = 500 \text{ mm} \)
Char. tensile strength (reinf'ment) \( fyk = 500 \text{ N/mm}^2 \)

The minimum reinforcement area to control crack spacing will be evaluated in accordance with BS EN 1992-1-1:2004 Clause 7.3.2(2). The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack. The section behaves elastically until the tensile fibre stress reaches \( f_{ctm} \). For a rectangular section, the area in tension is half the slab depth. Area of conc. within tensile zone \( A_{ct} = b \times h/2 = 250000 \text{ mm}^2/\text{m} \)
Stress distribution coefficient \( k_c = 0.9 \)
The coefficient \( (k) \) below allows for the effect of non-uniform self equilibrating stresses, which lead to a reduction of restraint forces.
Coefficient \( (\text{Clause 7.3.2(2)}) \) \( k' = \text{TABLE 6.13 for } h = 500 \)
\[ = 0.86 \]
Coefficient value to be adopted \( k = 1 \)
Minimum reinforcement area \( A_{smin} = k_c \times k \times A_{ct} \times f_{cteff}/f_yk = 1303.4 \text{ mm}^2/\text{m} \)
Permissible crack width \( w_k = 0.21 \text{ mm} \)
Temp. fall: hydrat. peak & ambient \( T_1 = 27 \degree C \)
Long-term drop in temperature \( T_2 = 20 \degree C \)
Bond coefficient \( k_1 = 1.14 \)

Crack inducing strain

Coefficient of thermal expansion \( a_{Tc} = 12 \mu \varepsilon/\degree C \)
Restrainment factor (short-term) \( R_1 = 0.62 \text{ (thermal)} \)
Restrainment factor (long-term) \( R_2 = 0.62 \text{ (thermal)} \)
Restrainment factor (long-term) \( R_3 = 0.62 \text{ (drying)} \)
Shrinkage strain (short-term) \( \epsilon_{cas} = 15 \mu \varepsilon \)
Shrinkage strain (long-term) \( \epsilon_{cal} = 50 \mu \varepsilon \)
Tensile strain capacity \( \epsilon_{tus} = 76 \mu \varepsilon \text{ (short-term)} \)
Tensile strain capacity \( \epsilon_{tul} = 108 \mu \varepsilon \text{ (long-term)} \)
The coefficient below deals with the effect of stress relaxation due to creep under sustained loading.
Creep coefficient \( k_l = 0.65 \)
Short-term crack inducing strain \( \epsilon_{crs} = K_1 \times a_{Tc} \times T_1 + \epsilon_{cas} \times R_1 - 0.5 \times \epsilon_{tus} = 98.617 \mu \varepsilon \)
Long-term crack inducing strain \( \epsilon_{crl} = K_1 \times (c_1 + c_2 + c_3) - 0.5 \times \epsilon_{tul} = 251.07 \mu \varepsilon \)

Check crack width

Min steel area each face \( A_{smin} = 1303.4 \text{ mm}^2/\text{m} \)
Diameter of distribution steel \( \text{dia} = 20 \text{ mm} \)
Spacing of distribution steel \( \text{spacd} = 175 \text{ mm} \)
Area of distribution steel  
\[ A_{\text{dis}} = \text{INT}(250 \times \pi \times \text{dia}^2 / \text{spacd}) \]  
\[ = 1795 \, \text{mm}^2 / \text{m} \]

Cover to reinforcement  
\[ c = 40 \, \text{mm} \]

Design crack width (short-term)  
\[ w_{ks} = \text{srmax} \times \text{ecrs} / 10^6 = 0.079958 \, \text{mm} \]

Design crack width (long-term)  
\[ w_{kl} = \text{srmax} \times \text{ecrl} / 10^6 = 0.20357 \, \text{mm} \]

The short-term crack width 0.079958 mm is less than the permissible value of 0.21 mm hence satisfactory.
The long-term crack width 0.20357 mm is less than the permissible value of 0.21 mm hence satisfactory.

**Arrangement of reinforcement**

(External edge restraint (Fig L.1(a), EN 1992-3))

Reinforcement shall be provided parallel to the restraint. Where the length to height ratio exceeds 2:1 this reinforcement shall be provided over the full height of the section. When the length to height ratio is 2:1 or less, and there is no end restraint, reinforcement need only be placed in the half of the section adjacent to the restraint.

The area of reinforcement, \( A_s = 1795 \, \text{mm}^2 / \text{m} \) shall be provided in the faces of the section.

In addition to the reinforcement parallel to the edge restraint, reinforcement of the same unit area shall also be provided to control the effects of warping restraint at the ends. This reinforcement shall be perpendicular to the edge restraint and be provided over a length at the end equivalent to the lesser of the height of the section, or 0.2 times the overall length.

**SUMMARY**

Section is subject to (external) edge restraint. Reinforcement \( A_s = 1795 \, \text{mm}^2 / \text{m} \) should be provided parallel to the edge restraint. Furthermore provide warping reinforcement.
Assessment of Half-Joints at Serviceability Limit State
to DoT Advice Note BA 39/93

Breadth of half-joint $b=1100$ mm
Depth of half-joint $h=710$ mm
Length of half-joint $k=540$ mm
Splay dimensions $s=100$ mm

Young's modulus of reinforcement $E_s=200000$ N/mm²
Number of reinforcement groups $nog=4$

Reinforcement group 1:
- Anti-clockwise angle from x axis $\text{ang}(1)=60°$
- x coordinate $x(1)=522.94$ mm
- y coordinate $y(1)=727.06$ mm
- Area of reinforcement $A_s(1)=2510$ mm²
- Diameter of bars in group $d(1)=20$ mm
- Spacing of bars in group $s(1)=140$ mm
- Effective area of reinforcement group normal to crack line $A_e(1)=A_s(i) \times (\cos(\pi/4 - \text{RAD}(\text{ang}(i))))^2=2341.9$ mm²

Reinforcement group 2:
- Anti-clockwise angle from x axis $\text{ang}(2)=60°$
- x coordinate $x(2)=571$ mm
- y coordinate $y(2)=679$ mm
- Area of reinforcement $A_s(2)=2510$ mm²
- Diameter of bars in group $d(2)=20$ mm
- Spacing of bars in group $s(2)=140$ mm
- Effective area of reinforcement group normal to crack line $A_e(2)=A_s(i) \times (\cos(\pi/4 - \text{RAD}(\text{ang}(i))))^2=2341.9$ mm²

Reinforcement group 3:
- Anti-clockwise angle from x axis $\text{ang}(3)=0°$
- y coordinate $y(3)=649$ mm
- Area of reinforcement $A_s(3)=452$ mm²
Diameter of bars in group d(3)=12 mm
Spacing of bars in group s(3)=275 mm
Effective area of reinforcement group normal to crack line
Ae(3)=As(i)*(COS(PI/4 -RAD(ang(i))))^2=226 mm²

Reinforcement group 4 :
Anti-clockwise angle from x axis ang(4)=90°
x coordinate x(4)=640 mm
Area of reinforcement As(4)=452 mm²
Diameter of bars in group d(4)=12 mm
Spacing of bars in group s(4)=275 mm
Effective area of reinforcement group normal to crack line
Ae(4)=As(i)*(COS(PI/4 -RAD(ang(i))))^2=226 mm²

Concrete cube strength fcu=30 N/mm²
Young's modulus Ec=28000 N/mm²

Vertical applied loading:
Load FAV(1)=-1057 kN
x coordinate xR(1)=235 mm
Horizontal applied loading
Number of applied horiz. loads noh=0
Partial safety factor for material strength at Serviceability
Limit State gamma m gm=1

SUMMARY
Concrete compressive strain 0.23398E-3
Crack width (from Expression 2) 0.4739 mm
**Location: Ex2 - Beam A**

**Assessment of Half-Joints at Serviceability Limit State**

_to DoT Advice Note BA 39/93_

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Breadth of half-joint</td>
<td>b=1100 mm</td>
</tr>
<tr>
<td>Depth of half-joint</td>
<td>h=710 mm</td>
</tr>
<tr>
<td>Length of half-joint</td>
<td>k=540 mm</td>
</tr>
<tr>
<td>Splay dimensions</td>
<td>s=100 mm</td>
</tr>
</tbody>
</table>

![Diagram of half-joint](image)

Breadth of half-joint 1100 mm

Young's modulus of reinforcement, \( E_s = 200000 \text{ N/mm}^2 \)

Number of reinforcement groups, \( n_{og} = 1 \)

Reinforcement group 1:
- Anti-clockwise angle from x axis, \( \text{ang}(1) = 0^\circ \)
- y coordinate, \( y(1) = 649 \text{ mm} \)
- Area of reinforcement, \( A_s(1) = 452 \text{ mm}^2 \)
- Diameter of bars in group, \( d(1) = 12 \text{ mm} \)
- Spacing of bars in group, \( s(1) = 275 \text{ mm} \)
- Effective area of reinforcement group normal to crack line, \( A_e(1) = A_s(1) \times (\cos(\pi/4 - \text{RAD(\text{ang}(1))}))^2 = 226 \text{ mm}^2 \)

**WARNING:**
Vertical or inclined reinforcement has not been specified.
**Location:** Ex3 - Beam B

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**Assessment of Half-Joints at Serviceability Limit State**

to DoT Advice Note BA 39/93

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**Breadth of half-joint**  \( b = 1100 \text{ mm} \)

**Depth of half-joint**  \( h = 710 \text{ mm} \)

**Length of half-joint**  \( k = 540 \text{ mm} \)

**Splay dimensions**  \( s = 100 \text{ mm} \)

---

**Young's modulus of reinforcement**  \( E_s = 200000 \text{ N/mm}^2 \)

**Number of reinforcement groups**  \( n_{og} = 4 \)

**Reinforcement group 1**

- **Anti-clockwise angle from x axis**  \( \text{ang}(1) = 60^\circ \)
- **x coordinate**  \( x(1) = 522.94 \text{ mm} \)
- **y coordinate**  \( y(1) = 727.06 \text{ mm} \)
- **Area of reinforcement**  \( A_s(1) = 2510 \text{ mm}^2 \)
- **Diameter of bars in group**  \( d(1) = 20 \text{ mm} \)
- **Spacing of bars in group**  \( s(1) = 140 \text{ mm} \)
- **Effective area of reinforcement group normal to crack line**  \( A_e(1) = A_s(1) \times (\cos(\pi/4 - \text{RAD}(\text{ang}(1))))^2 = 2341.9 \text{ mm}^2 \)

**Reinforcement group 2**

- **Anti-clockwise angle from x axis**  \( \text{ang}(2) = 60^\circ \)
- **x coordinate**  \( x(2) = 571 \text{ mm} \)
- **y coordinate**  \( y(2) = 679 \text{ mm} \)
- **Area of reinforcement**  \( A_s(2) = 2510 \text{ mm}^2 \)
- **Diameter of bars in group**  \( d(2) = 20 \text{ mm} \)
- **Spacing of bars in group**  \( s(2) = 140 \text{ mm} \)
- **Effective area of reinforcement group normal to crack line**  \( A_e(2) = A_s(2) \times (\cos(\pi/4 - \text{RAD}(\text{ang}(1))))^2 = 2341.9 \text{ mm}^2 \)

**Reinforcement group 3**

- **Anti-clockwise angle from x axis**  \( \text{ang}(3) = 0^\circ \)
- **y coordinate**  \( y(3) = 649 \text{ mm} \)
- **Area of reinforcement**  \( A_s(3) = 452 \text{ mm}^2 \)

---
Diameter of bars in group $d(3)=12 \text{ mm}$
Spacing of bars in group $s(3)=275 \text{ mm}$
Effective area of reinforcement group normal to crack line $A_\text{e}(3)=A_\text{s}(i)\cdot(\cos(\frac{\pi}{4}-\text{RAD}(\text{ang}(i))))^2=226 \text{ mm}^2$

Reinforcement group 4:
Anti-clockwise angle from x axis $\text{ang}(4)=90^\circ$
x coordinate $x(4)=640 \text{ mm}$
Area of reinforcement $A_\text{s}(4)=452 \text{ mm}^2$
Diameter of bars in group $d(4)=12 \text{ mm}$
Spacing of bars in group $s(4)=275 \text{ mm}$
Effective area of reinforcement group normal to crack line $A_\text{e}(4)=A_\text{s}(i)\cdot(\cos(\frac{\pi}{4}-\text{RAD}(\text{ang}(i))))^2=226 \text{ mm}^2$

Concrete cube strength $f_\text{cu}=30 \text{ N/mm}^2$
Young's modulus $E_\text{c}=28000 \text{ N/mm}^2$

Vertical applied loading:
Load $F_\text{AV}(1)=-100 \text{ kN}$
x coordinate $x_R(1)=200 \text{ mm}$

Horizontal applied loading
Number of applied horiz. loads $\text{no}_\text{h}=1$

Horizontal applied loading 1:
Load $F_\text{AH}(1)=1000 \text{ kN}$
y coordinate $y_H(1)=350 \text{ mm}$

WARNING:
Section is in compression. Crack width is zero.
Location: Ex4 - Beam C

Assessment of Half-Joints at Serviceability Limit State

to DoT Advice Note BA 39/93

Breadth of half-joint         b=1100 mm
Depth of half-joint            h=710 mm
Length of half-joint           k=540 mm
Splay dimensions              s=100 mm

Young's modulus of reinforcement          Es=200000 N/mm²
Number of reinforcement groups         nog=4
Reinforcement group 1 :
Anti-clockwise angle from x axis       ang(1)=60°
x coordinate                           x(1)=522.94 mm
y coordinate                           y(1)=727.06 mm
Area of reinforcement                  As(1)=2510 mm²
 Diameter of bars in group             d(1)=20 mm
 Spacing of bars in group              s(1)=140 mm
Effective area of reinforcement group normal to crack line
Ae(1)=As(i)*(COS(Pi/4-RAD(ang(i))))^2=2341.9 mm²

Reinforcement group 2 :
Anti-clockwise angle from x axis       ang(2)=60°
x coordinate                           x(2)=571 mm
y coordinate                           y(2)=679 mm
Area of reinforcement                  As(2)=2510 mm²
 Diameter of bars in group             d(2)=20 mm
 Spacing of bars in group              s(2)=140 mm
Effective area of reinforcement group normal to crack line
Ae(2)=As(i)*(COS(Pi/4-RAD(ang(i))))^2=2341.9 mm²

Reinforcement group 3 :
Anti-clockwise angle from x axis       ang(3)=0°
y coordinate                           y(3)=649 mm
Area of reinforcement                  As(3)=452 mm²
Diameter of bars in group \(d(3)=12\) mm
Spacing of bars in group \(s(3)=275\) mm
Effective area of reinforcement group normal to crack line \(Ae(3)=As(i)\cdot(COS(\pi/4-RAD(\text{ang}(i))))^2=226\) mm²
Reinforcement group 4:
Anti-clockwise angle from x axis \(\text{ang}(4)=90°\)
x coordinate \(x(4)=640\) mm
Area of reinforcement \(As(4)=452\) mm²
Diameter of bars in group \(d(4)=12\) mm
Spacing of bars in group \(s(4)=275\) mm
Effective area of reinforcement group normal to crack line \(Ae(4)=As(i)\cdot(COS(\pi/4-RAD(\text{ang}(i))))^2=226\) mm²
Concrete cube strength \(f_{cu}=30\) N/mm²
Young's modulus \(Ec=28000\) N/mm²
Vertical applied loading:
Load \(FAV(1)=-100\) kN
x coordinate \(xR(1)=200\) mm
Horizontal applied loading
Number of applied horiz. loads \(\text{noh}=1\)
Horizontal applied loading 1:
Load \(FAH(1)=-1000\) kN
y coordinate \(yH(1)=350\) mm

WARNING:
Section is in tension. Theory does not cover this case.
Assessment of Half-Joints at Serviceability Limit State

to DoT Advice Note BA 39/93

Breadth of half-joint 1100 mm

Young's modulus of reinforcement $E_s = 200000$ N/mm²
Number of reinforcement groups $nog = 4$

Reinforcement group 1:
- Anti-clockwise angle from x axis $\text{ang}(1) = 60°$
- x coordinate $x(1) = 522.94$ mm
- y coordinate $y(1) = 727.06$ mm
- Area of reinforcement $A_s(1) = 2510$ mm²
- Diameter of bars in group $d(1) = 20$ mm
- Spacing of bars in group $s(1) = 140$ mm
- Effective area of reinforcement group normal to crack line $A_e(1) = A_s(1) \times \left(\cos\left(\frac{\pi}{4} - \text{RAD}(\text{ang}(1))\right)\right)^2 = 2341.9$ mm²

Reinforcement group 2:
- Anti-clockwise angle from x axis $\text{ang}(2) = 60°$
- x coordinate $x(2) = 571$ mm
- y coordinate $y(2) = 679$ mm
- Area of reinforcement $A_s(2) = 2510$ mm²
- Diameter of bars in group $d(2) = 20$ mm
- Spacing of bars in group $s(2) = 140$ mm
- Effective area of reinforcement group normal to crack line $A_e(2) = A_s(2) \times \left(\cos\left(\frac{\pi}{4} - \text{RAD}(\text{ang}(2))\right)\right)^2 = 2341.9$ mm²

Reinforcement group 3:
- Anti-clockwise angle from x axis $\text{ang}(3) = 0°$
- y coordinate $y(3) = 649$ mm
- Area of reinforcement $A_s(3) = 452$ mm²
Diameter of bars in group  
Spacing of bars in group  
Effective area of reinforcement group normal to crack line  

Reinforcement group 4:
Anti-clockwise angle from x axis  
x coordinate  
Area of reinforcement  
Diameter of bars in group  
Spacing of bars in group  
Effective area of reinforcement group normal to crack line  

Concrete cube strength  
Young's modulus  

Vertical applied loading:
Load  
x coordinate  

Horizontal applied loading
Number of applied horiz. loads  
Horizontal applied loading 1:
Load  
y coordinate  

WARNING:
Re-entrant corner in compression. Theory does not cover this case.
Location: Ex6 - Beam E

Assessment of Half-Joints at Serviceability Limit State
to DoT Advice Note BA 39/93

Breadth of half-joint             b=1100 mm
Depth of half-joint               h=710 mm
Length of half-joint              k=540 mm
Splay dimensions                  s=100 mm

Young's modulus of reinforcement  Es=200000 N/mm²
Number of reinforcement groups    nog=4
Reinforcement group 1 :
Anti-clockwise angle from x axis  ang(1)=60°
x coordinate                      x(1)=522.94 mm
y coordinate                      y(1)=727.06 mm
Area of reinforcement             As(1)=2510 mm²
Diameter of bars in group         d(1)=20 mm
Spacing of bars in group          s(1)=140 mm
Effective area of reinforcement   Ae(1)=As(i)*(COS(PI/4-RAD(ang(i))))^2=2341.9 mm²

Reinforcement group 2 :
Anti-clockwise angle from x axis  ang(2)=60°
x coordinate                      x(2)=571 mm
y coordinate                      y(2)=679 mm
Area of reinforcement             As(2)=2510 mm²
Diameter of bars in group         d(2)=20 mm
Spacing of bars in group          s(2)=140 mm
Effective area of reinforcement   Ae(2)=As(i)*(COS(PI/4-RAD(ang(i))))^2=2341.9 mm²

Reinforcement group 3 :
Anti-clockwise angle from x axis  ang(3)=0°
y coordinate                      y(3)=649 mm
Area of reinforcement             As(3)=452 mm²
Diameter of bars in group         d(3)=12 mm
Spacing of bars in group          s(3)=275 mm
Effective area of reinforcement group normal to crack line
Ae(3)=As(i)*(COS(PI/4 -RAD(ang(i))))^2=226 mm²

Reinforcement group 4:
Anti-clockwise angle from x axis  ang(4)=90°
x coordinate                      x(4)=640 mm
Area of reinforcement             As(4)=452 mm²
Diameter of bars in group         d(4)=12 mm
Spacing of bars in group          s(4)=275 mm
Effective area of reinforcement group normal to crack line
Ae(4)=As(i)*(COS(PI/4 -RAD(ang(i))))^2=226 mm²

Concrete cube strength            fcu=30 N/mm²
Young's modulus                   Ec=28000 N/mm²

Vertical applied loading:
Load                              FAV(1)=-1057 kN
x coordinate                      xR(1)=235 mm
Horizontal applied loading
Number of applied horiz. loads    noh=1
Horizontal applied loading 1:
Load                              FAH(1)=-500 kN
y coordinate                      yH(1)=380 mm
Partial safety factor for material strength at Serviceability Limit State  gamma m  gm=1

SUMMARY

Concrete compressive strain  0.13455E-3
Crack width (from Expression 2)  0.58682 mm
Location: Ex1 - Beam A

Torsion in reinforced concrete sections to BS5400-4:1990

The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

Rectangular section - Clause 5.3.4.4(b)

Breadth of concrete section $b=600$ mm
Depth of concrete section $h=1200$ mm
Char. strength of concrete $f_{cu}=40$ N/mm$^2$
Smaller centre line link dim. $x_1=520$ mm
Larger centre line link dim. $y_1=1120$ mm
Torsion moment due to ult. loads $T=290$ kNm
Applied shear force $V=500$ kN
Tensile r'ment effective depth $d=1139$ mm

RESULTS FROM RECTANGULAR SECTION ANALYSIS FOR TORSION

Link r'ment adequate $A_{st}$ required 77.764 mm$^2$
$A_{st}$ provided 78.54 mm$^2$
Long r'ment adequate $A_{sl}$ required 605.01 mm$^2$
$A_{sl}$ provided 804.25 mm$^2$

TORSION REINFORCEMENT ADEQUATE
Location: Ex2 - Box Section A

Torsion in RC sections to BS5400-4:1990

The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

Box section - Clause 5.3.4.4(a)

![Diagram of box section]

Breadth of concrete section \( b = 2000 \text{ mm} \)
Depth of concrete section \( h = 1000 \text{ mm} \)
Char. strength of concrete \( f_{cu} = 40 \text{ N/mm}^2 \)
Smaller centre line link dim. \( x_1 = 900 \text{ mm} \)
Larger centre line link dim. \( y_1 = 1900 \text{ mm} \)
Torsion moment due to ult. loads \( T = 500 \text{ kNm} \)
Applied shear force to wall 2 \( V_2 = 50 \text{ kN} \)
Applied shear force to wall 4 \( V_4 = 75 \text{ kN} \)
Effective depth wall 2 \( d_2 = 920 \text{ mm} \)
Effective depth wall 4 \( d_4 = 900 \text{ mm} \)

RESULTS FROM BOX SECTION ANALYSIS FOR TORSION

Link r'ment adequate
- Ast required 131.56 mm²
- Ast provided 201.06 mm²

Long r'ment adequate
- Asl required 119.6 mm²
- Asl provided 314.16 mm²

TORSION REINFORCEMENT ADEQUATE
Location: Ex3 - Box Section B

Torsion in RC sections to BS5400-4:1990

The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

Box section - Clause 5.3.4.4(a)

<table>
<thead>
<tr>
<th>Wall 1</th>
<th>Wall 4</th>
<th>Wall 2</th>
<th>Wall 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>b=600 mm</td>
<td>h=500 mm</td>
<td>x1=500 mm</td>
<td>y1=400 mm</td>
</tr>
</tbody>
</table>

Breadth of concrete section b=600 mm
Depth of concrete section h=500 mm
Char. strength of concrete fcu=40 N/mm²
Smaller centre line link dim. x1=500 mm
Larger centre line link dim. y1=400 mm
Torsion moment due to ult. loads T=60 kNm
Applied shear force to wall 2 V2=10 kN
Applied shear force to wall 4 V4=20 kN
Effective depth wall 2 d2=420 mm
Effective depth wall 4 d4=440 mm

RESULTS FROM BOX SECTION ANALYSIS FOR TORSION

Link reinforcement inadequate : increase area or reduce spacing
Ast required 157.35 mm²
Ast provided 78.54 mm²

Long r'ment adequate Asl required 106.9 mm²
Asl provided 201.06 mm²

TORSION REINFORCEMENT INADEQUATE
Location: Ex4 - I Section

Torsion in RC sections to BS5400-4:1990

The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

I section - Clause 5.3.4.4(c)

Breadth of flange \( bf = 750 \text{ mm} \)
Breadth of web \( bw = 200 \text{ mm} \)
Depth of flange \( d_1 = 250 \text{ mm} \)
Depth of web \( d_2 = 1200 \text{ mm} \)
Torsion moment due to ult. loads \( T = 50 \text{ kNm} \)
Char. strength of concrete \( f_{cu} = 30 \text{ N/mm}^2 \)
Smaller centre line link dim. \( x_{1a} = 100 \text{ mm} \)
Larger centre line link dim. \( y_{1a} = 650 \text{ mm} \)
Torsional shear stress from Equation 9(a) \[
v_{ta} = \frac{2 \cdot T_{a} \cdot 1E6}{(h_{min_a}^2 \cdot (h_{max_a} - h_{min_a})/3)) = 0.85131 \text{ N/mm}^2\]
Smaller centre line link dim. \( x_{1b} = 100 \text{ mm} \)
Larger centre line link dim. \( y_{1b} = 1100 \text{ mm} \)
Torsional shear stress from Equation 9(a) \[
v_{tb} = \frac{2 \cdot T_{b} \cdot 1E6}{(h_{min_b}^2 \cdot (h_{max_b} - h_{min_b})/3)) = 0.64098 \text{ N/mm}^2\]
Applied shear force \( V = 150 \text{ kN} \)
Tensile r'ment effective depth \( d = 1600 \text{ mm} \)
Minimum ultimate torsion stress for which reinforcement is required (Table 10) \( v_{tmin} = \text{TABLE 1 for } f_{cu} = 30 \)
\( = 0.37 \text{ N/mm}^2 \)

RESULTS FROM I SECTION ANALYSIS FOR TORSION

Rectangle A:
Link reinforcement adequate
Ast required 74.571 mm²
Ast provided 113.1 mm²
Long. reinforcement adequate
Asl required 85.224 mm²
Asl provided 314.16 mm²

TORSION REINFORCEMENT ADEQUATE
### Rectangle B:

<table>
<thead>
<tr>
<th>Type</th>
<th>Ast required</th>
<th>Ast provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>Link reinforcement adequate</td>
<td>56.725 mm²</td>
<td>113.1 mm²</td>
</tr>
<tr>
<td>Long. reinforcement adequate</td>
<td>41.255 mm²</td>
<td>314.16 mm²</td>
</tr>
</tbody>
</table>

**TORSION REINFORCEMENT ADEQUATE**
Location: Ex5 - T Section

Torsion in RC sections to BS5400-4:1990

The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

T section - Clause 5.3.4.4(c)

Breadth of flange \( b_f = 1500 \text{ mm} \)
Breadth of web \( b_w = 350 \text{ mm} \)
Depth of flange \( d_1 = 450 \text{ mm} \)
Depth of web \( d_2 = 2000 \text{ mm} \)
Torsion moment due to ult. loads \( T = 150 \text{ kNm} \)
Char. strength of concrete \( f_{cu} = 30 \text{ N/mm}^2 \)
Smaller centre line link dim. \( x_{1a} = 350 \text{ mm} \)
Larger centre line link dim. \( y_{1a} = 1400 \text{ mm} \)
Torsional shear stress from Equation 9(a) \( v_{ta} = \frac{2T_a \times 10^6}{h_{mina}^2 \times (h_{maxa} - h_{mina}/3)} = 0.67435 \text{ N/mm}^2 \)
Smaller centre line link dim. \( x_{1b} = 250 \text{ mm} \)
Larger centre line link dim. \( y_{1b} = 1900 \text{ mm} \)
Torsional shear stress from Equation 9(a) \( v_{tb} = \frac{2T_b \times 10^6}{h_{minb}^2 \times (h_{maxb} - h_{minb}/3)} = 0.50128 \text{ N/mm}^2 \)
Applied shear force \( V = 100 \text{ kN} \)
Tensile r'ment effective depth \( d = 2250 \text{ mm} \)
Minimum ultimate torsion stress for which reinforcement is required (Table 10) \( v_{tmin} = \text{TABLE 1 for } f_{cu} = 30 = 0.37 \text{ N/mm}^2 \)

RESULTS FROM T SECTION ANALYSIS FOR TORSION

Rectangle A:
Link reinforcement adequate \( A_{st} \text{ required } 80.789 \text{ mm}^2 \)
\( A_{st} \text{ provided } 113.1 \text{ mm}^2 \)
Long. reinforcement adequate \( A_{sl} \text{ required } 58.756 \text{ mm}^2 \)
\( A_{sl} \text{ provided } 314.16 \text{ mm}^2 \)

TORSION REINFORCEMENT ADEQUATE
Rectangle B:

Link reinforcement adequate  
Ast required 52.283 mm²  
Ast provided 113.1 mm²

Long. reinforcement adequate  
A sl required 38.024 mm²  
A sl provided 314.16 mm²

TORSION REINFORCEMENT ADEQUATE
Location: Ex6 - L Section

Torsion in RC sections to BS5400-4:1990

The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

L section - Clause 5.3.4.4(c)

![Diagram of L section]

Breadth of flange \(bf=1500\text{ mm}\)
Breadth of web \(bw=350\text{ mm}\)
Depth of flange \(d1=400\text{ mm}\)
Depth of web \(d2=1500\text{ mm}\)
Torsion moment due to ult. loads \(T=150\text{ kNm}\)
Char. strength of concrete \(fcu=30\text{ N/mm}^2\)
Smaller centre line link dim. \(x1a=300\text{ mm}\)
Larger centre line link dim. \(y1a=1400\text{ mm}\)
Torsional shear stress from Equation 9(a) \(vta=2*Ta*1E6/(hmina^2*(hmaxa-hmina/3))=0.82157\text{ N/mm}^2\)
Smaller centre line link dim. \(x1b=250\text{ mm}\)
Larger centre line link dim. \(y1b=1400\text{ mm}\)
Torsional shear stress from Equation 9(a) \(vtb=2*Tb*1E6/(hminb^2*(hmaxb-hminb/3))=0.71021\text{ N/mm}^2\)
Applied shear force \(V=100\text{ kN}\)
Tensile r'ment effective depth \(d=1750\text{ mm}\)
Minimum ultimate torsion stress for which reinforcement is required (Table 10) \(vtmin=TABLE\text{ 1 for }fcu=30=0.37\text{ N/mm}^2\)

RESULTS FROM L SECTION ANALYSIS FOR TORSION

Rectangle A:
Link reinforcement adequate \(Ast\text{ required }91.85\text{ mm}^2\)
\(Ast\text{ provided }113.1\text{ mm}^2\)
Long. reinforcement adequate \(Asl\text{ required }66.8\text{ mm}^2\)
\(Asl\text{ provided }314.16\text{ mm}^2\)

TORSION REINFORCEMENT ADEQUATE
Rectangle B:
Link reinforcement adequate  Ast required 73.839 mm²
                        Ast provided 113.1 mm²
Long. reinforcement adequate  Asl required 53.701 mm²
                        Asl provided 314.16 mm²

TORSION REINFORCEMENT ADEQUATE
Location: Ex1 - Rectangular section

Torsion in RC sections to EN 1992-1-1:2004

Calculations are in accordance with BS EN 1992-1-1:2004 Clause 6.3 and The Concrete Centre (TCC) publication "Concise Eurocode 2 for Bridges". The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

The proforma does not include the following adjustment which may be made to the calculated area of torsion reinforcement:

The area of longitudinal reinforcement in the compressive zone may be reduced in proportion to the available compressive force. No guidance is given on what extent of compression zone should be considered, so it is recommended that a depth equal to twice the cover to the torsion links be considered, as was utilised in BS5400-4.

The angle $\theta$ for the compression struts (EC2-1-1, Figure 6.5) should be in the range given in the NA to EN 1992-1-1, Clause 6.2.3(2) such that $1 \leq \cot(\theta) \leq 2.5$ (i.e. angle $\theta$ values are $22^\circ$ - $45^\circ$).

Rectangular section

![Rectangular section diagram]

Breadth of concrete section $\text{b}=600$ mm
Depth of concrete section $\text{h}=1200$ mm
Smaller centre line link dim. $\text{x}_1=494$ mm
Larger centre line link dim. $\text{y}_1=1094$ mm
Design torsion moment $\text{T}_e=290$ kNm
Design torsional shear stress $\text{v}_t=2*\text{T}_e*1\text{E}6/(\text{h}_{\text{min}}^2*2*(\text{h}_{\text{max}}-\text{h}_{\text{min}}/3))=1.611 N/mm^2$
Design shear force $\text{V}_e=500$ kN
Tensile r'ment effective depth $\text{d}=1147$ mm
Angle of inclination of strut $\theta'=22^\circ$
Compute cot($\theta$) $\cot(\theta)=2.4751$
Compute tan($\theta$) $\tan(\theta)=0.40403$

Crushing limit for combined shear and torsion

The following check is in accordance with EN 1992-2 Clause 6.3.2(104).

Max torsional moment resistance $\text{T}_{\text{Rdmax}}=2*\text{v}_1*\text{fck}/1.5*\text{A}_k*\text{tef}^{*}\text{comp}_{1}/1\text{E}6=620.28$ kNm
Maximum shear resistance:
\[ VR_{d_{\text{max}}} = 0.36 \times b \times d \times \left(1 - \frac{f_{ck}}{250}\right) \times \frac{f_{ck}}{((ct+tt) \times 1000)} = 2401.2 \text{ kN} \]

Interaction condition \( \text{uni} = T_{Ed}/T_{Rd_{\text{max}}} + V_{Ed}/V_{Rd_{\text{max}}} = 0.67576 \)

Therefore concrete section is adequate.

RESULTS FROM RECTANGULAR SECTION ANALYSIS FOR TORSION

| Link reinforcement adequate | Ast required 33.669 mm² |
|                            | Ast provided 113.1 mm²  |
| Long. reinforcement adequate| Asl required 412.51 mm²  |
|                            | Asl provided 490.87 mm²  |

TORSION REINFORCEMENT ADEQUATE
Location: Ex2 - Box Section

Torsion in RC sections to EN 1992-1-1:2004

Calculations are in accordance with BS EN 1992-1-1:2004 Clause 6.3 and The Concrete Centre (TCC) publication "Concise Eurocode 2 for Bridges". The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

The proforma does not include the following adjustment which may be made to the calculated area of torsion reinforcement:

The area of longitudinal reinforcement in the compressive zone may be reduced in proportion to the available compressive force. No guidance is given on what extent of compression zone should be considered, so it is recommended that a depth equal to twice the cover to the torsion links be considered, as was utilised in BS5400-4.

The angle $\theta$ for the compression struts (EC2-1-1, Figure 6.5) should be in the range given in the NA to EN 1992-1-1, Clause 6.2.3(2) such that $1 \leq \cot(\theta) \leq 2.5$ (i.e. angle $\theta$ values are $22^\circ - 45^\circ$).

Box section

Breadth of concrete section $b=2000$ mm
Depth of concrete section $h=1000$ mm
Smaller centre line link dim. $x_1=900$ mm
Larger centre line link dim. $y_1=1900$ mm
Design torsion moment $T_{Ed}=500$ kNm
Design shear force to wall 2 $V_{Ed2}=50$ kN
Design shear force to wall 4 $V_{Ed4}=75$ kN
Effective depth wall 2 $d_2=920$ mm
Effective depth wall 4 $d_4=900$ mm
Angle of inclination of strut $\theta'=22^\circ$
Compute $\cot(\theta)$ $ct=\cot(\theta)=2.4751$
Compute $\tan(\theta)$ $tt=\tan(\theta)=0.40403$
Crushing limit for combined shear and torsion

The following check is in accordance with EN 1992-2 Clause 6.3.2(104).

Max torsional moment resistance

\[ TR_{d\text{max}} = \frac{2 \cdot v_1 \cdot f_{ck}}{1.5 \cdot A_k \cdot t_{ef} \cdot \text{comp} \cdot 1 \cdot 1_6} = 2278 \text{ kNm} \]

Maximum shear resistance:

\[ VR_{d\text{max}} = 0.36 \cdot h_w \cdot d_4 \cdot (1 - f_{ck}/250) \cdot f_{ck}/((c_t + t_t) \cdot 1000) = 863.55 \text{ kN} \]

Interaction condition

\[ \text{uni} = \frac{T_E d}{T_{d\text{max}}} + \frac{V_E d_4}{V_{d\text{max}}} = 0.30634 \]

Therefore concrete section is adequate.

RESULTS FROM BOX SECTION ANALYSIS FOR TORSION

| Link reinforcement adequate | Ast required 48.902 mm²  |
|                            | Ast provided 201.06 mm²  |
| Long.reinforcement adequate | Asl required 217.87 mm²  |
|                            | Asl provided 314.16 mm²  |

TORSION REINFORCEMENT ADEQUATE

Inner and outer link option

Provide inner and outer link legs to webs and flanges of the box section to resist torsion as detailed below:

Ast required per face =24.451 mm² (walls & flanges)
Ast provided per face =78.54 mm² (walls & flanges)
Link r'ment diameter =10 mm
Location: Ex3 - Box Section

Torsion in RC sections to EN 1992-1-1:2004

Calculations are in accordance with BS EN 1992-1-1:2004 Clause 6.3 and The Concrete Centre (TCC) publication "Concise Eurocode 2 for Bridges". The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

The proforma does not include the following adjustment which may be made to the calculated area of torsion reinforcement:

The area of longitudinal reinforcement in the compressive zone may be reduced in proportion to the available compressive force. No guidance is given on what extent of compression zone should be considered, so it is recommended that a depth equal to twice the cover to the torsion links be considered, as was utilised in BS5400-4.

The angle $\theta$ for the compression struts (EC2-1-1, Figure 6.5) should be in the range given in the NA to EN 1992-1-1, Clause 6.2.3(2) such that $1 \leq \cot(\theta) \leq 2.5$ (i.e. angle $\theta$ values are $22^\circ$ - $45^\circ$).

Box section

Breadth of concrete section $b=600$ mm
Depth of concrete section $h=500$ mm
Smaller centre line link dim. $x_1=500$ mm
Larger centre line link dim. $y_1=400$ mm
Design torsion moment $T_{Ed}=60$ kNm
Design shear force to wall $2$ $V_{Ed2}=10$ kN
Design shear force to wall $4$ $V_{Ed4}=20$ kN
Effective depth wall $2$ $d_2=420$ mm
Effective depth wall $4$ $d_4=440$ mm
Angle of inclination of strut $\theta'=22^\circ$
Compute $\cot(\theta)$ $ct=\cot(\theta)=2.4751$
Compute $\tan(\theta)$ $tt=\tan(\theta)=0.40403$
Crushing limit for combined shear and torsion

The following check is in accordance with EN 1992-2 Clause 6.3.2(104).
Max torsional moment resistance \( TR_{d_{max}} = \frac{2 \times v \times f_{ck}}{1.5 \times A_k \times t_{ef} \times c_{mpl1}} / 1 \times 10^6 \)
= 135.93 kNm

Maximum shear resistance:
\( VR_{d_{max}} = 0.36 \times h_w^4 \times d_4 \times (1 - \frac{f_{ck}}{250}) \times f_{ck} / ((c_t + t_t) \times 1000) = 214.93 \) kN

Interaction condition
\( uni = T_{Ed}/T_{Rd_{max}} + V_{Ed4}/V_{Rd_{max}} = 0.53446 \)
Therefore concrete section is adequate.

RESULTS FROM BOX SECTION ANALYSIS FOR TORSION

- Link reinforcement adequate
  - Ast required: 31.788 mm²
  - Ast provided: 113.1 mm²

- Long.reinforcement adequate
  - Asl required: 243.42 mm²
  - Asl provided: 314.16 mm²

TORSION REINFORCEMENT ADEQUATE
Torsion in RC sections to EN 1992-1-1:2004

Calculations are in accordance with BS EN 1992-1-1:2004 Clause 6.3 and The Concrete Centre (TCC) publication "Concise Eurocode 2 for Bridges". The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

The proforma does not include the following adjustment which may be made to the calculated area of torsion reinforcement:

The area of longitudinal reinforcement in the compressive zone may be reduced in proportion to the available compressive force. No guidance is given on what extent of compression zone should be considered, so it is recommended that a depth equal to twice the cover to the torsion links be considered, as was utilised in BS5400-4.

The angle \( \theta \) for the compression struts (EC2-1-1, Figure 6.5) should be in the range given in the NA to EN 1992-1-1, Clause 6.2.3(2) such that \( 1 \leq \cot(\theta) \leq 2.5 \) (i.e. angle \( \theta \) values are \( 22^\circ - 45^\circ \)).

I section

Breadth of flange \( bf \) = 750 mm
Breadth of web \( bw \) = 200 mm
Depth of flange \( d1 \) = 250 mm
Depth of web \( d2 \) = 1200 mm
Design torsion moment $T_{Ed}=50 \text{kNm}$
Smaller centre line link dim. $x_{la}=150 \text{ mm}$
Larger centre line link dim. $y_{la}=650 \text{ mm}$
Design torsional shear stress $v_{ta}=2T_{a}/h_{min}^2(h_{max}-h_{min})/3=0.82083 \text{ N/mm}^2$
Smaller centre line link dim. $x_{lb}=100 \text{ mm}$
Larger centre line link dim. $y_{lb}=1100 \text{ mm}$
Design torsional shear stress $v_{tb}=2T_{b}/h_{min}^2(h_{max}-h_{min})/3=0.6970 \text{ N/mm}^2$

Design shear force $V_{Ed}=150 \text{ kN}$

Tensile r'ment effective depth $d=1600 \text{ mm}$

Angle of inclination of strut $\theta'=22^\circ$

Compute cot($\theta$) $ct=\cot(\theta)=2.4751$
 Compute tan($\theta$) $tt=\tan(\theta)=0.40403$

Min ultimate torsion shear stress $v_{min}=\text{TABLE 1.1 for } f_{ck}=25$

$=0.37 \text{ N/mm}^2$

Max torsional moment resistance $T_{Rd_{max}}=2v_{1}f_{ck}/1.5A_{k}t_{ef}*\text{comp}1/1E6$

$=60.956 \text{ kNm}$

Max torsional moment resistance $T_{Rdmb}=2v_{1}f_{ck}/1.5A_{k}t_{ef}*\text{comp}1/1E6$

$=68.771 \text{ kNm}$

Crushing limit for combined shear and torsion

The following check is in accordance with EN 1992-2 Clause 6.3.2(104).

Max torsional moment resistance $T_{Rd_{max}}=60.956 \text{ kNm}$
Max torsional moment resistance $T_{Rdmb}=68.771 \text{ kNm}$
Minimum value governs $T_{Rd_{max}}=60.956 \text{ kNm}$

Maximum shear resistance:

$V_{Rd_{max}}=0.36bw*d*(1-f_{ck}/250)*f_{ck}/((ct+tt)*1000)=900.28 \text{ kN}$

Interaction condition $\text{uni}=T_{Ed}/T_{Rd_{max}}+V_{Ed}/V_{Rd_{max}}=0.98688$

Therefore concrete section is adequate.

RESULTS FROM I SECTION ANALYSIS FOR TORSION

**Rectangle A:**
Link reinforcement adequate
Long. reinforcement adequate

**TORSION REINFORCEMENT ADEQUATE**

**Rectangle B:**
Link reinforcement adequate
Long. reinforcement adequate

**TORSION REINFORCEMENT ADEQUATE**
Location: Ex5 - T Section

Torsion in RC sections to EN 1992-1-1:2004

Calculations are in accordance with BS EN 1992-1-1:2004 Clause 6.3 and The Concrete Centre (TCC) publication "Concise Eurocode 2 for Bridges". The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

The proforma does not include the following adjustment which may be made to the calculated area of torsion reinforcement:

The area of longitudinal reinforcement in the compressive zone may be reduced in proportion to the available compressive force. No guidance is given on what extent of compression zone should be considered, so it is recommended that a depth equal to twice the cover to the torsion links be considered, as was utilised in BS5400-4.

The angle θ for the compression struts (EC2-1-1, Figure 6.5) should be in the range given in the NA to EN 1992-1-1, Clause 6.2.3(2) such that $1 \leq \cot(\theta) \leq 2.5$ (i.e. angle θ values are 22° - 45°).

T section

Breadth of flange $bf=1500$ mm
Breadth of web $bw=350$ mm
Depth of flange $d1=450$ mm
Depth of web $d2=2000$ mm
Design torsion moment $TEd=150$ kNm
Smaller centre line link dim. $x1a=350$ mm
Larger centre line link dim. $y1a=1400$ mm
Design torsional shear stress $vta=2*Ta*1E6/(hmina^2*(hmaxa-hmina/3))=0.65012$ N/mm²
Smaller centre line link dim. $x1b=250$ mm
Larger centre line link dim. $y1b=1900$ mm
Design torsional shear stress $vtb=2*Tb*1E6/(hminb^2*(hmaxb-hminb/3))=0.52999$ N/mm²
Design shear force $VEd=100$ kN
Tensile r'ment effective depth \( d = 2250 \text{ mm} \)

Angle of inclination of strut \( \theta' = 22^\circ \)

Compute \( \cot(\theta) \)
\[
\cot(\theta) = 2.4751
\]

Compute \( \tan(\theta) \)
\[
\tan(\theta) = 0.40403
\]

Min ultimate torsion shear stress \( \nu_{\text{min}} = \text{TABLE 1.1 for } f_{\text{ck}} = 25 \)
\[
= 0.37 \text{ N/mm}^2
\]

Max torsional moment resistance \( TR_{\text{dma}} = 2 \times v_1 \times f_{\text{ck}} / 1.5 \times A_k \times t_{\text{ef}} \times \text{compl} / 1 \times 10^6 \)
\[
= 397.61 \text{ kNm}
\]

Max torsional moment resistance \( TR_{\text{dmb}} = 2 \times v_1 \times f_{\text{ck}} / 1.5 \times A_k \times t_{\text{ef}} \times \text{compl} / 1 \times 10^6 \)
\[
= 346.55 \text{ kNm}
\]

Crushing limit for combined shear and torsion

The following check is in accordance with EN 1992-2 Clause 6.3.2(104).

Max torsional moment resistance \( TR_{\text{dmax}} = 397.61 \text{ kNm} \)

Max torsional moment resistance \( TR_{\text{dmax}} = 346.55 \text{ kNm} \)

Minimum value governs \( TR_{\text{dmax}} = 346.55 \text{ kNm} \)

Maximum shear resistance:
\[
VR_{\text{dmax}} = 0.36 \times b_w \times d \times (1 - f_{\text{ck}} / 250) \times f_{\text{ck}} / ((\cot(\theta) + \tan(\theta)) \times 1000) = 2215.5 \text{ kN}
\]

Interaction condition \( \text{uni} = T_{\text{Ed}} / TR_{\text{dmax}} + V_{\text{Ed}} / VR_{\text{dmax}} = 0.47797 \)

Therefore concrete section is adequate.

RESULTS FROM T SECTION ANALYSIS FOR TORSION

**Rectangle A:**

- Link reinforcement adequate
  - \( A_{st} \) required 30.885 mm²
  - \( A_{st} \) provided 113.1 mm²

- Long. reinforcement adequate
  - \( A_{sl} \) required 137.6 mm²
  - \( A_{sl} \) provided 314.16 mm²

**TORSION REINFORCEMENT ADEQUATE**

**Rectangle B:**

- Link reinforcement adequate
  - \( A_{st} \) required 20.978 mm²
  - \( A_{st} \) provided 113.1 mm²

- Long. reinforcement adequate
  - \( A_{sl} \) required 93.464 mm²
  - \( A_{sl} \) provided 314.16 mm²

**TORSION REINFORCEMENT ADEQUATE**
Location: Ex6 - L Section

Torsion in RC sections to EN 1992-1-1:2004

Calculations are in accordance with BS EN 1992-1-1:2004 Clause 6.3 and The Concrete Centre (TCC) publication "Concise Eurocode 2 for Bridges". The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

The proforma does not include the following adjustment which may be made to the calculated area of torsion reinforcement:

The area of longitudinal reinforcement in the compressive zone may be reduced in proportion to the available compressive force. No guidance is given on what extent of compression zone should be considered, so it is recommended that a depth equal to twice the cover to the torsion links be considered, as was utilised in BS5400-4.

The angle \( \theta \) for the compression struts (EC2-1-1, Figure 6.5) should be in the range given in the NA to EN 1992-1-1, Clause 6.2.3(2) such that \( 1 \leq \cot(\theta) \leq 2.5 \) (i.e. angle \( \theta \) values are \( 22^\circ - 45^\circ \)).

L section

Breadth of flange \( \text{bf} = 1500 \text{ mm} \)
Breadth of web \( \text{bw} = 350 \text{ mm} \)
Depth of flange \( d_1 = 400 \text{ mm} \)
Depth of web \( d_2 = 1500 \text{ mm} \)
Design torsion moment \( T_E = 150 \text{ kNm} \)
Smaller centre line link dim. \( x_{1a} = 300 \text{ mm} \)
Larger centre line link dim. \( y_{1a} = 1400 \text{ mm} \)
Design torsional shear stress \( v_{ta} = \frac{2*Ta*1E6}{h_{mina}^2*(h_{maxa}-h_{mina}/3)} = 0.81289 \text{ N/mm}^2 \)
Smaller centre line link dim. \( x_{1b} = 250 \text{ mm} \)
Larger centre line link dim. \( y_{1b} = 1400 \text{ mm} \)
Design torsional shear stress \( v_{tb} = \frac{2*Tb*1E6}{h_{minb}^2*(h_{maxb}-h_{minb}/3)} = 0.72141 \text{ N/mm}^2 \)
Design shear force \( V_E = 100 \text{ kN} \)
Tensile r'ment effective depth \( d = 1750 \text{ mm} \)
Angle of inclination of strut \( \theta' = 22^\circ \)
Compute \( \cot(\theta) \) \( \cot(\theta) = 2.4751 \)
Compute \( \tan(\theta) \) \( \tan(\theta) = 0.40403 \)
Min ultimate torsion shear stress \( v_{t\text{min}} = \text{TABLE 1.1 for } f_{ck} = 25 \)
\[ = 0.37 \text{ N/mm}^2 \]
Max torsional moment resistance \( T_{R\text{dma}} = 2 \times v_1 \times f_{ck} / 1.5 \times A_k \times t_{ef} \times \text{comp} / 1E6 \)
\[ = 320.75 \text{ kNm} \]
Max torsional moment resistance \( T_{R\text{dmb}} = 2 \times v_1 \times f_{ck} / 1.5 \times A_k \times t_{ef} \times \text{comp} / 1E6 \)
\[ = 250.72 \text{ kNm} \]

**Crushing limit for combined shear and torsion**

The following check is in accordance with EN 1992-2 Clause 6.3.2(104).

Max torsional moment resistance \( T_{R\text{dma}} = 320.75 \text{ kNm} \)
Max torsional moment resistance \( T_{R\text{dmb}} = 250.72 \text{ kNm} \)
Minimum value governs \( T_{R\text{dmax}} = 250.72 \text{ kNm} \)
Maximum shear resistance:
\[ V_{R\text{dmax}} = 0.36 \times b_w \times d \times (1 - f_{ck}/250) \times f_{ck} / ((\cot(\theta) + \tan(\theta)) \times 1000) = 1723.2 \text{ kN} \]
Interaction condition \( \text{uni} = T_{Ed} / T_{R\text{dmax}} + V_{Ed} / V_{R\text{dmax}} = 0.6563 \)
Therefore concrete section is adequate.

**RESULTS FROM L SECTION ANALYSIS FOR TORSION**

**Rectangle A:**
- Link reinforcement adequate
  - Ast required 34.931 mm²
  - Ast provided 113.1 mm²
- Long. reinforcement adequate
  - Asl required 155.63 mm²
  - Asl provided 314.16 mm²
**TORSION REINFORCEMENT ADEQUATE**

**Rectangle B:**
- Link reinforcement adequate
  - Ast required 27.619 mm²
  - Ast provided 113.1 mm²
- Long. reinforcement adequate
  - Asl required 123.05 mm²
  - Asl provided 314.16 mm²
**TORSION REINFORCEMENT ADEQUATE**
Torsion in RC sections to EN 1992-1-1:2004

Calculations are in accordance with BS EN 1992-1-1:2004 Clause 6.3 and The Concrete Centre (TCC) publication "Concise Eurocode 2 for Bridges". The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

The proforma does not include the following adjustment which may be made to the calculated area of torsion reinforcement:

The area of longitudinal reinforcement in the compressive zone may be reduced in proportion to the available compressive force. No guidance is given on what extent of compression zone should be considered, so it is recommended that a depth equal to twice the cover to the torsion links be considered, as was utilised in BS5400-4.

The angle $\theta$ for the compression struts (EC2-1-1, Figure 6.5) should be in the range given in the NA to EN 1992-1-1, Clause 6.2.3(2) such that $1 \leq \cot(\theta) \leq 2.5$ (i.e. angle $\theta$ values are $22^\circ$ - $45^\circ$).

**Rectangular section**

\[
\text{Breadth of concrete section} \quad b = 300 \text{ mm} \\
\text{Depth of concrete section} \quad h = 600 \text{ mm} \\
\text{Smaller centre line link dim.} \quad x_1 = 224 \text{ mm} \\
\text{Larger centre line link dim.} \quad y_1 = 524 \text{ mm} \\
\text{Design torsion moment} \quad T_{Ed} = 24 \text{ kNm} \\
\text{Design torsional shear stress} \quad v_t = 2 \times T_{Ed} \times 10^6 / (h_{\text{min}}^2 \times (h_{\text{max}} - h_{\text{min}}/3)) = 1.0667 \text{ N/mm}^2 \\
\text{Max ultimate torsion shear stress} \quad v_{tu} = \text{TABLE 2.1 for fck=30} = 4.5643 \text{ N/mm}^2 \\
\text{Limiting torsion shear stress} \quad l_{ts} = v_{tu} \times y_1/550 = 4.3485 \text{ N/mm}^2 \\
\text{Design shear force} \quad V_{Ed} = 250 \text{ kN} \\
\text{Tensile r'ment effective depth} \quad d = 540 \text{ mm} \\
\text{Angle of inclination of strut} \quad \theta' = 22^\circ \\
\text{Compute } \cot(\theta) \quad c_t = \cot(\theta) = 2.4751 \\
\text{Compute } \tan(\theta) \quad t_t = \tan(\theta) = 0.40403
Crushing limit for combined shear and torsion

The following check is in accordance with EN 1992-2 Clause 6.3.2(104).

Max torsional moment resistance: \[ TR_{d_{\text{max}}} = 2 \times v_1 \times f_{ck} / 1.5 \times A_k \times t_{ef} \times \text{comp1} / 1 \times 10^6 \]

= 73.356 kNm

Maximum shear resistance:

\[ VR_{d_{\text{max}}} = 0.36 \times b \times d \times (1 - f_{ck} / 250) \times f_{ck} / ((c_t + t_t) \times 1000) = 534.76 \text{ kN} \]

Interaction condition

\[ \text{uni} = T_{Ed} / TR_{d_{\text{max}}} + V_{Ed} / VR_{d_{\text{max}}} = 0.79467 \]

Therefore concrete section is adequate.

RESULTS FROM RECTANGULAR SECTION ANALYSIS FOR TORSION

<table>
<thead>
<tr>
<th>Link reinforcement adequate</th>
<th>Ast required 13.932 mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ast provided 50.265 mm²</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Long. reinforcement adequate</th>
<th>Asl required 170.7 mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asl provided 201.06 mm²</td>
<td></td>
</tr>
</tbody>
</table>

TORSION REINFORCEMENT ADEQUATE
Location: Ex8 - Box Section (torsion reinforcement not required)

Torsion in RC sections to EN 1992-1-1:2004

Calculations are in accordance with BS EN 1992-1-1:2004 Clause 6.3 and The Concrete Centre (TCC) publication "Concise Eurocode 2 for Bridges". The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

The proforma does not include the following adjustment which may be made to the calculated area of torsion reinforcement:

- The area of longitudinal reinforcement in the compressive zone may be reduced in proportion to the available compressive force.
- No guidance is given on what extent of compression zone should be considered, so it is recommended that a depth equal to twice the cover to the torsion links be considered, as was utilised in BS5400-4.

The angle $\theta$ for the compression struts (EC2-1-1, Figure 6.5) should be in the range given in the NA to EN 1992-1-1, Clause 6.2.3(2) such that $1 \leq \cot(\theta) \leq 2.5$ (i.e. angle $\theta$ values are $22^\circ$ - $45^\circ$).

Box section

- Breadth of concrete section  $b=2000$ mm
- Depth of concrete section  $h=2000$ mm
- Wall 1 thickness  $h_{w1}=400$ mm
- Smaller centre line link dim.  $x_1=1900$ mm
- Larger centre line link dim.  $y_1=1900$ mm
- Design torsion moment  $T_{Ed}=500$ kNm
- Design shear force  $V_{Ed}=100$ kN
- Effective depth wall 2  $d_2=1920$ mm
- Effective depth wall 4  $d_4=1920$ mm
- Angle of inclination of strut  $\theta'=22^\circ$
- Compute $\cot(\theta)$  $c_t=\cot(\theta)=2.4751$
- Compute $\tan(\theta)$  $t_t=\tan(\theta)=0.40403$
RESULTS FROM BOX SECTION ANALYSIS FOR TORSION

TORSION REINFORCEMENT NOT REQUIRED
Location: Ex9 - I Section

Torsion in RC sections to EN 1992-1-1:2004

Calculations are in accordance with BS EN 1992-1-1:2004 Clause 6.3 and The Concrete Centre (TCC) publication "Concise Eurocode 2 for Bridges". The areas of longitudinal and link reinforcement calculated are for torsional strength only. The reinforcement is additional to any requirements for shear and bending but may contribute to the area of reinforcement required for the control of early thermal cracking.

The proforma does not include the following adjustment which may be made to the calculated area of torsion reinforcement:

The area of longitudinal reinforcement in the compressive zone may be reduced in proportion to the available compressive force. No guidance is given on what extent of compression zone should be considered, so it is recommended that a depth equal to twice the cover to the torsion links be considered, as was utilised in BS5400-4.

The angle \( \theta \) for the compression struts (EC2-1-1, Figure 6.5) should be in the range given in the NA to EN 1992-1-1, Clause 6.2.3(2) such that \( 1 \leq \cot(\theta) \leq 2.5 \) (i.e. angle \( \theta \) values are \( 22^\circ - 45^\circ \)).

I section

- Breadth of flange \( bf=1800 \text{ mm} \)
- Breadth of web \( bw=500 \text{ mm} \)
- Depth of flange \( d_1=500 \text{ mm} \)
- Depth of web \( d_2=1800 \text{ mm} \)
Design torsion moment $T_{Ed}=140 \text{ kNm}$
Smaller centre line link dim. $x_{la}=400 \text{ mm}$
Larger centre line link dim. $y_{la}=1700 \text{ mm}$
Design torsional shear stress $\tau_{ta}=2\times T_{a}\times 1\times 10^6/\left(h_{min}^2\times \left(h_{max} - h_{min}/3\right)\right)=0.22857 \text{ N/mm}^2$
Smaller centre line link dim. $x_{lb}=400 \text{ mm}$
Larger centre line link dim. $y_{lb}=1700 \text{ mm}$
Design torsional shear stress $\tau_{tb}=2\times T_{b}\times 1\times 10^6/\left(h_{min}^2\times \left(h_{max} - h_{min}/3\right)\right)=0.22857 \text{ N/mm}^2$
Design shear force $V_{Ed}=150 \text{ kN}$
Tensile r'ment effective depth $d=2700 \text{ mm}$
Angle of inclination of strut $\theta'=22^\circ$
Compute $\cot(\theta)$ $ct=\cot(\theta)=2.4751$
Compute $\tan(\theta)$ $tt=\tan(\theta)=0.40403$
Min ultimate torsion shear stress $\tau_{min}=\text{TABLE 1.1 for fck=40}$ $=0.42 \text{ N/mm}^2$

RESULTS FROM I SECTION ANALYSIS FOR TORSION

Rectangle A:
TORISON REINFORCEMENT NOT REQUIRED

Rectangle B:
TORISON REINFORCEMENT NOT REQUIRED
Location: Ex1 - Beam A

Torsional strength calculation to Departmental Standard BD 44/15

Rectangular section ( Clause 5.3.4.4(b) )
Breadth of concrete section          b=600 mm
Depth of concrete section           h=1200 mm
Strength of concrete                fcu=40 N/mm²
Factor for concrete                 gmc=1.5
Applied shear force                 V=500 kN
Tensile r'ment effective depth     d=1139 mm
Strength of links                   fyv=460 N/mm²
Strength of long. r'ment            fyl=460 N/mm²
Factor for reinforcement            gms=1.15
Area of all longitudinal reinf.     sumAl=5150 mm²
Area of one leg of closed link      Ast=201 mm²
Smaller centre-line dim. of link    x1=520 mm
Larger centre-line dim. of link     y1=1120 mm
Average spacing of long. reinf.     sl=200 mm
Spacing of links along member       sv=100 mm

RESULTS FROM TORSION ASSESSMENT

Assessed torsional strength      662.16 kNm
Location: Ex2 - Box Section A

Torsional strength calculation to Departmental Standard BD 44/15

Box section (Clause 5.3.4.4(a))

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of concrete section</td>
<td>( b = 2000 \text{ mm} )</td>
</tr>
<tr>
<td>Depth of concrete section</td>
<td>( h = 1000 \text{ mm} )</td>
</tr>
<tr>
<td>Wall 1 thickness</td>
<td>( h_{w1} = 250 \text{ mm} )</td>
</tr>
<tr>
<td>Wall 2 thickness</td>
<td>( h_{w2} = 300 \text{ mm} )</td>
</tr>
<tr>
<td>Wall 3 thickness</td>
<td>( h_{w3} = 225 \text{ mm} )</td>
</tr>
<tr>
<td>Wall 4 thickness</td>
<td>( h_{w4} = 275 \text{ mm} )</td>
</tr>
<tr>
<td>Strength of concrete</td>
<td>( f_{cu} = 40 \frac{N}{mm^2} )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor for concrete</td>
<td>( g_{mc} = 1.5 )</td>
</tr>
<tr>
<td>Applied shear force to wall 2</td>
<td>( V_2 = 50 \text{ kN} )</td>
</tr>
<tr>
<td>Applied shear force to wall 4</td>
<td>( V_4 = 75 \text{ kN} )</td>
</tr>
<tr>
<td>Effective depth wall 2</td>
<td>( d_2 = 920 \text{ mm} )</td>
</tr>
<tr>
<td>Effective depth wall 4</td>
<td>( d_4 = 900 \text{ mm} )</td>
</tr>
<tr>
<td>Strength of links</td>
<td>( f_{yv} = 460 \frac{N}{mm^2} )</td>
</tr>
<tr>
<td>Strength of long. r'ment</td>
<td>( f_{yl} = 460 \frac{N}{mm^2} )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor for reinforcement</td>
<td>( g_{ms} = 1.15 )</td>
</tr>
<tr>
<td>Area of all longitudinal reinf.</td>
<td>( \text{sumAl} = 7034 \text{ mm}^2 )</td>
</tr>
<tr>
<td>Area of one leg of closed link</td>
<td>( A_{st} = 113 \text{ mm}^2 )</td>
</tr>
<tr>
<td>Smaller centre-line dim. of link</td>
<td>( x_1 = 900 \text{ mm} )</td>
</tr>
<tr>
<td>Larger centre-line dim. of link</td>
<td>( y_1 = 1900 \text{ mm} )</td>
</tr>
<tr>
<td>Average spacing of long. reinf.</td>
<td>( s_l = 250 \text{ mm} )</td>
</tr>
<tr>
<td>Spacing of links along member</td>
<td>( s_v = 275 \text{ mm} )</td>
</tr>
</tbody>
</table>

RESULTS FROM TORSION ASSESSMENT

Assessed torsional strength 750.48 kNm
Location: Ex3 - Box Section B

Torsional strength calculation to Departmental Standard BD 44/15

Box section (Clause 5.3.4.4(a))

- Breadth of concrete section: b = 600 mm
- Depth of concrete section: h = 500 mm
- Wall 1 thickness: hw1 = 100 mm
- Wall 2 thickness: hw2 = 125 mm
- Wall 3 thickness: hw3 = 150 mm
- Wall 4 thickness: hw4 = 140 mm
- Strength of concrete: fcu = 40 N/mm²

- Factor for concrete: gmc = 1.5
- Applied shear force to wall 2: V2 = 10 kN
- Applied shear force to wall 4: V4 = 20 kN
- Effective depth wall 2: d2 = 420 mm
- Effective depth wall 4: d4 = 440 mm
- Strength of links: fyv = 250 N/mm²
- Strength of long. r'ment: fyl = 460 N/mm²

- Factor for reinforcement: gms = 1.15
- Area of all longitudinal reinf.: sumAl = 1447 mm²
- Area of one leg of closed link: Ast = 78 mm²
- Smaller centre-line dim. of link: x1 = 400 mm
- Larger centre-line dim. of link: y1 = 500 mm
- Average spacing of long. reinf.: sl = 250 mm
- Spacing of links along member: sv = 200 mm

RESULTS FROM TORSION ASSESSMENT

- Assessed torsional strength: 57.893 kNm
Location: Ex4 - I Section

Torsional strength calculation to Departmental Standard BD 44/15

I section ( Clause 5.3.4.4(c) )

- Breadth of flange: err = 750 mm
- Breadth of web: bw = 200 mm
- Depth of flange: d1 = 250 mm
- Depth of web: d2 = 1200 mm
- Strength of concrete: fcu = 30 N/mm²
- Factor for concrete: gmc = 1.5

Rectangle A

- Strength of links: fyva = 460 N/mm²
- Strength of long. r'ment: fyla = 460 N/mm²
- Factor for reinforcement: gmsa = 1.15
- Area of all longitudinal reinf.: sumAla = 2355 mm²
- Area of one leg of closed link: Asta = 113 mm²
- Smaller centre-line dim. of link: x1a = 100 mm
- Larger centre-line dim. of link: y1a = 650 mm
- Average spacing of long. reinf.: sla = 200 mm
- Spacing of links along member: sva = 175 mm

Rectangle B

- Applied shear force: Vb = 150 kN
- Tensile r'ment effective depth: db = 1600 mm
- Strength of links: fyvb = 460 N/mm²
- Strength of long. r'ment: fylb = 460 N/mm²
- Factor for reinforcement: gmsb = 1.15
- Area of all longitudinal reinf.: sumAlb = 3768 mm²
- Area of one leg of closed link: Astb = 113 mm²
- Smaller centre-line dim. of link: x1b = 100 mm
- Larger centre-line dim. of link: y1b = 1100 mm
- Average spacing of long. reinf.: slb = 200 mm
- Spacing of links along member: svb = 275 mm

Rectangle C

- Strength of links: fyvc = 460 N/mm²
- Strength of long. r'ment: fylc = 460 N/mm²
- Factor for reinforcement: gmsc = 1.15
- Area of all longitudinal reinf.: sumAlc = 1020 mm²
- Area of one leg of closed link: Astc = 113 mm²
- Smaller centre-line dim. of link: x1c = 150 mm
- Larger centre-line dim. of link: y1c = 175 mm
- Average spacing of long. reinf.: slc = 200 mm
- Spacing of links along member: svc = 175 mm

WARNING:
Spacing of links 175 mm is > than (x1c+y1c)/4=81.25 mm
Link spacing does not comply with Cl. 5.3.4.5 requirements.
Rectangle D

Applied shear force               Vd=150 kN
Tensile r'ment effective depth    dd=1600 mm
Strength of links                 fyvd=460 N/mm²
Strength of long. r'ment          fyld=460 N/mm²

Factor for reinforcement          gmsd=1.15
Area of all longitudinal r'ment   sumAld=5338 mm²
Area of one leg of closed link    Astd=113 mm²
Larger centre-line dim. of link   y1d=1600 mm
Smaller centre-line dim. of link  x1d=100 mm
Larger centre-line dim. of link   y1d=1600 mm
Average spacing of long. r'ment   sld=200 mm
Spacing of links along member     svd=275 mm

RESULTS FROM TORSION ASSESSMENT

Option 1: Assessed torsional strengths
Rectangle A                    41.885 kNm
Rectangle B                    56.545 kNm
Assessed torsional strength    140.32 kNm

Option 2:
Assessed torsional strengths
Rectangle C                    7.8411 kNm

WARNING:
Link spacing does not comply with Cl. 5.3.4.5 requirements.

Rectangle D                    82.247 kNm
Assessed torsional strength    113.61 kNm
Location: Ex5 - T Section

Torsional strength calculation to Departmental Standard BD 44/15

T section (Clause 5.3.4.4 (c))
Breadth of flange \( b_f = 1500 \text{ mm} \)
Breadth of web \( b_w = 350 \text{ mm} \)
Depth of flange \( d_1 = 450 \text{ mm} \)
Depth of web \( d_2 = 2000 \text{ mm} \)
Strength of concrete \( f_{cu} = 30 \text{ N/mm}^2 \)
Factor for concrete \( g_{mc} = 1.5 \)

Rectangle A
Strength of links \( f_{yva} = 460 \text{ N/mm}^2 \)
Strength of long. r'ment \( f_{yla} = 460 \text{ N/mm}^2 \)
Factor for reinforcement \( g_{msa} = 1.15 \)
Area of all longitudinal reinf. \( \sum A_{la} = 5495 \text{ mm}^2 \)
Area of one leg of closed link \( A_{ta} = 113 \text{ mm}^2 \)
Smaller centre-line dim. of link \( x_{1a} = 350 \text{ mm} \)
Larger centre-line dim. of link \( y_{1a} = 1400 \text{ mm} \)
Average spacing of long. reinf. \( s_{la} = 200 \text{ mm} \)
Spacing of links along member \( s_{va} = 275 \text{ mm} \)

Rectangle B
Applied shear force \( V_b = 75 \text{ kN} \)
Tensile r'ment effective depth \( d_b = 2300 \text{ mm} \)
Strength of links \( f_{yvb} = 460 \text{ N/mm}^2 \)
Strength of long. r'ment \( f_{ylb} = 460 \text{ N/mm}^2 \)
Factor for reinforcement \( g_{msb} = 1.15 \)
Area of all longitudinal reinf. \( \sum A_{lb} = 6751 \text{ mm}^2 \)
Area of one leg of closed link \( A_{tb} = 113 \text{ mm}^2 \)
Smaller centre-line dim. of link \( x_{1b} = 250 \text{ mm} \)
Larger centre-line dim. of link \( y_{1b} = 1900 \text{ mm} \)
Average spacing of long. reinf. \( s_{lb} = 200 \text{ mm} \)
Spacing of links along member \( s_{vb} = 275 \text{ mm} \)

Rectangle C
Strength of links \( f_{yvc} = 460 \text{ N/mm}^2 \)
Strength of long. r'ment \( f_{ylc} = 460 \text{ N/mm}^2 \)
Factor for reinforcement \( g_{msc} = 1.15 \)
Area of all longitudinal reinf. \( \sum A_{lc} = 2669 \text{ mm}^2 \)
Area of one leg of closed link \( A_{tc} = 113 \text{ mm}^2 \)
Smaller centre-line dim. of link \( x_{1c} = 350 \text{ mm} \)
Larger centre-line dim. of link \( y_{1c} = 500 \text{ mm} \)
Average spacing of long. reinf. \( s_{lc} = 200 \text{ mm} \)
Spacing of links along member \( s_{vc} = 275 \text{ mm} \)

WARNING:
Spacing of links 275 mm is > than \((x_{1c}+y_{1c})/4=212.5 \text{ mm}\)
Link spacing does not comply with Cl. 5.3.4.5 requirements.
Rectangle D

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied shear force</td>
<td>$V_d = 75$ kN</td>
</tr>
<tr>
<td>Tensile r'ment effective depth</td>
<td>$d_d = 2300$ mm</td>
</tr>
<tr>
<td>Strength of links</td>
<td>$f_{vyd} = 460$ N/mm$^2$</td>
</tr>
<tr>
<td>Strength of long. r'ment</td>
<td>$f_{yld} = 460$ N/mm$^2$</td>
</tr>
<tr>
<td>Factor for reinforcement</td>
<td>$g_{msd} = 1.15$</td>
</tr>
<tr>
<td>Area of all longitudinal r'ment</td>
<td>$s_{um Ald} = 8164$ mm$^2$</td>
</tr>
<tr>
<td>Area of one leg of closed link</td>
<td>$A_{std} = 113$ mm$^2$</td>
</tr>
<tr>
<td>Larger centre-line dim. of link</td>
<td>$y_{ld} = 2350$ mm</td>
</tr>
<tr>
<td>Smaller centre-line dim. of link</td>
<td>$x_{ld} = 250$ mm</td>
</tr>
<tr>
<td>Larger centre-line dim. of link</td>
<td>$y_{ld} = 2350$ mm</td>
</tr>
<tr>
<td>Average spacing of long. r'ment</td>
<td>$s_{ld} = 200$ mm</td>
</tr>
<tr>
<td>Spacing of links along member</td>
<td>$s_{vd} = 275$ mm</td>
</tr>
</tbody>
</table>

**RESULTS FROM TORSION ASSESSMENT**

**Option 1:** Assessed torsional strengths
- Rectangle A: $251.88$ kNm
- Rectangle B: $244.17$ kNm
- Assessed torsional strength: $496.06$ kNm

**Option 2:**
- Assessed torsional strengths
  - Rectangle C: $89.958$ kNm

**WARNING:**
Link spacing does not comply with Cl. 5.3.4.5 requirements.

- Rectangle D: $302$ kNm
- Assessed torsional strength: $481.92$ kNm
Location: Ex6 - L Section

Torsional strength calculation to Departmental Standard BD 44/15

<table>
<thead>
<tr>
<th>L section ( Clause 5.3.4.4(c) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of flange</td>
</tr>
<tr>
<td>Breadth of web</td>
</tr>
<tr>
<td>Depth of flange</td>
</tr>
<tr>
<td>Depth of web</td>
</tr>
<tr>
<td>Strength of concrete</td>
</tr>
<tr>
<td>Factor for concrete</td>
</tr>
</tbody>
</table>

**Rectangle A**

| Strength of links                        | fyva=460 N/mm²|
| Strength of long. r'ment                 | fyla=460 N/mm²|
| Factor for reinforcement                 | gmsa=1.15    |
| Area of all longitudinal reinf.          | sumAa=5338 mm²|
| Area of one leg of closed link           | Asta=113 mm² |
| Smaller centre-line dim. of link         | x1a=300 mm   |
| Larger centre-line dim. of link          | y1a=1400 mm  |
| Average spacing of long. reinf.          | sla=200 mm   |
| Spacing of links along member            | sva=275 mm   |

**Rectangle B**

| Applied shear force                      | Vb=100 kN    |
| Tensile r'ment effective depth          | db=1850 mm   |
| Strength of links                        | fylvb=460 N/mm²|
| Strength of long. r'ment                 | fylb=460 N/mm²|
| Factor for reinforcement                 | gmsb=1.15    |
| Area of all longitudinal reinf.          | sumAlb=5181 mm²|
| Area of one leg of closed link           | Astb=113 mm² |
| Smaller centre-line dim. of link         | x1b=250 mm   |
| Larger centre-line dim. of link          | y1b=1400 mm  |
| Average spacing of long. reinf.          | slb=200 mm   |
| Spacing of links along member            | svb=275 mm   |

**Rectangle C**

| Strength of links                        | fyvc=460 N/mm²|
| Strength of long. r'ment                 | fylc=460 N/mm²|
| Factor for reinforcement                 | gmsc=1.15    |
| Area of all longitudinal reinf.          | sumAlc=4082 mm²|
| Area of one leg of closed link           | Astc=113 mm² |
| Smaller centre-line dim. of link         | x1c=300 mm   |
| Larger centre-line dim. of link          | y1c=1000 mm  |
| Average spacing of long. reinf.          | slc=200 mm   |
| Spacing of links along member            | svc=275 mm   |

**Rectangle D**

| Applied shear force                      | Vd=100 kN    |
| Tensile r'ment effective depth          | dd=1750 mm   |
Strength of links \( f_{yvd} = 460 \, \text{N/mm}^2 \)
Strength of long. r'ment \( f_{yld} = 460 \, \text{N/mm}^2 \)

Factor for reinforcement \( g_{msd} = 1.15 \)
Area of all longitudinal r'ment \( \sum A_{ld} = 6437 \, \text{mm}^2 \)
Area of one leg of closed link \( A_{std} = 113 \, \text{mm}^2 \)
Larger centre-line dim. of link \( y_{ld} = 1800 \, \text{mm} \)
Smaller centre-line dim. of link \( x_{ld} = 250 \, \text{mm} \)
Larger centre-line dim. of link \( y_{ld} = 1800 \, \text{mm} \)
Average spacing of long. r'ment \( s_{ld} = 200 \, \text{mm} \)
Spacing of links along member \( s_{vd} = 275 \, \text{mm} \)

**RESULTS FROM TORSION ASSESSMENT**

<table>
<thead>
<tr>
<th>Option 1: Assessed torsional strengths</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangle A</td>
<td>215.9 kNm</td>
</tr>
<tr>
<td>Rectangle B</td>
<td>179.92 kNm</td>
</tr>
<tr>
<td>Assessed torsional strength</td>
<td>395.82 kNm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Option 2: Assessed torsional strengths</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangle C</td>
<td>154.21 kNm</td>
</tr>
<tr>
<td>Rectangle D</td>
<td>231.32 kNm</td>
</tr>
<tr>
<td>Assessed torsional strength</td>
<td>385.54 kNm</td>
</tr>
</tbody>
</table>
Location: Ex1 - One metre deep slab

Differential temperature effects in concrete bridge decks to BD 37/01

![Diagram of solid slab with surfacing](image)

- Structural depth: $h = 1000$ mm
- Depth of surfacing: $ds = 100$ mm
- Value of Young's modulus: $EM = 28000$ N/mm²
- Coefficient of expansion: $\alpha = 1.2 \times 10^{-6}$ /°C
- Number of cross-sections: $ncs = 1$
- Breadth at top: $x_{bt(11)} = 1000$ mm
- Breadth at bottom: $x_{bb(11)} = 1000$ mm
- Depth: $x_{d(11)} = 1000$ mm

Positive Temperature Calculations

Temperature gradient - increments of depth (Figure 9, BD 37/01)
Temperature values (Figure 9 & Table 23 in BD 37/01):

- TP1 = 13.5
- $h_p1 = 150$ mm
- TP2 = 3
- $h_p2 = 250$ mm
- TP3 = 2.5
- $h_p3 = 400$ mm
- $h_p4 = 200$ mm

Positive temperature diagram
### STRESSES (N/mm²) (tension is positive)

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained force</th>
<th>Releasing moment</th>
<th>Self equilibrating force</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-4.536</td>
<td>0.6258</td>
<td>1.0832</td>
</tr>
<tr>
<td>150</td>
<td>-1.008</td>
<td>0.6258</td>
<td>0.75823</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.6258</td>
<td>0.21664</td>
</tr>
<tr>
<td>800</td>
<td>0</td>
<td>0.6258</td>
<td>-0.64991</td>
</tr>
<tr>
<td>1000</td>
<td>-0.84</td>
<td>0.6258</td>
<td>-1.0832</td>
</tr>
</tbody>
</table>

Member length \( L = 10 \) m

Partial load factor (temperature) \( \gamma = 0.8 \)

### Plane frame analysis

Axial and bending effects can be analysed in a plane frame model.

Bending effects for hogging releasing moment:

NOTE: The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[
\text{<members> ROTATION Z \(-0.61896E-3\)}
\]

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

Longitudinal effects for tensile releasing force:

NOTE: The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[
\text{<members> DISTORTION X 0.1788E-3}
\]

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

### Reverse Temperature Calculations

Temperature gradient – increments of depth (Figure 9, BD 37/01)

Temperature values (Figure 9 & Table 23 in BD 37/01):

\[
\begin{align*}
\text{TR1} &= -8 \\
\text{TR2} &= -1.5 \\
\text{TR3} &= -1.5 \\
\text{TR4} &= -6.3
\end{align*}
\]

Reverse temperature diagram
<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained force</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Equilibrating moment</th>
<th>Self equilibrating forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.688</td>
<td>-0.68208</td>
<td>-0.14851</td>
<td>1.8574</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>0.504</td>
<td>-0.68208</td>
<td>-0.089107</td>
<td>-0.26719</td>
<td></td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>-0.68208</td>
<td>-0.029702</td>
<td>-0.71178</td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>0</td>
<td>-0.68208</td>
<td>0.029702</td>
<td>-0.65238</td>
<td></td>
</tr>
<tr>
<td>800</td>
<td>0.504</td>
<td>-0.68208</td>
<td>0.089107</td>
<td>-0.088973</td>
<td></td>
</tr>
<tr>
<td>1000</td>
<td>2.1168</td>
<td>-0.68208</td>
<td>0.14851</td>
<td>1.5832</td>
<td></td>
</tr>
</tbody>
</table>

Member length $L=10$ m
Partial load factor (temperature) $\gamma=0.8$

**Plane frame analysis**

Axial and bending effects can be analysed in a plane frame model. Bending effects for sagging releasing moment:
NOTE: The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

```
<members> ROTATION Z 84.864E-6
```

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated. Longitudinal effects for compressive releasing force:
NOTE: The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

```
<members> DISTORTION X -0.19488E-3
```

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
Location: Ex2 - I beam with 150mm surfacing

Differential temperature effects in concrete bridge decks to BD 37/01

![Diagram of bridge deck with surfacing and dimensions]

Structural depth $h=1500$ mm
Depth of surfacing $ds=150$ mm
Number of cross-sections $ncs=5$
Breadth at top $xbt(11)=1000$ mm
Breadth at bottom $xbb(11)=1000$ mm
Depth $xd(11)=250$ mm
Young's modulus $E(11)=20000$ N/mm²
Coefficient of expansion $coe(11)=10E-6 /{^\circ}C$
Breadth at top $xbt(21)=300$ mm
Breadth at bottom $xbb(21)=200$ mm
Depth $xd(21)=100$ mm
Young's modulus $E(21)=22000$ N/mm²
Coefficient of expansion $coe(21)=12E-6 /{^\circ}C$
Breadth at top $xbt(31)=200$ mm
Breadth at bottom $xbb(31)=200$ mm
Depth $xd(31)=700$ mm
Young's modulus $E(31)=24000$ N/mm²
Coefficient of expansion $coe(31)=11E-6 /{^\circ}C$
Breadth at top $xbt(41)=200$ mm
Breadth at bottom $xbb(41)=300$ mm
Depth $xd(41)=150$ mm
Young's modulus $E(41)=28000$ N/mm²
Coefficient of expansion $coe(41)=15E-6 /{^\circ}C$
Breadth at top $xbt(51)=1000$ mm
Breadth at bottom $xbb(51)=1000$ mm
Depth $xd(51)=300$ mm
Young's modulus $E(51)=31000$ N/mm²
Coefficient of expansion $coe(51)=13E-6 /{^\circ}C$

Positive Temperature Calculations

Temperature gradient – increments of depth (Figure 9, BD 37/01)
Temperature values (Figure 9 & Table 23 in BD 37/01):
Temperature effects

Positive temperature diagram

STRESSES (N/mm²) (tension is positive)

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-2</td>
<td>0.34863</td>
<td>0.41505</td>
<td>-1.2363</td>
</tr>
<tr>
<td>150</td>
<td>-0.5</td>
<td>0.34863</td>
<td>0.34402</td>
<td>0.19265</td>
</tr>
<tr>
<td>150</td>
<td>-0.5</td>
<td>0.34863</td>
<td>0.34402</td>
<td>0.19265</td>
</tr>
<tr>
<td>250</td>
<td>-0.3</td>
<td>0.34863</td>
<td>0.29667</td>
<td>0.3453</td>
</tr>
<tr>
<td>250</td>
<td>-0.396</td>
<td>0.38349</td>
<td>0.32633</td>
<td>0.31383</td>
</tr>
<tr>
<td>350</td>
<td>-0.132</td>
<td>0.38349</td>
<td>0.27425</td>
<td>0.52574</td>
</tr>
<tr>
<td>350</td>
<td>-0.132</td>
<td>0.41836</td>
<td>0.29918</td>
<td>0.58553</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.41836</td>
<td>0.27077</td>
<td>0.68912</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.41836</td>
<td>0.27077</td>
<td>0.68912</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>0.41836</td>
<td>-0.098579</td>
<td>0.31978</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>0.48808</td>
<td>-0.11501</td>
<td>0.37307</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
<td>0.48808</td>
<td>-0.21445</td>
<td>0.27363</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
<td>0.54038</td>
<td>-0.23742</td>
<td>0.30295</td>
</tr>
<tr>
<td>1250</td>
<td>0</td>
<td>0.54038</td>
<td>-0.27412</td>
<td>0.26625</td>
</tr>
<tr>
<td>1250</td>
<td>0</td>
<td>0.54038</td>
<td>-0.27412</td>
<td>0.26625</td>
</tr>
<tr>
<td>1500</td>
<td>-0.806</td>
<td>0.54038</td>
<td>-0.45761</td>
<td>-0.72324</td>
</tr>
</tbody>
</table>

Member length L=12 m
Partial load factor (temperature) gamma=1

Plane frame analysis

Axial and bending effects can be analysed in a plane frame model.
Bending effects for hogging releasing moment:
NOTE: The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> ROTATION Z -0.28411E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
Longitudinal effects for tensile releasing force:
NOTE: The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> DISTORTION X 0.20918E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

Reverse Temperature Calculations

Temperature gradient – increments of depth (Figure 9, BD 37/01)
Temperature values (Figure 9 & Table 23 in BD 37/01):

<table>
<thead>
<tr>
<th>Depth from top (mm)</th>
<th>Restrainted force (N/mm²)</th>
<th>Releasing force (N/mm²)</th>
<th>Releasing moment (N-mm)</th>
<th>Self equilibrating force (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.3</td>
<td>-0.59098</td>
<td>0.20802</td>
<td>0.91704</td>
</tr>
<tr>
<td>250</td>
<td>0.08</td>
<td>-0.59098</td>
<td>0.14869</td>
<td>-0.36229</td>
</tr>
<tr>
<td>250</td>
<td>0.1056</td>
<td>-0.65008</td>
<td>0.16356</td>
<td>-0.38092</td>
</tr>
<tr>
<td>350</td>
<td>0.0528</td>
<td>-0.65008</td>
<td>0.13745</td>
<td>-0.45983</td>
</tr>
<tr>
<td>350</td>
<td>0.0528</td>
<td>-0.70918</td>
<td>0.14995</td>
<td>-0.50643</td>
</tr>
<tr>
<td>450</td>
<td>0</td>
<td>-0.70918</td>
<td>0.12147</td>
<td>-0.58771</td>
</tr>
<tr>
<td>450</td>
<td>0</td>
<td>-0.70918</td>
<td>0.12147</td>
<td>-0.58771</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>-0.70918</td>
<td>-0.049407</td>
<td>-0.75858</td>
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<tr>
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<td>0.3465</td>
<td>-0.82737</td>
<td>-0.10748</td>
<td>-0.58835</td>
</tr>
<tr>
<td>1200</td>
<td>0.33248</td>
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<td>-0.119</td>
<td>-0.70254</td>
</tr>
<tr>
<td>1250</td>
<td>0.4433</td>
<td>-0.91602</td>
<td>-0.13739</td>
<td>-0.61011</td>
</tr>
<tr>
<td>1250</td>
<td>0.4433</td>
<td>-0.91602</td>
<td>-0.13739</td>
<td>-0.61011</td>
</tr>
<tr>
<td>1500</td>
<td>2.4986</td>
<td>-0.91602</td>
<td>-0.22935</td>
<td>1.3532</td>
</tr>
</tbody>
</table>

Member length 
L=12 m
Partial load factor (temperature) gamma=1

Plane frame analysis

Axial and bending effects can be analysed in a plane frame model.
Bending effects for hogging releasing moment:
NOTE: The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> ROTATION Z -0.1424E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

Longitudinal effects for compressive releasing force:
NOTE: The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> DISTORTION X -0.35459E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
Location: Ex3 - I beam with waterproofing

Differential temperature effects in concrete bridge decks to

BD 37/01

Positive Temperature Calculations

Temperature gradient – increments of depth (Figure 9, BD 37/01)
Temperature values (Figure 9 & Table 23 in BD 37/01)
**Positive temperature diagram**

**STRESSES (N/mm²) (tension is positive)**

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-4.72</td>
<td>0.57309</td>
<td>1.2666</td>
<td>-2.8803</td>
</tr>
<tr>
<td>150</td>
<td>-1</td>
<td>0.57309</td>
<td>1.0498</td>
<td>0.62293</td>
</tr>
<tr>
<td>150</td>
<td>-1</td>
<td>0.57309</td>
<td>1.0498</td>
<td>0.62293</td>
</tr>
<tr>
<td>250</td>
<td>-0.6</td>
<td>0.57309</td>
<td>0.90534</td>
<td>0.87843</td>
</tr>
<tr>
<td>250</td>
<td>-0.792</td>
<td>0.6304</td>
<td>0.99587</td>
<td>0.83427</td>
</tr>
<tr>
<td>350</td>
<td>-0.264</td>
<td>0.6304</td>
<td>0.83692</td>
<td>1.2033</td>
</tr>
<tr>
<td>350</td>
<td>-0.264</td>
<td>0.68771</td>
<td>0.913</td>
<td>1.3367</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.68771</td>
<td>0.8263</td>
<td>1.514</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.68771</td>
<td>0.8263</td>
<td>1.514</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>0.68771</td>
<td>-0.30083</td>
<td>0.38688</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>0.80233</td>
<td>-0.35097</td>
<td>0.45136</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
<td>0.80233</td>
<td>-0.65443</td>
<td>0.1479</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
<td>0.88829</td>
<td>-0.72455</td>
<td>0.16375</td>
</tr>
<tr>
<td>1401</td>
<td>0</td>
<td>0.88829</td>
<td>-1.1747</td>
<td>-0.28646</td>
</tr>
<tr>
<td>1401</td>
<td>0</td>
<td>0.88829</td>
<td>-1.1747</td>
<td>-0.28646</td>
</tr>
<tr>
<td>1500</td>
<td>-0.5642</td>
<td>0.88829</td>
<td>-1.3965</td>
<td>-1.0724</td>
</tr>
</tbody>
</table>

**Member length** \( L = 12 \text{ m} \)

**Partial load factor (temperature) gamma=1**

**Grid analysis**
Bending effects for hogging releasing moment:
NOTE: The distortion should be input into the grid analysis using the MEMBER DISTORTIONS option with the following format.

<members> ROTATION Y 0.86702E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

Reverse Temperature Calculations

Temperature gradient — increments of depth (Figure 9, BD 37/01)
Temperature values (Figure 9 & Table 23 in BD 37/01):
Bending effects for sagging releasing moment:
NOTE: The distortion should be input into the grid analysis using the MEMBER DISTORTIONS option with the following format.

<members> ROTATION Y -0.20103E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
Location: Ex1 - One metre deep slab

Differential temperature effects in concrete bridge decks

\[ \text{Surface} \quad \text{h} \quad \text{Solid slab} \]

- Structural depth \( h = 1000 \text{ mm} \)
- Depth of surfacing \( d_s = 100 \text{ mm} \)
- Value of Young's modulus \( E_M = 28000 \text{ N/mm}^2 \)
- Coefficient of expansion \( \alpha = 12 \text{E-6/°C} \)
- Number of cross-sections \( n_c = 1 \)
- Breadth at top \( x_{bt(11)} = 1000 \text{ mm} \)
- Breadth at bottom \( x_{bb(11)} = 1000 \text{ mm} \)
- Depth \( x_d(11) = 1000 \text{ mm} \)

Positive Temperature Calculations

Temperature gradient – increments of depth (Figure 6.2c, EN 1991-1-5)
Temperature values (Figure 6.2c & Table B.3 in EN 1991-1-5):

\[ \begin{array}{cccc}
TP1 = 13.5 & hp1 = 150 \text{ mm} \\
TP2 = 3 & hp2 = 250 \text{ mm} \\
TP3 = 2.5 & hp4 = 400 \text{ mm} \\
\end{array} \]

Positive temperature diagram

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained force</th>
<th>Releasing moment</th>
<th>Releasing self</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-4.536</td>
<td>0.6258</td>
<td>1.0832</td>
<td>-2.827</td>
</tr>
<tr>
<td>150</td>
<td>-1.008</td>
<td>0.6258</td>
<td>0.75823</td>
<td>0.37603</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.6258</td>
<td>0.21664</td>
<td>0.84244</td>
</tr>
<tr>
<td>800</td>
<td>0</td>
<td>0.6258</td>
<td>-0.64991</td>
<td>-0.024108</td>
</tr>
<tr>
<td>1000</td>
<td>-0.84</td>
<td>0.6258</td>
<td>-1.0832</td>
<td>-1.2974</td>
</tr>
</tbody>
</table>

Member length \( L = 10 \text{ m} \)
Partial load factor (temperature) \( \gamma = 0.8 \)
Plane frame analysis

Axial and bending effects can be analysed in a plane frame model. Bending effects for hogging releasing moment:

NOTE: The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> ROTATION Z -0.61896E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

Longitudinal effects for tensile releasing force:

NOTE: The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> DISTORTION X 0.1788E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

Reverse Temperature Calculations

Temperature gradient - increments of depth (Figure 6.2c, EN 1991-1-5)
Temperature values (Figure 6.2c & Table B.3 in EN 1991-1-5):

Reverse temperature diagram

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.688</td>
<td>-0.68208</td>
<td>-0.14851</td>
<td>1.8574</td>
</tr>
<tr>
<td>200</td>
<td>0.504</td>
<td>-0.68208</td>
<td>-0.089107</td>
<td>-0.26719</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>-0.68208</td>
<td>-0.029702</td>
<td>-0.71178</td>
</tr>
<tr>
<td>600</td>
<td>0</td>
<td>-0.68208</td>
<td>0.029702</td>
<td>-0.65238</td>
</tr>
<tr>
<td>800</td>
<td>0.504</td>
<td>-0.68208</td>
<td>0.089107</td>
<td>-0.088973</td>
</tr>
<tr>
<td>1000</td>
<td>2.1168</td>
<td>-0.68208</td>
<td>0.14851</td>
<td>1.5832</td>
</tr>
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</table>

Member length L=10 m
Partial load factor (temperature) gamma=0.8
Plane frame analysis

Axial and bending effects can be analysed in a plane frame model.
Bending effects for sagging releasing moment:
NOTE: The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> ROTATION Z 84.864E-6

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

Longitudinal effects for compressive releasing force:
NOTE: The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> DISTORTION X -0.19488E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
Location: Ex2 - I beam with 150mm surfacing

Differential temperature effects in concrete bridge decks

![Diagram of concrete bridge deck with surfacing and structural depth](image)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural depth</td>
<td>h=1500 mm</td>
</tr>
<tr>
<td>Depth of surfacing</td>
<td>ds=150 mm</td>
</tr>
<tr>
<td>Number of cross-sections</td>
<td>ncs=5</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(11)=1000 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(11)=1000 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(11)=250 mm</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>E(11)=20000 N/mm²</td>
</tr>
<tr>
<td>Coefficient of expansion</td>
<td>coe(11)=10E-6 /°C</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(21)=300 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(21)=200 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(21)=100 mm</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>E(21)=22000 N/mm²</td>
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<tr>
<td>Coefficient of expansion</td>
<td>coe(21)=12E-6 /°C</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(31)=200 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(31)=200 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(31)=700 mm</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>E(31)=24000 N/mm²</td>
</tr>
<tr>
<td>Coefficient of expansion</td>
<td>coe(31)=11E-6 /°C</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(41)=200 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(41)=300 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(41)=150 mm</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>E(41)=28000 N/mm²</td>
</tr>
<tr>
<td>Coefficient of expansion</td>
<td>coe(41)=15E-6 /°C</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(51)=1000 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(51)=1000 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(51)=300 mm</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>E(51)=31000 N/mm²</td>
</tr>
<tr>
<td>Coefficient of expansion</td>
<td>coe(51)=13E-6 /°C</td>
</tr>
</tbody>
</table>

Positive Temperature Calculations

Temperature gradient – increments of depth (Figure 6.2c, EN 1991-1-5)
Temperature values (Figure 6.2c & Table B.3 in EN 1991-1-5):
Positive temperature diagram

STRESSES ( N/mm² ) (tension is positive)

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-2</td>
<td>0.34863</td>
<td>0.41505</td>
<td>-1.2363</td>
</tr>
<tr>
<td>150</td>
<td>-0.5</td>
<td>0.34863</td>
<td>0.34402</td>
<td>0.19265</td>
</tr>
<tr>
<td>150</td>
<td>-0.5</td>
<td>0.34863</td>
<td>0.34402</td>
<td>0.19265</td>
</tr>
<tr>
<td>250</td>
<td>-0.3</td>
<td>0.34863</td>
<td>0.29667</td>
<td>0.3453</td>
</tr>
<tr>
<td>250</td>
<td>-0.396</td>
<td>0.38349</td>
<td>0.32633</td>
<td>0.31383</td>
</tr>
<tr>
<td>350</td>
<td>-0.132</td>
<td>0.38349</td>
<td>0.27425</td>
<td>0.52574</td>
</tr>
<tr>
<td>350</td>
<td>-0.132</td>
<td>0.41836</td>
<td>0.29918</td>
<td>0.58553</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.41836</td>
<td>0.27077</td>
<td>0.68912</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.41836</td>
<td>0.27077</td>
<td>0.68912</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>0.41836</td>
<td>-0.098579</td>
<td>0.31978</td>
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<tr>
<td>1050</td>
<td>0</td>
<td>0.48808</td>
<td>-0.11501</td>
<td>0.37307</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
<td>0.48808</td>
<td>-0.21445</td>
<td>0.27363</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
<td>0.54038</td>
<td>-0.23742</td>
<td>0.30295</td>
</tr>
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<td>1250</td>
<td>0</td>
<td>0.54038</td>
<td>-0.27412</td>
<td>0.26625</td>
</tr>
<tr>
<td>1250</td>
<td>0</td>
<td>0.54038</td>
<td>-0.27412</td>
<td>0.26625</td>
</tr>
<tr>
<td>1500</td>
<td>-0.806</td>
<td>0.54038</td>
<td>-0.45761</td>
<td>-0.72324</td>
</tr>
</tbody>
</table>

Member length L=12 m
Partial load factor (temperature) gamma=1

**Plane frame analysis**

Axial and bending effects can be analysed in a plane frame model. Bending effects for hogging releasing moment:

**NOTE:** The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> ROTATION Z -0.28411E-3

**NOTE:** The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
Longitudinal effects for tensile releasing force:
NOTE: The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> DISTORTION X 0.20918E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

Reverse Temperature Calculations

Temperature gradient – increments of depth (Figure 6.2c, EN 1991-1-5)
Temperature values (Figure 6.2c & Table B.3 in EN 1991-1-5):

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Strain restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.3</td>
<td>-0.59098</td>
<td>0.20802</td>
<td>0.91704</td>
</tr>
<tr>
<td>250</td>
<td>0.08</td>
<td>-0.59098</td>
<td>0.14869</td>
<td>-0.36229</td>
</tr>
<tr>
<td>250</td>
<td>0.1056</td>
<td>-0.65008</td>
<td>0.16356</td>
<td>-0.38902</td>
</tr>
<tr>
<td>350</td>
<td>0.0528</td>
<td>-0.65008</td>
<td>0.13745</td>
<td>-0.45983</td>
</tr>
<tr>
<td>350</td>
<td>0.0528</td>
<td>-0.70918</td>
<td>0.14995</td>
<td>-0.50643</td>
</tr>
<tr>
<td>450</td>
<td>0</td>
<td>-0.70918</td>
<td>0.12147</td>
<td>-0.58771</td>
</tr>
<tr>
<td>450</td>
<td>0</td>
<td>-0.70918</td>
<td>0.12147</td>
<td>-0.58771</td>
</tr>
<tr>
<td>1050</td>
<td>0.3465</td>
<td>-0.82737</td>
<td>-0.10748</td>
<td>-0.58835</td>
</tr>
<tr>
<td>1200</td>
<td>0.33248</td>
<td>-0.91602</td>
<td>-0.119</td>
<td>-0.70254</td>
</tr>
<tr>
<td>1250</td>
<td>0.4433</td>
<td>-0.91602</td>
<td>-0.13739</td>
<td>-0.61011</td>
</tr>
<tr>
<td>1250</td>
<td>0.4433</td>
<td>-0.91602</td>
<td>-0.13739</td>
<td>-0.61011</td>
</tr>
<tr>
<td>1500</td>
<td>2.4986</td>
<td>-0.91602</td>
<td>-0.22935</td>
<td>1.3532</td>
</tr>
</tbody>
</table>

Member length     L=12 m
Partial load factor (temperature) gamma=1

Plane frame analysis

Axial and bending effects can be analysed in a plane frame model.
Bending effects for hogging releasing moment:
NOTE: The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> ROTATION Z -0.1424E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

Longitudinal effects for compressive releasing force:
NOTE: The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

<members> DISTORTION X -0.35459E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
Location: Ex3 - I beam with waterproofing

Differential temperature effects in concrete bridge decks

![Diagram of concrete bridge deck with surfacing and depth dimensions.]

- Structural depth: \( h = 1500 \text{ mm} \)
- Depth of surfacing: \( d_s = -1 \text{ mm} \)
- Number of cross-sections: \( n_{cs} = 5 \)
- Breadth at top: \( x_{bt}(11) = 1000 \text{ mm} \)
- Breadth at bottom: \( x_{bb}(11) = 1000 \text{ mm} \)
- Depth: \( x_d(11) = 250 \text{ mm} \)
- Young's modulus: \( E(11) = 20000 \text{ N/mm}^2 \)
- Coefficient of expansion: \( \text{coe}(11) = 10 \times 10^{-6} /{^\circ C} \)
- Breadth at top: \( x_{bt}(21) = 300 \text{ mm} \)
- Breadth at bottom: \( x_{bb}(21) = 200 \text{ mm} \)
- Depth: \( x_d(21) = 100 \text{ mm} \)
- Young's modulus: \( E(21) = 22000 \text{ N/mm}^2 \)
- Coefficient of expansion: \( \text{coe}(21) = 12 \times 10^{-6} /{^\circ C} \)
- Breadth at top: \( x_{bt}(31) = 200 \text{ mm} \)
- Breadth at bottom: \( x_{bb}(31) = 200 \text{ mm} \)
- Depth: \( x_d(31) = 700 \text{ mm} \)
- Young's modulus: \( E(31) = 24000 \text{ N/mm}^2 \)
- Coefficient of expansion: \( \text{coe}(31) = 11 \times 10^{-6} /{^\circ C} \)
- Breadth at top: \( x_{bt}(41) = 200 \text{ mm} \)
- Breadth at bottom: \( x_{bb}(41) = 300 \text{ mm} \)
- Depth: \( x_d(41) = 150 \text{ mm} \)
- Young's modulus: \( E(41) = 28000 \text{ N/mm}^2 \)
- Coefficient of expansion: \( \text{coe}(41) = 15 \times 10^{-6} /{^\circ C} \)
- Breadth at top: \( x_{bt}(51) = 1000 \text{ mm} \)
- Breadth at bottom: \( x_{bb}(51) = 1000 \text{ mm} \)
- Depth: \( x_d(51) = 300 \text{ mm} \)
- Young's modulus: \( E(51) = 31000 \text{ N/mm}^2 \)
- Coefficient of expansion: \( \text{coe}(51) = 13 \times 10^{-6} /{^\circ C} \)

Positive Temperature Calculations

Temperature gradient – increments of depth (Figure 6.2c, EN 1991-1-5)
Temperature values (Figure 6.2c & Table B.3 in EN 1991-1-5):
## Positive temperature diagram

**STRESSES ( N/mm² )** (tension is positive)

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing Force</th>
<th>Releasing Moment</th>
<th>Self Equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-4.72</td>
<td>0.57309</td>
<td>1.2666</td>
<td>-2.8803</td>
</tr>
<tr>
<td>150</td>
<td>-1</td>
<td>0.57309</td>
<td>1.0498</td>
<td>0.62293</td>
</tr>
<tr>
<td>150</td>
<td>-1</td>
<td>0.57309</td>
<td>1.0498</td>
<td>0.62293</td>
</tr>
<tr>
<td>250</td>
<td>-0.6</td>
<td>0.57309</td>
<td>0.90534</td>
<td>0.87843</td>
</tr>
<tr>
<td>250</td>
<td>-0.792</td>
<td>0.6304</td>
<td>0.99587</td>
<td>0.83427</td>
</tr>
<tr>
<td>350</td>
<td>-0.264</td>
<td>0.6304</td>
<td>0.83692</td>
<td>1.2033</td>
</tr>
<tr>
<td>350</td>
<td>-0.264</td>
<td>0.68771</td>
<td>0.913</td>
<td>1.3367</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.68771</td>
<td>0.8263</td>
<td>1.514</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.68771</td>
<td>0.8263</td>
<td>1.514</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>0.68771</td>
<td>-0.30083</td>
<td>0.38688</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>0.80233</td>
<td>-0.35097</td>
<td>0.45136</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
<td>0.80233</td>
<td>-0.65443</td>
<td>0.1479</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
<td>0.88829</td>
<td>-0.72455</td>
<td>0.16375</td>
</tr>
<tr>
<td>1401</td>
<td>0</td>
<td>0.88829</td>
<td>-1.1747</td>
<td>-0.28646</td>
</tr>
<tr>
<td>1401</td>
<td>0</td>
<td>0.88829</td>
<td>-1.1747</td>
<td>-0.28646</td>
</tr>
<tr>
<td>1500</td>
<td>0</td>
<td>0.88829</td>
<td>-1.3965</td>
<td>-1.0724</td>
</tr>
</tbody>
</table>

Member length \( L = 12 \text{ m} \)

Partial load factor (temperature) \( \gamma = 1 \)

### Grid analysis
Bending effects for hogging releasing moment:
NOTE: The distortion should be input into the grid analysis using
the MEMBER DISTORTIONS option with the following format.

\[
\text{members} \ \text{ROTATION Y 0.86702E-3}
\]

NOTE: The equilibrating stresses should be taken into account when the
stresses at a section due to combinations of loading are calculated.

Reverse Temperature Calculations

Temperature gradient – increments of depth (Figure 6.2c, EN 1991-1-5)
Temperature values (Figure 6.2c & Table B.3 in EN 1991-1-5):

\[
\begin{array}{cccc}
\text{TR1} & \text{hn1} & \text{TR2} & \text{hn2} & \text{TR3} & \text{hn3} & \text{TR4} & \text{hn4} \\
-13.7 & 250 & -1 & 200 & -0.6 & 250 & -6.7 & 250 \\
\end{array}
\]

Reverse temperature diagram

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Stresses (N/mm²)</th>
<th>Restrained</th>
<th>Releasing</th>
<th>Equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.74</td>
<td>-0.78502</td>
<td>-0.29368</td>
<td>1.6613</td>
</tr>
<tr>
<td>250</td>
<td>0.2</td>
<td>-0.78502</td>
<td>-0.20992</td>
<td>-0.79494</td>
</tr>
<tr>
<td>250</td>
<td>0.264</td>
<td>-0.86352</td>
<td>-0.23091</td>
<td>-0.83043</td>
</tr>
<tr>
<td>350</td>
<td>0.132</td>
<td>-0.86352</td>
<td>-0.19405</td>
<td>-0.92558</td>
</tr>
<tr>
<td>350</td>
<td>0.132</td>
<td>-0.94203</td>
<td>-0.2117</td>
<td>-1.0217</td>
</tr>
<tr>
<td>450</td>
<td>0</td>
<td>-0.94203</td>
<td>-0.17149</td>
<td>-1.1135</td>
</tr>
<tr>
<td>450</td>
<td>0</td>
<td>-0.94203</td>
<td>-0.17149</td>
<td>-1.1135</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>-0.94203</td>
<td>0.069753</td>
<td>-0.87227</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>-1.099</td>
<td>0.081379</td>
<td>-1.0177</td>
</tr>
<tr>
<td>1200</td>
<td>0.189</td>
<td>-1.099</td>
<td>0.15174</td>
<td>-0.75829</td>
</tr>
<tr>
<td>1200</td>
<td>0.18135</td>
<td>-1.2168</td>
<td>0.168</td>
<td>-0.86744</td>
</tr>
<tr>
<td>1250</td>
<td>0.2418</td>
<td>-1.2168</td>
<td>0.19397</td>
<td>-0.78102</td>
</tr>
<tr>
<td>1250</td>
<td>0.2418</td>
<td>-1.2168</td>
<td>0.19397</td>
<td>-0.78102</td>
</tr>
<tr>
<td>1500</td>
<td>2.7001</td>
<td>-1.2168</td>
<td>0.3238</td>
<td>1.8071</td>
</tr>
</tbody>
</table>

Member length \( L = 12 \) m
Partial load factor (temperature) \( \gamma = 1 \)

Grid analysis
Bending effects for sagging releasing moment:
NOTE: The distortion should be input into the grid analysis using
the MEMBER DISTORTIONS option with the following format.
  <members> ROTATION Y -0.20103E-3

NOTE: The equilibrating stresses should be taken into account when the
stresses at a section due to combinations of loading are calculated.
Location: Ex4 - Deck on box girder

Differential temperature effects in concrete bridge decks

---

**Surfacing**

---

**Deck on box girders**

<table>
<thead>
<tr>
<th>Structural depth</th>
<th>h=1500 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of surfacing</td>
<td>ds=100 mm</td>
</tr>
<tr>
<td>Value of Young's modulus</td>
<td>EM=20000 N/mm²</td>
</tr>
<tr>
<td>Number of cross-sections</td>
<td>ncs=3</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(11)=1000 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(11)=1000 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(11)=250 mm</td>
</tr>
<tr>
<td>Coefficient of expansion</td>
<td>coe(11)=10E-6 /°C</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(21)=400 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(21)=400 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(21)=1000 mm</td>
</tr>
<tr>
<td>Coefficient of expansion</td>
<td>coe(21)=12E-6 /°C</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(31)=800 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(31)=800 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(31)=250 mm</td>
</tr>
<tr>
<td>Coefficient of expansion</td>
<td>coe(31)=11E-6 /°C</td>
</tr>
</tbody>
</table>

**Positive Temperature Calculations**

Temperature gradient – increments of depth (Figure 6.2c, EN 1991-1-5)

Temperature values (Figure 6.2c & Table B.3 in EN 1991-1-5):

---

Positive temperature diagram
STRESSES ( N/mm² ) (tension is positive)

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-2.7</td>
<td>0.41466</td>
<td>0.544</td>
<td>-1.7413</td>
</tr>
<tr>
<td>150</td>
<td>-0.6</td>
<td>0.41466</td>
<td>0.42959</td>
<td>0.24425</td>
</tr>
<tr>
<td>250</td>
<td>-0.36</td>
<td>0.41466</td>
<td>0.35332</td>
<td>0.40798</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>0.41466</td>
<td>0.23891</td>
<td>0.65357</td>
</tr>
<tr>
<td>1250</td>
<td>0</td>
<td>0.41466</td>
<td>-0.4094</td>
<td>0.0052576</td>
</tr>
<tr>
<td>1300</td>
<td>0</td>
<td>0.41466</td>
<td>-0.44754</td>
<td>-0.032878</td>
</tr>
<tr>
<td>1500</td>
<td>-0.55</td>
<td>0.41466</td>
<td>-0.60008</td>
<td>-0.73542</td>
</tr>
</tbody>
</table>

Member length L=12 m
Partial load factor (temperature) gamma=1

Grid analysis

Bending effects for hogging releasing moment:

NOTE: The distortion should be input into the grid analysis using the MEMBER DISTORTIONS option with the following format.

<members> ROTATION Y 0.45763E-3

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.

Reverse Temperature Calculations

Temperature gradient – increments of depth (Figure 6.2c, EN 1991-1-5)
Temperature values (Figure 6.2c & Table B.3 in EN 1991-1-5):

Reverse temperature diagram
### STRESSES (N/mm²) (tension is positive)

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.68</td>
<td>-0.47282</td>
<td>-0.075523</td>
<td>1.1317</td>
</tr>
<tr>
<td>250</td>
<td>0.1</td>
<td>-0.47282</td>
<td>-0.049051</td>
<td>-0.42187</td>
</tr>
<tr>
<td>250</td>
<td>0.12</td>
<td>-0.47282</td>
<td>-0.049051</td>
<td>-0.40187</td>
</tr>
<tr>
<td>450</td>
<td>0</td>
<td>-0.47282</td>
<td>-0.027873</td>
<td>-0.5007</td>
</tr>
<tr>
<td>450</td>
<td>0</td>
<td>-0.47282</td>
<td>-0.027873</td>
<td>-0.5007</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>-0.47282</td>
<td>0.035659</td>
<td>-0.43716</td>
</tr>
<tr>
<td>1050</td>
<td>0</td>
<td>-0.47282</td>
<td>0.035659</td>
<td>-0.43716</td>
</tr>
<tr>
<td>1250</td>
<td>0.24</td>
<td>-0.47282</td>
<td>0.056837</td>
<td>-0.17599</td>
</tr>
<tr>
<td>1250</td>
<td>0.22</td>
<td>-0.47282</td>
<td>0.056837</td>
<td>-0.19599</td>
</tr>
<tr>
<td>1500</td>
<td>1.43</td>
<td>-0.47282</td>
<td>0.083309</td>
<td>1.0405</td>
</tr>
</tbody>
</table>

Member length: \( L = 12 \text{ m} \)

Partial load factor (temperature) \( \gamma = 1 \)

### Grid analysis

Bending effects for sagging releasing moment:

NOTE: The distortion should be input into the grid analysis using the MEMBER DISTORTIONS option with the following format.

\[ \text{<members> ROTATION Y } -63.533E-6 \]

NOTE: The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
**Location:** Ex1 - Single carriageway

**Type HA uniformly distributed loading (UDL)**

and **knife edge loading (KEL)**

Calculations are in accordance with Departmental Standard BD 37/01 "Loads for Highway Structures". References are to clauses in the composite version of BS5400-2 contained in Appendix A of BD 37/01.

Number of carriageways on superstructure $noc=1$

<table>
<thead>
<tr>
<th>Width of carriageway $cw(1)=2.8$ m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loaded length for HA UDL $ll=40$ m</td>
</tr>
<tr>
<td>Loaded length is less than 50m</td>
</tr>
</tbody>
</table>

Nominal HA UDL expressed in kN/m (per notional lane).

- **HA UDL** $W=336*(1/ll)^0.67=28.377$ kN/m
- **Knife Edge Load (KEL)** $KEL=120$ kN (per notional lane)

**Loading for carriageway**

<table>
<thead>
<tr>
<th></th>
<th>Factor $β$</th>
<th>Nominal HA UDL (kN/m) per lane</th>
<th>Factored HA UDL (kN/m) per lane</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1</td>
<td>28.377</td>
<td>28.377</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Remaining area</td>
<td>1.0</td>
<td>$kN/m^2$</td>
<td>$kN/m^2$</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Lane Loadings
Location: Ex2 - Dual carriageway

Type HA uniformly distributed loading (UDL)

and knife edge loading (KEL)

Calculations are in accordance with Departmental Standard BD 37/01 "Loads for Highway Structures". References are to clauses in the composite version of BS5400-2 contained in Appendix A of BD 37/01.

Number of carriageways on superstructure noc=2

Width of carriageway               cw(1)=15 m  
Width of carriageway               cw(2)=15 m  
Loaded length for HA UDL           ll=120 m  
Loaded length is greater than or equal to 50m

Nominal HA UDL expressed in kN/m (per notional lane).

HA UDL  W=36*(1/ll)^0.10=22.304 kN/m

Knife Edge Load (KEL)              KEL=120 kN (per notional lane)

Loading for carriageways

<table>
<thead>
<tr>
<th></th>
<th>Factor β</th>
<th>Nominal HA UDL (kN/m) per lane</th>
<th>Factored HA UDL (kN/m) per lane</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1</td>
<td>22.304</td>
<td>22.304</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1</td>
<td>22.304</td>
<td>22.304</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.6</td>
<td>22.304</td>
<td>13.382</td>
<td>120</td>
<td>72</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.6</td>
<td>22.304</td>
<td>13.382</td>
<td>120</td>
<td>72</td>
</tr>
</tbody>
</table>

Lane Loadings
Location: Ex3 - Dual carriageway

Type HA uniformly distributed loading (UDL) and knife edge loading (KEL)

Calculations are in accordance with Departmental Standard BD 37/01 "Loads for Highway Structures". References are to clauses in the composite version of BS5400-2 contained in Appendix A of BD 37/01.

Number of carriageways on superstructure noc=2
Width of carriageway cw(1)=4.5 m
Width of carriageway cw(2)=4.5 m
Loaded length for HA UDL ll=140 m
Loaded length is greater than or equal to 50m
Nominal HA UDL expressed in kN/m (per notional lane).
HA UDL W=36*(1/ll)^0.10=21.963 kN/m
Knife Edge Load (KEL) KEL=120 kN (per notional lane)

Loading for carriageways

<table>
<thead>
<tr>
<th></th>
<th>Factor β</th>
<th>Nominal HA UDL (kN/m) per lane</th>
<th>Factored HA UDL (kN/m) per lane</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1</td>
<td>21.963</td>
<td>21.963</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1</td>
<td>21.963</td>
<td>21.963</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Remaining area</td>
<td>1.0</td>
<td>kN/m²</td>
<td>kN/m²</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.00</td>
<td>5.00</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Lane Loadings
Location: Ex4 - Dual carriageway

Type HA uniformly distributed loading (UDL)

and knife edge loading (KEL)

Calculations are in accordance with Departmental Standard BD 37/01 "Loads for Highway Structures". References are to clauses in the composite version of BS5400-2 contained in Appendix A of BD 37/01.

Number of carriageways on superstructure $n_{oc}=2$
Width of carriageway $cw(1)=11$ m
Width of carriageway $cw(2)=15$ m
Loaded length for HA UDL $ll=142$ m

Loaded length is greater than or equal to 50m

Nominal HA UDL expressed in kN/m (per notional lane).

HA UDL $W=36 \times \left(\frac{1}{ll}\right)^{0.10}=21.932$ kN/m

Knife Edge Load (KEL) $KEL=120$ kN (per notional lane)
### Loading for carriageway 1

<table>
<thead>
<tr>
<th>Lane Type</th>
<th>Factor $\beta$</th>
<th>Nominal HA UDL (kN/m) per lane</th>
<th>Factored HA UDL (kN/m) per lane</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1</td>
<td>21.932</td>
<td>21.932</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1</td>
<td>21.932</td>
<td>21.932</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.6</td>
<td>21.932</td>
<td>13.159</td>
<td>120</td>
<td>72</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.6</td>
<td>21.932</td>
<td>13.159</td>
<td>120</td>
<td>72</td>
</tr>
</tbody>
</table>

### Loading for carriageway 2

<table>
<thead>
<tr>
<th>Lane Type</th>
<th>Factor $\beta$</th>
<th>Nominal HA UDL (kN/m) per lane</th>
<th>Factored HA UDL (kN/m) per lane</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1</td>
<td>21.932</td>
<td>21.932</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1</td>
<td>21.932</td>
<td>21.932</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.6</td>
<td>21.932</td>
<td>13.159</td>
<td>120</td>
<td>72</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.6</td>
<td>21.932</td>
<td>13.159</td>
<td>120</td>
<td>72</td>
</tr>
</tbody>
</table>

Lane Loadings
Location: Ex5 - Dual carriageway

Type HA uniformly distributed loading (UDL)

and knife edge loading (KEL)

Calculations are in accordance with Departmental Standard BD 37/01 "Loads for Highway Structures". References are to clauses in the composite version of BS5400-2 contained in Appendix A of BD 37/01.

Number of carriageways on superstructure \( \text{noc}=2 \)

Width of carriageway \( \text{cw}(1)=4.5 \text{ m} \)

Width of carriageway \( \text{cw}(2)=3.5 \text{ m} \)

Loaded length for HA UDL \( \text{ll}=45 \text{ m} \)

Loaded length is less than 50m

Nominal HA UDL expressed in kN/m (per notional lane).

HA UDL \( W=336\times(1/\text{ll})^{0.67}=26.223 \text{ kN/m} \)

Knife Edge Load (KEL) \( \text{KEL}=120 \text{ kN} \) (per notional lane)

Loading for carriageways

<table>
<thead>
<tr>
<th></th>
<th>Factor ( \beta )</th>
<th>Nominal HA UDL (kN/m) per lane</th>
<th>Factored HA UDL (kN/m) per lane</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1</td>
<td>26.223</td>
<td>26.223</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1</td>
<td>26.223</td>
<td>26.223</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Remaining area</td>
<td>1.0</td>
<td>kN/m²</td>
<td>kN/m²</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Lane Loadings
Location: Ex6 - Dual carriageway

Type HA uniformly distributed loading (UDL) and knife edge loading (KEL)

Calculations are in accordance with Departmental Standard BD 37/01 "Loads for Highway Structures". References are to clauses in the composite version of BS5400-2 contained in Appendix A of BD 37/01.

Number of carriageways on superstructure \( \text{noc}=2 \)

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Width of carriageway ( \text{cw}(1) )</td>
<td>7.8 m</td>
</tr>
<tr>
<td>Width of carriageway ( \text{cw}(2) )</td>
<td>4 m</td>
</tr>
<tr>
<td>Loaded length for HA UDL ( \text{ll} )</td>
<td>30 m</td>
</tr>
</tbody>
</table>

Loaded length is less than 50m

Nominal HA UDL expressed in kN/m (per notional lane).

- HA UDL \( W=336 \times (1/11)^0.67=34.409 \text{ kN/m} \)
- Knife Edge Load (KEL) \( \text{KEL}=120 \text{ kN (per notional lane)} \)
**Loading for carriageway 1**

<table>
<thead>
<tr>
<th></th>
<th>Factor $\beta$</th>
<th>Nominal HA UDL (kN/m) per lane</th>
<th>Factored HA UDL (kN/m) per lane</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.85616</td>
<td>34.409</td>
<td>29.459</td>
<td>120</td>
<td>102.74</td>
</tr>
<tr>
<td>Second Lane</td>
<td>0.85616</td>
<td>34.409</td>
<td>29.459</td>
<td>120</td>
<td>102.74</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.6</td>
<td>34.409</td>
<td>20.645</td>
<td>120</td>
<td>72</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.5137</td>
<td>34.409</td>
<td>17.676</td>
<td>120</td>
<td>61.644</td>
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</tbody>
</table>

**Loading for carriageway 2**

<table>
<thead>
<tr>
<th></th>
<th>Factor $\beta$</th>
<th>Nominal HA UDL (kN/m) per lane</th>
<th>Factored HA UDL (kN/m) per lane</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.84247</td>
<td>34.409</td>
<td>28.988</td>
<td>120</td>
<td>101.1</td>
</tr>
<tr>
<td>Second Lane</td>
<td>0.84247</td>
<td>34.409</td>
<td>28.988</td>
<td>120</td>
<td>101.1</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.84247</td>
<td>34.409</td>
<td>28.988</td>
<td>120</td>
<td>101.1</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.84247</td>
<td>34.409</td>
<td>28.988</td>
<td>120</td>
<td>101.1</td>
</tr>
<tr>
<td>Remaining area</td>
<td>1.0</td>
<td>kN/m²</td>
<td>kN/m²</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Location: Ex7 - Dual carriageway

Type HA uniformly distributed loading (UDL) and knife edge loading (KEL)

Calculations are in accordance with Departmental Standard BD 37/01 "Loads for Highway Structures". References are to clauses in the composite version of BS5400-2 contained in Appendix A of BD 37/01.

Number of carriageways on superstructure noc=2

Width of carriageway cw(1)=4.2 m
Width of carriageway cw(2)=8.5 m
Loaded length for HA UDL ll=5 m
Loaded length is less than 50m
Nominal HA UDL expressed in kN/m (per notional lane).

HA UDL W=336*(1/ll)^0.67=114.3 kN/m

Knife Edge Load (KEL) KEL=120 kN (per notional lane)
### Loading for carriageway 1

<table>
<thead>
<tr>
<th></th>
<th>Factor $\beta$</th>
<th>Nominal HA UDL (kN/m)</th>
<th>Factored HA UDL (kN/m)</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.68493</td>
<td>114.3</td>
<td>78.285</td>
<td>120</td>
<td>82.192</td>
</tr>
<tr>
<td>Second Lane</td>
<td>0.68493</td>
<td>114.3</td>
<td>78.285</td>
<td>120</td>
<td>82.192</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.68493</td>
<td>114.3</td>
<td>78.285</td>
<td>120</td>
<td>82.192</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.68493</td>
<td>114.3</td>
<td>78.285</td>
<td>120</td>
<td>82.192</td>
</tr>
<tr>
<td>Remaining area</td>
<td>1.0</td>
<td>$5.00$</td>
<td>$5.00$</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

#### Lane Loadings

### Loading for carriageway 2

<table>
<thead>
<tr>
<th></th>
<th>Factor $\beta$</th>
<th>Nominal HA UDL (kN/m)</th>
<th>Factored HA UDL (kN/m)</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.77626</td>
<td>114.3</td>
<td>88.723</td>
<td>120</td>
<td>93.151</td>
</tr>
<tr>
<td>Second Lane</td>
<td>0.77626</td>
<td>114.3</td>
<td>88.723</td>
<td>120</td>
<td>93.151</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.6</td>
<td>114.3</td>
<td>68.577</td>
<td>120</td>
<td>72</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.46575</td>
<td>114.3</td>
<td>53.234</td>
<td>120</td>
<td>55.89</td>
</tr>
</tbody>
</table>

#### Lane Loadings
Location: Ex8 - Single carriageway

Type HA uniformly distributed loading (UDL) and knife edge loading (KEL)

Calculations are in accordance with Departmental Standard BD 37/01 "Loads for Highway Structures". References are to clauses in the composite version of BS5400-2 contained in Appendix A of BD 37/01.

Number of carriageways on superstructure \( noc = 1 \)

Width of carriageway \( cw(1) = 15 \text{ m} \)

Loaded length for HA UDL \( ll = 60 \text{ m} \)

Loaded length is greater than or equal to 50m

Nominal HA UDL expressed in kN/m (per notional lane).

HA UDL \( W = 36 \times (1/ll)^{0.10} = 23.905 \text{ kN/m} \)

Knife Edge Load (KEL) \( KEL = 120 \text{ kN} \) (per notional lane)

Loading for carriageway

<table>
<thead>
<tr>
<th></th>
<th>Factor ( \beta )</th>
<th>Nominal HA UDL (kN/m) per lane</th>
<th>Factored HA UDL (kN/m) per lane</th>
<th>Nominal KEL (kN) per lane</th>
<th>Factored KEL (kN) per lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1</td>
<td>23.905</td>
<td>23.905</td>
<td>120</td>
<td>120</td>
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<tr>
<td>Second Lane</td>
<td>1</td>
<td>23.905</td>
<td>23.905</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.6</td>
<td>23.905</td>
<td>14.343</td>
<td>120</td>
<td>72</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.6</td>
<td>23.905</td>
<td>14.343</td>
<td>120</td>
<td>72</td>
</tr>
</tbody>
</table>

Lane Loadings
Location: Ex1 - Single carriageway

Load Model 1 (LM1) - concentrated & uniformly distributed loading

The calculations are in accordance with BS EN 1991-2:2003 and the NA to BS EN 1991-2:2003 entitled Eurocode 1: Actions on structures Part 2: Traffic loads on bridges. Load model 1 represents normal loading (normal traffic) and comprises of two components: a UDL over the full width of the traffic lane and a pair of axles (this is also referred to as a Tandem System TS). In each lane (up to a maximum of three lanes) a pair of axles should be positioned centrally in the lane at a position along the lane that causes maximum adverse effect (Clause 4.3.1 & 4.3.2). Load Model 3 (LM3) represents abnormal vehicles, also needs to be considered where appropriate. The latter is beyond the scope of this proforma.

Number of carriageways: noc=1
Width of carriageway: cw(1)=3 m
Loaded length for UDL: ll=40 m

Loading for carriageway - characteristic values

<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor $\alpha q_1$</th>
<th>UDL system ($kN/m^2$)</th>
<th>Adjustment factor $\alpha Q_1$</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
</tbody>
</table>

Lane Loadings

NOTE: It is probably much easier to apply a Knife Edge Load (KEL) for each axle (i.e. distribute the weight of one axle along a line across the lane) than two isolated wheel loads, and this is considered to be adequate for global analysis. For carriageway Lane 1, the four wheels of the tandems together weigh 600 kN (i.e. 150 kN per wheel).

Carriageway 1:
KEL=$Q_1/k/wl(1)=100$ kN
UDL=$5.49$ kN/m
Axles spaced at 1.2 m apart

Lane 1: Loads per metre width of deck
Location: Ex2 - Dual carriageway

Load Model 1 (LM1) - concentrated & uniformly distributed loading

The calculations are in accordance with BS EN 1991-2:2003 and the NA to BS EN 1991-2:2003 entitled Eurocode 1: Actions on structures Part 2: Traffic loads on bridges. Load model 1 represents normal loading (normal traffic) and comprises of two components: a UDL over the full width of the traffic lane and a pair of axles (this is also referred to as a Tandem System TS). In each lane (up to a maximum of three lanes) a pair of axles should be positioned centrally in the lane at a position along the lane that causes maximum adverse effect (Clause 4.3.1 & 4.3.2). Load Model 3 (LM3) represents abnormal vehicles, also needs to be considered where appropriate. The latter is beyond the scope of this proforma.

Number of carriageways  noc=2
Width of carriageway  cw(1)=15 m
Width of carriageway  cw(2)=15 m
Loaded length for UDL  ll=120 m

Loading for carriageways - characteristic values

<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor ( \alpha_{q1} )</th>
<th>UDL system (kN/m²)</th>
<th>Adjustment factor ( \alpha_{Q1} )</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Third Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>100 kN</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

Lane Loadings

NOTE: It is probably much easier to apply a Knife Edge Load (KEL) for each axle (i.e. distribute the weight of one axle along a line across the lane) than two isolated wheel loads, and this is considered to be adequate for global analysis. For carriageway Lane 1, the four wheels of the tandems together weigh 600 kN (i.e. 150 kN per wheel).

KEL
\[ 1.2 \]

Carriageway 1:
KEL\( =Q_{k}/wl(1)\)=100 kN
UDL=5.49 kN/m

Carriageway 2:
KEL\( =Q_{k}/wl(2)\)=100 kN
UDL=5.49 kN/m

Lane 1: Loads per metre width of deck
axles spaced at 1.2 m apart
**Location: Ex3 - Dual carriageway**

**Load Model 1 (LM1) - concentrated & uniformly distributed loading**

The calculations are in accordance with BS EN 1991-2:2003 and the NA to BS EN 1991-2:2003 entitled Eurocode 1: Actions on structures Part 2: Traffic loads on bridges. Load model 1 represents normal loading (normal traffic) and comprises of two components: a UDL over the full width of the traffic lane and a pair of axles (this is also referred to as a Tandem System TS). In each lane (up to a maximum of three lanes) a pair of axles should be positioned centrally in the lane at a position along the lane that causes maximum adverse effect (Clause 4.3.1 & 4.3.2). Load Model 3 (LM3) represents abnormal vehicles, also needs to be considered where appropriate. The latter is beyond the scope of this proforma.

Number of carriageways: noc=2
Width of carriageway: cw(1)=4.5 m
Width of carriageway: cw(2)=4.5 m
Loaded length for UDL: ll=140 m

**Loading for carriageways - characteristic values**

<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor α_q1</th>
<th>UDL system (kN/m^2)</th>
<th>Adjustment factor α_Q1</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Remaining area</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

**Lane Loadings**

NOTE: It is probably much easier to apply a Knife Edge Load (KEL) for each axle (i.e. distribute the weight of one axle along a line across the lane) than two isolated wheel loads, and this is considered to be adequate for global analysis. For carriageway Lane 1, the four wheels of the tandems together weigh 600 kN (i.e. 150 kN per wheel).

Carriageway 1:
KEL=Q1k/wl(1)=100 kN
UDL=5.49 kN/m

Carriageway 2:
KEL=Q1k/wl(2)=100 kN
UDL=5.49 kN/m

Lane 1: Loads per metre width of deck
axles spaced at 1.2 m apart
**Location: Ex4 - Dual carriageway**

**Load Model 1 (LM1) - concentrated & uniformly distributed loading**

The calculations are in accordance with BS EN 1991-2:2003 and the NA to BS EN 1991-2:2003 entitled Eurocode 1: Actions on structures Part 2: Traffic loads on bridges. Load model 1 represents normal loading (normal traffic) and comprises of two components: a UDL over the full width of the traffic lane and a pair of axles (this is also referred to as a Tandem System TS). In each lane (up to a maximum of three lanes) a pair of axles should be positioned centrally in the lane at a position along the lane that causes maximum adverse effect (Clause 4.3.1 & 4.3.2). Load Model 3 (LM3) represents abnormal vehicles, also needs to be considered where appropriate. The latter is beyond the scope of this proforma.

- Number of carriageways: noc=2
- Width of carriageway: cw(1)=11 m
- Width of carriageway: cw(2)=15 m
- Loaded length for UDL: l=142 m

**Loading for carriageways - characteristic values**

<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor αq1</th>
<th>UDL system (kN/m²)</th>
<th>Adjustment factor αQ1</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Third Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>100 kN</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Remaining area</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

**Lane Loadings**

NOTE: It is probably much easier to apply a Knife Edge Load (KEL) for each axle (i.e. distribute the weight of one axle along a line across the lane) than two isolated wheel loads, and this is considered to be adequate for global analysis. For carriageway Lane 1, the four wheels of the tandems together weigh 600 kN (i.e. 150 kN per wheel).
Lane 1: Loads per metre width of deck
axles spaced at 1.2 m apart

Carriageway 1:
KEL = Q1k/wl(1) = 100 kN
UDL = 5.49 kN/m

Carriageway 2:
KEL = Q1k/wl(2) = 100 kN
UDL = 5.49 kN/m
**Location: Ex5 - Dual carriageway**

**Load Model 1 (LM1) - concentrated & uniformly distributed loading**

The calculations are in accordance with BS EN 1991-2:2003 and the NA to BS EN 1991-2:2003 entitled Eurocode 1: Actions on structures Part 2: Traffic loads on bridges. Load model 1 represents normal loading (normal traffic) and comprises of two components: a UDL over the full width of the traffic lane and a pair of axles (this is also referred to as a Tandem System TS). In each lane (up to a maximum of three lanes) a pair of axles should be positioned centrally in the lane at a position along the lane that causes maximum adverse effect (Clause 4.3.1 & 4.3.2). Load Model 3 (LM3) represents abnormal vehicles, also needs to be considered where appropriate. The latter is beyond the scope of this proforma.

Number of carriageways \( \text{noc} = 2 \)

Width of carriageway \( \text{cw}(1) = 4.5 \text{ m} \)

Width of carriageway \( \text{cw}(2) = 3.5 \text{ m} \)

Loaded length for UDL \( l_l = 45 \text{ m} \)

### Loading for carriageways - characteristic values

<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor ( \alpha_{Q1} )</th>
<th>UDL system ( Q_1 ) ( (\text{kN/m}^2) )</th>
<th>Adjustment factor ( \alpha_{Q1} )</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Remaining area</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

**Lane Loadings**

NOTE: It is probably much easier to apply a Knife Edge Load (KEL) for each axle (i.e. distribute the weight of one axle along a line across the lane) than two isolated wheel loads, and this is considered to be adequate for global analysis. For carriageway Lane 1, the four wheels of the tandems together weigh 600 kN (i.e. 150 kN per wheel).

\[
\begin{align*}
\text{KEL} & \quad 1.2 \quad \text{KEL} \\
\text{Carriageway 1:} & \\
& \text{KEL} = Q_1 k / w l(1) = 100 \text{ kN} \\
& \text{UDL} = 5.49 \text{ kN/m} \\
& \text{Carriageway 2:} \\
& \text{KEL} = Q_1 k / w l(2) = 100 \text{ kN} \\
& \text{UDL} = 5.49 \text{ kN/m} \\
\end{align*}
\]

Lane 1: Loads per metre width of deck axles spaced at 1.2 m apart
Location: Ex6 - Dual carriageway

Load Model 1 (LM1) - concentrated & uniformly distributed loading

The calculations are in accordance with BS EN 1991-2:2003 and the NA to BS EN 1991-2:2003 entitled Eurocode 1: Actions on structures Part 2: Traffic loads on bridges. Load model 1 represents normal loading (normal traffic) and comprises of two components: a UDL over the full width of the traffic lane and a pair of axles (this is also referred to as a Tandem System TS). In each lane (up to a maximum of three lanes) a pair of axles should be positioned centrally in the lane at a position along the lane that causes maximum adverse effect (Clause 4.3.1 & 4.3.2). Load Model 3 (LM3) represents abnormal vehicles, also needs to be considered where appropriate. The latter is beyond the scope of this proforma.

Number of carriageways: noc=2
Width of carriageway: cw(1)=7.8 m, cw(2)=4 m
Loaded length for UDL: ll=30 m

Loading for carriageway 1 - characteristic values

<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor αq1</th>
<th>UDL system (kN/m²)</th>
<th>Adjustment factor αQ1</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Third Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>100 kN</td>
</tr>
<tr>
<td>Remaining area</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

Lane Loadings
<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor $q_{1}$</th>
<th>UDL system $(kN/m^2)$</th>
<th>Adjustment factor $q_{1}$</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Third Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>100 kN</td>
</tr>
<tr>
<td>Remaining area</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

**Lane Loadings**

NOTE: It is probably much easier to apply a Knife Edge Load (KEL) for each axle (i.e. distribute the weight of one axle along a line across the lane) than two isolated wheel loads, and this is considered to be adequate for global analysis. For carriageway Lane 1, the four wheels of the tandems together weigh 600 kN (i.e. 150 kN per wheel).

Lane 1: Loads per metre width of deck
axles spaced at 1.2 m apart

Carriageway 1:
- KEL = $Q_{1}k/w(1)$ = 100 kN
- UDL = 5.49 kN/m
- KEL = $Q_{1}k/w(2)$ = 100 kN
- UDL = 5.49 kN/m
Location: Ex7 - Dual carriageway

Load Model 1 (LM1) - concentrated & uniformly distributed loading

The calculations are in accordance with BS EN 1991-2:2003 and the NA to BS EN 1991-2:2003 entitled Eurocode 1: Actions on structures Part 2: Traffic loads on bridges. Load model 1 represents normal loading (normal traffic) and comprises of two components: a UDL over the full width of the traffic lane and a pair of axles (this is also referred to as a Tandem System TS). In each lane (up to a maximum of three lanes) a pair of axles should be positioned centrally in the lane at a position along the lane that causes maximum adverse effect (Clause 4.3.1 & 4.3.2). Load Model 3 (LM3) represents abnormal vehicles, also needs to be considered where appropriate. The latter is beyond the scope of this proforma.

Number of carriageways: noc=2
Width of carriageway: cw(1)=4.2 m
Width of carriageway: cw(2)=8.5 m
Loaded length for UDL: ll=5 m

Loading for carriageway 1 - characteristic values

<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor ( \alpha_{Q1} )</th>
<th>UDL system (kN/m²)</th>
<th>Adjustment factor ( \alpha_{Q1} )</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Third Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>100 kN</td>
</tr>
<tr>
<td>Remaining area</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

Lane Loadings
### Loading for carriageway 2 - characteristic values

<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor $\alpha_{Q1}$</th>
<th>UDL system (kN/m²)</th>
<th>Adjustment factor $\alpha_{Q1}$</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Third Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>100 kN</td>
</tr>
<tr>
<td>Remaining area</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

#### Lane Loadings

NOTE: It is probably much easier to apply a Knife Edge Load (KEL) for each axle (i.e. distribute the weight of one axle along a line across the lane) than two isolated wheel loads, and this is considered to be adequate for global analysis. For carriageway Lane 1, the four wheels of the tandems together weigh 600 kN (i.e. 150 kN per wheel).

- **Carriageway 1:**
  - KEL = $Q_1k/wl(1) = 100$ kN
  - UDL = 5.49 kN/m

- **Carriageway 2:**
  - KEL = $Q_1k/wl(2) = 100$ kN
  - UDL = 5.49 kN/m

Lane 1: Loads per metre width of deck axes spaced at 1.2 m apart
Location: Ex8 - Single carriageway

Load Model 1 (LM1) - concentrated & uniformly distributed loading

The calculations are in accordance with BS EN 1991-2:2003 and the NA to BS EN 1991-2:2003 entitled Eurocode 1: Actions on structures Part 2: Traffic loads on bridges. Load model 1 represents normal loading (normal traffic) and comprises of two components: a UDL over the full width of the traffic lane and a pair of axles (this is also referred to as a Tandem System TS). In each lane (up to a maximum of three lanes) a pair of axles should be positioned centrally in the lane at a position along the lane that causes maximum adverse effect (Clause 4.3.1 & 4.3.2). Load Model 3 (LM3) represents abnormal vehicles, also needs to be considered where appropriate. The latter is beyond the scope of this proforma.

Number of carriageways \( n_{oc} = 1 \)
Width of carriageway \( c_w(1) = 15 \text{ m} \)
Loaded length for UDL \( l_l = 60 \text{ m} \)

Loading for carriageway - characteristic values

<table>
<thead>
<tr>
<th>Lane</th>
<th>Adjustment factor ( \alpha_q )</th>
<th>UDL system ( \text{(kN/m²)} )</th>
<th>Adjustment factor ( \alpha_{Q1} )</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Third Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>100 kN</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

Lane Loadings

NOTE: It is probably much easier to apply a Knife Edge Load (KEL) for each axle (i.e. distribute the weight of one axle along a line across the lane) than two isolated wheel loads, and this is considered to be adequate for global analysis. For carriageway Lane 1, the four wheels of the tandems together weigh 600 kN (i.e. 150 kN per wheel).

\[
\text{KEL = } 1.2 \times 100 \text{ kN} \\
\text{KEL = } 5.49 \text{ kN/m} \\
\text{Axles spaced at } 1.2 \text{ m apart}
\]

Lane 1: Loads per metre width of deck

SCALE 5.48 Office 1007 Proforma 130
Location: Ex9 - Dual carriageway

Load Model 1 (LM1) - concentrated & uniformly distributed loading

The calculations are in accordance with BS EN 1991-2:2003 and the NA to BS EN 1991-2:2003 entitled Eurocode 1: Actions on structures Part 2: Traffic loads on bridges. Load model 1 represents normal loading (normal traffic) and comprises of two components: a UDL over the full width of the traffic lane and a pair of axles (this is also referred to as a Tandem System TS). In each lane (up to a maximum of three lanes) a pair of axles should be positioned centrally in the lane at a position along the lane that causes maximum adverse effect (Clause 4.3.1 & 4.3.2). Load Model 3 (LM3) represents abnormal vehicles, also needs to be considered where appropriate. The latter is beyond the scope of this proforma.

Number of carriageways  

noc=2

Width of carriageway  

cw(1)=7.8 m

Width of carriageway  

cw(2)=5.4 m

Loaded length for UDL  

ll=30 m

Loading for carriageway 1 - characteristic values

<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor $\alpha q_l$</th>
<th>UDL system (kN/m²)</th>
<th>Adjustment factor $\alpha Q_1$</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Third Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>100 kN</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Remaining area</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

Lane Loadings
### Loading for carriageway 2 - characteristic values

<table>
<thead>
<tr>
<th></th>
<th>Adjustment factor $\alpha_q$</th>
<th>UDL system (kN/m²)</th>
<th>Adjustment factor $\alpha Q_l$</th>
<th>Tandem system TS Axle loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>0.61</td>
<td>5.49</td>
<td>1</td>
<td>300 kN</td>
</tr>
<tr>
<td>Second Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>200 kN</td>
</tr>
<tr>
<td>Third Lane</td>
<td>2.2</td>
<td>5.5</td>
<td>1</td>
<td>100 kN</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Remaining area</td>
<td>2.2</td>
<td>5.5</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

**Lane Loadings**

NOTE: It is probably much easier to apply a Knife Edge Load (KEL) for each axle (i.e. distribute the weight of one axle along a line across the lane) than two isolated wheel loads, and this is considered to be adequate for global analysis. For carriageway Lane 1, the four wheels of the tandems together weigh 600 kN (i.e. 150 kN per wheel).

**Carriageway 1:**
- $KEL = Q_1 k/wl(1) = 100$ kN
- $UDL = 5.49$ kN/m

**Carriageway 2:**
- $KEL = Q_1 k/wl(2) = 111.11$ kN
- $UDL = 5.49$ kN/m
Location: Single carriageway

Type HA uniformly distributed loading (UDL) and knife edge loading (KEL); Load Reduction Factors K

Highway carriageway and lanes

Number of carriageways on superstructure: noc=1
Width of carriageway: cw(1)=2.8 m

Loading for carriageway

Loaded length for HA UDL: ll=40 m

Adjustment Factor (AF) = 1

Nominal HA Load x Lane Factor

Factored HA Load = \frac{\text{Nominal HA Load} \times \text{Lane Factor}}{\text{Adjustment Factor}}

<table>
<thead>
<tr>
<th>Lane Factor</th>
<th>Nominal HA UDL kN/lin.m</th>
<th>Factored HA UDL kN/lin.m</th>
<th>Nominal HA KEL kN/lane</th>
<th>Factored HA KEL kN/lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1.0</td>
<td>28.377</td>
<td>28.377</td>
<td>120</td>
</tr>
</tbody>
</table>

The lane loading for any lane shall be applied to occupy a width of 2.5m in the most onerous transverse position in that lane. The remainder of any notional lane shall not be loaded with live loading.
Reduction Factor (K) for HA UDL and KEL (Clause 5.20).
Traffic flow is categorised High (H)
Road surface profile is categorised Good (g)

<table>
<thead>
<tr>
<th>Assessment Live Loadings</th>
<th>Reduction Factors for Type HA UDL and KEL Loaded length 40 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 tonnes</td>
<td>0.81</td>
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<tr>
<td>26 tonnes</td>
<td>0.78</td>
</tr>
<tr>
<td>18 tonnes</td>
<td>0.61</td>
</tr>
<tr>
<td>7.5 tonnes</td>
<td>0.311</td>
</tr>
<tr>
<td>3.0 tonnes</td>
<td>0.311</td>
</tr>
<tr>
<td>Group 1 Fire Engine</td>
<td>0.5</td>
</tr>
<tr>
<td>Group 2 Fire Engine</td>
<td>0.311</td>
</tr>
</tbody>
</table>

Figure 5/5. K Factors for Heavy Traffic Good Surface (Hg)
**Location:** Dual carriageway

**Type HA uniformly distributed loading (UDL) and knife edge loading (KEL); Load Reduction Factors K**

**Highway carriageway and lanes**

- Number of carriageways on superstructure: $\text{noc} = 2$
- Width of carriageway: $\text{cw}(1)=15 \text{ m}$
- Width of carriageway: $\text{cw}(2)=15 \text{ m}$
- Number of notional traffic lanes: $\text{nl}(2)=5$

**Loading for carriageways**

- Loaded length for HA UDL: $\text{ll}=35 \text{ m}$

Adjustment Factor (AF) = 1.115

$$\text{Factored HA Load} = \frac{\text{Nominal HA Load} \times \text{Lane Factor}}{\text{Adjustment Factor}}$$

<table>
<thead>
<tr>
<th>Lane Factor</th>
<th>Nominal HA UDL kN/lin.m</th>
<th>Factored HA UDL kN/lin.m</th>
<th>Nominal HA KEL kN/lane</th>
<th>Factored HA KEL kN/lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1.0</td>
<td>31.032</td>
<td>27.832</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1.0</td>
<td>31.032</td>
<td>27.832</td>
<td>120</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.5</td>
<td>31.032</td>
<td>13.916</td>
<td>120</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.4</td>
<td>31.032</td>
<td>11.133</td>
<td>120</td>
</tr>
</tbody>
</table>

The lane loading for any lane shall be applied to occupy a width of 2.5m in the most onerous transverse position in that lane. The remainder of any notional lane shall not be loaded with live loading.
Reduction Factor (K) for HA UDL and KEL (Clause 5.20).

Traffic flow is categorised High (H)

Road surface profile is categorised Poor (p)

<table>
<thead>
<tr>
<th>Assessment Live Loadings</th>
<th>Reduction Factors for Type HA UDL and KEL Loaded length 35 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 tonnes</td>
<td>0.91</td>
</tr>
<tr>
<td>26 tonnes</td>
<td>0.79</td>
</tr>
<tr>
<td>18 tonnes</td>
<td>0.63</td>
</tr>
<tr>
<td>7.5 tonnes</td>
<td>0.291</td>
</tr>
<tr>
<td>3.0 tonnes</td>
<td>0.291</td>
</tr>
<tr>
<td>Group 1 Fire Engine</td>
<td>0.5</td>
</tr>
<tr>
<td>Group 2 Fire Engine</td>
<td>0.291</td>
</tr>
</tbody>
</table>

Figure 5/2. K Factors for Heavy Traffic Poor Surface (Hp)
Location: Dual carriageway

Type HA uniformly distributed loading (UDL) and knife edge loading (KEL); Load Reduction Factors K

Highway carriageway and lanes

Number of carriageways on superstructure \( n_{oc} = 2 \)
Width of carriageway \( cw(1) = 4.5 \text{ m} \)
Width of carriageway \( cw(2) = 4.5 \text{ m} \)
Number of notional traffic lanes \( n_{l}(2) = 1 \)

Loading for carriageways

Loaded length for HA UDL \( ll = 10 \text{ m} \)

Adjustment Factor (AF) = 1.46

\[
\text{Factored HA Load} = \frac{\text{Nominal HA Load} \times \text{Lane Factor}}{\text{Adjustment Factor}}
\]

<table>
<thead>
<tr>
<th>Lane Factor</th>
<th>Nominal HA UDL kN/lin.m</th>
<th>Factored HA UDL kN/lin.m</th>
<th>Nominal HA KEL kN/lane</th>
<th>Factored HA KEL kN/lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1.0</td>
<td>71.836</td>
<td>49.202</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1.0</td>
<td>71.836</td>
<td>49.202</td>
<td>120</td>
</tr>
</tbody>
</table>

The lane loading for any lane shall be applied to occupy a width of 2.5m in the most onerous transverse position in that lane. The remainder of any notional lane shall not be loaded with live loading.
Reduction Factor (K) for HA UDL and KEL (Clause 5.20).
Traffic flow is categorised Medium (M)
Road surface profile is categorised Good (g)

<table>
<thead>
<tr>
<th>Assessment Live Loadings</th>
<th>Reduction Factors for Type HA UDL and KEL Loaded length 10 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 tonnes</td>
<td>0.79</td>
</tr>
<tr>
<td>26 tonnes</td>
<td>0.74</td>
</tr>
<tr>
<td>18 tonnes</td>
<td>0.61</td>
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<tr>
<td>7.5 tonnes</td>
<td>0.37</td>
</tr>
<tr>
<td>3.0 tonnes</td>
<td>0.2</td>
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<tr>
<td>Group 1 Fire Engine</td>
<td>0.468</td>
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<tr>
<td>Group 2 Fire Engine</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Figure 5/6. K Factors for Medium Traffic Good Surface (Mg)
Location: Dual carriageway

Type HA uniformly distributed loading ( UDL ) and knife edge loading ( KEL ); Load Reduction Factors K

Highway carriageway and lanes

Number of carriageways on superstructure noc=2
Width of carriageway cw(1)=11 m
Number of notional traffic lanes nl(1)=4
Width of carriageway cw(2)=15 m
Number of notional traffic lanes nl(2)=5

Loading for carriageways

Loaded length for HA UDL ll=14 m

Adjustment Factor (AF) = 1.46

Factored HA Load = \( \frac{\text{Nominal HA Load} \times \text{Lane Factor}}{\text{Adjustment Factor}} \)

<table>
<thead>
<tr>
<th>Lane Factor</th>
<th>Nominal HA UDL kN/lin.m</th>
<th>Factored HA UDL kN/lin.m</th>
<th>Nominal HA KEL kN/lane</th>
<th>Factored HA KEL kN/lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1.0</td>
<td>57.337</td>
<td>39.272</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1.0</td>
<td>57.337</td>
<td>39.272</td>
<td>120</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.5</td>
<td>57.337</td>
<td>19.636</td>
<td>120</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.4</td>
<td>57.337</td>
<td>15.709</td>
<td>120</td>
</tr>
</tbody>
</table>

The lane loading for any lane shall be applied to occupy a width of 2.5m in the most onerous transverse position in that lane. The remainder of any notional lane shall not be loaded with live loading.
Reduction Factor (K) for HA UDL and KEL (Clause 5.20).
Traffic flow is categorised Medium (M)
Road surface profile is categorised Poor (p)

<table>
<thead>
<tr>
<th>Assessment Live Loadings</th>
<th>Reduction Factors for Type HA UDL and KEL Loaded length 14 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 tonnes</td>
<td>0.892</td>
</tr>
<tr>
<td>26 tonnes</td>
<td>0.84</td>
</tr>
<tr>
<td>18 tonnes</td>
<td>0.63333</td>
</tr>
<tr>
<td>7.5 tonnes</td>
<td>0.36667</td>
</tr>
<tr>
<td>3.0 tonnes</td>
<td>0.208</td>
</tr>
<tr>
<td>Group 1 Fire Engine</td>
<td>0.462</td>
</tr>
<tr>
<td>Group 2 Fire Engine</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Figure 5/3. K Factors for Medium Traffic Poor Surface (Mp)
Location: Dual carriageway

Type HA uniformly distributed loading (UDL) and knife edge loading (KEL); Load Reduction Factors K

Highway carriageway and lanes

Number of carriageways on superstructure noc=2
Width of carriageway cw(1)=4.5 m
Width of carriageway cw(2)=3.5 m

Loading for carriageways

Loaded length for HA UDL ll=39 m

Adjustment Factor (AF) = 1.023

Factored HA Load = \[
\frac{\text{Nominal HA Load} \times \text{Lane Factor}}{\text{Adjustment Factor}}
\]

<table>
<thead>
<tr>
<th>Lane Factor</th>
<th>Nominal HA UDL kN/lin.m</th>
<th>Factored HA UDL kN/lin.m</th>
<th>Nominal HA KEL kN/lane</th>
<th>Factored HA KEL kN/lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1.0</td>
<td>28.862</td>
<td>28.213</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1.0</td>
<td>28.862</td>
<td>28.213</td>
<td>120</td>
</tr>
</tbody>
</table>

The lane loading for any lane shall be applied to occupy a width of 2.5m in the most onerous transverse position in that lane. The remainder of any notional lane shall not be loaded with live loading.
Reduction Factor \((K)\) for HA UDL and KEL (Clause 5.20).
Traffic flow is categorised Low (L)
Road surface profile is categorised Good (g)

<table>
<thead>
<tr>
<th>Assessment Live Loadings</th>
<th>Reduction Factors for Type HA UDL and KEL Loaded length 39 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 tonnes</td>
<td>0.76</td>
</tr>
<tr>
<td>26 tonnes</td>
<td>0.69</td>
</tr>
<tr>
<td>18 tonnes</td>
<td>0.54</td>
</tr>
<tr>
<td>7.5 tonnes</td>
<td>0.307</td>
</tr>
<tr>
<td>3.0 tonnes</td>
<td>0.307</td>
</tr>
<tr>
<td>Group 1 Fire Engine</td>
<td>0.5</td>
</tr>
<tr>
<td>Group 2 Fire Engine</td>
<td>0.307</td>
</tr>
</tbody>
</table>

Figure 5/7. K Factors for Low Traffic Good Surface (Lg)
Location: Dual carriageway

Type HA uniformly distributed loading ( UDL ) and knife edge loading ( KEL ); Load Reduction Factors K

Highway carriageway and lanes

Number of carriageways on superstructure  noc=2
Width of carriageway  cw(1)=7.8 m
Width of carriageway  cw(2)=4 m

Loading for carriageways

Loaded length for HA UDL  ll=30 m

Adjustment Factor (AF) = 1.23

Factored HA Load = \frac{\text{Nominal HA Load} \times \text{Lane Factor}}{\text{Adjustment Factor}}

<table>
<thead>
<tr>
<th>Lane Factor</th>
<th>Nominal HA UDL kN/lin.m</th>
<th>Factored HA UDL kN/lin.m</th>
<th>Nominal HA KEL kN/lane</th>
<th>Factored HA KEL kN/lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1.0</td>
<td>34.409</td>
<td>27.975</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1.0</td>
<td>34.409</td>
<td>27.975</td>
<td>120</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.5</td>
<td>34.409</td>
<td>13.987</td>
<td>120</td>
</tr>
<tr>
<td>Fourth Lane</td>
<td>0.4</td>
<td>34.409</td>
<td>11.19</td>
<td>120</td>
</tr>
</tbody>
</table>

The lane loading for any lane shall be applied to occupy a width of 2.5m in the most onerous transverse position in that lane. The remainder of any notional lane shall not be loaded with live loading.
Reduction Factor (K) for HA UDL and KEL (Clause 5.20).
Traffic flow is categorised Low (L)
Road surface profile is categorised Poor (p)

<table>
<thead>
<tr>
<th>Assessment Live Loadings</th>
<th>Reduction Factors for Type HA UDL and KEL Loaded length 30 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 tonnes</td>
<td>0.86</td>
</tr>
<tr>
<td>26 tonnes</td>
<td>0.72</td>
</tr>
<tr>
<td>18 tonnes</td>
<td>0.56</td>
</tr>
<tr>
<td>7.5 tonnes</td>
<td>0.271</td>
</tr>
<tr>
<td>3.0 tonnes</td>
<td>0.271</td>
</tr>
<tr>
<td>Group 1 Fire Engine</td>
<td>0.5</td>
</tr>
<tr>
<td>Group 2 Fire Engine</td>
<td>0.271</td>
</tr>
</tbody>
</table>

Figure 5/4. K Factors for Low Traffic Poor Surface (Lp)
Location: Dual carriageway

Type HA uniformly distributed loading (UDL) and knife edge loading (KEL); Load Reduction Factors K

Highway carriageway and lanes

Number of carriageways on superstructure noc=2
Width of carriageway cw(1)=4.5 m
Width of carriageway cw(2)=8.5 m
Number of notional traffic lanes nl(2)=3

Loading for carriageways

Loaded length for HA UDL ll=7.5 m

Adjustment Factor (AF) = 1.46

Factored HA Load = \[ \frac{\text{Nominal HA Load} \times \text{Lane Factor}}{\text{Adjustment Factor}} \]

<table>
<thead>
<tr>
<th>Lane Factor</th>
<th>Nominal HA UDL kn/lin.m</th>
<th>Factored HA UDL kn/lin.m</th>
<th>Nominal HA KEL kn/lane</th>
<th>Factored HA KEL kn/lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1.0</td>
<td>87.106</td>
<td>59.662</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1.0</td>
<td>87.106</td>
<td>59.662</td>
<td>120</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.5</td>
<td>87.106</td>
<td>29.831</td>
<td>120</td>
</tr>
<tr>
<td>Fourth Lane</td>
<td>0.4</td>
<td>87.106</td>
<td>23.865</td>
<td>120</td>
</tr>
</tbody>
</table>

The lane loading for any lane shall be applied to occupy a width of 2.5m in the most onerous transverse position in that lane. The remainder of any notional lane shall not be loaded with live loading.
Reduction Factor (K) for HA UDL and KEL (Clause 5.20).
Traffic flow is categorised High (H)
Road surface profile is categorised Poor (p)

<table>
<thead>
<tr>
<th>Assessment Live Loadings</th>
<th>Reduction Factors for Type HA UDL and KEL Loaded length 7.5 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 tonnes</td>
<td>0.91</td>
</tr>
<tr>
<td>26 tonnes</td>
<td>0.89</td>
</tr>
<tr>
<td>18 tonnes</td>
<td>0.73</td>
</tr>
<tr>
<td>7.5 tonnes</td>
<td>0.39</td>
</tr>
<tr>
<td>3.0 tonnes</td>
<td>0.2</td>
</tr>
<tr>
<td>Group 1 Fire Engine</td>
<td>0.49</td>
</tr>
<tr>
<td>Group 2 Fire Engine</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Figure 5/2. K Factors for Heavy Traffic Poor Surface (Hp)
**Location:** Single carriageway

**Type HA uniformly distributed loading (UDL) and**

knife edge loading (KEL); Load Reduction Factors K

**Highway carriageway and lanes**

- Number of carriageways on superstructure, $n_{oc} = 1$
- Width of carriageway, $cw(1) = 15$ m
- Number of notional traffic lanes, $n_{l}(1) = 5$

**Loading for carriageway**

- Loaded length for HA UDL, $ll = 50$ m
- Adjustment Factor (AF) = 1
- Factored HA Load = \( \frac{Nominal \ HA \ Load \times Lane \ Factor}{Adjustment \ Factor} \)

<table>
<thead>
<tr>
<th>Lane Factor</th>
<th>Nominal HA UDL (kN/lin.m)</th>
<th>Factored HA UDL (kN/lin.m)</th>
<th>Nominal HA KEL (kN/lane)</th>
<th>Factored HA KEL (kN/lane)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Lane</td>
<td>1.0</td>
<td>24.436</td>
<td>24.436</td>
<td>120</td>
</tr>
<tr>
<td>Second Lane</td>
<td>1.0</td>
<td>24.436</td>
<td>24.436</td>
<td>120</td>
</tr>
<tr>
<td>Third Lane</td>
<td>0.5</td>
<td>24.436</td>
<td>12.218</td>
<td>60</td>
</tr>
<tr>
<td>Fourth and subsequent lanes</td>
<td>0.4</td>
<td>24.436</td>
<td>9.7744</td>
<td>48</td>
</tr>
</tbody>
</table>

The lane loading for any lane shall be applied to occupy a width of 2.5m in the most onerous transverse position in that lane. The remainder of any notional lane shall not be loaded with live loading.
Reduction Factor (K) for HA UDL and KEL (Clause 5.20).
Traffic flow is categorised Medium (M)
Road surface profile is categorised Good (g)

<table>
<thead>
<tr>
<th>Assessment Live Loadings</th>
<th>Reduction Factors for Type HA UDL and KEL Loaded length 50 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 tonnes</td>
<td>0.78</td>
</tr>
<tr>
<td>26 tonnes</td>
<td>0.73</td>
</tr>
<tr>
<td>18 tonnes</td>
<td>0.58</td>
</tr>
<tr>
<td>7.5 tonnes</td>
<td>0.35</td>
</tr>
<tr>
<td>3.0 tonnes</td>
<td>0.35</td>
</tr>
<tr>
<td>Group 1 Fire Engine</td>
<td>0.5</td>
</tr>
<tr>
<td>Group 2 Fire Engine</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Figure 5/6. K Factors for Medium Traffic Good Surface (Mg)
Location: Ex1 - Internal beam

Ultimate limit state: flexure of prestressed concrete section

Calculations are in accordance with BS5400:Part 4:1990 as implemented by DOT Departmental Standard BD 44/15.

Deck on Beams

Char. strength of concrete \( f_{cum} = 40 \text{ N/mm}^2 \)
Number of cross-sections \( ncs = 3 \)

Cross-section 1
Top of cross-section is the compression face.
Breadth at top \( x_{bt}(1) = 1000 \text{ mm} \)
Breadth at bottom \( x_{bb}(1) = 1000 \text{ mm} \)
Depth \( x_d(1) = 300 \text{ mm} \)

Cross-section 2
Breadth at top \( x_{bt}(2) = 400 \text{ mm} \)
Breadth at bottom \( x_{bb}(2) = 400 \text{ mm} \)
Depth \( x_d(2) = 550 \text{ mm} \)

Cross-section 3
Bottom of cross-section is the tension face.
Breadth at top \( x_{bt}(3) = 1200 \text{ mm} \)
Breadth at bottom \( x_{bb}(3) = 1200 \text{ mm} \)
Depth \( x_d(3) = 350 \text{ mm} \)

Number of layers of prestressing tendons and additional reinforcement \( n_{ols} = 2 \)

Layer 1
Area of tendon or reinforcement \( A_s(1) = 2412 \text{ mm}^2 \)
Depth from compression face \( d_s(1) = 1050 \text{ mm} \)
Characteristic strength \( f_y(1) = 1860 \text{ N/mm}^2 \)
Stress after all losses \( f_{al}(1) = 975 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E(1) = 200000 \text{ N/mm}^2 \)
Layer 2

Area of tendon or reinforcement \( A_s(2) = 4825 \text{ mm}^2 \)
Depth from compression face \( d_s(2) = 1130 \text{ mm} \)
Characteristic strength \( f_y(2) = 460 \text{ N/mm}^2 \)
Stress after all losses \( f_{al}(2) = 0 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E(2) = 200000 \text{ N/mm}^2 \)
Total moment from steel forces \( 3886.5 \text{ kNm} \)
Moment from concrete forces \( 1252.1 \text{ kNm} \)
Moment of resistance of section \( M_{MR} + M_C = 5138.6 \text{ kNm} \)
Depth to Neutral Axis \( 381.78 \text{ mm} \)

Ductility requirements

Strain in outermost layer of tendons 0.011001 is less than the limiting strain 0.013087 from Clause 6.3.3.1.
The calculated ultimate moment 5138.6 kNm should be at least 1.15 times the required value (Clause 6.3.3.1).

DESIGN SUMMARY

Char.compr.strength of concrete \( 40 \text{ N/mm}^2 \)
Tendon material factor \( 1.15 \)
Section moment resistance \( 5138.6 \text{ kNm} \)
Depth to Neutral Axis \( 381.78 \text{ mm} \)
The calculated ultimate moment 5138.6 kNm should be at least 1.15 times the required value (Clause 6.3.3.1).
Location: Internal beam

Ultimate limit state: flexure of prestressed concrete section

Calculations are in accordance with BS5400:Part 4:1990 as implemented by DOT Departmental Standard BD 44/15.

Breadth of section \( b = 1000 \text{ mm} \)
Effective depth to tension r'ment \( d = 1100 \text{ mm} \)
Char. strength of tendons \( f_{pu} = 1860 \text{ N/mm}^2 \)
Area of tendons in tension zone \( A_{ps} = 3000 \text{ mm}^2 \)
Char. strength of concrete \( f_{cu} = 40 \text{ N/mm}^2 \)
Depth to neutral axis \( x = t0^2*d = 303.01 \text{ mm} \)

Moment of resistance of section
\[
M_u = f_{pu}A_{ps}(d-0.5\times x)/1E6 = 4604.6 \text{ kNm}
\]

DESIGN SUMMARY

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Char. strength of tendons</td>
<td>1860 N/mm²</td>
</tr>
<tr>
<td>Char. compr. strength of concrete</td>
<td>40 N/mm²</td>
</tr>
<tr>
<td>Tendon material factor</td>
<td>1.15</td>
</tr>
<tr>
<td>Section moment resistance</td>
<td>4604.6 kNm</td>
</tr>
<tr>
<td>Depth to Neutral Axis</td>
<td>303.01 mm</td>
</tr>
</tbody>
</table>
Location: Internal beam

Ultimate limit state: flexure of prestressed concrete section

Calculations are in accordance with BS5400:Part 4:1990 as implemented by DOT Departmental Standard BD 44/15.

Breadth of section \( b = 1000 \text{ mm} \)

Effective depth to tension reinforcement \( d = 500 \text{ mm} \)

Char. strength of tendons \( f_{pu} = 1820 \text{ N/mm}^2 \)

Area of tendons in tension zone \( A_{ps} = 2310 \text{ mm}^2 \)

Char. strength of concrete \( f_{cu} = 40 \text{ N/mm}^2 \)

Depth to neutral axis \( x = t_c 2 \times d = 213.74 \text{ mm} \)

Moment of resistance of section \( M_u = f_{pb} A_{ps} (d-0.5*x)/1E6 = 1351.4 \text{ kNm} \)

The depth to Neutral Axis 213.74 mm is too low to provide the elongation given in Clause 6.3.3.1.

The ultimate moment calculated 1351.4 kNm should be at least 1.15 times the required value. Table 27.

DESIGN SUMMARY

- Char. strength of tendons: 1820 N/mm²
- Char. compr. strength of concrete: 40 N/mm²
- Tendon material factor: 1.15
- Section moment resistance: 1351.4 kNm
- Depth to Neutral Axis: 213.74 mm

The ultimate moment calculated 1351.4 kNm should be at least 1.15 times the required value. Table 27.
Location: Beam A

Ultimate limit state: flexure of prestressed concrete section

Calculations are in accordance with BS5400:Part 4:1990 as implemented by DOT Departmental Standard BD 44/15.

Number of cross-sections ncs=4
Cross-section 1
Top of cross-section is the compression face.
Breadth at top xbt(1)=1200 mm
Breadth at bottom xbb(1)=1000 mm
Depth xd(1)=250 mm
Char. strength of concrete fcu(1)=40 N/mm²
Cross-section 2
Breadth at top xbt(2)=600 mm
Breadth at bottom xbb(2)=450 mm
Depth xd(2)=150 mm
Char. strength of concrete fcu(2)=30 N/mm²
Cross-section 3
Breadth at top xbt(3)=300 mm
Breadth at bottom xbb(3)=300 mm
Depth xd(3)=600 mm
Char. strength of concrete fcu(3)=50 N/mm²
Cross-section 4
Bottom of cross-section is the tension face.
Breadth at top xbt(4)=750 mm
Breadth at bottom xbb(4)=1000 mm
Depth xd(4)=300 mm
Char. strength of concrete fcu(4)=45 N/mm²
Number of layers of prestressing tendons and additional reinforcement nols=5
Layer 1
Area of tendon or reinforcement As(1)=3272 mm²
Depth from compression face ds(1)=100 mm
Characteristic strength fy(1)=460 N/mm²
Stress after all losses fal(1)=0 N/mm²
Young's modulus for reinforcement E(1)=200000 N/mm²
Layer 2
Area of tendon or reinforcement As(2)=1620 mm²
Depth from compression face ds(2)=400 mm
Characteristic strength fy(2)=1820 N/mm²
Stress after all losses fal(2)=1000 N/mm²
Young's modulus for reinforcement E(2)=200000 N/mm²

Layer 3
Area of tendon or reinforcement As(3)=1200 mm²
Depth from compression face ds(3)=800 mm
Characteristic strength fy(3)=1860 N/mm²
Stress after all losses fal(3)=950 N/mm²
Young's modulus for reinforcement E(3)=200000 N/mm²

Layer 4
Area of tendon or reinforcement As(4)=1800 mm²
Depth from compression face ds(4)=1150 mm
Characteristic strength fy(4)=1770 N/mm²
Stress after all losses fal(4)=1025 N/mm²
Young's modulus for reinforcement E(4)=200000 N/mm²

Layer 5
Area of tendon or reinforcement As(5)=2513 mm²
Depth from compression face ds(5)=1250 mm
Characteristic strength fy(5)=460 N/mm²
Stress after all losses fal(5)=0 N/mm²
Young's modulus for reinforcement E(5)=200000 N/mm²

Total moment from steel forces 3738.9 kNm
Moment from concrete forces 1536.5 kNm
Moment of resistance of section M=MR+MC=5275.4 kNm
Depth to Neutral Axis 415.45 mm

Ductility requirements

Strain in outermost layer of tendons 0.011313 is less than the limiting strain 0.012696 from Clause 6.3.3.1.
The calculated ultimate moment 5275.4 kNm should be at least 1.15 times the required value (Clause 6.3.3.1).

DESIGN SUMMARY

Tendon material factor 1.15
Section moment resistance 5275.4 kNm
Depth to Neutral Axis 415.45 mm
The calculated ultimate moment 5275.4 kNm should be at least 1.15 times the required value (Clause 6.3.3.1).
Ultimate limit state: flexure of prestressed concrete section


Number of cross-sections \( ncs = 4 \)

Cross-section 1
Concrete compressive strength \( f_{ck}^{(1)} = 30 \text{ N/mm}^2 \)
Material factor \( \gamma_{mc} = 1.5 \)

Cross-section 2
Concrete compressive strength \( f_{ck}^{(2)} = 40 \text{ N/mm}^2 \)

Cross-section 3
Concrete compressive strength \( f_{ck}^{(3)} = 40 \text{ N/mm}^2 \)

Cross-section 4
Concrete compressive strength \( f_{ck}^{(4)} = 40 \text{ N/mm}^2 \)

Number of layers of prestressing tendons \( n_{olpt} = 4 \)
Number of layers of non prestressed reinforcement \( n_{ols} = 0 \)

Prestressing tendons:
Characteristic tensile strength \( f_{pk} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress \( f_{pl} = 1468 \text{ N/mm}^2 \)
Young's modulus of tendons \( E_p = 195000 \text{ N/mm}^2 \)
Partial material factor (tendons) \( \gamma_{sp} = 1.15 \)

Tendon stress-strain relationship:
Horizontal top branch with no strain limits

Layer 1
Area of tendon reinforcement \( A_{s(1)} = 278 \text{ mm}^2 \)
Depth from compression face \( d_{s(1)} = 600 \text{ mm} \)
Stress after all losses \( s_{tr(1)} = 901.8 \text{ N/mm}^2 \)
Characteristic tensile strength \( f_{pk(1)} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress \( f_{pl(1)} = 1468 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E(1) = 195000 \text{ N/mm}^2 \)

Layer 2
Area of tendon reinforcement \( A_{s(2)} = 278 \text{ mm}^2 \)
Depth from compression face \( d_{s(2)} = 770 \text{ mm} \)
Stress after all losses \( s_{tr(2)} = 901.8 \text{ N/mm}^2 \)
Characteristic tensile strength \( f_{pk(2)} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress \( f_{pl(2)} = 1468 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E(2) = 195000 \text{ N/mm}^2 \)

Layer 3
Area of tendon reinforcement \( A_{s(3)} = 278 \text{ mm}^2 \)
Depth from compression face \( d_{s(3)} = 820 \text{ mm} \)
Stress after all losses \( s_{tr(3)} = 901.8 \text{ N/mm}^2 \)
Characteristic tensile strength \( f_{pk(3)} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress \( f_{pl(3)} = 1468 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E(3) = 195000 \text{ N/mm}^2 \)

Layer 4
Area of tendon reinforcement \( A_{s(4)} = 2085 \text{ mm}^2 \)
Depth from compression face \( d_{s(4)} = 870 \text{ mm} \)
Stress after all losses \( s_{tr(4)} = 901.8 \text{ N/mm}^2 \)
Characteristic tensile strength \( f_{pk(4)} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress \( f_{pl(4)} = 1468 \text{ N/mm}^2 \)
Young's modulus for reinforcement $E(4) = E_p = 195000 \text{ N/mm}^2$

**Ultimate limit state : flexure of general prestressed concrete section**

The calculation is based on BS EN 1992-1-1 Clause 6.1(2)P. Strain compatibility is ensured at the centroid of the tension reinforcement or, if the characteristic strength of the reinforcement is not constant, at the centroid of each layer. No limit is placed on either the depth of concrete in compression or the value of the lever arm.

Total moment from steel forces $M_1 = 1845.3 \text{ kNm}$

The concrete forces are obtained by integrating the product of the concrete area and concrete stress for each cross-section.

Moment from concrete forces $M_2 = 859.09 \text{ kN}$

Section moment resistance $M_{yRd} = M_1 + M_2 = 2704.4 \text{ kNm}$

Depth to Neutral Axis $334.77 \text{ mm}$

**DESIGN SUMMARY**

| Char. tensile strength of tendons | 1670 N/mm² |
| Characteristic 0.1% proof stress | 1468 N/mm² |
| Concrete material partial factor | 1.5 |
| Tendon partial material factor | 1.15 |
| Section moment resistance | 2704.4 kNm |
| Depth to Neutral Axis | 334.77 mm |
Location: Ex2 - M beam Example 6.1.5 Inclined Top Branch

Ultimate limit state: flexure of prestressed concrete section

Calculations are in accordance with BS EN 1991-1-1, BS EN 1992-2 and

Number of cross-sections \( n_{cs} = 4 \)

Cross-section 1
Concrete compressive strength \( f_{ck}^{(1)} = 30 \text{ N/mm}^2 \)
Material factor \( \gamma_{mc} = 1.5 \)

Cross-section 2
Concrete compressive strength \( f_{ck}^{(2)} = 40 \text{ N/mm}^2 \)

Cross-section 3
Concrete compressive strength \( f_{ck}^{(3)} = 40 \text{ N/mm}^2 \)

Cross-section 4
Concrete compressive strength \( f_{ck}^{(4)} = 40 \text{ N/mm}^2 \)

Number of layers of non prestressed reinforcement \( n_{ols} = 0 \)

Prestressing tendons:
Characteristic tensile strength \( f_{pk} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress \( f_{pl} = 1468 \text{ N/mm}^2 \)
Young's modulus of tendons \( E_p = 195000 \text{ N/mm}^2 \)
Partial material factor (tendons) \( \gamma_{mpt} = 1.15 \)

Layer 1
Area of tendon reinforcement \( A_s^{(1)} = 278 \text{ mm}^2 \)
Depth from compression face \( d_s^{(1)} = 600 \text{ mm} \)
Stress after all losses \( s_r^{(1)} = 901.8 \text{ N/mm}^2 \)
Characteristic tensile strength \( f_{pk}^{(1)} = f_{pk} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress \( f_{pl}^{(1)} = f_{pl} = 1468 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E^{(1)} = E_p = 195000 \text{ N/mm}^2 \)

Layer 2
Area of tendon reinforcement \( A_s^{(2)} = 278 \text{ mm}^2 \)
Depth from compression face \( d_s^{(2)} = 770 \text{ mm} \)
Stress after all losses \( s_r^{(2)} = 901.8 \text{ N/mm}^2 \)
Characteristic tensile strength \( f_{pk}^{(2)} = f_{pk} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress \( f_{pl}^{(2)} = f_{pl} = 1468 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E^{(2)} = E_p = 195000 \text{ N/mm}^2 \)

Layer 3
Area of tendon reinforcement \( A_s^{(3)} = 278 \text{ mm}^2 \)
Depth from compression face \( d_s^{(3)} = 820 \text{ mm} \)
Stress after all losses \( s_r^{(3)} = 901.8 \text{ N/mm}^2 \)
Characteristic tensile strength \( f_{pk}^{(3)} = f_{pk} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress \( f_{pl}^{(3)} = f_{pl} = 1468 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E^{(3)} = E_p = 195000 \text{ N/mm}^2 \)

Layer 4
Area of tendon reinforcement \( A_s^{(4)} = 2085 \text{ mm}^2 \)
Depth from compression face \( d_s^{(4)} = 870 \text{ mm} \)
Stress after all losses \( s_r^{(4)} = 901.8 \text{ N/mm}^2 \)
Characteristic tensile strength \( f_{pk}^{(4)} = f_{pk} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress \( f_{pl(4)} = f_{pl} = 1468 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E(4) = E_p = 195000 \text{ N/mm}^2 \)

**Ultimate limit state: flexure of general prestressed concrete section**

The calculation is based on BS EN 1992-1-1 Clause 6.1(2)P. Strain compatibility is ensured at the centroid of the tension reinforcement or, if the characteristic strength of the reinforcement is not constant, at the centroid of each layer. No limit is placed on either the depth of concrete in compression or the value of the lever arm.

Total moment from steel forces \( M_1 = 1845.1 \text{ kNm} \)
The concrete forces are obtained by integrating the product of the concrete area and concrete stress for each cross-section.

Moment from concrete forces \( M_2 = 918.17 \text{ kN} \)
Section moment resistance \( M_{yd} = M_1 + M_2 = 2763.3 \text{ kNm} \)
Depth to Neutral Axis \( 347.8 \text{ mm} \)

**DESIGN SUMMARY**

- Char.tensile strength of tendons \( 1670 \text{ N/mm}^2 \)
- Characteristic 0.1% proof stress \( 1468 \text{ N/mm}^2 \)
- Concrete material partial factor \( 1.5 \)
- Tendon partial material factor \( 1.15 \)
- Section moment resistance \( 2763.3 \text{ kNm} \)
- Depth to Neutral Axis \( 347.8 \text{ mm} \)
Location: Ex3 – M beam Prestress and Reinf. Horizontal Top Branch

Ultimate limit state : flexure of prestressed concrete section


Number of cross-sections ncs=4
Cross-section 1
Concrete compressive strength fck(1)=30 N/mm²
Material factor gamc=1.5
Cross-section 2
Concrete compressive strength fck(2)=40 N/mm²
Cross-section 3
Concrete compressive strength fck(3)=40 N/mm²
Cross-section 4
Concrete compressive strength fck(4)=40 N/mm²
Number of layers of prestressing tendons nolpt=4
Number of layers of non prestressed reinforcement nols=1

Prestressing tendons:
Characteristic tensile strength fpk=1670 N/mm²
Characteristic 0.1% proof stress fpl=1468 N/mm²
Young's modulus of tendons Ep=195000 N/mm²
Partial material factor (tendons) gamsp=1.15
Tendon stress-strain relationship:
Horizontal top branch with no strain limits
Non prestressed reinforcement
Characteristic yield stress fykr=500 N/mm²
Young's modulus of reinforcement Es=200000 N/mm²
Material factor for reinforcement gamsr=1.15
Reinforcement stress-strain relationship
Horizontal top branch with no strain limits
Layer 1
Area of tendon reinforcement As(1)=278 mm²
Depth from compression face ds(1)=600 mm
Stress after all losses str(1)=901.8 N/mm²
Characteristic tensile strength fpk(1)=fpk=1670 N/mm²
Characteristic 0.1% proof stress fpl(1)=fpl=1468 N/mm²
Young's modulus for reinforcement E(1)=Ep=195000 N/mm²
Layer 2
Area of tendon reinforcement As(2)=278 mm²
Depth from compression face ds(2)=770 mm
Stress after all losses str(2)=901.8 N/mm²
Characteristic tensile strength fpk(2)=fpk=1670 N/mm²
Characteristic 0.1% proof stress fpl(2)=fpl=1468 N/mm²
Young's modulus for reinforcement E(2)=Ep=195000 N/mm²
Layer 3
Area of tendon reinforcement As(3)=278 mm²
Depth from compression face ds(3)=820 mm
Stress after all losses str(3)=901.8 N/mm²
Characteristic tensile strength fpk(3)=fpk=1670 N/mm²
Characteristic 0.1% proof stress fpl(3)=fpl=1468 N/mm²
Young's modulus for reinforcement E(3)=Ep=195000 N/mm²
Layer 4
Area of tendon reinforcement $A_s(4)=2085 \text{ mm}^2$
Depth from compression face $d_s(4)=870 \text{ mm}$
Stress after all losses $\sigma(4)=901.8 \text{ N/mm}^2$
Characteristic tensile strength $f_{pk}(4)=f_{pk}=1670 \text{ N/mm}^2$
Characteristic 0.1% proof stress $f_{pl}(4)=f_{pl}=1468 \text{ N/mm}^2$
Young's modulus for reinforcement $E(4)=E_p=195000 \text{ N/mm}^2$

Layer 5
Area of bar reinforcement $A_s(5)=791 \text{ mm}^2$
Depth from compression face $d_s(5)=870 \text{ mm}$
Characteristic strength $f_{yk}(5)=f_{ykr}=500 \text{ N/mm}^2$
Young's modulus for reinforcement $E(5)=E_s=200000 \text{ N/mm}^2$
Material factor for reinforcement $g_{ms}(5)=g_{amsr}=1.15$

**Ultimate limit state: flexure of general prestressed concrete section**

The calculation is based on BS EN 1992-1-1 Clause 6.1(2)P. Strain compatibility is ensured at the centroid of the tension reinforcement or, if the characteristic strength of the reinforcement is not constant, at the centroid of each layer. No limit is placed on either the depth of concrete in compression or the value of the lever arm.

Total moment from steel forces $M_1=1838.8 \text{ kNm}$
The concrete forces are obtained by integrating the product of the concrete area and concrete stress for each cross-section.

Moment from concrete forces $M_2=1078.9 \text{ kN}$
Section moment resistance $M_{yRd}=M_1+M_2=2917.7 \text{ kNm}$
Depth to Neutral Axis $381.59 \text{ mm}$

**DESIGN SUMMARY**

| Characteristic strength of tendons $f_{pk}$ | 1670 N/mm² |
| Characteristic 0.1% proof stress $f_{pl}$ | 1468 N/mm² |
| Concrete material partial factor | 1.5 |
| Tendon partial material factor | 1.15 |
| Section moment resistance | 2917.7 kNm |
| Depth to Neutral Axis | 381.59 mm |
**Location: Ex4 - M beam Prestress and Reinf. Inclined Top Branch**

**Ultimate limit state: flexure of prestressed concrete section**


Number of cross-sections $ncs=4$

Cross-section 1
Concrete compressive strength $f_{ck}(1)=30$ N/mm$^2$
Material factor $\gamma_{mc}=1.5$

Cross-section 2
Concrete compressive strength $f_{ck}(2)=40$ N/mm$^2$

Cross-section 3
Concrete compressive strength $f_{ck}(3)=40$ N/mm$^2$

Cross-section 4
Concrete compressive strength $f_{ck}(4)=40$ N/mm$^2$

Number of layers of prestressing tendons $nolpt=4$
Number of layers of non prestressed reinforcement $nols=1$

Prestressing tendons:
Characteristic tensile strength $f_{pk}=1670$ N/mm$^2$
Characteristic 0.1% proof stress $f_{pl}=1468$ N/mm$^2$
Young's modulus of tendons $E_p=195000$ N/mm$^2$
Partial material factor (tendons) $\gamma_{mpt}=1.15$
Tendon stress-strain relationship:
Inclined top branch with strain limits
Limiting strain $\varepsilon_{udp}=0.02$

Non prestressed reinforcement
Characteristic yield stress $f_{ykr}=500$ N/mm$^2$
Young's modulus of reinforcement $E_s=200000$ N/mm$^2$
Material factor for reinforcement $\gamma_{msr}=1.15$
Reinforcement stress-strain relationship
Inclined top branch with strain limits
Ductility class is constant for whole section
Ductility Class (1=A, 2=B, 3=C) $dcr=2$
Strain at maximum force $\varepsilon_{ukr}=0.05$
Multiplication factor $kr=1.08$

Layer 1
Area of tendon reinforcement $A_s(1)=278$ mm$^2$
Depth from compression face $d_s(1)=600$ mm
Stress after all losses $\sigma(1)=901.8$ N/mm$^2$
Characteristic tensile strength $f_{pk}(1)=f_{pk}=1670$ N/mm$^2$
Characteristic 0.1% proof stress $f_{pl}(1)=f_{pl}=1468$ N/mm$^2$
Young's modulus for reinforcement $E(1)=E_p=195000$ N/mm$^2$

Layer 2
Area of tendon reinforcement $A_s(2)=278$ mm$^2$
Depth from compression face $d_s(2)=770$ mm
Stress after all losses $\sigma(2)=901.8$ N/mm$^2$
Characteristic tensile strength $f_{pk}(2)=f_{pk}=1670$ N/mm$^2$
Characteristic 0.1% proof stress $f_{pl}(2)=f_{pl}=1468$ N/mm$^2$
Young's modulus for reinforcement $E(2)=E_p=195000$ N/mm$^2$

Layer 3
Area of tendon reinforcement $A_s(3)=278$ mm$^2$
Depth from compression face: \( d_s(3) = 820 \text{ mm} \)
Stress after all losses: \( s_{tr}(3) = 901.8 \text{ N/mm}^2 \)
Characteristic tensile strength: \( f_{pk}(3) = f_{pk} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress: \( f_{pl}(3) = f_{pl} = 1468 \text{ N/mm}^2 \)
Young's modulus for reinforcement: \( E(3) = E_p = 195000 \text{ N/mm}^2 \)

Layer 4
Area of tendon reinforcement: \( A_s(4) = 2085 \text{ mm}^2 \)
Depth from compression face: \( d_s(4) = 870 \text{ mm} \)
Stress after all losses: \( s_{tr}(4) = 901.8 \text{ N/mm}^2 \)
Characteristic tensile strength: \( f_{pk}(4) = f_{pk} = 1670 \text{ N/mm}^2 \)
Characteristic 0.1% proof stress: \( f_{pl}(4) = f_{pl} = 1468 \text{ N/mm}^2 \)
Young's modulus for reinforcement: \( E(4) = E_p = 195000 \text{ N/mm}^2 \)

Layer 5
Area of bar reinforcement: \( A_s(5) = 791 \text{ mm}^2 \)
Depth from compression face: \( d_s(5) = 870 \text{ mm} \)
Characteristic strength: \( f_{yk}(5) = f_{yk} = 500 \text{ N/mm}^2 \)
Young's modulus for reinforcement: \( E(5) = E_s = 200000 \text{ N/mm}^2 \)
Material factor for reinforcement: \( g_{ms}(5) = g_{ms} = 1.15 \)
Ductility Class for reinforcement: \( d_{cr}(5) = d_{cr} = 2 \)
Strain at maximum force: \( e_{uk}(5) = e_{uk} = 0.05 \)
Multiplication factor: \( k(5) = k = 1.08 \)

**Ultimate limit state : flexure of general prestressed concrete section**

The calculation is based on BS EN 1992-1-1 Clause 6.1(2)P. Strain compatibility is ensured at the centroid of the tension reinforcement or, if the characteristic strength of the reinforcement is not constant, at the centroid of each layer. No limit is placed on either the depth of concrete in compression or the value of the lever arm.

Total moment from steel forces: \( M_1 = 1833.8 \text{ kNm} \)
The concrete forces are obtained by integrating the product of the concrete area and concrete stress for each cross-section.

Moment from concrete forces: \( M_2 = 1122.7 \text{ kN} \)
Section moment resistance: \( M_{yRd} = M_1 + M_2 = 2956.5 \text{ kNm} \)
Depth to Neutral Axis: \( 390.45 \text{ mm} \)

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char. tensile strength of tendons</td>
<td>1670 N/mm²</td>
</tr>
<tr>
<td>Characteristic 0.1% proof stress</td>
<td>1468 N/mm²</td>
</tr>
<tr>
<td>Concrete material partial factor</td>
<td>1.5</td>
</tr>
<tr>
<td>Tendon partial material factor</td>
<td>1.15</td>
</tr>
<tr>
<td>Section moment resistance</td>
<td>2956.5 kNm</td>
</tr>
<tr>
<td>Depth to Neutral Axis</td>
<td>390.45 mm</td>
</tr>
</tbody>
</table>
Location: Ex1 - Internal beam

Ultimate limit state: flexure of prestressed concrete section

Ultimate moment calculation to Cl. 6.3.3.1 of BD 44/15 Appendix A

Concrete strength \( f_{\text{cum}} = 40 \text{ mm}^2 \)
Factor for concrete \( g_{\text{cm}} = 1.5 \)
Factor for steel \( g_{\text{sm}} = 1.15 \)
Number of cross-sections \( n_{\text{cs}} = 3 \)

Cross-section 1
Top of cross-section is the compression face.
Breadth at top \( x_{\text{bt}}(1) = 1000 \text{ mm} \)
Breadth at bottom \( x_{\text{bb}}(1) = 1000 \text{ mm} \)
Depth \( x_{\text{d}}(1) = 300 \text{ mm} \)

Cross-section 2
Breadth at top \( x_{\text{bt}}(2) = 400 \text{ mm} \)
Breadth at bottom \( x_{\text{bb}}(2) = 400 \text{ mm} \)
Depth \( x_{\text{d}}(2) = 550 \text{ mm} \)

Cross-section 3
Bottom of cross-section is the tension face.
Breadth at top \( x_{\text{bt}}(3) = 1200 \text{ mm} \)
Breadth at bottom \( x_{\text{bb}}(3) = 1200 \text{ mm} \)
Depth \( x_{\text{d}}(3) = 350 \text{ mm} \)

No. of layers of prestressing tendons and additional r'ment \( n_{\text{ols}} = 2 \)

Layer 1:
Area of tendon or reinforcement \( A_{s}(1) = 2412 \text{ mm}^2 \)
Depth from compression face \( d_{s}(1) = 1050 \text{ mm} \)
Strength of steel \( f_{y}(1) = 1860 \text{ N/mm}^2 \)
Stress after all losses \( f_{\text{al}}(1) = 975 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E(1) = 200000 \text{ N/mm}^2 \)

Layer 2:
Area of tendon or reinforcement \( A_{s}(2) = 4825 \text{ mm}^2 \)
Depth from compression face \( d_{s}(2) = 1130 \text{ mm} \)
Strength of steel \( f_{y}(2) = 460 \text{ N/mm}^2 \)
Stress after all losses \( f_{\text{al}}(2) = 0 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E(2) = 200000 \text{ N/mm}^2 \)
Section Moment Resistance 5138.6 kNm

Depth to Neutral Axis 381.78 mm

Strain in outermost layer of tendons 0.011001 is less than the limiting strain of 0.013087 (BS 5400 Clause 6.3.3.1). Hence failure may be brittle.
Location: Ex2 - Inner beam

Ultimate limit state: flexure of prestressed concrete section

Ultimate moment calculation (Cl. 6.3.3.3 of BD 44/15 Appendix A)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of section</td>
<td>b=1000 mm</td>
</tr>
<tr>
<td>Effective depth to tendons</td>
<td>d=1100 mm</td>
</tr>
<tr>
<td>Tendon strength</td>
<td>fpu=1860 N/mm²</td>
</tr>
<tr>
<td>Young's modulus for tendon</td>
<td>Es=200000 N/mm²</td>
</tr>
<tr>
<td>Area of tendons in tension zone</td>
<td>Aps=3000 mm²</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>fcu=40 mm²</td>
</tr>
<tr>
<td>Factor for tendons</td>
<td>gms=1.15</td>
</tr>
<tr>
<td>Factor for concrete</td>
<td>gmc=1.5</td>
</tr>
</tbody>
</table>

Depth to neutral axis: 
\[ x = \frac{(f_{pb} \cdot A_{ps} \cdot g_{mc})}{(0.6 \cdot f_{cu} \cdot b)} = 303.26 \text{ mm} \]

Moment of resistance of section: 
\[ M_u = f_{pb} \cdot A_{ps} \cdot (d - 0.5 \cdot x) / 1 \times 10^6 \approx 4601.7 \text{ kNm} \]

Stress in tendons at failure is greater than fpu/gms. Hence failure should be ductile.
Location: Ex3 - Internal beam

Ultimate limit state: flexure of prestressed concrete section

Ultimate moment calculation (Cl. 6.3.3.3 of BD 44/15 Appendix A)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of section</td>
<td>b=1000 mm</td>
</tr>
<tr>
<td>Effective depth to tendons</td>
<td>d=500 mm</td>
</tr>
<tr>
<td>Tendon strength</td>
<td>fpu=1820 N/mm²</td>
</tr>
<tr>
<td>Young's modulus for tendon</td>
<td>Es=200000 N/mm²</td>
</tr>
<tr>
<td>Area of tendons in tension zone</td>
<td>Aps=2310 mm²</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>fcu=40 mm²</td>
</tr>
<tr>
<td>Factor for tendons</td>
<td>gms=1.15</td>
</tr>
<tr>
<td>Factor for concrete</td>
<td>gmc=1.5</td>
</tr>
<tr>
<td>Depth to neutral axis</td>
<td>x=(fpb<em>Aps</em>gmc)/(0.6<em>fcu</em>b)=214.73 mm</td>
</tr>
<tr>
<td>Moment of resistance of section</td>
<td>Mu=fpb<em>Aps</em>(d-0.5*x)/1E6=1349 kNm</td>
</tr>
<tr>
<td>Stress in tendons at failure is less than fpu/gms. Hence failure may be brittle.</td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex4 - Beam A

Ultimate limit state: flexure of prestressed concrete section

Ultimate moment calculation to Cl. 6.3.3.1 of BD 44/15 Appendix A

Number of cross-sections \( ncs = 4 \)

Cross-section 1

Top of cross-section is the compression face.

- Breadth at top: \( xbt(1) = 1200 \text{ mm} \)
- Breadth at bottom: \( xbb(1) = 1000 \text{ mm} \)
- Depth: \( xd(1) = 250 \text{ mm} \)
- Strength of concrete: \( fcu(1) = 40 \text{ N/mm}^2 \)
- Factor for concrete: \( gmc(1) = 1.2 \)

Cross-section 2

- Breadth at top: \( xbt(2) = 600 \text{ mm} \)
- Breadth at bottom: \( xbb(2) = 450 \text{ mm} \)
- Depth: \( xd(2) = 150 \text{ mm} \)
- Strength of concrete: \( fcu(2) = 30 \text{ N/mm}^2 \)
- Factor for concrete: \( gmc(2) = 1.5 \)

Cross-section 3

- Breadth at top: \( xbt(3) = 300 \text{ mm} \)
- Breadth at bottom: \( xbb(3) = 300 \text{ mm} \)
- Depth: \( xd(3) = 600 \text{ mm} \)
- Strength of concrete: \( fcu(3) = 50 \text{ N/mm}^2 \)
- Factor for concrete: \( gmc(3) = 1.5 \)

Cross-section 4

Bottom of cross-section is the tension face.

- Breadth at top: \( xbt(4) = 750 \text{ mm} \)
- Breadth at bottom: \( xbb(4) = 1000 \text{ mm} \)
- Depth: \( xd(4) = 300 \text{ mm} \)
- Strength of concrete: \( fcu(4) = 45 \text{ N/mm}^2 \)
- Factor for concrete: \( gmc(4) = 1.2 \)

No. of layers of prestressing tendons and additional r'ment: \( nols = 5 \)

Layer 1:

- Area of tendon or reinforcement: \( As(1) = 3272 \text{ mm}^2 \)
- Depth from compression face: \( ds(1) = 100 \text{ mm} \)
- Strength of steel: \( fy(1) = 460 \text{ N/mm}^2 \)
- Stress after all losses: \( fal(1) = 0 \text{ N/mm}^2 \)
- Factor for steel: \( gms(1) = 1.15 \)
- Young's modulus for reinforcement: \( E(1) = 200000 \text{ N/mm}^2 \)
Layer 2:
Area of tendon or reinforcement: $A_s(2) = 1620 \text{ mm}^2$
Depth from compression face: $d_s(2) = 250 \text{ mm}$
Strength of steel: $f_y(2) = 1820 \text{ N/mm}^2$
Stress after all losses: $f_{al}(2) = 1000 \text{ N/mm}^2$
Factor for steel: $g_{ms}(2) = 1.1$
Stress 'a': $t_{ra}(2) = 1365 \text{ N/mm}^2$
Strain 'a': $s_{a}(2) = 0.007$
Stress 'b': $t_{rb}(2) = 1820 \text{ N/mm}^2$
Strain 'b': $s_{b}(2) = 0.0133$

Layer 3:
Area of tendon or reinforcement: $A_s(3) = 1200 \text{ mm}^2$
Depth from compression face: $d_s(3) = 800 \text{ mm}$
Strength of steel: $f_y(3) = 1860 \text{ N/mm}^2$
Stress after all losses: $f_{al}(3) = 950 \text{ N/mm}^2$
Factor for steel: $g_{ms}(3) = 1.15$
Young's modulus for reinforcement: $E(3) = 200000 \text{ N/mm}^2$

Layer 4:
Area of tendon or reinforcement: $A_s(4) = 1800 \text{ mm}^2$
Depth from compression face: $d_s(4) = 1150 \text{ mm}$
Strength of steel: $f_y(4) = 1770 \text{ N/mm}^2$
Stress after all losses: $f_{al}(4) = 1025 \text{ N/mm}^2$
Factor for steel: $g_{ms}(4) = 1.15$
Young's modulus for reinforcement: $E(4) = 175000 \text{ N/mm}^2$

Layer 5:
Area of tendon or reinforcement: $A_s(5) = 2513 \text{ mm}^2$
Depth from compression face: $d_s(5) = 1250 \text{ mm}$
Strength of steel: $f_y(5) = 460 \text{ N/mm}^2$
Stress after all losses: $f_{al}(5) = 0 \text{ N/mm}^2$
Factor for steel: $g_{ms}(5) = 1.15$
Young's modulus for reinforcement: $E(5) = 200000 \text{ N/mm}^2$
Section Moment Resistance: $5445.6 \text{ kNm}$

Depth to Neutral Axis: $288.22 \text{ mm}$

Strain in outermost layer of tendons 0.017147 is greater than the limiting strain of 0.013795 (BS 5400 Clause 6.3.3.1). Hence failure should be ductile.
Location: EX1 - Edge beam

Ultimate limit state - shear capacity of a prestressed concrete section

Calculations are for Class 1, 2 & 3 members in accordance with BS5400 Part 4:1990 as implemented by DOT Departmental Standard BD 44/15.

At any section the ultimate shear resistance of the concrete alone, Vc, should be considered for the section both uncracked (Clause 6.3.4.2) and cracked (Clause 6.3.4.3) in flexure and if necessary shear reinforcement should be provided (Clause 6.3.4.4).

Flanged member

<table>
<thead>
<tr>
<th>Breadth of member</th>
<th>b = 250 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall depth of member</td>
<td>h = 1035 mm</td>
</tr>
<tr>
<td>Char strength of concrete</td>
<td>f_{cu} = 50 N/mm²</td>
</tr>
</tbody>
</table>

Compressive stress at centroidal axis due to prestress (f_{cp}). The value of f_{cp} should be derived from the prestressing force after all losses have occurred multiplied by the appropriate partial safety factor from Clause 4.2.3.

Stress due to prestress $f_{cp} = 6.88$ N/mm²

Shear capacity of uncracked sections (V_{co})

Component of prestressing force $V_{vc} = 0$ kN

Uncracked section capacity

$V_{co} = V_{vc} + 0.67 \times b \times h \times \sqrt{f_{t}^2 + f_{cp} \times ft} / 1000 = 661.41$ kN

Cracked section Class 1 / 2 members (Cl. 6.3.4.3(a))

Depth to centroid of section from extreme compression fibre: $d_c = 517.5$ mm

Depth to extreme tensile fibre from centroid of concrete section: $y = h - d_c = 517.5$ mm

Second moment of area $I = 63.969E9$ mm⁴

Depth from extreme compression fibre to centroid of all tendons: $d = 893$ mm

Total area of tensioned and untensioned steel in tension zone: $A_s = 5280$ mm²

Depth from compression face to centroid of steel area in tension zone (d₁). The steel area includes tensioned and untensioned steel. $d_1 = 945.5$ mm

$f_{pt}$ is the stress due to prestress at the extreme tensile fibre and should be derived from the prestressing force after all losses have occurred and multiplied by the appropriate partial safety factor from Clause 4.2.3.

Stress $f_{pt} = 16.53$ N/mm²

Cracking moment of section

$M_{cr} = (0.37 \times \sqrt{f_{cu}} + f_{pt}) \times I / (1E6 \times y) = 2366.7$ kNm
Results from shear capacity calculations

<table>
<thead>
<tr>
<th>Class 1 / 2 member</th>
<th>Uncracked Section Shear Capacity</th>
<th>661.41 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cracked Section Shear Capacity</td>
<td>734.61 kN</td>
</tr>
</tbody>
</table>

The shear force $V_c$ which can be carried by the concrete alone is the lesser of 661.41 kN and 734.61 kN. The shear force $V_c$ carried by concrete is 661.41 kN.

Shear reinforcement (Cl. 6.3.4.4)

Shear reinforcement design is for a flanged beam. The shear force due to ultimate loads $V=900$ kN is not greater than $1.8 \times V_c = 1190.5$ kN. The spacing of links along the member should be less than the smallest of $0.75 \times dt = 728.25$ mm and $4 \times bw = 1000$ mm (Clause 6.3.4.4). Shear force $V=900$ kN exceeds $V_c=661.41$ kN. The area of shear reinforcement required is $198.69$ mm$^2$ and the area provided is $226.19$ mm$^2$.

Reinforcement summary

Flanged beam section. Link reinforcement adequate. Area of effectively anchored longitudinal tensile reinforcement and prestressing tendons (excluding debonded tendons) additional to that required at the ultimate limit state for other purposes. Minimum area required $= 1124.4$ mm$^2$. 
Location: Ex2 - Beam A

Ultimate limit state - shear capacity of a prestressed concrete section

Calculations are for Class 1, 2 & 3 members in accordance with BS5400 Part 4:1990 as implemented by DOT Departmental Standard BD 44/15.

At any section the ultimate shear resistance of the concrete alone, \( V_c \), should be considered for the section both uncracked (Clause 6.3.4.2) and cracked (Clause 6.3.4.3) in flexure and if necessary shear reinforcement should be provided (Clause 6.3.4.4).

Rectangular member

- Breadth of member \( b = 250 \text{ mm} \)
- Overall depth of member \( h = 1035 \text{ mm} \)
- Char strength of concrete \( f_{cu} = 50 \text{ N/mm}^2 \)

Compressive stress at centroidal axis due to prestress \( f_{cp} \).

**Stress due to prestress** \( f_{cp} = 6.88 \text{ N/mm}^2 \)

Shear capacity of uncracked sections \( V_{co} \)

- Component of prestressing force \( V_{vc} = 44 \text{ kN} \)
- Uncracked section capacity

\[
V_{co} = V_{vc} + 0.67 \times b \times h \times \sqrt{f_t + f_{cp}} \times 1000 = 705.41 \text{ kN}
\]

Cracked section Class 1 / 2 members (Cl. 6.3.4.3(a))

- Depth to centroid of section from extreme compression fibre: \( d_c = 517.5 \text{ mm} \)
- Depth to extreme tensile fibre from centroid of concrete section: \( y = h - d_c = 517.5 \text{ mm} \)
- Second moment of area \( I = 23.098E9 \text{ mm}^4 \)
- Depth from extreme compression fibre to centroid of all tendons: \( d = 900 \text{ mm} \)
- Total area of tensioned and untensioned steel in tension zone: \( A_s = 4500 \text{ mm}^2 \)
- Depth from compression face to centroid of steel area in tension zone (\( d_1 \)). The steel area includes tensioned and untensioned steel. \( d_1 = 940 \text{ mm} \)

**f_{pt}** is the stress due to prestress at the extreme tensile fibre and should be derived from the prestressing force after all losses have occurred and multiplied by the appropriate partial safety factor from Clause 4.2.3.

**Stress f_{pt}** \( f_{pt} = 15.2 \text{ N/mm}^2 \)

**Cracking moment of section**

\[
M_{cr} = \frac{(0.37 \times \sqrt{f_{cu}} + f_{pt}) \times I}{1E6 \times y} = 795.21 \text{ kNm}
\]
Cracked section Class 3 members (Cl. 6.3.4.3(b))

Depth to centroid of section from extreme compression fibre:
\[ dc = 517.5 \text{ mm} \]

Second moment of area
\[ I = 23.098E9 \text{ mm}^4 \]

Depth from compression face to centroid of tensioned and untensioned steel area in tension zone
\[ d_1 = 940 \text{ mm} \]

Depth to centroid of tensioned and untensioned steel area in tension zone from centroid of concrete section:
\[ y_s = d_1 - d_c = 422.5 \text{ mm} \]

\( f_{pts} \) is the stress due to prestress at the depth 940 mm distance 422.5 mm from the centroid of the concrete section and should be derived from the prestressing force after all losses have occurred and multiplied by the appropriate partial safety factor from Clause 4.2.3.

Stress \( f_{pts} \)
\[ f_{pts} = 13.67 \text{ N/mm}^2 \]

Total area of tensioned and untensioned steel in tension zone
\[ A_s = 4500 \text{ mm}^2 \]

Effective force
\[ P_f = 1780 \text{ kN} \]

Area of tensioned steel
\[ A_{st} = 2000 \text{ mm}^2 \]

Area of untensioned steel
\[ A_{su} = 2500 \text{ mm}^2 \]

Char strength of tensioned steel
\[ f_{put} = 1800 \text{ N/mm}^2 \]

Char strength (untensioned steel)
\[ f_{yu} = 460 \text{ N/mm}^2 \]

Results from shear capacity calculations

Results for a Class 1 / 2 member and a Class 3 member

Uncracked section shear capacity
\[ 705.41 \text{ kN} \]

Cracked section shear capacity Class 1 / 2
\[ 337.19 \text{ kN} \]

Cracked section shear capacity Class 3
\[ 432.71 \text{ kN} \]

The shear force \( V_c \) which can be carried by the concrete alone is the least of 705.41 kN and 337.19 kN and 432.71 kN

Shear force \( V_c \) carried by concrete
\[ 337.19 \text{ kN} \]

Shear reinforcement (Cl. 6.3.4.4)

Shear reinforcement design is for a slab.

Shear force due to ultimate loads \( V = 700 \text{ kN} \)
Calculations are for a slab.

Effective depth for shear calc.
\[ d_t = 940 \text{ mm} \]

Diameter of link reinforcement
\[ b_d = 12 \text{ mm} \]

Characteristic strength of links
\[ f_{yv} = 460 \text{ N/mm}^2 \]

Section width
\[ b_w = 250 \text{ mm} \]

Shear force \( V = 700 \text{ kN} \) exceeds 1.8*\( V_c = 606.94 \text{ kN} \)
Spacing of links along the member should be less than 0.50*\( d_t = 470 \text{ mm} \) (Clause 6.3.4.4).
Spacing of links along the member \( s_v = 125 \text{ mm} \)
Lateral spacing of individual legs of the links at a cross-section should not exceed 0.75*\( d_t = 705 \text{ mm} \) (Clause 6.3.4.4).
Number of legs of links across the member at right angles to the span
\[ s_v^2 = 2 \]
Cross-sectional area of all legs of the links at a member cross-section
\[ A_{sv} = s_v^2 \pi b_d^2 / 4 = 226.19 \text{ mm}^2 \]
Area of shear reinforcement required (Clause 6.3.4.4).
Shear force \( V = 700 \text{ kN} \) exceeds \( V_c = 337.19 \text{ kN} \)
Area required  \[ A_{svr} = \left( V - V_c \right) \times 1000 + 0.4 \times \text{bw} \times \text{dt} \times \text{sv} / (0.87 \times \text{fyv} \times \text{dt}) = 151.79 \text{ mm}^2 \]

Link reinforcement adequate

Area required 151.79 mm\(^2\)
Area provided 226.19 mm\(^2\)

**Reinforcement summary**

**Slab section.**

Link reinforcement adequate  
Area required 151.79 mm\(^2\)
Area provided 226.19 mm\(^2\)

Area of effectively anchored longitudinal tensile r'ment and prestressing tendons (excluding debonded tendons) additional to that required at the ultimate limit state for other purposes.

Minimum area required = 874.56 mm\(^2\)
Location: Ex3 - Beam B

Ultimate limit state - shear capacity of a prestressed concrete section

Calculations are for Class 1, 2 & 3 members in accordance with BS5400 Part 4:1990 as implemented by DOT Departmental Standard BD 44/15.

Transmission length \( l_t = 1300 \text{ mm} \)

At any section the ultimate shear resistance of the concrete alone, \( V_c \), should be considered for the section both uncracked (Clause 6.3.4.2) and cracked (Clause 6.3.4.3) in flexure and if necessary shear reinforcement should be provided (Clause 6.3.4.4).

**Flanged member**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of member</td>
<td>( b = 300 \text{ mm} )</td>
</tr>
<tr>
<td>Overall depth of member</td>
<td>( h = 950 \text{ mm} )</td>
</tr>
<tr>
<td>Char strength of concrete</td>
<td>( f_{cu} = 40 \text{ N/mm}^2 )</td>
</tr>
</tbody>
</table>

Compressive stress at centroidal axis due to prestress (\( f_{cp} \)). The value of \( f_{cp} \) should be derived from the prestressing force after all losses have occurred multiplied by the appropriate partial safety factor from Clause 4.2.3.

Stress due to prestress \( f_{cp} = 8.6 \text{ N/mm}^2 \)

**Cracked section Class 1 / 2 members (Cl. 6.3.4.3(a))**

Depth to centroid of section from extreme compression fibre: \( d_c = 400 \text{ mm} \)

Depth to extreme tensile fibre from centroid of concrete section: \( y = h - d_c = 550 \text{ mm} \)

Second moment of area \( I = 35.6E9 \text{ mm}^4 \)

Depth from extreme compression fibre to centroid of all tendons: \( d = 850 \text{ mm} \)

Total area of tensioned and untensioned steel in tension zone: \( A_s = 3600 \text{ mm}^2 \)

Depth from compression face to centroid of steel area in tension zone (\( d_1 \)). The steel area includes tensioned and untensioned steel. \( d_1 = 900 \text{ mm} \)

\( f_{pt} \) is the stress due to prestress at the extreme tensile fibre and should be derived from the prestressing force after all losses have occurred and multiplied by the appropriate partial safety factor from Clause 4.2.3.

Stress \( f_{pt} \) at end of transmission length: \( f_{pt} = 14 \text{ N/mm}^2 \)
Cracked section Class 3 members (Cl. 6.3.4.3(b))

Depth to centroid of section from extreme compression fibre:
\[ dc = 400 \text{ mm} \]

Second moment of area \[ I = 35.6E9 \text{ mm}^4 \]

Depth from compression face to centroid of tensioned and untensioned steel area in tension zone \[ d_1 = 900 \text{ mm} \]

Depth to centroid of tensioned and untensioned steel area in tension zone from centroid of concrete section:
\[ y_s = d_1 - dc = 500 \text{ mm} \]

\( f_{pts} \) is the stress due to prestress at the depth 900 mm distance 500 mm from the centroid of the concrete section and should be derived from the prestressing force after all losses have occurred and multiplied by the appropriate partial safety factor from Clause 4.2.3.

Stress \( f_{pts} \) at end of transmission length:
\[ f_{pts} = 13.1 \text{ N/mm}^2 \]

Total area of tensioned and untensioned steel in tension zone:
\[ A_s = 3600 \text{ mm}^2 \]

At end of transmission length
Effective prestressing force after all losses have occurred multiplied by the appropriate partial safety factor from Clause 4.2.3.
\[ \text{Effective force} = P_f = 1500 \text{ kN} \]

Area of tensioned steel \[ A_{st} = 2500 \text{ mm}^2 \]

Area of untensioned steel \[ A_{su} = 1100 \text{ mm}^2 \]

Char strength of tensioned steel \[ f_{put} = 1720 \text{ N/mm}^2 \]

Char strength (untensioned steel) \[ f_{yL} = 460 \text{ N/mm}^2 \]

Distance from start of transmission length (zero stress end):
\[ d_{lt(1)} = 500 \text{ mm} \]

Component of prestressing force \[ V_{vc} = 0 \text{ kN} \]

Total area of untensioned steel in tension zone
\[ A_{ut} = 1100 \text{ mm}^2 \]

Depth from compression face to centroid of untensioned steel
\[ d_{ut} = 960 \text{ mm} \]

Section width \[ b_w = 300 \text{ mm} \]

Distance of section from support \[ a_v = 400 \text{ mm} \]

Shear capacity from reinforced concrete design
\[ V_{rc} = e_h s * b_w * d_{ut} / 1000 = 629.43 \text{ kN} \]

Results from shear capacity calculations

Results for a Class 1 / 2 member and a Class 3 member
Uncracked section shear capacity \[ 516.79 \text{ kN} \]
Cracked section shear capacity Class 1 / 2 \[ 2309.7 \text{ kN} \]
Cracked section shear capacity Class 3 \[ 1796.2 \text{ kN} \]
Prestressed concrete shear capacity \[ 516.79 \text{ kN} \]
Reinforced concrete shear capacity \[ 629.43 \text{ kN} \]

The shear force \( V_c \) which can be carried by the concrete alone is the greater of 516.79 kN and 629.43 kN

Shear force \( V_c \) carried by concrete \[ 629.43 \text{ kN} \]
Shear reinforcement (Cl. 6.3.4.4)

Shear reinforcement design is for a flanged beam.

Shear force due to ultimate loads $V=1100$ kN
Effective depth for shear calc. $dt=900$ mm
Diameter of link reinforcement $bd_1=12$ mm
Characteristic strength of links $f_{yv}=460$ N/mm²
Calculations are for a flanged beam.
Web thickness $bw=300$ mm
Shear force $V=1100$ kN is not greater than $1.8*V_c=1133$ kN
Spacing of links along the member should be less than the smallest of $0.75*dt=675$ mm and $4*bw=1200$ mm (Clause 6.3.4.4).
Spacing of links along the member $sv=125$ mm
Lateral spacing of individual legs of the links at a cross-section should not exceed $0.75*dt=675$ mm (Clause 6.3.4.4).
Number of legs of links across the member at right angles to the span $sv^2=2$
Cross-sectional area of all legs of the links at a member cross-section $A_{sv}=sv^2*\pi*bd_1^2/4=226.19$ mm²
Area of shear reinforcement required (Clause 6.3.4.4).
Shear force $V=1100$ kN exceeds $V_c=629.43$ kN
Area required $A_{svr}=((V-V_c)*1000+0.4*bw*dt)*sv/(0.87*f_{yv}*dt)=200.79$ mm²
Link reinforcement adequate
Area required 200.79 mm²
Area provided 226.19 mm²

Reinforcement summary

Flanged beam section.
Link reinforcement adequate Area required 200.79 mm²
Area provided 226.19 mm²
Area of effectively anchored longitudinal tensile r'ment and prestressing tendons (excluding debonded tendons) additional to that required at the ultimate limit state for other purposes. Minimum area required =1374.3 mm²
Location: EX1 - Edge beam (I Beam)

Design shear resistance of a prestressed concrete section


Depth of NA from compression face \( dc = \frac{h}{2} = 517.5 \text{ mm} \)
Consider a level just underneath the top slab. First moment of area of excluded area above this level and about the centroidal axis of the section

\[
S_{\text{uf}} = b_f f_t \left( dc - \frac{f_t}{2}\right) / 1E9 = 0.089906 \text{ m}^2
\]

Shear resistance of sections uncracked in flexure (\( VR_{dc1} \))

The following shear resistance check is at the section centroid level. Prestressing force below is after all losses have occurred.

Prestressing force after loses \( N_{Ed} = 3947 \text{ kN} \)
Section cross-sectional area \( A = 0.57375 \text{ m}^2 \)
Second moment of area \( I = 0.063969 \text{ m}^4 \)
The compressive stress \( (scp) \) due to axial loading or prestressing (after losses and including appropriate partial safety factors) at the level considered is as follows:

Compressive stress \( scp = \frac{N_{Ed} \times 1E3}{A \times 1E6} = 6.8793 \text{ N/mm}^2 \)
First moment of area of excluded area above and about the centroidal axis of the section \( S = 0.093413 \text{ m}^3 \)
Prestressing coefficient \( a_1 = 0.9 \) (pre-tensioned)
compute \( comp = (fctd^2 + a_1 \times scp \times fctd)^{0.5} = 3.5804 \text{ kN} \)
Uncracked shear resistance \( VR_{dc1} = I \times bw / S \times comp = 612.96 \text{ kN} \)

Shear resistance of sections uncracked in flexure (\( VR_{dc2} \))

In certain sections such as I-beams, where the section width varies over height the maximum principal stress may occur at a level other than at the centroidal axis. In such cases EN 1992-1-1, Clause 6.2.2(2) requires that the minimum value of shear resistance is determined by calculating \( VR_{dc} \) at various levels in the cross-section. The following shear resistance check is at the underside of the top slab.

Depth below is to centroid of section from extreme compression fibre.

Depth to centroid of section \( dc = 517.5 \text{ mm} \)
Depth below is to extreme tensile fibre from centroid of section.
Depth to extreme tensile fibre \( z_{na} = h - dc = 517.5 \text{ mm} \)
Top flange thickness \( tf = 350 \text{ mm} \)
Compressive stress \( scp = 6.8793 \text{ N/mm}^2 \)
Height of the centroid of prestressing force from extreme compression fibre of the section \( z_{pf} = 893 \text{ mm} \)
Flexural stress \( sbend = N_{Ed} / comp / (I \times 1E9) = 3.8808 \text{ N/mm}^2 \)
Consider a level just underneath the top slab. First moment of area of excluded area above this level and about the centroidal axis of the section

\[
S_{\text{uf}} = 0.089906 \text{ m}^3
\]
compute \( comp = (fctd^2 + (a_1 \times scp + sbend) \times fctd)^{0.5} = 4.3788 \text{ kN} \)
Uncracked shear resistance \( VR_{dc2} = \frac{I \cdot bw}{S_{uf} \cdot \text{comp}} = 778.89 \text{ kN} \)

**Shear resistance of sections cracked in flexure** (VR_{dc3})

The area of tensile reinforcement and bonded prestressing steel may be included in the calculation of \( A_{sl} \) below (i.e. the total area of tensioned and untensioned steel in tension zone).

Area of tensioned steel \( A_{st} = 5280 \text{ mm}^2 \)

Area of untensioned steel \( A_{su} = 0 \text{ mm}^2 \)

Depth from compression face to centroid of steel area in tension zone. The steel area includes tensioned and untensioned steel.

Effective depth (tensile reinforcement) \( d = 945.5 \text{ mm} \)

Limiting compressive stress \( scp' = 5.3333 \text{ N/mm}^2 \)

compute \( \text{comp} = (CR_{dc} \cdot ks \cdot vR_{dc} + 0.15 \cdot scp') = 1.5549 \)

Shear resistance \( V_{R_{dc3}} = \text{comp} \cdot bw \cdot d / 1000 = 367.53 \text{ kN} \)

Minimum shear resistance \( V_{R_{dm}} = v_{min} \cdot bw \cdot d / 1000 = 92.298 \text{ kN} \)

Shear resistance \( V_{R_{dc3}} = 367.53 \text{ kN} \)

**Results from shear resistance calculations**

- Uncracked section shear resistance 612.96 kN
- Uncracked section shear resistance 778.89 kN
- Cracked section shear resistance 367.53 kN

The shear force \( VR_{dc} \) which can be carried by the concrete alone is the lesser of the above three values.

Shear resistance for concrete section (VR_{dc}) 367.53 kN

Design shear force (\( V_{Ed} \)) 650 kN

As \( V_{Ed} > VR_{dc} \) (650 kN > 367.53 kN), the design shear force exceeds the shear resistance. Shear reinforcement is required.

**Shear reinforcement - in accordance with Clause 6.2.3(3)**

Shear reinforcement design is for a flanged beam.

Stress coefficient to be adopted \( acw = 1 \)

Maximum shear resistance:

\[
VR_{dmax} = acw \cdot 0.18 \cdot bw \cdot d \cdot fck \cdot (1 - fck/250) \cdot \sin(\text{RAD(2*theta'))}) / 1000
\]

\[
= 993.08 \text{ kN}
\]

Design shear force \( V_{Ed} = 650 \text{ kN} \)

Effective depth for shear calc. \( dt = 971 \text{ mm} \)

Diameter of link reinforcement \( bd1 = 12 \text{ mm} \)

Characteristic strength of links \( fyk = 500 \text{ N/mm}^2 \)

Design yield strength of links \( fyd = fyk / 1.15 = 434.78 \text{ N/mm}^2 \)

Calculations are for a flanged beam section.

Spacing of links along the member should be less than the smallest of 0.75*\( dt = 728.25 \text{ mm} \) and 300 mm.

Spacing of links along the member \( sv = 125 \text{ mm} \)

Lateral spacing of individual legs of the links at a cross-section should be less than the smallest of 600 mm or 0.75*\( dt = 728.25 \text{ mm} \).

Number of legs of links across the member at right angles to the span \( sv2 = 2 \)

Cross-sectional area of all legs of the links at a member cross-section \( Asv = sv2 \cdot \pi \cdot bd1^2 / 4 = 226.19 \text{ mm}^2 \)
Area of shear reinforcement required:

\[ Asvr = VEd * \frac{1 \times 10^3 \times sv}{0.9 \times dt \times fyd \times 1 / \tan(\text{RAD}(\theta'))} = 86.397 \, \text{mm}^2 \]

Link reinforcement adequate

Area required 86.397 mm²
Area provided 226.19 mm²

**Additional longitudinal tensile force - Clause 6.2.3(7)**

The additional longitudinal tensile force, Ftd, in the longitudinal reinforcement due to shear VEd is calculated using expression 6.18 and could be provided for by extending the curtailment point of the mid-span longitudinal reinforcement as necessary to achieve the required curtailment anchorage.

\[ \cot(\theta) = \frac{1}{\tan(\text{RAD}(\theta'))} = 2.4751 \]

Additional long.tensile force \( Ftd = 0.5 \times VEd \times ctheta = 804.4 \, \text{kN} \)

**NOTE:** As an alternative to the above curtailment option, additional longitudinal reinforcement could be provided to resist the above longitudinal tensile force.

**Reinforcement summary**

**Member under consideration is a flanged beam section.**
Link reinforcement adequate

Area required 86.397 mm²
Area provided 226.19 mm²
Use 2 No/12 mm diameter legs of links across the member spaced at 125 mm centres along the member

The additional longitudinal tensile force, Ftd=804.4 kN, in the longitudinal reinforcement due to shear VEd=650 kN is catered for by extending the curtailment point of the mid-span longitudinal reinforcement as necessary to achieve the required curtailment anchorage. Alternatively additional longitudinal reinforcement could be provided to resist this additional tensile force which is beyond the scope of this proforma.
Design shear resistance of a prestressed concrete section


First moment of area of excluded area above and about the centroidal axis of the section: $S=b\cdot h^2/(8\cdot 1E9)=0.033476 \text{ m}^3$

Principal tensile stress at centroidal axis.

Shear resistance of sections uncracked in flexure (VRdc1)

The following shear resistance check is at the section centroid level.

Prestressing force after losses $N_{Ed}=1780 \text{ kN}$

The compressive stress ($scp$) due to axial loading or prestressing (after losses and including appropriate partial safety factors) at the level considered is as follows:

Compressive stress $scp=N_{Ed}\cdot 1E3/(A\cdot 1E6)=6.8792 \text{ N/mm}^2$

First moment of area of excluded area above and about the centroidal axis of the section $S=0.033476 \text{ m}^3$

Prestressing coefficient $a_1=1$ (pre-tensioned)

Compute $comp=(fctd^2+a_1\cdot scp\cdot fctd)^0.5=3.7344 \text{ kN}$

Uncracked shear resistance $VRdc1=I\cdot bw/S\cdot comp=644.18 \text{ kN}$

Shear resistance of sections cracked in flexure (VRdc3)

The area of tensile reinforcement and bonded prestressing steel may be included in the calculation of $A_{sl}$ below (i.e. the total area of tensioned and untensioned steel in tension zone).

Area of tensioned steel $A_{st}=2000 \text{ mm}^2$

Area of untensioned steel $A_{su}=2500 \text{ mm}^2$

Depth from compression face to centroid of steel area in tension zone.

The steel area includes tensioned and untensioned steel.

Effective depth (tensile r'ment) $d=940 \text{ mm}$

Limiting compressive stress $scp'=5.3333 \text{ N/mm}^2$

Compute $comp=(CR_{dc} \cdot ks \cdot v_{Rdc}+0.15 \cdot scp')=1.5447$

Shear resistance $VRdc3=comp\cdot bw\cdot d/1000=363 \text{ kN}$

Minimum shear resistance $VRdm=v_{min}\cdot bw\cdot d/1000=91.888 \text{ kN}$

Shear resistance $VRdc3=363 \text{ kN}$

Results from shear resistance calculations

Uncracked section shear resistance $644.18 \text{ kN}$

Cracked section shear resistance $363 \text{ kN}$

The shear force $VRdc$ which can be carried by the concrete alone is the lesser of the above two values.

Shear resistance for concrete section (VRdc) $363 \text{ kN}$

Design shear force ($V_{Ed}$) $660 \text{ kN}$
As \( V_{Ed} > V_{Rdc} \) (660 kN > 363 kN), the design shear force exceeds the shear resistance. Shear reinforcement is required.

**Shear reinforcement - in accordance with Clause 6.2.3(3)**

Shear reinforcement design is for a slab.

Stress coefficient to be adopted \( acw = 1 \)

Maximum shear resistance:

\[
V_{Rd,\text{max}} = acw \times 0.18 \times bw \times d \times f_{ck} \times (1 - f_{ck}/250) \times \sin(\text{RAD}(2 \times \theta'))/1000
\]

\[= 987.3 \text{ kN} \]

Design shear force \( V_{Ed} = 660 \text{ kN} \)

Calculations are for a slab section.

- Effective depth for shear calc. \( dt = 940 \text{ mm} \)
- Diameter of link reinforcement \( bd_1 = 12 \text{ mm} \)
- Characteristic strength of links \( f_{yk} = 500 \text{ N/mm}^2 \)
- Design yield strength of links \( f_{yd} = f_{yk}/1.15 = 434.78 \text{ N/mm}^2 \)
- Shear force \( V_{Ed} = 660 \text{ kN} \) exceeds \( V_{Rdc} = 363 \text{ kN} \)
- Spacing of links along the member should be less than 0.75*\( dt = 705 \text{ mm} \) or 300 mm whichever is least.
- Spacing of links along the member \( sv = 125 \text{ mm} \)
- Lateral spacing of individual legs of the links at a cross-section should be less than the smallest of 600 mm or 0.75*\( dt = 705 \text{ mm} \).
- Number of legs of links across the member at right angles to the span \( sv^2 = 2 \)
- Cross-sectional area of all legs of the links at a member cross-section \( A_{sv} = sv^2 \times \pi \times bd_1^2/4 = 226.19 \text{ mm}^2 \)

Area of shear reinforcement required:

\[
A_{svr} = V_{Ed} \times 1E3 \times sv/(0.9 \times dt \times f_{yd} \times 1/TAN(\text{RAD}(\theta')))
\]

\[= 90.619 \text{ mm}^2 \]

Link reinforcement adequate

Area required 90.619 mm²

Area provided 226.19 mm²

**Additional longitudinal tensile force - Clause 6.2.3(7)**

The additional longitudinal tensile force, \( F_{td} \), in the longitudinal reinforcement due to shear \( V_{Ed} \) is calculated using expression 6.18 and could be provided for by extending the curtailment point of the mid-span longitudinal reinforcement as necessary to achieve the required curtailment anchorage.

\[c_{theta} = 1/TAN(\text{RAD}(\theta')) = 2.4751\]

\[F_{td} = 0.5 \times V_{Ed} \times c_{theta} = 816.78 \text{ kN}\]

NOTE: As an alternative to the above curtailment option, additional longitudinal reinforcement could be provided to resist the above longitudinal tensile force.
Reinforcement summary

Member under consideration is a slab section.
Link reinforcement adequate
Area required 90.619 mm²
Area provided 226.19 mm²
Use 2 No/12 mm diameter legs
of links across the member spaced
at 125 mm centres along the member

The additional longitudinal tensile force, Ftd=816.78 kN, in the
longitudinal reinforcement due to shear VEd=660 kN is catered for
by extending the curtailment point of the mid-span longitudinal
reinforcement as necessary to achieve the required curtailment
anchorage. Alternatively additional longitudinal reinforcement
could be provided to resist this additional tensile force which
is beyond the scope of this proforma.
Design shear resistance of a prestressed concrete section


First moment of area:
$$ S = \frac{(bf \cdot tf \cdot (dc - tf/2) + comp1 + comp2 + comp3)}{1E9} = 11.364 \text{ m}^3 $$

Consider a level just underneath the top slab. First moment of area of excluded area above this level and about the centroidal axis of the section
$$ S_{uf} = \frac{bf \cdot tf \cdot (dc - tf/2)}{1E9} = 10.29 \text{ m}^3 $$

Shear resistance of sections uncracked in flexure (VRdc1)

The following shear resistance check is at the section centroid level. Prestressing force below is after all losses have occurred.

Prestressing force after loses $N_{Ed} = 92600 \text{ kN}$

Section cross-sectional area $A = 17 \text{ m}^2$

Second moment of area $I = 33.5 \text{ m}^4$

The compressive stress ($scp$) due to axial loading or prestressing (after losses and including appropriate partial safety factors) at the level considered is as follows:

Compressive stress $scp = \frac{N_{Ed} \cdot 1E3}{A \cdot 1E6} = 5.4471 \text{ N/mm}^2$

First moment of area of excluded area above and about the centroidal axis of the section $S = 11.364 \text{ m}^3$

compute $comp = (fctd^2 + a1 \cdot scp \cdot fctd)^0.5 = 3.2255 \text{ kN}$

Uncracked shear resistance $VR_{dc1} = I \cdot bw / S \cdot comp = 11857 \text{ kN}$

Shear resistance of sections uncracked in flexure (VRdc2)

In certain sections such as I-beams, where the section width varies over height the maximum principal stress may occur at a level other than at the centroidal axis. In such cases EN 1992-1-1, Clause 6.2.2(2) requires that the minimum value of shear resistance is determined by calculating VRdc at various levels in the cross-section. The following shear resistance check is at the underside of the top slab.

Depth below is to centroid of section from extreme compression fibre.

Depth to centroid of section $dc = 1588.5 \text{ mm}$

Depth below is to extreme tensile fibre from centroid of section.

Depth to extreme tensile fibre $z_{na} = h - dc = 1911.5 \text{ mm}$

Top flange thickness $tf = 350 \text{ mm}$

Compressive stress $scp = 5.4471 \text{ N/mm}^2$

Height of the centroid of prestressing force from extreme compression fibre of the section $z_{pf} = 644 \text{ mm}$

Flexural stress $sbend = \frac{N_{Ed} \cdot comp}{(I \cdot 1E9)} = -4.3392 \text{ N/mm}^2$

Consider a level just underneath the top slab. First moment of area of excluded area above this level and about the centroidal axis of the section $S_{uf} = 10.29 \text{ m}^3$

compute $comp = (fctd^2 + a1 \cdot scp + sbend) * fctd)^{0.5} = 1.9757 \text{ kN}$
Uncracked shear resistance \[ VR_{dc2} = \frac{I \times bw}{S_{uf}} \times \text{comp} = 8020.7 \text{ kN} \]

**Shear resistance of sections cracked in flexure (VR_{dc3})**

The area of tensile reinforcement and bonded prestressing steel may be included in the calculation of \( A_{sl} \) below (i.e. the total area of tensioned and untensioned steel in tension zone).

Area of tensioned steel \[ A_{st} = 105640 \text{ mm}^2 \]

Area of untensioned steel \[ A_{su} = 0 \text{ mm}^2 \]

Depth from compression face to centroid of steel area in tension zone.

The steel area includes tensioned and untensioned steel.

Effective depth (tensile reinforcement) \[ d = 3250 \text{ mm} \]

Limiting compressive stress \[ \text{scp}' = 4.6667 \text{ N/mm}^2 \]

compute \[ \text{comp} = (CR_{dc} \times ks \times vr_{dc} + 0.15 \times scp') = 1.3172 \]

Shear resistance \[ VR_{dc3} = \frac{\text{comp} \times bw \times d}{1000} = 5338.5 \text{ kN} \]

Minimum shear resistance \[ VR_{dm} = \frac{v_{min} \times bw \times d}{1000} = 1170.1 \text{ kN} \]

Shear resistance \[ VR_{dc3} = 5338.5 \text{ kN} \]

**Results from shear resistance calculations**

Uncracked section shear resistance 11857 kN

Uncracked section shear resistance 8020.7 kN

Cracked section shear resistance 5338.5 kN

The shear force \( VR_{dc} \) which can be carried by the concrete alone is the lesser of the above three values.

Shear resistance for concrete section (VR_{dc}) 5338.5 kN

Design shear force (V_{Ed}) 6000 kN

As \( V_{Ed} > VR_{dc} (6000 \text{ kN} > 5338.5 \text{ kN}) \), the design shear force exceeds the shear resistance. Shear reinforcement is required.

**Shear reinforcement - in accordance with Clause 6.2.3(3)**

Shear reinforcement design is for a flanged beam.

Stress coefficient to be adopted \( ac_w = 1 \)

Maximum shear resistance:

\[ VR_{dmax} = ac_w \times 0.18 \times bw \times d \times f_{ck} \times (1 - f_{ck}/250) \times \sin(\text{RAD}(2\times \theta'))/1000 \]

\[ = 15253 \text{ kN} \]

Design shear force \( V_{Ed} = 6000 \text{ kN} \)

Effective depth for shear calc. \( dt = 3250 \text{ mm} \)

Diameter of link reinforcement \( bd_1 = 12 \text{ mm} \)

Characteristic strength of links \( f_{yk} = 500 \text{ N/mm}^2 \)

Design yield strength of links \( f_{yd} = f_{yk}/1.15 = 434.78 \text{ N/mm}^2 \)

Calculations are for a flanged beam section.

Spacing of links along the member should be less than the smallest of 0.75*\( dt = 2437.5 \text{ mm} \) and 300 mm.

Spacing of links along the member \( sv = 300 \text{ mm} \)

Lateral spacing of individual legs of the links at a cross-section should be less than the smallest of 600 mm or 0.75*\( dt = 2437.5 \text{ mm} \).

Number of legs of links across the member at right angles to the span \( sv_2 = 6 \)

Cross-sectional area of all legs of the links at a member cross-section \( A_{sv} = sv_2 \times \pi \times bd_1^2/4 = 678.58 \text{ mm}^2 \)
Area of shear reinforcement required:

\[ Asvr = VEd * 1E3 * sv / (0.9 * dt * fyd * 1 / \text{TAN}(\text{RAD}(\theta'))) \]

\[ = 571.85 \text{ mm}^2 \]

Link reinforcement adequate

Area required 571.85 mm\(^2\)
Area provided 678.58 mm\(^2\)

**Additional longitudinal tensile force - Clause 6.2.3(7)**

The additional longitudinal tensile force, \( Ftd \), in the longitudinal reinforcement due to shear \( VEd \) is calculated using expression 6.18 and could be provided for by extending the curtailment point of the mid-span longitudinal reinforcement as necessary to achieve the required curtailment anchorage.

\[ \text{compute cot}(\theta) \]
\[ \text{ctheta} = 1 / \text{TAN}(\text{RAD}(\theta')) = 2.4751 \]

Additional long. tensile force \( Ftd = 0.5 * VEd * \text{ctheta} = 7425.3 \text{ kN} \)

**NOTE:** As an alternative to the above curtailment option, additional longitudinal reinforcement could be provided to resist the above longitudinal tensile force.

**Reinforcement summary**

Member under consideration is a flanged beam section.

Link reinforcement adequate

Area required 571.85 mm\(^2\)
Area provided 678.58 mm\(^2\)
Use 6 No/12 mm diameter legs of links across the member spaced at 300 mm centres along the member

The additional longitudinal tensile force, \( Ftd = 7425.3 \text{ kN} \), in the longitudinal reinforcement due to shear \( VEd = 6000 \text{ kN} \) is catered for by extending the curtailment point of the mid-span longitudinal reinforcement as necessary to achieve the required curtailment anchorage. Alternatively, additional longitudinal reinforcement could be provided to resist this additional tensile force which is beyond the scope of this proforma.
Location: EX4 - As Ex3 but with minimum shear links

Design shear resistance of a prestressed concrete section


First moment of area:
\[ S = \frac{bf \times tf \times (dc - tf/2) + comp1 + comp2 + comp3}{1E9} = 11.364 \text{ m}^3 \]

Consider a level just underneath the top slab. First moment of area of excluded area above this level and about the centroidal axis of the section
\[ S_{uf} = \frac{bf \times tf \times (dc - tf/2)}{1E9} = 10.29 \text{ m}^3 \]

Shear resistance of sections uncracked in flexure (VRdc1)

The following shear resistance check is at the section centroid level.

Prestressing force below is after all losses have occurred.
- Prestressing force after loses \( N_{Ed} = 92600 \text{ kN} \)
- Section cross-sectional area \( A = 17 \text{ m}^2 \)
- Second moment of area \( I = 33.5 \text{ m}^4 \)

The compressive stress (scp) due to axial loading or prestressing (after losses and including appropriate partial safety factors) at the level considered is as follows:
- Compressive stress \( scp = \frac{N_{Ed} \times 1E3}{A \times 1E6} = 5.4471 \text{ N/mm}^2 \)
- First moment of area of excluded area above and about the centroidal axis of the section \( S = 11.364 \text{ m}^3 \)
- compute \( comp = (fctd^2 + a1 \times scp \times fctd)^0.5 = 3.2255 \text{ kN} \)
- Uncracked shear resistance \( VR_{dc1} = I \times bw/S \times comp = 11857 \text{ kN} \)

Shear resistance of sections uncracked in flexure (VRdc2)

In certain sections such as I-beams, where the section width varies over height the maximum principal stress may occur at a level other than at the centroidal axis. In such cases EN 1992-1-1, Clause 6.2.2(2) requires that the minimum value of shear resistance is determined by calculating VRdc at various levels in the cross-section. The following shear resistance check is at the underside of the top slab.

Depth below is to centroid of section from extreme compression fibre.
- Depth to centroid of section \( dc = 1588.5 \text{ mm} \)
- Depth below is to extreme tensile fibre from centroid of section.
- Depth to extreme tensile fibre \( z_{na} = h - dc = 1911.5 \text{ mm} \)
- Top flange thickness \( tf = 350 \text{ mm} \)
- Compressive stress \( scp = 5.4471 \text{ N/mm}^2 \)
- Height of the centroid of prestressing force from extreme compression fibre of the section \( z_{pf} = 644 \text{ mm} \)
- Flexural stress \( sbend = \frac{N_{Ed} \times comp}{(I \times 1E9)} = -4.3392 \text{ N/mm}^2 \)

Consider a level just underneath the top slab. First moment of area of excluded area above this level and about the centroidal axis of the section
- compute \( comp = (fctd^2 + (a1 \times scp + sbend) \times fctd)^0.5 = 1.9757 \text{ kN} \)
Uncracked shear resistance \[ VRdc2 = \frac{I \times bw}{Suf \times \text{comp}} = 8020.7 \text{kN} \]

**Shear resistance of sections cracked in flexure (VRdc3)**

The area of tensile reinforcement and bonded prestressing steel may be included in the calculation of \( A_{sl} \) below (i.e. the total area of tensioned and untensioned steel in tension zone).

- Area of tensioned steel \( A_{st} = 105640 \text{ mm}^2 \)
- Area of untensioned steel \( A_{su} = 0 \text{ mm}^2 \)

Depth from compression face to centroid of steel area in tension zone.

The steel area includes tensioned and untensioned steel.

- Effective depth \( d = 3250 \text{ mm} \)
- Limiting compressive stress \( scp' = 4.6667 \text{ N/mm}^2 \)

Compute: \[ \text{comp} = (CRdc \times ks \times V_{Rdc} + 0.15 \times scp') = 1.3172 \]

Shear resistance \( VRdc3 = \text{comp} \times bw \times d / 1000 = 5338.5 \text{ kN} \)

Minimum shear resistance \( VR_{dm} = v_{min} \times bw \times d / 1000 = 1170.1 \text{ kN} \)

Shear resistance \( VRdc3 = 5338.5 \text{ kN} \)

**Results from shear resistance calculations**

Uncracked section shear resistance \( 11857 \text{ kN} \)
Uncracked section shear resistance \( 8020.7 \text{ kN} \)
Cracked section shear resistance \( 5338.5 \text{ kN} \)

The shear force \( VRdc \) which can be carried by the concrete alone is the lesser of the above three values.

Shear resistance for concrete section \( (VRdc) = 5338.5 \text{ kN} \)
Design shear force \( (VEd) = 5300 \text{ kN} \)

As \( VEd \leq VRdc \) (5300 kN \( \leq 5338.5 \text{ kN} \)), the design shear force is less than the shear resistance of the section. Hence, provide minimum shear reinforcement.

**Shear reinforcement - in accordance with Clause 6.2.3(3)**

Shear reinforcement design is for a flanged beam.

Stress coefficient to be adopted \( acw = 1 \)

Maximum shear resistance:

\[ VRdmax = acw \times 0.18 \times bw \times d \times fck \times (1 - fck/250) \times \sin(\text{RAD}(2 \times \text{theta}')) / 1000 \]
\[ = 15253 \text{ kN} \]

Design shear force \( VEd = 5300 \text{ kN} \)
Effective depth for shear calc. \( dt = 3250 \text{ mm} \)
Diameter of link reinforcement \( bd1 = 10 \text{ mm} \)
Characteristic strength of links \( f_{yk} = 500 \text{ N/mm}^2 \)
Design yield strength of links \( f_{yd} = f_{yk} / 1.15 = 434.78 \text{ N/mm}^2 \)

Calculations are for a flanged beam section.

Spacing of links along the member should be less than the smallest of 0.75\( \times dt = 2437.5 \text{ mm} \) and 300 mm.

Spacing of links along the member \( sv = 300 \text{ mm} \)
Lateral spacing of individual legs of the links at a cross-section should be less than the smallest of 600 mm or 0.75\( \times dt = 2437.5 \text{ mm} \).

Number of legs of links across the member at right angles to the span \( sv2 = 6 \)
Cross-sectional area of all legs of the links at a
member cross-section \[ \text{Asv}=\text{sv}^2\pi\text{bd}_1^2/4=471.24 \text{ mm}^2 \]
Minimum area of shear r'ment \[ \text{Asvr}=0.08*\text{fck}^{0.5}*(\text{sv}/\text{fyk})=397.56 \text{ mm}^2 \]
Link reinforcement adequate \[ \text{Area required}=397.56 \text{ mm}^2 \]
\[ \text{Area provided}=471.24 \text{ mm}^2 \]

**Additional longitudinal tensile force - Clause 6.2.3(7)**

The additional longitudinal tensile force, \( F_{td} \), in the longitudinal reinforcement due to shear \( V_{Ed} \) is calculated using expression 6.18 and could be provided for by extending the curtailment point of the mid-span longitudinal reinforcement as necessary to achieve the required curtailment anchorage.

\[ \text{compute } \cot(\theta) \quad \theta=\arctan(\text{Asv}/\pi\text{bd}^2/2) \]
\[ \text{ctheta}=1/\tan(\theta)=2.4751 \]

**Additional long.tensile force** \[ F_{td}=0.5*V_{Ed}*\text{ctheta}=6559 \text{ kN} \]

**NOTE:** As an alternative to the above curtailment option, additional longitudinal reinforcement could be provided to resist the additional longitudinal tensile force.

**Reinforcement summary**

Member under consideration is a flanged beam section.

**Link reinforcement adequate**

Area required 397.56 mm\(^2\)
Area provided 471.24 mm\(^2\)
Use 6 No/10 mm diameter legs of links across the member spaced at 300 mm centres along the member

The additional longitudinal tensile force, \( F_{td}=6559 \text{ kN} \), in the longitudinal reinforcement due to shear \( V_{Ed}=5300 \text{ kN} \) is catered for by extending the curtailment point of the mid-span longitudinal reinforcement as necessary to achieve the required curtailment anchorage. Alternatively additional longitudinal reinforcement could be provided to resist this additional tensile force which is beyond the scope of this proforma.
Location: EX5 - As Ex3 but using 2 Web Box

Design shear resistance of a prestressed concrete section


First moment of area:
\[ S = \frac{b_f t_f (d_c - t_f/2) + c_{comp1} + c_{comp2}}{1E9} = 10.981 \text{ m}^3 \]
Consider a level just underneath the top slab. First moment of area of excluded area above this level and about the centroidal axis of the section
\[ S_{uf} = \frac{b_f t_f (d_c - t_f/2)}{1E9} = 10.29 \text{ m}^3 \]

Shear resistance of sections uncracked in flexure (VR_{dc1})

The following shear resistance check is at the section centroid level. Prestressing force below is after all losses have occurred.
- Prestressing force after losses \( N_Ed = 92600 \text{ kN} \)
- Section cross-sectional area \( A = 17 \text{ m}^2 \)
- Second moment of area \( I = 33.5 \text{ m}^4 \)
The compressive stress \( (\sigma_{cp}) \) due to axial loading or prestressing (after losses and including appropriate partial safety factors) at the level considered is as follows:
- Compressive stress \( \sigma_{cp} = \frac{N_Ed}{A} = 5.4471 \text{ N/mm}^2 \)
- First moment of area of excluded area above and about the centroidal axis of the section \( S = 10.981 \text{ m}^3 \)
- Compute \( \text{comp} = (f_{ctd}^2 + a_1 \sigma_{cp} f_{ctd})^{0.5} = 3.2255 \text{ kN} \)
Uncracked shear resistance \( \text{VR}_{dc1} = I b_w/S \text{comp} = 7852.6 \text{ kN} \)

Shear resistance of sections uncracked in flexure (VR_{dc2})

In certain sections such as I-beams, where the section width varies over height the maximum principal stress may occur at a level other than at the centroidal axis. In such cases EN 1992-1-1, Clause 6.2.2(2) requires that the minimum value of shear resistance is determined by calculating VRdc at various levels in the cross-section. The following shear resistance check is at the underside of the top slab. Depth below is to centroid of section from extreme compression fibre.
- Depth to centroid of section \( d_c = 1588.5 \text{ mm} \)
- Depth below to extreme tensile fibre from centroid of section \( z_{na} = h - d_c = 1911.5 \text{ mm} \)
- Top flange thickness \( t_f = 350 \text{ mm} \)
- Compressive stress \( \sigma_{cp} = 5.4471 \text{ N/mm}^2 \)
- Height of the centroid of prestressing force from extreme compression fibre of the section \( z_{pf} = 644 \text{ mm} \)
- Flexural stress \( \sigma_{bend} = \frac{N_Ed}{I} = -4.3392 \text{ N/mm}^2 \)
Consider a level just underneath the top slab. First moment of area of excluded area above this level and about the centroidal axis of the section
\[ S_{uf} = 10.29 \text{ m}^3 \]
- Compute \( \text{comp} = (f_{ctd}^2 + (a_1 \sigma_{cp} + \sigma_{bend}) f_{ctd})^{0.5} = 1.9757 \text{ kN} \)
Uncracked shear resistance \( VR_{dc2} = I \times bw / S_{uf} \times \text{comp} = 5132.7 \text{ kN} \)

**Shear resistance of sections cracked in flexure (VR_{dc3})**

The area of tensile reinforcement and bonded prestressing steel may be included in the calculation of \( A_{sl} \) below (i.e. the total area of tensioned and untensioned steel in tension zone).

Area of tensioned steel \( A_{st} = 105640 \text{ mm}^2 \)
Area of untensioned steel \( A_{su} = 0 \text{ mm}^2 \)

Depth from compression face to centroid of steel area in tension zone.
The steel area includes tensioned and untensioned steel.

Effective depth (tensile r'ment) \( d = 3250 \text{ mm} \)
Limiting compressive stress \( scp' = 4.6667 \text{ N/mm}^2 \)

compute \( \text{comp} = (CR_{dc} \times ks \times VR_{dc} + 0.15 \times scp') = 1.3172 \)

Shear resistance \( VR_{dc3} = \text{comp} \times bw \times d / 1000 = 3416.3 \text{ kN} \)

Minimum shear resistance \( VR_{dm} = \text{vmin} \times bw \times d / 1000 = 748.77 \text{ kN} \)

Shear resistance \( VR_{dc3} = 3416.3 \text{ kN} \)

**Results from shear resistance calculations**

| Uncracked section shear resistance | 7852.6 kN |
| Uncracked section shear resistance | 5132.7 kN |
| Cracked section shear resistance  | 3416.3 kN |

The shear force \( VR_{dc} \) which can be carried by the concrete alone is the lesser of the above three values.

Shear resistance for concrete section (VR_{dc}) \( 3416.3 \text{ kN} \)
Design shear force (VE_{d}) \( 6000 \text{ kN} \)

As VE_{d} > VR_{dc} \( (6000 \mathrm{kN} > 3416.3 \mathrm{kN}) \), the design shear force exceeds the shear resistance. Shear reinforcement is required.

**Shear reinforcement - in accordance with Clause 6.2.3(3)**

Shear reinforcement design is for a flanged beam.
Stress coefficient to be adopted \( acw = 1 \)
Maximum shear resistance:

\[
VR_{dmax} = acw \times 0.18 \times bw \times d \times f_{ck} \times (1 - f_{ck}/250) \times \sin(\text{RAD}(2 \times \theta')) / 1000 = 9761 \text{ kN}
\]

Design shear force \( VE_{d} = 6000 \text{ kN} \)
Effective depth for shear calc. \( dt = 3250 \text{ mm} \)
Diameter of link reinforcement \( bd_1 = 12 \text{ mm} \)
Characteristic strength of links \( f_{yk} = 500 \text{ N/mm}^2 \)
Design yield strength of links \( f_{yd} = f_{yk} / 1.15 = 434.78 \text{ N/mm}^2 \)

Calculations are for a flanged beam section.
Spacing of links along the member should be less than the smallest of 0.75*dt=2437.5 mm and 300 mm.
Spacing of links along the member \( sv = 300 \text{ mm} \)
Lateral spacing of individual legs of the links at a cross-section should be less than the smallest of 600 mm or 0.75*dt=2437.5 mm.
Number of legs of links across the member at right angles to the span \( sv_2 = 6 \)
Cross-sectional area of all legs of the links at a member cross-section \( A_{sv} = sv_2 \times PI \times bd_1^2 / 4 = 678.58 \text{ mm}^2 \)
Area of shear reinforcement required:

\[ A_{svr} = \frac{V_E d_s \times 10^3 \times s_v}{0.9 \times d_t \times f_y d} \times \frac{1}{\tan(\theta')} \]

= 571.85 mm²

Link reinforcement adequate

Area required 571.85 mm²
Area provided 678.58 mm²

**Additional longitudinal tensile force - Clause 6.2.3(7)**

The additional longitudinal tensile force, \( F_{td} \), in the longitudinal reinforcement due to shear \( V_E d \) is calculated using expression 6.18 and could be provided for by extending the curtailment point of the mid-span longitudinal reinforcement as necessary to achieve the required curtailment anchorage.

\[ c\theta = \frac{1}{\tan(\theta')} = 2.4751 \]

Additional long. tensile force \( F_{td} = 0.5 \times V_E d \times c\theta = 7425.3 \) kN

**NOTE:** As an alternative to the above curtailment option, additional longitudinal reinforcement could be provided to resist the above longitudinal tensile force.

**Reinforcement summary**

Member under consideration is a flanged beam section.

Link reinforcement adequate

Area required 571.85 mm²
Area provided 678.58 mm²

Use 6 No/12 mm diameter legs of links across the member spaced at 300 mm centres along the member.

The additional longitudinal tensile force, \( F_{td} = 7425.3 \) kN, in the longitudinal reinforcement due to shear \( V_E d = 6000 \) kN is catered for by extending the curtailment point of the mid-span longitudinal reinforcement as necessary to achieve the required curtailment anchorage. Alternatively additional longitudinal reinforcement could be provided to resist this additional tensile force which is beyond the scope of this proforma.
Location: Ex1 - Edge Beam

Ultimate limit state: shear capacity of a prestressed concrete section

Calculations are in accordance with Departmental Standard BD 44/15 "The Assessment of Concrete Highway Bridges and Structures". Reference is made to BS5400:Part 4:1990 implemented by DOT to BD 21/01 "The Assessment of Highway Bridges and Structures". Unless noted otherwise, references are to Appendix A of BD 44/15.

Flanged member

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of member</td>
<td>250 mm</td>
</tr>
<tr>
<td>Overall depth of member</td>
<td>1035 mm</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>50 N/mm²</td>
</tr>
<tr>
<td>Factor for concrete</td>
<td>1.5</td>
</tr>
<tr>
<td>Factor for reinforcement</td>
<td>1.15</td>
</tr>
<tr>
<td>Factor for shear in concrete</td>
<td>1.25</td>
</tr>
</tbody>
</table>

Compressive stress at centroidal axis due to prestress (fcp).

\[ f_{cp} = 6.88 \text{ N/mm}^2 \]

Component of prestressing force Vvc=0 kN

Depth to centroid of section from extreme compression fibre:
\[ d_c = 517.5 \text{ mm} \]

Depth to extreme tensile fibre from centroid of concrete section:
\[ y = h - d_c = 517.5 \text{ mm} \]

Second moment of area I=63,969E9 mm⁴

fpt is the stress due to prestress at the extreme tensile fibre

\[ f_{pt} = 16.53 \text{ N/mm}^2 \]
\[ f_{pu} = 1820 \text{ N/mm}^2 \]

Effective prestress fpe=776 N/mm²

Depth from extreme compression fibre to centroid of all tendons:
\[ d = 893 \text{ mm} \]

Total area of tensioned and untensioned steel in tension zone:
\[ A_s = 5280 \text{ mm}^2 \]

Depth from compression face to centroid of steel area in tension zone (As). The steel area includes tensioned and untensioned steel.

\[ d_s = 946 \text{ mm} \]

fpt1 is the stress due to prestress at depth 893 mm

\[ f_{pt1} = 13 \text{ N/mm}^2 \]

Shear resistance from reinforcement

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective depth</td>
<td>1144 mm</td>
</tr>
<tr>
<td>Distance of section from support</td>
<td>1500 mm</td>
</tr>
</tbody>
</table>
Angle of inclination of shear reinforcement $\alpha = 90^\circ$

- $A_{sv} = 226 \text{ mm}^2$
- $f_{yv} = 250 \text{ N/mm}^2$

- In direction of span $s_{v1} = 175 \text{ mm}$
- At right angles to span $s_{v2} = 150 \text{ mm}$

- Applied moment at Section 'S' $M_{AMB} = 3150 \text{ kNm}$
- Area of tensioned steel $A_{st} = 5280 \text{ mm}^2$
- Area of untensioned steel $A_{su} = 0 \text{ mm}^2$
- Strength of tensioned steel $f_{put} = 1820 \text{ N/mm}^2$

Results from Shear Capacity Calculations

Concrete Section:
Uncracked Section Shear Capacity 696.14 kN

Equation 30A used to calculate cracked section capacity

Cracked Section Shear Capacity 898.73 kN

The shear force $V_c$ which can be carried by the concrete alone is the lesser of 696.14 kN and 898.73 kN.

Shear force $V_c$ carried by concrete 696.14 kN

Shear Reinforcement:
Shear capacity 321.17 kN

Total shear capacity of section 1017.3 kN

Maximum shear force as per BD 44/15 Clause 6.3.4.5.
Maximum shear force 1256.2 kN

Total shear capacity of section 1017.3 kN is less than maximum shear force 1256.2 kN.

Tensile capacity of longitudinal reinforcement 8356.2 kN is greater than the tensile capacity for bending 3699.8 kN.

Tensile capacity of longitudinal reinforcement is adequate.

Ensure that effective reinforcement extends at least 572 mm from Section 'S' in direction of decreasing bending moment.
**Location: Ex2 - Beam A**

**Ultimate limit state: shear capacity of a prestressed concrete section**

Calculations are in accordance with Departmental Standard BD 44/15 "The Assessment of Concrete Highway Bridges and Structures". Reference is made to BS5400:Part 4:1990 implemented by DOT to BD 21/01 "The Assessment of Highway Bridges and Structures". Unless noted otherwise, references are to Appendix A of BD 44/15.

**Flanged member**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of member</td>
<td>b=250 mm</td>
</tr>
<tr>
<td>Overall depth of member</td>
<td>h=1035 mm</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>fcu=50 N/mm²</td>
</tr>
<tr>
<td>Factor for concrete</td>
<td>gmc=1.5</td>
</tr>
<tr>
<td>Factor for reinforcement</td>
<td>gms=1.15</td>
</tr>
<tr>
<td>Factor for shear in concrete</td>
<td>gmv=1.25</td>
</tr>
<tr>
<td>Compressive stress at centroidal axis due to prestress (fcp).</td>
<td></td>
</tr>
<tr>
<td>Stress due to prestress</td>
<td>fcp=6.88 N/mm²</td>
</tr>
<tr>
<td>Component of prestressing force</td>
<td>Vvc=50 kN</td>
</tr>
<tr>
<td>Depth to centroid of section from extreme compression fibre:</td>
<td></td>
</tr>
<tr>
<td>dc</td>
<td>517.5 mm</td>
</tr>
<tr>
<td>Depth to extreme tensile fibre from centroid of concrete section:</td>
<td></td>
</tr>
<tr>
<td>y=h-dc</td>
<td>517.5 mm</td>
</tr>
<tr>
<td>Second moment of area</td>
<td>I=23.098E9 mm⁴</td>
</tr>
<tr>
<td>fpt is the stress due to prestress at the extreme tensile fibre</td>
<td></td>
</tr>
<tr>
<td>Stress fpt</td>
<td>fpt=15.2 N/mm²</td>
</tr>
<tr>
<td>Area of tensioned steel</td>
<td>Ast=2000 mm²</td>
</tr>
<tr>
<td>Area of untensioned steel</td>
<td>Asu=2500 mm²</td>
</tr>
<tr>
<td>fput=1800 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Char strength or worst credible strength of untensioned steel:</td>
<td>fyLu=460 N/mm²</td>
</tr>
<tr>
<td>Effective prestressing force after all losses have occurred multiplied by the appropriate partial safety factor from Clause 4.2.3.</td>
<td></td>
</tr>
<tr>
<td>Effective force</td>
<td>Pf=1780 kN</td>
</tr>
<tr>
<td>Depth from extreme compression fibre to centroid of all tendons:</td>
<td>d=900 mm</td>
</tr>
<tr>
<td>Total area of tensioned and untensioned steel in tension zone:</td>
<td>As=4500 mm²</td>
</tr>
<tr>
<td>Depth from compression face to centroid of steel area in tension zone (As). The steel area includes tensioned and untensioned steel.</td>
<td></td>
</tr>
<tr>
<td>Depth ds</td>
<td>ds=940 mm</td>
</tr>
<tr>
<td>fpt1 is the stress due to prestress at depth 900 mm</td>
<td></td>
</tr>
<tr>
<td>Stress fpt1</td>
<td>fpt1=13.03 N/mm²</td>
</tr>
</tbody>
</table>
Shear resistance from reinforcement

Effective depth \( dt = 980 \text{ mm} \)
Distance of section from support \( av = 1500 \text{ mm} \)
Angle of inclination of shear r'ment \( \alpha = 75^\circ \)
  
  Asv = 226 \text{ mm}^2
  fyv = 250 \text{ N/mm}^2
In direction of span \( sv_1 = 175 \text{ mm} \)
At right angles to span \( sv_2 = 150 \text{ mm} \)
Value of K for calculations \( K = 1.2679 \)
Applied moment at Section 'B' \( AM = 2000 \text{ kNm} \)
Co-existent TOTAL shear force at Section 'B' \( AV = 250 \text{ kN} \)

Concrete Section:
Uncracked Section Shear Capacity \( 746.14 \text{ kN} \)
Equation 30A used to calculate cracked section capacity
Calculations for Mo to BD 44/15
Cracked Section Shear Capacity \( 412.12 \text{ kN} \)
The shear force \( V_c \) which can be carried by the concrete alone
is the lesser of 746.14 kN and 412.12 kN.

Shear force \( V_c \) carried by concrete \( 412.12 \text{ kN} \)

Results from Shear Capacity Calculations

Shear Reinforcement:
Shear capacity \( 336.96 \text{ kN} \)
Total shear capacity of section \( 749.09 \text{ kN} \)
Maximum shear force as per BD 44/15 Clause 6.3.4.5.
Maximum shear force \( 1248.2 \text{ kN} \)
Total shear capacity of section \( 749.09 \text{ kN} \) is less than maximum shear force 1248.2 kN.

Tensile capacity of longitudinal r'ment 4130.4 kN exceeds required tensile capacity of 2391.1 kN required by the bending moment and co-existent shear force carried by the shear reinforcement at Section B.
Tensile capacity of longitudinal reinforcement is adequate.
Location: Ex3 - Beam B

Ultimate limit state: shear capacity of a prestressed concrete section

Calculations are in accordance with Departmental Standard BD 44/15 "The Assessment of Concrete Highway Bridges and Structures". Reference is made to BS5400:Part 4:1990 implemented by DOT to BD 21/01 "The Assessment of Highway Bridges and Structures". Unless noted otherwise, references are to Appendix A of BD 44/15.

Rectangular member

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of member</td>
<td>b=250 mm</td>
</tr>
<tr>
<td>Overall depth of member</td>
<td>h=1035 mm</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>fcu=50 N/mm²</td>
</tr>
<tr>
<td>Factor for concrete</td>
<td>gmc=1.5</td>
</tr>
<tr>
<td>Factor for reinforcement</td>
<td>gms=1.15</td>
</tr>
<tr>
<td>Factor for shear in concrete</td>
<td>gmv=1.25</td>
</tr>
<tr>
<td>Compressive stress at centroidal axis due to prestress (fcp).</td>
<td>fcp=6.88 N/mm²</td>
</tr>
<tr>
<td>Stress due to prestress</td>
<td>fcp=6.88 N/mm²</td>
</tr>
</tbody>
</table>

Component of prestressing force Vvc=50 kN

Depth to centroid of section from extreme compression fibre: dc=517.5 mm

Depth to extreme tensile fibre from centroid of concrete section: y=h-dc=517.5 mm

Second moment of area I=23.098E9 mm⁴

fpt is the stress due to prestress at the extreme tensile fibre

Stress fpt fpt=15.2 N/mm²

Area of tensioned steel Ast=1500 mm²

Area of untensioned steel Asu=500 mm²

fput=1800 N/mm²

Char strength or worst credible strength of untensioned steel: fyLu=460 N/mm²

Effective prestressing force after all losses have occurred multiplied by the appropriate partial safety factor from Clause 4.2.3.

Effective force Pf=1780 kN

Depth from extreme compression fibre to centroid of all tendons: d=600 mm

Depth from compression face to centroid of steel area in tension zone (As). The steel area includes tensioned and untensioned steel. Depth ds ds=940 mm

Shear resistance from reinforcement

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective depth</td>
<td>dt=940 mm</td>
</tr>
<tr>
<td>Distance of section from support</td>
<td>av=1500 mm</td>
</tr>
</tbody>
</table>
Angle of inclination of shear r'ment $\alpha=90^\circ$

- $A_{sv}=226\text{ mm}^2$
- $f_{yv}=250\text{ N/mm}^2$
- In direction of span $sv_1=300\text{ mm}$
- At right angles to span $sv_2=150\text{ mm}$
- Applied moment at Section B $AM=2000\text{ kNm}$
- Co-existent TOTAL shear force $AV=550\text{ kN}$
- Area of steel $A_s=2000\text{ mm}^2$
- Depth $d_s=940\text{ mm}$
- Constant $K_{VC}=0.24$
- Area of tensioned steel $A_{st}=1500\text{ mm}^2$
- Area of untensioned steel $A_{su}=500\text{ mm}^2$
- Strength of tensioned steel $f_{put}=1800\text{ N/mm}^2$
- Strength of untensioned steel $f_{yu}=460\text{ N/mm}^2$
- Maximum moment in region $M_{max}=4000\text{ kNm}$
- Associated effective depth $d_{tmax}=940\text{ mm}$

Results from Shear Capacity Calculations

Concrete Section:
- Uncracked Section Shear Capacity $746.14\text{ kN}$
- Equation 29A used to calculate cracked section capacity
- Cracked Section Shear Capacity $284.91\text{ kN}$
- The shear force $V_c$ which can be carried by the concrete alone is the lesser of 746.14 kN and 284.91 kN.
- Shear force $V_c$ carried by concrete $284.91\text{ kN}$
- Shear Reinforcement:
  - Shear capacity $59.942\text{ kN}$
  - Total shear capacity of section $344.86\text{ kN}$
  - Maximum shear force as per BD 44/15 Clause 6.3.4.5.
  - Maximum shear force $1248.2\text{ kN}$
  - Total shear capacity of section $344.86\text{ kN}$ is less than maximum shear force $1248.2\text{ kN}$.

Tensile capacity of longitudinal r'ment $2547.8\text{ kN}$ is less than the tensile capacity of $2571.9\text{ kN}$ required by the bending moment and co-existent shear force carried by the shear reinforcement at Section B.

Tensile capacity of longitudinal r'ment $2547.8\text{ kN}$ is less than required tensile capacity for $M_{max}$ of $4728.1\text{ kN}$.

Tensile capacity of longitudinal reinforcement not adequate.
**Location: Ex4 - Beam C**

**Ultimate limit state : shear capacity of a prestressed concrete section**

Calculations are in accordance with Departmental Standard BD 44/15 "The Assessment of Concrete Highway Bridges and Structures". Reference is made to BS5400:Part 4:1990 implemented by DOT to BD 21/01 "The Assessment of Highway Bridges and Structures". Unless noted otherwise, references are to Appendix A of BD 44/15.

Transmission length \( lt = 1300 \text{ mm} \)

**Flanged member**

Breadth of member \( b = 300 \text{ mm} \)
Overall depth of member \( h = 950 \text{ mm} \)
Concrete strength \( f_{cu} = 40 \text{ N/mm}^2 \)
Factor for concrete \( g_{mc} = 1.2 \)
Factor for reinforcement \( g_{ms} = 1.1 \)
Factor for shear in concrete \( g_{mv} = 1.15 \)
Compressive stress at centroidal axis due to prestress \( f_{cp} \).
Stress due to prestress \( f_{cp} = 8.6 \text{ N/mm}^2 \)

Distance from start of transmission length (zero stress end):
\( d_{lt(1)} = 500 \text{ mm} \)

Component of prestressing force \( V_{vc} = 0 \text{ kN} \)
Depth to centroid of section from extreme compression fibre:
\( d_{c} = 400 \text{ mm} \)
Depth to extreme tensile fibre from centroid of concrete section:
\( y = h - d_{c} = 550 \text{ mm} \)
Second moment of area \( I = 35.6E9 \text{ mm}^4 \)
\( f_{pt} \) is the stress due to prestress at the extreme tensile fibre
Stress \( f_{pt} \) at end of transmission length:
\( f_{pt} = 14 \text{ N/mm}^2 \)

Area of tensioned steel \( A_{st} = 2500 \text{ mm}^2 \)
Area of untensioned steel \( A_{su} = 1100 \text{ mm}^2 \)
Char strength or worst credible strength of tensioned steel:
\( f_{put} = 1720 \text{ N/mm}^2 \)
Char strength or worst credible strength of untensioned steel:
\( f_{yL} = 460 \text{ N/mm}^2 \)
Effective prestressing force after all losses have occurred multiplied by the appropriate partial safety factor from Clause 4.2.3
Effective force \( P_f = 1500 \text{ kN} \)
Depth from extreme compression fibre to centroid of all tendons:
\( d = 850 \text{ mm} \)
Total area of tensioned and untensioned steel in tension zone:
\( A_s = 3600 \text{ mm}^2 \)
Depth from compression face to centroid of steel area in tension zone (\( A_s \)). The steel area includes tensioned and untensioned steel.
Depth \( d_{s} = 940 \text{ mm} \)
Area of tension reinforcement: $A_s = 3600 \text{ mm}^2$

Effective depth of tensile reinforcement: $d = 850 \text{ mm}$

Distance of section from support: $a_v = 400 \text{ mm}$

**Shear resistance from reinforcement**

- Effective depth: $d_t = 900 \text{ mm}$
- Distance of section from support: $a_v = 400 \text{ mm}$
- Resistance ($\Sigma \text{WRV}$): $\text{WRV} = 400 \text{ kN}$

**Results from Shear Capacity Calculations**

**Results for section 500 mm from start of transmission length.**

**Prestressed Concrete Section:**
- Uncracked Section Shear Capacity: $589.3 \text{ kN}$
- Applied moment is less than cracking moment.
- The shear force $V_c$ which can be carried by the concrete alone is the uncracked capacity $589.3 \text{ kN}$.

**Reinforced Concrete Section:**
- Reinforced Concrete Shear Capacity: $1138.4 \text{ kN}$
- The shear force $V_c$ which can be carried by the concrete alone is the greater of $589.3 \text{ kN}$ and $1138.4 \text{ kN}$.
- Shear force carried by concrete: $1138.4 \text{ kN}$

**Shear Reinforcement:**
- Shear capacity: $400 \text{ kN}$

**Total shear capacity of section:** $1538.4 \text{ kN}$

**Maximum shear force as per BD 44/15 Clause 6.3.4.5:**
- Maximum shear force: $1497.9 \text{ kN}$

**Total shear capacity of section 1538.4 kN exceeds maximum shear force 1497.9 kN.**

**Reduced shear capacity**: $t_{sc} = V_{max1} = 1497.9 \text{ kN}$

**Tensile capacity of longitudinal reinforcement not assessed.**
**Location:** 13.5 metre span TY7 beam with solid infill

**Precast prestressed concrete TY beams with in-fill concrete**

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

<table>
<thead>
<tr>
<th>Span of beam</th>
<th>span=13.5 m</th>
</tr>
</thead>
</table>

**Beam TY 7 section properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=700 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=272194 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=272.9 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>Zt=29.42E6 mm³</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>Zb=46.04E6 mm³</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=25 kN/m³</td>
</tr>
</tbody>
</table>

**Concrete strengths**

<table>
<thead>
<tr>
<th>Strength</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength at transfer</td>
<td>fci=40 N/mm²</td>
</tr>
<tr>
<td>Characteristic strength</td>
<td>fcu=50 N/mm²</td>
</tr>
</tbody>
</table>

**Concrete moduli**

<table>
<thead>
<tr>
<th>Modulus of elasticity</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus at transfer for concrete strength 40 N/mm²</td>
<td>Eci=31 kN/mm²</td>
</tr>
<tr>
<td>Modulus for concrete strength 50 N/mm²</td>
<td>Ecb=34 kN/mm²</td>
</tr>
</tbody>
</table>

**Composite section properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char strength of in-situ concrete</td>
<td>fcuc=40 N/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity for concrete strength 40 N/mm²</td>
<td>Ecc=31 kN/mm²</td>
</tr>
</tbody>
</table>

**Graph**

- Centroids are related to the soffit of section.
- Precast beam shown dotted.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of rectangular section</td>
<td>b=765 mm</td>
</tr>
<tr>
<td>Depth of rectangular section</td>
<td>di=875 mm</td>
</tr>
<tr>
<td>Second moment of area</td>
<td>Ic=42.708E9 mm⁴</td>
</tr>
</tbody>
</table>
Initial force per strand $i_f$ = 116 kN
Modulus of elasticity $E_s$ = 200 kN/mm²

**Cross-section 1**

Distance from mid-span of beam $c_{sd}(1)$ = 0 m
Number of layers of strand $n_{ls}(1)$ = 4
Height above soffit $d_s(1)$ = 60 mm
Number of strands $n_s(1)$ = 11
Height above soffit $d_s(2)$ = 100 mm
Number of strands $n_s(2)$ = 9
Height above soffit $d_s(3)$ = 140 mm
Number of strands $n_s(3)$ = 5
Height above soffit $d_s(4)$ = 650 mm
Number of strands $n_s(4)$ = 2

**Short Term Losses**

Relaxation at transfer ( % ) $s_{tr}$ = 1

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force $kN$</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.32</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>264.08</td>
<td>8.4316</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>295.4</td>
<td>9.4316</td>
</tr>
</tbody>
</table>

**Losses at Transfer**

Initial force $3132$ kN
Force at transfer $P_t = P_0 - loft = 2836.6$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.1793</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.2697</td>
</tr>
<tr>
<td>Total</td>
<td>2.0904</td>
</tr>
</tbody>
</table>

**Stress Limitations**

Top of beam:
Compressive stress 2.0904 N/mm² is less than allowable stresses of 0.5*f_{ci}=20 N/mm² and 0.4*f_{cu}=20 N/mm².
Bottom of beam:
Compressive stress 15.744 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

**Long Term Losses**

Relaxation after transfer( % ) slr(1)=1.5
Shrinkage per unit length sul(1)=0.3E-3

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>31.32</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>264.08</td>
<td>8.4316</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>295.4</td>
<td>9.4316</td>
</tr>
<tr>
<td>Relaxation</td>
<td>46.98</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>150.66</td>
<td>4.8103</td>
</tr>
<tr>
<td>Creep</td>
<td>346.24</td>
<td>11.055</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>543.88</td>
<td>17.365</td>
</tr>
<tr>
<td>All Losses</td>
<td>839.28</td>
<td>26.797</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 3132 kN
Final force Pf=P0-loftl=2292.7 kN

Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-2.5697</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.2697</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>7.6895</td>
</tr>
<tr>
<td>Total</td>
<td>10.389</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Height from beam soffit to bottom of in-situ concrete hsis=135 mm

**Combination 1 loading**

Applied bending moment c1b(1)=320 kNm
### Combination 1 Loading - Stresses

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 1</td>
<td>3.6879</td>
</tr>
<tr>
<td>Total</td>
<td>3.6879</td>
</tr>
</tbody>
</table>

### Combination 2 to 5 Loading

**Applied bending moment**  
c2b(1)=400 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>4.0976</td>
</tr>
<tr>
<td>Total</td>
<td>4.0976</td>
</tr>
</tbody>
</table>

### Combination 3 loading

- Top of in-situ concrete: Compressive stress 4.0976 N/mm² is less than the maximum allowable stress of 0.5*fcuc=20 N/mm².  
- Top of beam: Compressive stress 12.848 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².  
- Bottom of in-situ concrete: Tensile stress 2.8332 N/mm² is less than the maximum allowable tensile stress of 4.4 N/mm² from Table 32.  
- Bottom of beam: Compressive stress 3.0691 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².
Location: 13.5 metre span TY7 edge beam with solid infill

Precast prestressed concrete TY beams with in-fill concrete

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam                  span=13.5 m

Beam TY 7 section properties

Depth of beam                 d=700 mm
Cross-sectional area         Ac=272194 mm²
Centroid above soffit        Yb=272.9 mm
Section modulus top          Zt=29.42E6 mm³
Section modulus bottom       Zb=46.04E6 mm³
Unit weight of beam          uwb=25 kN/m³

Concrete strengths

Strength at transfer         fci=40 N/mm²
Characteristic strength      fcu=50 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 40 N/mm² is:
Ec = 31 kN/mm²

Modulus of elasticity for concrete strength 50 N/mm² is:
Ec = 34 kN/mm²

Composite section properties

Char strength of in-situ concrete fcuc=30 N/mm²
Modulus of elasticity for concrete strength 30 N/mm² is:
Ec = 28 kN/mm²

Item weighting              iw1=0.82353
X co-ordinate               X(1)=0 mm
Y co-ordinate               Y(1)=0 mm
X co-ordinate               X(2)=765 mm
Y co-ordinate               Y(2)=0 mm
X co-ordinate               X(3)=765 mm
Y co-ordinate               Y(3)=250 mm
X co-ordinate               X(4)=915 mm
Y co-ordinate               Y(4)=350 mm
X co-ordinate               X(5)=915 mm
Y co-ordinate               Y(5)=875 mm
X co-ordinate               X(6)=415 mm
Y co-ordinate               Y(6)=875 mm
X co-ordinate               X(7)=415 mm
Y co-ordinate               Y(7)=775 mm
X co-ordinate               X(8)=0 mm
Y co-ordinate               Y(8)=775 mm
X co-ordinate               X(9)=0 mm
Y co-ordinate               Y(9)=0 mm
Second moment of area \( I_{XX2} = 12.563 \times 10^9 \text{ mm}^4 \)

Area of item \( A_2 = 272247 \text{ mm}^2 \)

Centroid from X-X axis \( Y_{C2} = 272.94 \text{ mm} \)

Item weighting \( i\omega = 0.17647 \)

Depth of composite section \( d_i = 775 \text{ mm} \)

Nominal diameter \( n_\text{od} = 12 \text{ mm} \)

Nominal tensile strength \( n_\text{omt} = 1700 \text{ N/mm}^2 \)

Nominal steel area \( n_\text{oma} = 100 \text{ mm}^2 \)

Characteristic breaking load \( c\text{bl} = 150 \text{ kN} \)

Initial force per strand \( i\text{f} = 100 \text{ kN} \)

Modulus of elasticity \( E_s = 200 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( c_{sd}(1) = 2 \text{ m} \)

Number of layers of strand \( n_{ls}(1) = 4 \)

Height above soffit \( d_{s}(1) = 60 \text{ mm} \)

Number of strands \( n_{s}(1) = 11 \)

Height above soffit \( d_{s}(2) = 100 \text{ mm} \)

Number of strands \( n_{s}(2) = 13 \)

Height above soffit \( d_{s}(3) = 140 \text{ mm} \)

Number of strands \( n_{s}(3) = 8 \)

Height above soffit \( d_{s}(4) = 650 \text{ mm} \)

Number of strands \( n_{s}(4) = 2 \)

Bending moment from temp. loads \( t_{lb}(1) = 80 \text{ kN} \)

Bending moment from other loads \( o_{lb}(1) = 50 \text{ kN} \)

**Short Term Losses**

Relaxation at transfer ( % ) \( s_{tr} = 1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>34</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>393.24</td>
<td>11.566</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>427.24</td>
<td>12.566</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \( 3400 \text{ kN} \)

Force at transfer \( P_t = P_0 - lof_t = 2972.8 \text{ kN} \)
Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 5.5883 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Bottom of beam:
Compressive stress 14.329 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

**Long Term Losses**

Relaxation after transfer( % )  slr(1)=1.5
Shrinkage per unit length  sul(1)=0.3E-3

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>34</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>393.24</td>
<td>11.566</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>427.24</td>
<td>12.566</td>
</tr>
<tr>
<td>Relaxation</td>
<td>51</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>204</td>
<td>6</td>
</tr>
<tr>
<td>Creep</td>
<td>490.77</td>
<td>14.434</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>745.77</td>
<td>21.934</td>
</tr>
<tr>
<td>All Losses</td>
<td>1173</td>
<td>34.5</td>
</tr>
</tbody>
</table>

**Losses - Long term**

Initial force  3400 kN
Final force   Pf=P0-loftl=2227 kN

Unit-weight of in-situ concrete  wc=25 kN/m³
<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td><strong>Prestress</strong></td>
<td>-2.7253</td>
</tr>
<tr>
<td><strong>Self weight of Beam</strong></td>
<td>4.8071</td>
</tr>
<tr>
<td><strong>In-situ concrete</strong></td>
<td>7.8038</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>9.8856</td>
</tr>
</tbody>
</table>

**Beam concrete stresses - Long term**

**Composite section**

- Height from beam soffit to bottom of in-situ concrete: \( h_{sis} = 135 \text{ mm} \)
- Percentage of shrinkage remaining: \( p_s = 40 \% \)
- Percentage: \( p_{cr} = 40 \% \)
- Percentage: \( stT = 50 \% \)

Apply moment to beam to straighten it as follows:

The net moment is: \( M_c = F \cdot e / 1E3 - M_b = -145.59 \text{ kNm} \)

Stress at top of in-situ concrete:
\[
ft_{isds} = \frac{F}{(A_i \cdot mr)} - \frac{F}{(A_i \cdot mr + Ac)} - M_c \cdot 1E3 \cdot (d_i - Y_c) / I_c \cdot mr \cdot 1E3 \\
= 0.10329 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
fb_{isds} = \frac{F}{(A_i \cdot mr)} - \frac{F}{(A_i \cdot mr + Ac)} - M_c \cdot 1E3 \cdot (hsis - Y_c) / I_c \cdot mr \cdot 1E3 \\
= 2.1815 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:
\[
ft_{pbds} = \frac{F}{(A_i \cdot mr + Ac)} + M_c \cdot 1E3 \cdot (d - Y_c) / I_c + M_b \cdot 1E3 \cdot Y_t / I_{xx} \cdot 1E3 \\
= -3.0552 \text{ N/mm}^2
\]

Stress at bottom of precast concrete beam:
\[
fb_{pbds} = \frac{F}{(A_i \cdot mr + Ac)} - M_c \cdot 1E3 \cdot Y_c / I_c - M_b \cdot 1E3 \cdot Y_b / I_{xx} \cdot 1E3 = -0.42523 \text{ N/mm}^2
\]

**Combination 1 loading**

Applied bending moment \( c_{lb(1)} = 550 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td><strong>Prestress, beam and in-situ</strong></td>
<td>0</td>
</tr>
<tr>
<td><strong>Dif. shrinkage Combination 1</strong></td>
<td>0.10329</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>4.8417</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses
Combination 2 to 5 loading

Applied bending moment: c2b(1)=625 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.10329</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>4.9358</td>
</tr>
<tr>
<td>Total</td>
<td>5.0391</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 5.0391 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².

Top of beam:
Compressive stress 11.554 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².

Bottom of in-situ concrete:
Tensile stress 1.8043 N/mm² is less than the maximum allowable tensile stress of 3.6 N/mm² from Table 32.

Bottom of beam:
Tensile stress 0.45734 N/mm² is less than the maximum allowable tensile stress of 3.182 N/mm².
Location: 15.0 metre span TY9 beam

Precast prestressed concrete TY beams with in-fill concrete

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=15 m

Beam TY 9 section properties

Depth of beam d=800 mm
Cross-sectional area Ac=308163 mm²
Centroid above soffit Yb=328.6 mm
Section modulus top Zt=42.11E6 mm³
Section modulus bottom Zb=60.4E6 mm³
Unit weight of beam uwb=25 kN/m³

Concrete strengths

Strength at transfer fci=35 N/mm²
Characteristic strength fcu=40 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 35 N/mm² is:
Eci=29.5 kN/mm²
Modulus of elasticity for concrete strength 40 N/mm² is:
Ec=31 kN/mm²

Composite section properties

Char strength of in-situ concrete fcuc=35 N/mm²
Modulus of elasticity for concrete strength 35 N/mm² is:
Ecc=29.5 kN/mm²

TY 9 beam

In-situ concrete

Centroids are related to the soffit of section. Precast beam shown dotted.

Breadth of rectangular section b=765 mm
Depth of rectangular section di=875 mm
Second moment of area Ic=Ixx+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi-Yc)^2)=42.708E9 mm⁴
Initial force per strand \( if = 130 \, \text{kN} \)
Modulus of elasticity \( E_s = 200 \, \text{kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( csd(1) = 6 \, \text{m} \)
Number of layers of strand \( nls(1) = 6 \)
Height above soffit \( ds(1) = 60 \, \text{mm} \)
Number of strands \( ns(1) = 9 \)
Height above soffit \( ds(2) = 100 \, \text{mm} \)
Number of strands \( ns(2) = 9 \)
Height above soffit \( ds(3) = 140 \, \text{mm} \)
Number of strands \( ns(3) = 5 \)
Height above soffit \( ds(4) = 650 \, \text{mm} \)
Number of strands \( ns(4) = 2 \)
Height above soffit \( ds(5) = 700 \, \text{mm} \)
Number of strands \( ns(5) = 2 \)
Height above soffit \( ds(6) = 750 \, \text{mm} \)
Number of strands \( ns(6) = 2 \)

**Short Term Losses**

Relaxation at transfer ( % ) \( str = 1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>37.7</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>282.84</td>
<td>7.5025</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>320.54</td>
<td>8.5025</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \( 3770 \, \text{kN} \)
Force at transfer \( Pt = P_0 - loft = 3449.5 \, \text{kN} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>2.1832</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>1.8527</td>
</tr>
<tr>
<td>Total</td>
<td>4.0358</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer
**Stress Limitations**

Top of beam:
Compressive stress 4.0358 N/mm² is less than allowable stresses of 0.5*fci=17.5 N/mm² and 0.4*fcu=16 N/mm².

Bottom of beam:

**WARNING:**
Compressive stress 16.183 N/mm² is less than allowable stress of 0.5*fci=17.5 N/mm² but exceeds 0.4*fcu=16 N/mm².

**Long Term Losses**

Relaxation after transfer (%) slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>37.7</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>282.84</td>
<td>7.5025</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>320.54</td>
<td>8.5025</td>
</tr>
<tr>
<td>Relaxation</td>
<td>75.4</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>174</td>
<td>4.6154</td>
</tr>
<tr>
<td>Creep</td>
<td>488.5</td>
<td>12.958</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>737.9</td>
<td>19.573</td>
</tr>
<tr>
<td>All Losses</td>
<td>1058.4</td>
<td>28.075</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 3770 kN
Final force Pf=P0-loftl=2711.6 kN

Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>1.7161</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>1.8527</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.1716</td>
</tr>
<tr>
<td>Total</td>
<td>5.7404</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term
Height from beam soffit to bottom of in-situ concrete hsis=135 mm

**Combination 1 loading**

Applied bending moment \( c_{1b(1)}=200 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Combination 1</td>
<td>2.3049</td>
</tr>
<tr>
<td>Total</td>
<td>2.3049</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

**Combination 2 to 5 loading**

Applied bending moment \( c_{2b(1)}=250 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Combination 2-5</td>
<td>2.561</td>
</tr>
<tr>
<td>Total</td>
<td>2.561</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 2.561 N/mm² is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm².
Top of beam:
Compressive stress 7.8624 N/mm² is less than the maximum allowable stress of 0.4*fcu=16 N/mm².
Bottom of in-situ concrete:
Tensile stress 1.7708 N/mm² is less than the maximum allowable tensile stress of 4 N/mm² from Table 32.
Bottom of beam:
Compressive stress 8.3702 N/mm² is less than the maximum allowable stress of 0.4*fcu=16 N/mm².
Precast prestressed concrete TY beams with in-fill concrete


Span of beam span=13.5 m

Beam TY 7 section properties

- Depth of beam d=700 mm
- Cross-sectional area Ac=272194 mm²
- Centroid above soffit Yb=272.9 mm
- Section modulus top Zt=29.42E6 mm⁴
- Section modulus bottom Zb=46.04E6 mm⁴
- Unit weight of beam uwb=25 kN/m³

Concrete compressive strengths

- Characteristic cylinder strength fck=40 N/mm²
- Characteristic cylinder strength fcki=32 N/mm² (transfer)

Concrete moduli (short-term)

- Concrete modulus of elasticity Ecmb=35 kN/mm²
- Concrete modulus of elasticity Ecmi=33 kN/mm² (transfer)

Composite section properties

- Char strength (in-situ concrete) fckc=32 N/mm²
- Short-term modulus of elasticity for in-situ concrete Ecmc=33 kN/mm²

Breadth of rectangular section b=765 mm
Depth of rectangular section di=875 mm
Second moment of area \[ Ic=I_{yy}+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi-Yc)^2)=42.708E9 \text{ mm}^4 \]
Modulus of elasticity (strand) Ep=195 kN/mm²
**Cross-section 1**

Distance from mid-span of beam \( c_{sd}(1) = 0 \) m  
Number of layers of strand \( n_{ls}(1) = 4 \)  
Height above soffit \( d_s(1) = 60 \) mm  
Number of strands \( n_s(1) = 11 \)  
Height above soffit \( d_s(2) = 100 \) mm  
Number of strands \( n_s(2) = 9 \)  
Height above soffit \( d_s(3) = 140 \) mm  
Number of strands \( n_s(3) = 5 \)  
Height above soffit \( d_s(4) = 650 \) mm  
Number of strands \( n_s(4) = 2 \)  

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer ( % ) \( str = 1 \)  

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.32</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>224.37</td>
<td>7.1638</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>255.69</td>
<td>8.1638</td>
</tr>
</tbody>
</table>

**Losses at Transfer**

Initial force 3132 kN  
Force at transfer \( Pt = P_0 - loft = 2876.3 \) kN  

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-3.2238</td>
<td>19.379</td>
<td>-3.3671</td>
<td>-1.7403</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.2697</td>
<td>-3.3671</td>
<td>-1.7403</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>2.0459</td>
<td>16.012</td>
<td>13.381</td>
<td></td>
</tr>
</tbody>
</table>

**Beam Concrete Stresses at Transfer**

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:  
Compressive stress 2.0459 N/mm² is less than allowable stresses of 0.7\( f_{ck}\) = 22.4 N/mm² and 0.6\( f_{ck}\) = 24 N/mm².
Bottom of beam:
Compressive stress 16.012 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

Long Term Losses ( BS EN 1992-1-1, Clause 5.10.6(1) )

Relaxation after transfer( % ) slr(1)=1.5
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(ω, to) crf=2

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.32</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>224.37</td>
<td>7.1638</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>255.69</td>
<td>8.1638</td>
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<tr>
<td>Relaxation</td>
<td>46.98</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>146.89</td>
<td>4.6901</td>
</tr>
<tr>
<td>Creep</td>
<td>403.79</td>
<td>12.892</td>
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<tr>
<td>Long-term Total</td>
<td>597.66</td>
<td>19.082</td>
</tr>
<tr>
<td>All Losses</td>
<td>853.35</td>
<td>27.246</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 3132 kN
Final force Pf=P0- loftl=2278.6 kN

Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-2.5539</td>
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<tr>
<td>Self weight of Beam</td>
<td>5.2697</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>7.6895</td>
</tr>
<tr>
<td>Total</td>
<td>10.405</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Height from beam soffit to bottom of in-situ concrete hsis=135 mm
Loading Combination 1

No tensile stress permitted in precast beam.

Characteristic bending moment \( c_{1b}(1) = 320 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 1</td>
<td>3.6879</td>
</tr>
<tr>
<td>Total</td>
<td>3.6879</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses

Loading Combination 2

Tensile stress permitted (no visible cracking).

Characteristic bending moment \( c_{2b}(1) = 400 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 2</td>
<td>4.0976</td>
</tr>
<tr>
<td>Total</td>
<td>4.0976</td>
</tr>
</tbody>
</table>

Loading Combination 2 - Stresses

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 4.0976 N/mm² is less than the maximum allowable stress of 0.6*fckc=19.2 N/mm².

Top of beam:
Compressive stress 12.864 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

Bottom of in-situ concrete:
Tensile stress 2.8332 N/mm² is less than the maximum allowable tensile stress of 3.0231 N/mm² (EN 1992-1-1 expression 3.23).

Bottom of beam:
Compressive stress 2.9742 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².
Location: Ex2 - 13.5 metre span TY7 edge beam with solid infill

Precast prestressed concrete TY beams with in-fill concrete


Span of beam span=13.5 m

Beam TY 7 section properties

Depth of beam d=700 mm
Cross-sectional area Ac=272194 mm²
Centroid above soffit Yb=272.9 mm
Section modulus top Zt=29.42E6 mm³
Section modulus bottom Zb=46.04E6 mm³
Unit weight of beam uwb=25 kN/m³

Concrete compressive strengths

Characteristic cylinder strength fcki=32 N/mm² (transfer)
Characteristic cylinder strength fck=40 N/mm²

Concrete moduli (short-term)

Concrete modulus of elasticity Ecmi=33 kN/mm² (transfer)
Concrete modulus of elasticity Ecmb=35 kN/mm²

Composite section properties

Char strength (in-situ concrete) fckc=24 N/mm²
Short-term modulus of elasticity for in-situ concrete Ecmc=31 kN/mm²
Item weighting iw1=0.88571
X co-ordinate X(1)=0 mm
Y co-ordinate Y(1)=0 mm
X co-ordinate X(2)=765 mm
Y co-ordinate Y(2)=0 mm
X co-ordinate X(3)=765 mm
Y co-ordinate Y(3)=250 mm
X co-ordinate X(4)=915 mm
Y co-ordinate Y(4)=350 mm
X co-ordinate X(5)=915 mm
Y co-ordinate Y(5)=875 mm
X co-ordinate X(6)=415 mm
Y co-ordinate Y(6)=875 mm
X co-ordinate X(7)=415 mm
Y co-ordinate Y(7)=775 mm
X co-ordinate X(8)=0 mm
Y co-ordinate Y(8)=775 mm
X co-ordinate X(9)=0 mm
Y co-ordinate Y(9)=0 mm
Second moment of area \( I_{XX2} = 12.563E9 \, \text{mm}^4 \)

Area of item \( A_2 = 272247 \, \text{mm}^2 \)

Centroid from X-X axis \( Y_{C2} = 272.94 \, \text{mm} \)

Item weighting \( i_{W2} = 0.17647 \)

Depth of composite section \( d_i = 775 \, \text{mm} \)

Nominal diameter \( d_{nom} = 12 \, \text{mm} \)

Nominal tensile strength \( d_{nom} = 1700 \, \text{N/mm}^2 \)

Nominal steel area \( n_{oma} = 100 \, \text{mm}^2 \)

Char breaking load (strand) \( f_{pk} = 150 \, \text{kN} \)

Initial force per strand \( i_{pf} = 100 \, \text{kN} \)

Modulus of elasticity (strand) \( E_p = 195 \, \text{kN/mm}^2 \)

Cross-section 1

Distance from mid-span of beam \( c_{sd(1)} = 2 \, \text{m} \)

Number of layers of strand \( n_{ls(1)} = 4 \)

Height above soffit \( d_{s(1)} = 60 \, \text{mm} \)

Number of strands \( n_s(1) = 11 \)

Height above soffit \( d_{s(2)} = 100 \, \text{mm} \)

Number of strands \( n_s(2) = 13 \)

Height above soffit \( d_{s(3)} = 140 \, \text{mm} \)

Number of strands \( n_s(3) = 8 \)

Height above soffit \( d_{s(4)} = 650 \, \text{mm} \)

Number of strands \( n_s(4) = 2 \)

Bending moment from temp. loads \( t_{lb(1)} = 80 \, \text{kNm} \)

Bending moment from other loads \( o_{lb(1)} = 50 \, \text{kNm} \)

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % ) \( s_{t} = 1 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>34</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>325.36</td>
<td>9.5694</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>359.36</td>
<td>10.569</td>
</tr>
</tbody>
</table>

Initial force \( 3400 \, \text{kN} \)

Force at transfer \( P_t = P_0 - I_{o\text{f}t} = 3040.6 \, \text{kN} \)
Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
Compressive stress 5.5052 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².
Bottom of beam:
Compressive stress 14.791 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer ( % )  slr(1)=1.5
Shrinkage per unit length  sul(1)=0.3E-3
Creep coefficient φ(•,to)  crf=2

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>34</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>325.36</td>
<td>9.5694</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>359.36</td>
<td>10.569</td>
</tr>
<tr>
<td>Relaxation</td>
<td>51</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>198.9</td>
<td>5.85</td>
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<tr>
<td>Creep</td>
<td>586.03</td>
<td>17.236</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>835.93</td>
<td>24.586</td>
</tr>
<tr>
<td>All Losses</td>
<td>1195.3</td>
<td>35.156</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force  3400 kN
Final force  Pf=P0-loftl=2204.7 kN
Unit-weight of in-situ concrete  wc=25 kN/m³
**Composite section**

Height from beam soffit to bottom of in-situ concrete \( hs_{is} = 135 \text{ mm} \)

Percentage of shrinkage remaining \( ps = 40\% \)

Percentage \( pcr = 40\% \)

Percentage \( stT = 50\% \)

The net moment is \( Mc = F*e/1E3 - Mb = -206.13 \text{ kNm} \)

Stress at top of in-situ concrete:
\[ ft_{isds} = -(F/(Ai*mr) - F/(Ai*mr+Ac) - Mc*1E3*(d-i)/Ic)*mr*1E3 \]
\[ = 0.061653 \text{ N/mm}^2 \]

Stress at bottom of in-situ concrete:
\[ fb_{isds} = -(F/(Ai*mr) - F/(Ai*mr+Ac) - Mc*1E3*(hs_{is}-Yc)/Ic)*mr*1E3 \]
\[ = 3.0225 \text{ N/mm}^2 \]

Stress at top of precast concrete beam:
\[ ftp_{pbs} = (F/(Ai*mr+Ac) + Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)*1E3 \]
\[ = -4.0271 \text{ N/mm}^2 \]

Stress at bottom of precast concrete beam:
\[ fb_{pbs} = (F/(Ai*mr+Ac) - Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)*1E3 \]
\[ = -0.66568 \text{ N/mm}^2 \]

**Loading Combination 1**

No tensile stress permitted in precast beam.

Characteristic bending moment \( cl_b(1) = 550 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>0.061653</td>
</tr>
<tr>
<td>Total</td>
<td>4.8191</td>
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</tbody>
</table>
### Loading Combination 2

Tensile stress permitted (no visible cracking).

Characteristic bending moment \( c_{2b(1)} = 625 \, \text{kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2</th>
<th>Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>9.9128</td>
</tr>
<tr>
<td>Diff. shrinkage</td>
<td>0.061653</td>
<td>-4.0271</td>
</tr>
<tr>
<td>Combination 2</td>
<td>4.9557</td>
<td>4.4074</td>
</tr>
<tr>
<td></td>
<td>5.0174</td>
<td>10.293</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bottom of in-situ</td>
<td>Bottom of Beam</td>
</tr>
<tr>
<td>Prestress, beam</td>
<td>0</td>
<td>6.9413</td>
</tr>
<tr>
<td>Diff. shrinkage</td>
<td>3.0225</td>
<td>-0.66568</td>
</tr>
<tr>
<td>Combination 2</td>
<td>4.0218</td>
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</tr>
<tr>
<td></td>
<td>-0.99928</td>
<td>-0.40314</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTE:** The stresses from temperature differences should be added to the total stresses.

**Top of in-situ concrete:**
Compressive stress 5.0174 N/mm² is less than the maximum allowable stress of 0.6*fckc=14.4 N/mm².

**Top of beam:**
Compressive stress 10.293 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

**Bottom of in-situ concrete:**
Tensile stress 0.99928 N/mm² is less than the maximum allowable tensile stress of 2.4956 N/mm² (EN 1992-1-1 expression 3.23).

**Bottom of beam:**
Tensile stress 0.40314 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
Location: Ex3 - 15.0 metre span TY9 beam

Precast prestressed concrete TY beams with in-fill concrete


Span of beam span=15 m

Beam TY 9 section properties

Depth of beam d=800 mm
Cross-sectional area Ac=308163 mm²
Centroid above soffit Yb=328.6 mm
Section modulus top Zt=42.11E6 mm³
Section modulus bottom Zb=60.4E6 mm³
Unit weight of beam uwb=25 kN/m³

Concrete compressive strengths

Characteristic cylinder strength fcki=28 N/mm² (transfer)
Characteristic cylinder strength fck=32 N/mm²

Concrete moduli (short-term)

Concrete modulus of elasticity Ecmi=32 kN/mm² (transfer)
Concrete modulus of elasticity Ecmb=33 kN/mm²

Composite section properties

Char strength (in-situ concrete) fckc=28 N/mm²
Short-term modulus of elasticity for in-situ concrete Ecmc=32 kN/mm²

Ty 9 beam

In-situ concrete

Centroids are related to the soffit of section.

Precast beam shown dotted

Breadth of rectangular section b=765 mm
Depth of rectangular section di=875 mm
Second moment of area Ic=Iyy+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi-Yc)^2)=42.708E9 mm⁴
Modulus of elasticity (strand) Ep=195 kN/mm²
Cross-section 1

Distance from mid-span of beam  \( csd(1)=6 \text{ m} \)
Number of layers of strand  \( nls(1)=6 \)
Height above soffit  \( ds(1)=60 \text{ mm} \)
Number of strands  \( ns(1)=9 \)
Height above soffit  \( ds(2)=100 \text{ mm} \)
Number of strands  \( ns(2)=9 \)
Height above soffit  \( ds(3)=140 \text{ mm} \)
Number of strands  \( ns(3)=5 \)
Height above soffit  \( ds(4)=650 \text{ mm} \)
Number of strands  \( ns(4)=2 \)
Height above soffit  \( ds(5)=700 \text{ mm} \)
Number of strands  \( ns(5)=2 \)
Height above soffit  \( ds(6)=750 \text{ mm} \)
Number of strands  \( ns(6)=2 \)

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % )  \( \text{str}=1 \)

\[
\begin{array}{|c|c|c|}
\hline
\text{Loss} & \text{Loss of Force} & \% \text{Loss} \\
& \text{kN} & \\
\hline
\text{Relaxation} & 37.7 & 1 \\
\text{Elastic Shortening} & 238.01 & 6.3134 \\
\text{Transfer Total} & 275.71 & 7.3134 \\
\hline
\end{array}
\]

Losses at Transfer

Initial force  \( 3770 \text{ kN} \)
Force at transfer  \( Pt=P0-\text{loft}=3494.3 \text{ kN} \)

\[
\begin{array}{|c|c|c|c|}
\hline
\text{Loading} & \text{Stresses at Transfer N/mm}^2 & \\
& \text{Top of Beam} & \text{Bottom of Beam} & \text{Centroid of Strands} \\
\hline
\text{Prestress} & 2.2115 & 17.702 & 13.469 \\
\text{Self weight of Beam} & 1.8527 & -1.2915 & -0.43224 \\
\hline
\text{Total} & 4.0642 & 16.41 & 13.036 \\
\hline
\end{array}
\]

Beam Concrete Stresses at Transfer

\textbf{Stress Limitations ( BS EN 1992-1-1, Clause 5.10.2.2 )}

Top of beam:
Compressive stress 4.0642 N/mm\(^2\) is less than allowable stresses of \(0.7*\text{fckl}=19.6 \text{ N/mm}^2\) and \(0.6*\text{fck}=19.2 \text{ N/mm}^2\).
Bottom of beam:
Compressive stress 16.41 N/mm² is less than allowable stresses of 0.7*fcki=19.6 N/mm² and 0.6*fck=19.2 N/mm².

**Long Term Losses ( BS EN 1992-1-1, Clause 5.10.6(1) )**

Relaxation after transfer( % ) slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(,to) crf=2

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>37.7</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>238.01</td>
<td>6.3134</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>275.71</td>
<td>7.3134</td>
</tr>
<tr>
<td>Relaxation</td>
<td>75.4</td>
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</tr>
<tr>
<td>Shrinkage</td>
<td>169.65</td>
<td>4.5</td>
</tr>
<tr>
<td>Creep</td>
<td>547.96</td>
<td>14.535</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>793.01</td>
<td>21.035</td>
</tr>
<tr>
<td>All Losses</td>
<td>1068.7</td>
<td>28.348</td>
</tr>
</tbody>
</table>

Losses - Long term
Initial force 3770 kN
Final force Pf=P0-loftl=2701.3 kN
Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>1.7096</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>1.8527</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.1716</td>
</tr>
<tr>
<td>Total</td>
<td>5.7339</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term
Height from beam soffit to bottom of in-situ concrete hsis=135 mm
Loading Combination 1

No tensile stress permitted in precast beam.

Characteristic bending moment \( c_{1b}(1) = 200 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 1</td>
<td>2.3049</td>
</tr>
<tr>
<td>Total</td>
<td>2.3049</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses

Loading Combination 2

Tensile stress permitted (no visible cracking).

Characteristic bending moment \( c_{2b}(1) = 250 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 2</td>
<td>2.561</td>
</tr>
<tr>
<td>Total</td>
<td>2.561</td>
</tr>
</tbody>
</table>

Loading Combination 2 - Stresses

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 2.561 N/mm² is less than the maximum allowable stress of \( 0.6 \times f_{ckc} = 16.8 \text{ N/mm}² \).

Top of beam:
Compressive stress 7.8559 N/mm² is less than the maximum allowable stress of \( 0.6 \times f_{ck} = 19.2 \text{ N/mm}² \).

Bottom of in-situ concrete:
Tensile stress 1.7708 N/mm² is less than the maximum allowable tensile stress of 2.7656 N/mm² (EN 1992-1-1 expression 3.23).

Bottom of beam:
Compressive stress 8.3181 N/mm² is less than the maximum allowable stress of \( 0.6 \times f_{ck} = 19.2 \text{ N/mm}² \).
Location: 16 metre span TY10 beam with in-situ slab

Precast prestressed concrete TY beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam \(\text{span}=16\,\text{m}\)

**Beam TY 10 section properties**

- Depth of beam \(d=850\,\text{mm}\)
- Cross-sectional area \(A_c=323206\,\text{mm}^2\)
- Centroid above soffit \(Y_b=351.52\,\text{mm}\)
- Section modulus top \(Z_t=46.9E6\,\text{mm}^3\)
- Section modulus bottom \(Z_b=66.47E6\,\text{mm}^3\)
- Unit weight of beam \(u_{wb}=24\,\text{kN/m}^3\)

**Concrete strengths**

- Strength at transfer \(f_{ci}=40\,\text{N/mm}^2\)
- Characteristic strength \(f_{cu}=50\,\text{N/mm}^2\)

**Concrete moduli**

- Modulus of elasticity at transfer for concrete strength 40 N/mm\(^2\) is: \(E_{ci}=31\,\text{kN/mm}^2\)
- Modulus of elasticity for concrete strength 50 N/mm\(^2\) is: \(E_{cb}=34\,\text{kN/mm}^2\)

**Composite section properties**

- Char strength of in-situ concrete \(f_{cuc}=40\,\text{N/mm}^2\)
- Modulus of elasticity for concrete strength 40 N/mm\(^2\) is: \(E_{cc}=31\,\text{kN/mm}^2\)
Breadth of slab $b=1000$ mm
Depth of slab $d_{is}=160$ mm
Depth of permanent formwork $d_{pf}=20$ mm

Second moment of area
$A_c=I_{x}+A_c \cdot (Y_b-Y_c)^2+mr \cdot (I_{i}+A_i \cdot (Y_i-Y_c)^2)=55.066E9$ mm$^4$

Depth of composite section $d_i=980$ mm
Initial force $I_f=123$ kN
Modulus of elasticity $E_s=200$ kN/mm$^2$

Cross-section 1
Distance from mid-span of beam $c_{sd}(1)=0$ m
Number of layers of strand $n_{ls}(1)=4$
Height above soffit $d_s(1)=60$ mm
Number of strands $n_s(1)=11$
Height above soffit $d_s(2)=100$ mm
Number of strands $n_s(2)=13$
Height above soffit $d_s(3)=140$ mm
Number of strands $n_s(3)=7$
Height above soffit $d_s(4)=800$ mm
Number of strands $n_s(4)=2$

Short Term Losses
Relaxation at transfer ( % ) $s_{tr}=1$

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.59</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>402.04</td>
<td>9.9048</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>442.63</td>
<td>10.905</td>
</tr>
</tbody>
</table>

Losses at Transfer
Initial force $4059$ kN
Force at transfer $P_t=P_0-lo_{ft}=3616.4$ kN
Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
The flexural tensile stress of 0.021518 N/mm² at transfer is less than the permissible value of 1 N/mm² (Clause 6.3.2.4(b)(1)).

Bottom of beam:
Compressive stress 19.095 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

**Long Term Losses**

Relaxation after transfer( % )  slr(1)=2
Shrinkage per unit length  sul(1)=0.3E-3

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.59</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>402.04</td>
<td>9.9048</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>442.63</td>
<td>10.905</td>
</tr>
<tr>
<td>Relaxation</td>
<td>81.18</td>
<td>2</td>
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<tr>
<td>Shrinkage</td>
<td>184.14</td>
<td>4.5366</td>
</tr>
<tr>
<td>Creep</td>
<td>573.26</td>
<td>14.123</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>838.58</td>
<td>20.66</td>
</tr>
<tr>
<td>All Losses</td>
<td>1281.2</td>
<td>31.565</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 4059 kN
Final force Pf=P0-loftl=2777.8 kN

Unit-weight of in-situ concrete wc=24 kN/m³
<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.0841</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.2956</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.4741</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>3.6855</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

**Composite section**

Height from beam soffit to bottom of in-situ concrete: $h_{sis}=820$ mm

Percentage of shrinkage remaining: $ps=25\%$

Percentage: $pcr=25$

Percentage: $stT=67\%$

The net moment is: $M_c=F_e/E-9145.46$ kNm

Stress at top of in-situ concrete:

$$f_{t,ds}=-(F/(A_i*mr)-F/(A_i*mr+Ac)-Mc*1E3*(d_i-Yc)/Ic)*mr*1E3$$

Stress at bottom of in-situ concrete:

$$f_{b,ds}=-(F/(A_i*mr)-F/(A_i*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3$$

Stress at top of precast concrete beam:

$$f_{tp,ds}=(F/(A_i*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Y_b/Ixx)*1E3$$

Stress at bottom of precast concrete beam:

$$f_{bp,ds}=(F/(A_i*mr+Ac)-Mc*1E3*Y_c/Ic-Mb*1E3*Y_b/Ixx)*1E3$$

**Combination 1 loading**

Applied bending moment: $c_{lb}(1)=900$ kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage</td>
<td>-0.94879</td>
</tr>
<tr>
<td>Combination 1</td>
<td>9.0411</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>8.0923</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses
Combination 2 to 5 loading

Applied bending moment \( c2b(1)=1200 \ \text{kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Dif. shrinkage Combination 2-5</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>-0.94879</td>
</tr>
<tr>
<td>Total</td>
<td>8.9143</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 8.9143 N/mm\(^2\) is less than the maximum allowable stress of 0.5*fcuc=20 N/mm\(^2\).

Top of beam:
Compressive stress 12.412 N/mm\(^2\) is less than the maximum allowable stress of 0.4*fcu=20 N/mm\(^2\).

Bottom of in-situ concrete:
Compressive stress 5.8502 N/mm\(^2\) is less than the maximum allowable stress of 0.5*fcuc=20 N/mm\(^2\).

Bottom of beam:
Tensile stress 0.037895 N/mm\(^2\) is less than the maximum allowable tensile stress of 3.182 N/mm\(^2\).
Location: 12.5 metre span TY6 beam with in-situ top slab

Precast prestressed concrete TY beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam: span=12.5 m

Beam TY 6 section properties

- Depth of beam: d=650 mm
- Cross-sectional area: Ac=251341 mm²
- Centroid above soffit: Yb=240.07 mm
- Section modulus top: Zt=22.13E6 mm⁴
- Section modulus bottom: Zb=37.75E6 mm⁴
- Unit weight of beam: uwb=24.5 kN/m³

Concrete strengths

- Strength at transfer: fci=40 N/mm²
- Characteristic strength: fcu=52.5 N/mm²

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 40 N/mm² is: Eci=31 kN/mm²
- Modulus of elasticity for concrete strength 52.5 N/mm² is: Ecb=34.5 kN/mm²

Composite section properties

- Char strength of in-situ concrete fcuc=40 N/mm²
- Modulus of elasticity for concrete strength 40 N/mm² is: Ecc=31 kN/mm²

Item weighting

- iw1=0.89855

X co-ordinate

- X(1)=125 mm
- Y(1)=0 mm
- X(2)=875 mm
- Y(2)=0 mm
- X(3)=875 mm
- Y(3)=135 mm
- X(4)=1007 mm
- Y(4)=400 mm
- X(5)=1207 mm
- Y(5)=400 mm
- X(6)=1207 mm
- Y(6)=900 mm
- X(7)=707 mm
- Y(7)=880 mm
- X(8)=707 mm
- Y(8)=780 mm
- X(9)=0 mm
- Y(9)=780 mm
X co-ordinate                     X(10)=0 mm
Y co-ordinate                     Y(10)=620 mm
X co-ordinate                     X(11)=391.5 mm
Y co-ordinate                     Y(11)=620 mm
X co-ordinate                     X(12)=391.5 mm
Y co-ordinate                     Y(12)=600 mm
X co-ordinate                     X(13)=351.5 mm
Y co-ordinate                     Y(13)=600 mm
X co-ordinate                     X(14)=407.5 mm
Y co-ordinate                     Y(14)=320 mm
X co-ordinate                     X(15)=357.5 mm
Y co-ordinate                     Y(15)=240 mm
X co-ordinate                     X(16)=130 mm
Y co-ordinate                     Y(16)=135 mm
X co-ordinate                     X(17)=125 mm
Y co-ordinate                     Y(17)=0 mm

Second moment of area             IXX2=9.0626E9 mm®
Area of item                      A2=251341 mmý
Centroid from X-X axis            YC2=240.07 mm
Item weighting                    iw2=0.10145
Depth of composite section        di=900 mm
Initial force                     if=146 kN
Modulus of elasticity             Es=200 kN/mmý

Cross-section 1

Distance from mid-span of beam    csd(1)=3.25 m
Number of layers of strand        nls(1)=4
Height above soffit              ds(1)=60 mm
Number of strands                 ns(1)=10
Height above soffit              ds(2)=100 mm
Number of strands                 ns(2)=8
Height above soffit              ds(3)=140 mm
Number of strands                 ns(3)=3
Height above soffit              ds(4)=600 mm
Number of strands                 ns(4)=2

Short Term Losses

Relaxation at transfer ( % )       str=1.5
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.37</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>290.47</td>
<td>8.65</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>340.84</td>
<td>10.15</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 3358 kN
Force at transfer Pt= P0-loft=3017.2 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-2.8396</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.9692</td>
</tr>
<tr>
<td>Total</td>
<td>1.1296</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
Compressive stress 1.1296 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=21 N/mm².

Bottom of beam:
Compressive stress 18.373 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=21 N/mm².

Long Term Losses

Relaxation after transfer ( % ) slr(1)=1
Shrinkage per unit length sul(1)=0.3E-3
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.37</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>290.47</td>
<td>8.65</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>340.84</td>
<td>10.15</td>
</tr>
<tr>
<td>Relaxation</td>
<td>33.58</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>154.56</td>
<td>4.6027</td>
</tr>
<tr>
<td>Creep</td>
<td>437.8</td>
<td>13.038</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>625.94</td>
<td>18.64</td>
</tr>
<tr>
<td>All Losses</td>
<td>966.78</td>
<td>28.79</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 3358 kN
Final force Pf=P0-loftl=2391.2 kN

Unit-weight of in-situ concrete wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-2.2505</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.9692</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>6.8424</td>
</tr>
<tr>
<td>Total</td>
<td>8.5611</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Height from beam soffit to bottom of in-situ concrete hsis=135 mm
Percentage of shrinkage remaining ps=45 %
Percentage pcr=45
Percentage stT=60 %
The net moment is Mc=F*e/1E3-Mb=-327.91 kNm

Stress at top of in-situ concrete: 
ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3 
=-1.3111 N/mm²

Stress at bottom of in-situ concrete: 
fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3 
=4.1552 N/mm²

Stress at top of precast concrete beam:
ftpbds=(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3

Stress at bottom of precast concrete beam:

\[ fbpbds = \left( \frac{F}{(Ai*mr+Ac)} - Mc*1E3*Yc/Ic - Mb*1E3*Yb/Ixx \right) \times 1E3 = -0.59322 \text{ N/mm}^2 \]

### Combination 1 loading

**Applied bending moment:** clb(1)=400 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>8.5611</td>
<td>0</td>
<td>10.072</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-1.3111</td>
<td>-3.0921</td>
<td>4.1552</td>
<td>-0.59322</td>
</tr>
<tr>
<td>Total</td>
<td>3.0225</td>
<td>7.5636</td>
<td>1.3391</td>
<td>5.0349</td>
</tr>
</tbody>
</table>

**Combination 1 Loading - Stresses**

### Combination 2 to 5 loading

**Applied bending moment:** c2b(1)=500 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>8.5611</td>
<td>0</td>
<td>10.072</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2-5</td>
<td>-1.3111</td>
<td>-3.0921</td>
<td>4.1552</td>
<td>-0.59322</td>
</tr>
<tr>
<td>Total</td>
<td>3.504</td>
<td>7.7963</td>
<td>0.63508</td>
<td>3.924</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 Loading - Stresses**

**Combination 3 loading.** The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:

Compressive stress 3.504 N/mm² is less than the maximum allowable stress of 0.5*fcuc=20 N/mm².

Top of beam:

Compressive stress 7.7963 N/mm² is less than the maximum allowable stress of 0.4*fcu=21 N/mm².

Bottom of in-situ concrete:

Compressive stress 0.63508 N/mm² is less than the maximum allowable stress of 0.5*fcuc=20 N/mm².
Bottom of beam:
Compressive stress 3.924 N/mm² is less than the maximum allowable stress of 0.4*fcu=21 N/mm².
Location: 8 metre span TY4 beam with in-situ top slab

Precast prestressed concrete TY beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam \( \text{span}=8 \text{ m} \)

**Beam TY 4 section properties**

- Depth of beam \( d=550 \text{ mm} \)
- Cross-sectional area \( A_c=221426 \text{ mm}^2 \)
- Centroid above soffit \( Y_b=193.45 \text{ mm} \)
- Section modulus top \( Z_t=13.95E6 \text{ mm}^3 \)
- Section modulus bottom \( Z_b=25.68E6 \text{ mm}^3 \)
- Unit weight of beam \( u_w_b=24.5 \text{ kN/m}^3 \)

**Concrete strengths**

- Strength at transfer \( f_{ci}=37.5 \text{ N/mm}^2 \)
- Characteristic strength \( f_{cu}=45 \text{ N/mm}^2 \)

**Concrete moduli**

- Modulus of elasticity at transfer for concrete strength 37.5 N/mm\(^2\) is: \( E_{ci}=30.25 \text{ kN/mm}^2 \)
- Modulus of elasticity for concrete strength 45 N/mm\(^2\) is: \( E_{cb}=32.5 \text{ kN/mm}^2 \)

**Composite section properties**

- Char strength of in-situ concrete \( f_{cu_c}=30 \text{ N/mm}^2 \)
- Modulus of elasticity for concrete strength 30 N/mm\(^2\) is: \( E_{cc}=28 \text{ kN/mm}^2 \)
Breadth of slab \( b = 1000 \text{ mm} \)
Depth of slab \( d_{is} = 160 \text{ mm} \)
Depth of permanent formwork \( d_{pf} = 20 \text{ mm} \)

Area 1 \( 177 \)

\[ \text{Transf'med second moment of area } I_c = I_{xx} + A_c(Y_b - Y_c)^2 + m_r(I_i + A_i(Y_i - Y_c)^2) = 19.144 \times 10^9 \text{ mm}^4 \]

Depth of composite section \( d_i = 680 \text{ mm} \)
Nominal diameter \( n_{omd} = 12 \text{ mm} \)
Nominal tensile strength \( n_{omt} = 1600 \text{ N/mm}^2 \)
Nominal steel area \( n_{oma} = 90 \text{ mm}^2 \)
Characteristic breaking load \( c_{bl} = 160 \text{ kN} \)
Initial force per strand \( i_f = 100 \text{ kN} \)
Modulus of elasticity \( E_s = 200 \text{ kN/mm}^2 \)

Cross-section 1

Distance from mid-span of beam \( c_{sd} = 2.5 \text{ m} \)
Number of layers of strand \( n_{ls} = 3 \)
Height above soffit \( d_s(1) = 60 \text{ mm} \)
Number of strands \( n_s(1) = 8 \)
Height above soffit \( d_s(2) = 100 \text{ mm} \)
Number of strands \( n_s(2) = 12 \)
Height above soffit \( d_s(3) = 180 \text{ mm} \)
Number of strands \( n_s(3) = 3 \)
Bending moment from temp.loads \( t_{lb} = 100 \text{ kNm} \)
Bending moment from other loads \( o_{lb} = 150 \text{ kNm} \)

Short Term Losses

Relaxation at transfer ( % ) \( s_t = 1 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>23 kN</td>
<td>1 %</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>199.67 kN</td>
<td>8.6814</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>222.67 kN</td>
<td>9.6814</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 2300 kN
Force at transfer Pt=P0-loft=2077.3 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-5.0699</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>1.8981</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>7.1772</td>
</tr>
<tr>
<td>Other Loads</td>
<td>10.766</td>
</tr>
<tr>
<td>Total</td>
<td>14.771</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
Compressive stress 14.771 N/mm² is less than allowable stresses of 0.5*fci=18.75 N/mm² and 0.4*fcu=18 N/mm².

Bottom of beam:
Compressive stress 6.4573 N/mm² is less than allowable stresses of 0.5*fci=18.75 N/mm² and 0.4*fcu=18 N/mm².

Long Term Losses

Relaxation after transfer( % ) slr(1)=0.8
Shrinkage per unit length sul(1)=0.3E-3
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>23</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>199.67</td>
<td>8.6814</td>
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<tr>
<td>Transfer Total</td>
<td>222.67</td>
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<tr>
<td>Relaxation</td>
<td>18.4</td>
<td>0.8</td>
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<td>Shrinkage</td>
<td>124.2</td>
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<tr>
<td>Creep</td>
<td>209.16</td>
<td>9.0939</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>351.76</td>
<td>15.294</td>
</tr>
<tr>
<td>All Losses</td>
<td>574.43</td>
<td>24.975</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2300 kN
Final force Pf=P0-loftl=1725.6 kN
Unit-weight of in-situ concrete wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.2114</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>1.8981</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>1.299</td>
</tr>
<tr>
<td>Total</td>
<td>-1.0143</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Height from beam soffit to bottom of in-situ concrete: hsis=520 mm
Percentage of shrinkage remaining: ps=50 %
Percentage: pcr=50
Percentage: stT=45 %
The net moment is: Mc=F*e/1E3-Mb=-225.48 kNm
Stress at top of in-situ concrete:
ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3
=-1.6713 N/mm²
Stress at bottom of in-situ concrete:
fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
=-0.047759 N/mm²
Stress at top of precast concrete beam:
Stress at bottom of precast concrete beam:
\[ f_{bpbds} = \left( \frac{F}{A_i \cdot m_r + A_c} - M_c \cdot 10^3 \frac{Y_c}{I_c} - M_b \cdot 10^3 \frac{Y_b}{I_{xx}} \right) \cdot 10^3 = -1.6816 \, \text{N/mm}^2 \]

Combination 1 loading

Applied bending moment \( c_{lb(1)} = 300 \, \text{kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>-1.0143</td>
<td>0</td>
<td>12.571</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-1.6713</td>
<td>4.8515</td>
<td>-0.047759</td>
<td>-1.6816</td>
</tr>
<tr>
<td>Combination 1</td>
<td>5.9937</td>
<td>4.2407</td>
<td>3.1135</td>
<td>-5.4384</td>
</tr>
<tr>
<td>Total</td>
<td>4.3223</td>
<td>8.0779</td>
<td>3.0657</td>
<td>5.4515</td>
</tr>
</tbody>
</table>

Combination 2 to 5 loading

Applied bending moment \( c_{2b(1)} = 500 \, \text{kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>-1.0143</td>
<td>0</td>
<td>12.571</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-1.6713</td>
<td>4.8515</td>
<td>-0.047759</td>
<td>-1.6816</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>7.4921</td>
<td>5.3008</td>
<td>3.8918</td>
<td>-9.064</td>
</tr>
<tr>
<td>Total</td>
<td>5.8207</td>
<td>9.1381</td>
<td>3.8441</td>
<td>1.8258</td>
</tr>
</tbody>
</table>

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 5.8207 N/mm² is less than the maximum allowable stress of 0.5*f_{cu}=15 N/mm².
Top of beam:
Compressive stress 9.1381 N/mm² is less than the maximum allowable stress of 0.4*f_{cu}=18 N/mm².
Bottom of in-situ concrete:
Compressive stress 3.8441 N/mm² is less than the maximum allowable stress of 0.5*f_{cu}=15 N/mm².
Bottom of beam:
Compressive stress 1.8258 N/mm² is less than the maximum allowable stress of 0.4*f_{cu}=18 N/mm².
Precast prestressed concrete TY beams with in-situ top slab


Beam span under consideration

**Beam TY 10 section properties**

- Depth of beam: $d=850\text{ mm}$
- Cross-sectional area: $A_c=323206\text{ mm}^2$
- Centroid above soffit: $y_b=351.52\text{ mm}$
- Section modulus top: $z_t=46.9E6\text{ mm}^3$
- Section modulus bottom: $z_b=66.47E6\text{ mm}^3$
- Unit weight of beam: $u_w=24\text{ kN/m}^3$

**Concrete compressive strengths**

- Characteristic cylinder strength: $f_{ck}=32\text{ N/mm}^2$ (transfer)
- Characteristic cylinder strength: $f_{ck}=40\text{ N/mm}^2$

**Concrete moduli (short-term)**

- Concrete modulus of elasticity: $e_{cm}=33\text{ kN/mm}^2$ (transfer)
- Concrete modulus of elasticity: $e_{cm}=35\text{ kN/mm}^2$

**Composite section properties**

- Char strength (in-situ concrete): $f_{ckc}=32\text{ N/mm}^2$
- Short-term modulus of elasticity for in-situ concrete: $e_{mc}=33\text{ kN/mm}^2$

---

**Location:** Ex1 - 16 metre span TY10 beam with in-situ slab

**Beam TY 10**

- Breadth of slab: $b=1000\text{ mm}$

---

 SCALE 5.48 Office 1007 Proforma 115
Depth of slab \( d_{is} = 160 \text{ mm} \)
Depth of permanent formwork \( d_{pf} = 20 \text{ mm} \)

\[
\text{Area} = \frac{\text{twob} \times (50 - d_{pf})}{2} \text{ mm}^2
\]

Second moment of area
\[
I_c = I_{yy} + A_c (Y_b - Y_c)^2 + m_r (I_i + A_i (Y_i - Y_c)^2) = 55.066 \times 10^9 \text{ mm}^4
\]

Depth of composite section \( d_i = 980 \text{ mm} \)
Modulus of elasticity (strand) \( E_p = 195 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( c_{sd}(1) = 0 \text{ m} \)
Number of layers of strand \( n_{ls}(1) = 4 \)
Height above soffit \( d_s(1) = 60 \text{ mm} \)
Number of strands \( n_s(1) = 11 \)
Height above soffit \( d_s(2) = 100 \text{ mm} \)
Number of strands \( n_s(2) = 13 \)
Height above soffit \( d_s(3) = 140 \text{ mm} \)
Number of strands \( n_s(3) = 7 \)
Height above soffit \( d_s(4) = 800 \text{ mm} \)
Number of strands \( n_s(4) = 2 \)

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer (\%) \( \text{str}=1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force ( kN )</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.59</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>337.32</td>
<td>8.3104</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>377.91</td>
<td>9.3104</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \( 4059 \text{ kN} \)
Force at transfer \( P_t = P_0 - \text{loft} = 3681.1 \text{ kN} \)
### Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

**Top of beam:**
The flexural tensile stress of 0.11667 N/mm² at transfer is less than the permissible value of 1 N/mm² (as was permitted in BS5400-4).

**Bottom of beam:**
Compressive stress 19.503 N/mm² is less than allowable stresses of 0.7*fck=22.4 N/mm² and 0.6*fck=24 N/mm².

### Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

- Relaxation after transfer (%): $slr(1)=2$
- Shrinkage per unit length: $sul(1)=0.3E-3$
- Creep coefficient $\varphi(\kappa, to): crf=2$

### Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.59</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>337.32</td>
<td>8.3104</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>377.91</td>
<td>9.3104</td>
</tr>
<tr>
<td>Relaxation</td>
<td>81.18</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>179.54</td>
<td>4.4232</td>
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<tr>
<td>Creep</td>
<td>709.26</td>
<td>17.474</td>
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<tr>
<td>Long-term Total</td>
<td>969.97</td>
<td>23.897</td>
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<tr>
<td>All Losses</td>
<td>1347.9</td>
<td>33.207</td>
</tr>
</tbody>
</table>

**Losses - Long term**

- Initial force: 4059 kN
- Final force: $P_f=P_0-loftl=2711.1$ kN
- Unit-weight of in-situ concrete: $w_c=24$ kN/m³
**Beam concrete stresses - Long term**

**Composite section**

Height from beam soffit to bottom of in-situ concrete: \( h_{sis} = 820 \text{ mm} \)
Perc'age of shrinkage remaining: \( p_s = 25 \% \)
Percentage: \( p_{cr} = 25 \% \)
Percentage: \( sT = 67 \% \)
The net moment is: \( M_c = F \times e/1E3 - M_b = -226.88 \text{ kNm} \)

**Stress at top of in-situ concrete:**
\[
ft_{isds} = -(F/(Ai \times mr) - F/(Ai \times mr + Ac)) - Mc \times 1E3 \times (d - Yc) / I_c \times mr \times 1E3
\]
\[
= -1.0282 \text{ N/mm}^2
\]

**Stress at bottom of in-situ concrete:**
\[
f_{bisds} = -(F/(Ai \times mr) - F/(Ai \times mr + Ac)) - Mc \times 1E3 \times (h_{sis} - Yc) / I_c \times mr \times 1E3
\]
\[
= 0.36893 \text{ N/mm}^2
\]

**Stress at top of precast concrete beam:**
\[
ft_{tpbsd} = (F/(Ai \times mr + Ac) + Mc \times 1E3 \times (d - Yc) / I_c + Mb \times 1E3 \times Yt / I_{yy}) \times 1E3
\]
\[
= 1.6316 \text{ N/mm}^2
\]

**Stress at bottom of precast concrete beam:**
\[
f_{bpbsd} = -(F/(Ai \times mr + Ac) - Mc \times 1E3 \times Yc / I_c - Mb \times 1E3 \times Yb / I_{yy}) \times 1E3
\]
\[
= -0.58136 \text{ N/mm}^2
\]

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment: \( c_{lb(1)} = 900 \text{ kNm} \)

**Loading Combination 1 - Stresses**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
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<tr>
<td>Prestress, beam and in-situ</td>
<td>8.013</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>9.0411</td>
</tr>
<tr>
<td>Total</td>
<td>17.054</td>
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</tbody>
</table>
**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment \( c_{2b}(1) = 1200 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
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<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage Combination 2</td>
<td>-1.0282</td>
</tr>
<tr>
<td>Total</td>
<td>8.8349</td>
</tr>
</tbody>
</table>

**NOTE:** The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 8.8349 N/mm\(^2\) is less than the maximum allowable stress of 0.6*f<sub>c</sub>k=19.2 N/mm\(^2\).

Top of beam:
Compressive stress 12.445 N/mm\(^2\) is less than the maximum allowable stress of 0.6*f<sub>c</sub>k=24 N/mm\(^2\).

Bottom of in-situ concrete:
Compressive stress 6.0074 N/mm\(^2\) is less than the maximum allowable stress of 0.6*f<sub>c</sub>k=19.2 N/mm\(^2\).

Bottom of beam:
Tensile stress 0.43931 N/mm\(^2\) is less than the maximum allowable tensile stress of 3.0263 N/mm\(^2\).
Location: Ex2 - 12.5 metre span TY6 beam with in-situ top slab

Precast prestressed concrete TY beams with in-situ top slab


Beam span under consideration span=12.5 m

Beam TY 6 section properties

Depth of beam d=650 mm
Cross-sectional area Ac=251341 mm²
Centroid above soffit Yb=240.07 mm
Section modulus top Zt=22.13E6 mm³
Section modulus bottom Zb=37.75E6 mm³
Unit weight of beam uwb=24.5 kN/m³

Concrete compressive strengths

Characteristic cylinder strength fcki=32 N/mm² (transfer)
Characteristic cylinder strength fck=42 N/mm²

Concrete moduli (short-term)

Concrete modulus of elasticity Ecmi=34 kN/mm² (transfer)
Concrete modulus of elasticity Ecmb=36 kN/mm²

Composite section properties

Char strength (in-situ concrete) fckc=32 N/mm²
Short-term modulus of elasticity for in-situ concrete Ecmc=33 kN/mm²
X co-ordinate X(1)=125 mm
Y co-ordinate Y(1)=0 mm
X co-ordinate X(2)=875 mm
Y co-ordinate Y(2)=0 mm
X co-ordinate X(3)=875 mm
Y co-ordinate Y(3)=135 mm
X co-ordinate X(4)=1007 mm
Y co-ordinate Y(4)=400 mm
X co-ordinate X(5)=1207 mm
Y co-ordinate Y(5)=400 mm
X co-ordinate X(6)=1207 mm
Y co-ordinate Y(6)=900 mm
X co-ordinate X(7)=707 mm
Y co-ordinate Y(7)=880 mm
X co-ordinate X(8)=707 mm
Y co-ordinate Y(8)=780 mm
X co-ordinate X(9)=0 mm
Y co-ordinate Y(9)=780 mm
X co-ordinate X(10)=0 mm
Y co-ordinate Y(10)=620 mm
X co-ordinate X(11)=391.5 mm
Y co-ordinate                     Y(11)=620 mm
X co-ordinate                     X(11)=391.5 mm
Y co-ordinate                     Y(12)=600 mm
X co-ordinate                     X(12)=351.5 mm
Y co-ordinate                     Y(13)=600 mm
X co-ordinate                     X(13)=407.5 mm
Y co-ordinate                     Y(14)=240 mm
X co-ordinate                     X(14)=130 mm
Y co-ordinate                     Y(15)=135 mm
X co-ordinate                     X(15)=125 mm
Y co-ordinate                     Y(16)=0 mm
Depth of composite section        di=900 mm
Area of in-situ concrete          Ai=442313 mm²
Centroid of in-situ concrete      Yi=595.94 mm
Modulus of elasticity (strand)    Ep=195 kN/mm²

Cross-section 1

Distance from mid-span of beam    csd(1)=3.25 m
Number of layers of strand        nls(1)=4
Height above soffit               ds(1)=60 mm
Number of strands                 ns(1)=10
Height above soffit               ds(2)=100 mm
Number of strands                 ns(2)=8
Height above soffit               ds(3)=140 mm
Number of strands                 ns(3)=3
Height above soffit               ds(4)=600 mm
Number of strands                 ns(4)=2

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % )      str=1.5

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.37</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>239.52</td>
<td>7.1327</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>289.89</td>
<td>8.6327</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force              3358 kN
Force at transfer          Pt=P0-loft=3068.1 kN
Stresses at Transfer N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-2.8875</td>
<td>21.047</td>
<td>16.212</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.9692</td>
<td>-2.3245</td>
<td>-1.0531</td>
</tr>
<tr>
<td>Total</td>
<td>1.0817</td>
<td>18.722</td>
<td>15.159</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations** *(BS EN 1992-1-1, Clause 5.10.2.2)*

Top of beam:
Compressive stress 1.0817 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=25.2 N/mm².

Bottom of beam:
Compressive stress 18.722 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=25.2 N/mm².

**Long Term Losses** *(BS EN 1992-1-1, Clause 5.10.6(1))*

Relaxation after transfer( % )    slr(1)=1
Shrinkage per unit length         sul(1)=0.3E-3
Creep coefficient φ(=,to)         crf=2

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.37</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>239.52</td>
<td>7.1327</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>289.89</td>
<td>8.6327</td>
</tr>
<tr>
<td>Relaxation</td>
<td>33.58</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>150.7</td>
<td>4.4877</td>
</tr>
<tr>
<td>Creep</td>
<td>518.04</td>
<td>15.427</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>702.32</td>
<td>20.915</td>
</tr>
<tr>
<td>All Losses</td>
<td>992.21</td>
<td>29.548</td>
</tr>
</tbody>
</table>

Initial force                     3358 kN
Final force                       Pf=P0-loftl=2365.8 kN
Unit-weight of in-situ concrete   wc=24 kN/m³
**Composite section**

Height from beam soffit to bottom of in-situ concrete: $h_{sis}=135$ mm

Percentage of shrinkage remaining: $p_s=45\%$

Percentage: $p_c=45$

Percentage: $s_T=60\%$

The net moment is: $M_c=F*e/1E3-M_b=-418.25$ kNm

Stress at top of in-situ concrete:

$$f_{tisds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3$$

Stress at bottom of in-situ concrete:

$$f_{bisds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3$$

Stress at top of precast concrete beam:

$$f_{tpbsd}=-(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)*1E3$$

Stress at bottom of precast concrete beam:

$$f_{bpbsd}=-(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)*1E3$$

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment: $c_l(1)=400$ kNm

**Final Stresses N/mm²**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-2.2265</td>
<td>16.229</td>
<td>12.501</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.9692</td>
<td>-2.3245</td>
<td>-1.0531</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>6.8424</td>
<td>-4.0072</td>
<td>-1.8155</td>
</tr>
<tr>
<td>Total</td>
<td>8.5851</td>
<td>9.8974</td>
<td>9.6323</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Loading Combination 1 - Stresses

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>8.5851</td>
<td>0</td>
<td>9.8974</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-1.8074</td>
<td>-3.7947</td>
<td>5.5544</td>
<td>-0.89926</td>
</tr>
<tr>
<td>Combination 1</td>
<td>4.832</td>
<td>1.8948</td>
<td>-3.0554</td>
<td>-4.2979</td>
</tr>
<tr>
<td>Total</td>
<td>2.6759</td>
<td>6.6852</td>
<td>2.499</td>
<td>4.7002</td>
</tr>
</tbody>
</table>
Loading Combination 2

(tensile stress permitted (no visible cracking))

Characteristic bending moment $c_2b(1)=500$ kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage Combination 2</td>
<td>-1.8074</td>
</tr>
<tr>
<td></td>
<td>4.9814</td>
</tr>
<tr>
<td>Total</td>
<td>3.174</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 3.174 N/mm² is less than the maximum allowable stress of $0.6*f_{ckc}=19.2$ N/mm².

Top of beam:
Compressive stress 6.8957 N/mm² is less than the maximum allowable stress of $0.6*f_{ck}=25.2$ N/mm².

Bottom of in-situ concrete:
Compressive stress 1.7351 N/mm² is less than the maximum allowable stress of $0.6*f_{ckc}=19.2$ N/mm².

Bottom of beam:
Compressive stress 3.6257 N/mm² is less than the maximum allowable stress of $0.6*f_{ck}=25.2$ N/mm².
Precast prestressed concrete TY beams with in-situ top slab


Beam span under consideration span=8 m

Beam TY 4 section properties

Depth of beam d=550 mm
Cross-sectional area Ac=221426 mm²
Centroid above soffit Yb=193.45 mm
Section modulus top Zt=13.95E6 mm⁴
Section modulus bottom Zb=25.68E6 mm⁴
Unit weight of beam uwb=24.5 kN/m³

Concrete compressive strengths

Characteristic cylinder strength fcki=30 N/mm² (transfer)
Characteristic cylinder strength fck=36 N/mm²

Concrete moduli (short-term)

Concrete modulus of elasticity Ecmi=33 kN/mm² (transfer)
Concrete modulus of elasticity Ecmb=34 kN/mm²

Composite section properties

Char strength (in-situ concrete) fckc=24 N/mm²
Short-term modulus of elasticity for in-situ concrete Ecmc=31 kN/mm²

Breadth of slab b=1000 mm
Depth of slab \( \text{dis}=160 \text{ mm} \)
Depth of permanent formwork \( \text{dpf}=20 \text{ mm} \)

\[
\text{Area} = \text{twob} \times (50 - \text{dpf}) \text{ mm}^2
\]

Transf'med second moment of area
\[
I_c = I_{yy} + A_c \times (Y_b - Y_c)^2 + m_r \times (I_i + A_i \times Y_i - Y_c)^2 = 19.655 \times 10^9 \text{ mm}^4
\]

Depth of composite section \( \text{di}=680 \text{ mm} \)
Nominal diameter \( \text{nomd}=12 \text{ mm} \)
Nominal tensile strength \( \text{nomt}=1600 \text{ N/mm}^2 \)
Nominal steel area \( \text{noma}=90 \text{ mm}^2 \)
Char breaking load (strand) \( \text{fpk}=160 \text{ kN} \)
Initial force per strand \( \text{ipf}=100 \text{ kN} \)
Modulus of elasticity (strand) \( \text{Ep}=195 \text{ kN/mm}^2 \)

Cross-section 1

Distance from mid-span of beam \( \text{csd}(1)=2.5 \text{ m} \)
Number of layers of strand \( \text{nls}(1)=3 \)
Height above soffit \( \text{ds}(1)=60 \text{ mm} \)
Number of strands \( \text{ns}(1)=8 \)
Height above soffit \( \text{ds}(2)=100 \text{ mm} \)
Number of strands \( \text{ns}(2)=12 \)
Height above soffit \( \text{ds}(3)=180 \text{ mm} \)
Number of strands \( \text{ns}(3)=3 \)
Bending moment from temp.loads \( \text{tlb}(1)=100 \text{ kNm} \)
Bending moment from other loads \( \text{olb}(1)=150 \text{ kNm} \)

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % ) \( \text{str}=1 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force ( \text{kN} )</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>23</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>165.49</td>
<td>7.1951</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>188.49</td>
<td>8.1951</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \( 2300 \text{ kN} \)
### Force at Transfer

\[ \text{Pt}=P_0-\text{loft}=2111.5 \text{ kN} \]

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-5.1533</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>1.8981</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>7.1772</td>
</tr>
<tr>
<td>Other Loads</td>
<td>10.766</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>14.688</strong></td>
</tr>
</tbody>
</table>

### Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

- **Top of beam:**
  - Compressive stress 14.688 N/mm² is less than allowable stresses of 0.7*fcki=21 N/mm² and 0.6*fck=21.6 N/mm².
- **Bottom of beam:**
  - Compressive stress 6.7408 N/mm² is less than allowable stresses of 0.7*fcki=21 N/mm² and 0.6*fck=21.6 N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

- Relaxation after transfer( % ) \( \text{slr}(1)=0.8 \)
- Shrinkage per unit length \( \text{sul}(1)=0.3E-3 \)
- Creep coefficient \( \psi(\infty,\text{to}) \) \( \text{crf}=2 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>23</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>165.49</td>
<td>7.1951</td>
</tr>
<tr>
<td><strong>Transfer Total</strong></td>
<td><strong>188.49</strong></td>
<td><strong>8.1951</strong></td>
</tr>
<tr>
<td>Relaxation</td>
<td>18.4</td>
<td>0.8</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>121.1</td>
<td>5.265</td>
</tr>
<tr>
<td>Creep</td>
<td>239.5</td>
<td>10.413</td>
</tr>
<tr>
<td><strong>Long-term Total</strong></td>
<td><strong>378.99</strong></td>
<td><strong>16.478</strong></td>
</tr>
<tr>
<td>All Losses</td>
<td>567.48</td>
<td>24.673</td>
</tr>
</tbody>
</table>

### Losses - Long term

- **Initial force** 2300 kN
- **Final force** \( \text{Pf}=P_0-\text{loft}=1732.5 \text{ kN} \)
Unit-weight of in-situ concrete \( wc = 24 \) kN/m\(^3\)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.2284</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>1.8981</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>1.299</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>-1.0312</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

**Composite section**

Height from beam soffit to bottom of in-situ concrete \( h_{sis} = 520 \) mm

Percentage of shrinkage remaining \( ps = 50 \) %

Percentage \( pcr = 50 \)

Percentage \( stT = 45 \) %

The net moment is \( Mc = F*e/1E3 - Mb = -244.06 \) kNm

Stress at top of in-situ concrete:
\[
ft_{isds} = -(F/(Ai*mr) - F/(Ai*mr+Ac) - Mc*1E3*(di-Yc)/Ic)*mr*1E3
\]
\[=-1.8236 \text{ N/mm}^2\]

Stress at bottom of in-situ concrete:
\[
fb_{isds} = -(F/(Ai*mr) - F/(Ai*mr+Ac) - Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
\]
\[=-0.012188 \text{ N/mm}^2\]

Stress at top of precast concrete beam:
\[
ft_{pbds} = (F/(Ai*mr+Ac) + Mc*1E3*(d-Yc)/Ic + Mb*1E3*Yt/Iyy)*1E3
\]
\[=5.1976 \text{ N/mm}^2\]

Stress at bottom of precast concrete beam:
\[
fb_{pbds} = (F/(Ai*mr+Ac) - Mc*1E3*Yc/Ic - Mb*1E3*Yb/Iyy)*1E3
\]
\[=-1.8036 \text{ N/mm}^2\]

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment \( clb(1) = 300 \) kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-1.8236</td>
</tr>
<tr>
<td></td>
<td>6.0767</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>4.253</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses
Loading Combination 2

(tensile stress permitted (no visible cracking))
Characteristic bending moment c2b(1)=500 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2</th>
<th>Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>-1.0312</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>-1.8236</td>
<td>5.1976</td>
</tr>
<tr>
<td></td>
<td>7.5958</td>
<td>5.0239</td>
</tr>
<tr>
<td>Total</td>
<td>5.7722</td>
<td>9.1902</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 5.7722 N/mm² is less than the maximum allowable stress of 0.6*fckc=14.4 N/mm².
Top of beam:
Compressive stress 9.1902 N/mm² is less than the maximum allowable stress of 0.6*fck=21.6 N/mm².
Bottom of in-situ concrete:
Compressive stress 3.8726 N/mm² is less than the maximum allowable stress of 0.6*fckc=14.4 N/mm².
Bottom of beam:
Compressive stress 1.858 N/mm² is less than the maximum allowable stress of 0.6*fck=21.6 N/mm².
Precast prestressed concrete TYE beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

**Span of beam**

span=16 m

**Beam TYE 10 section properties**

- **Depth of beam**
  
d=850 mm

- **Cross-sectional area**
  
Ac=480040 mm²

- **Centroid above soffit**
  
Yb=400.57 mm

- **Section modulus top**
  
Zt=69.86E6 mm©

- **Section modulus bottom**
  
Zb=78.4E6 mm©

- **Unit weight of beam**
  
Uwb=24 kN/m©

**Concrete strengths**

- **Strength at transfer**
  
fci=40 N/mm²

- **Characteristic strength**
  
fcu=50 N/mm²

**Concrete moduli**

- **Modulus of elasticity at transfer for concrete strength 40 N/mm²**
  
Eci=31 kN/mm²

- **Modulus of elasticity for concrete strength 50 N/mm²**
  
Ecb=34 kN/mm²

**Composite section properties**

- **Char strength (in-situ concrete)**
  
fcuc=40 N/mm²

- **Modulus of elasticity for concrete strength 40 N/mm²**
  
Ecc=31 kN/mm²

- **Second moment of area**
  
Ic=Ixx+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi-Yc)^2)=60.133E9 mm®

- **Depth of composite section**
  
di=980 mm

- **Initial force per strand**
  
If=123 kN

- **Modulus of elasticity**
  
Es=200 kN/mm²

**Cross-section 1**

- **Distance from mid-span of beam**
  
Csd(1)=0 m

- **Number of layers of strand**
  
Nls(1)=4

- **Height above soffit**
  
Ds(1)=60 mm

- **Number of strands**
  
Ns(1)=11

- **Height above soffit**
  
Ds(2)=100 mm

- **Number of strands**
  
Ns(2)=13

- **Height above soffit**
  
Ds(3)=140 mm

- **Number of strands**
  
Ns(3)=7

- **Height above soffit**
  
Ds(4)=800 mm

- **Number of strands**
  
Ns(4)=2
Short Term Losses

Relaxation at transfer (%): str=1

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.59</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>340.98</td>
<td>8.4006</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>381.57</td>
<td>9.4006</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force: 4059 kN
Force at transfer: Pt=P0-loss=3677.4 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-6.18</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>-5.276</td>
</tr>
<tr>
<td>Total</td>
<td>-0.90402</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
The flexural tensile stress of 0.90402 N/mm² at transfer is less than the permissible value of 1 N/mm² (Clause 6.3.2.4(b)(1)).

Bottom of beam:
Compressive stress 15.294 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Long Term Losses

Relaxation after transfer: slr(1)=2
Shrinkage per unit length: sul(1)=0.3E-3
### Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.59</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>340.98</td>
<td>8.4006</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>381.57</td>
<td>9.4006</td>
</tr>
<tr>
<td>Relaxation</td>
<td>81.18</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>184.14</td>
<td>4.5366</td>
</tr>
<tr>
<td>Creep</td>
<td>400.82</td>
<td>9.8748</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>666.14</td>
<td>16.411</td>
</tr>
<tr>
<td>All Losses</td>
<td>1047.7</td>
<td>25.812</td>
</tr>
</tbody>
</table>

#### Losses - Long term

**Initial force** 4059 kN  
**Final force** Pf=P0-loftl=3011.3 kN  
**Unit-weight of in-situ concrete** wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-5.0606</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.276</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>1.5858</td>
</tr>
<tr>
<td>Total</td>
<td>1.8012</td>
</tr>
</tbody>
</table>

#### Composite section

- **Height from beam soffit to bottom of in-situ concrete** hsis=820 mm  
- **Perc'age of shrinkage remaining** ps=25 %  
- **Percentage** pcr=25  
- **Percentage** stT=67 %  
- **The net moment is** Mc=F*e/1E3-Mb=-98.356 kNm  
- **Stress at top of in-situ concrete:**  
  \[
  f_{tisds} = \frac{F}{A_i m r} - \frac{F}{A_i m r + A_c} - M_c E_3 \left(\frac{d_i - Y_c}{I_c}\right) m r E_3 = -1.3412 \text{ N/mm}^2
  \]
- **Stress at bottom of in-situ concrete:**  
  \[
  f_{bisds} = \frac{F}{A_i m r} - \frac{F}{A_i m r + A_c} - M_c E_3 \left(\frac{h_{sis} - Y_c}{I_c}\right) m r E_3 = -1.0795 \text{ N/mm}^2
  \]
- **Stress at top of precast concrete beam:**
Stress at bottom of precast concrete beam:
\[ \text{fbpdb} = \frac{F}{(A_i \cdot r_A + A_c)} \cdot \frac{10^3 \cdot (d-Y_c)}{I_c} = -0.77732 \text{ N/mm}^2 \]

**Combination 1 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam</td>
<td>0</td>
</tr>
<tr>
<td>and in-situ</td>
<td></td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-1.3412</td>
</tr>
<tr>
<td>Combination 1</td>
<td>8.4582</td>
</tr>
<tr>
<td>Total</td>
<td>7.117</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam</td>
<td>0</td>
</tr>
<tr>
<td>and in-situ</td>
<td></td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-1.3412</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>9.2271</td>
</tr>
<tr>
<td>Total</td>
<td>7.8859</td>
</tr>
</tbody>
</table>

Combination 3 loading. The stresses from the proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 7.8859 N/mm$^2$ is less than the maximum allowable stress of 0.5*fucc=20 N/mm$^2$.

Top of beam:
Compressive stress 10.086 N/mm$^2$ is less than the maximum allowable stress of 0.4*fucc=20 N/mm$^2$.

Bottom of in-situ concrete:
Compressive stress 4.9547 N/mm$^2$ is less than the maximum allowable stress of 0.5*fucc=20 N/mm$^2$. 
Bottom of beam:
Tensile stress 0.84833 N/mm² is less than the maximum allowable tensile stress of 3.182 N/mm².
Location: Ex2 - 12.5 metre span TYE6 beam with in-situ top slab

Precast prestressed concrete TYE beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam  span=12.5 m

Beam TYE 6 section properties

Depth of beam  d=650 mm
Cross-sectional area  Ac=369053 mm²
Centroid above soffit  Yb=296.39 mm
Section modulus top  Zt=38.69E6 mm©
Section modulus bottom  Zb=46.16E6 mm©
Unit weight of beam  uwb=24.5 kN/m©

Concrete strengths

Strength at transfer  fci=40 N/mm²
Characteristic strength  fcu=52.5 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 40 N/mm² is:
Eci=31 kN/mm²
Modulus of elasticity for concrete strength 52.5 N/mm² is:
Modulus  Ecb=34.5 kN/mm²

Composite section properties

Char strength (in-situ concrete)  fcuc=40 N/mm²
Modulus of elasticity for concrete strength 40 N/mm² is:
Ecc=31 kN/mm²
Item weighting  iw1=0.89855
X co-ordinate  X(1)=500 mm
Y co-ordinate  Y(1)=0 mm
X co-ordinate  X(2)=1250 mm
Y co-ordinate  Y(2)=0 mm
X co-ordinate  X(3)=1245 mm
Y co-ordinate  Y(3)=135 mm
X co-ordinate  X(4)=1017.5 mm
Y co-ordinate  Y(4)=240 mm
X co-ordinate  X(5)=967.5 mm
Y co-ordinate  Y(5)=320 mm
X co-ordinate  X(6)=1023 mm
Y co-ordinate  Y(6)=600 mm
X co-ordinate  X(7)=983 mm
Y co-ordinate  Y(7)=600 mm
X co-ordinate  X(8)=983 mm
Y co-ordinate  Y(8)=620 mm
X co-ordinate  X(9)=1375 mm
Y co-ordinate  Y(9)=600 mm

SCALE 5.48
Office 1007
Proforma 116
X co-ordinate                     X(10)=1375 mm
Y co-ordinate                     Y(10)=780 mm
X co-ordinate                     X(11)=500 mm
Y co-ordinate                     Y(11)=780 mm
X co-ordinate                     X(12)=500 mm
Y co-ordinate                     Y(12)=880 mm
X co-ordinate                     X(13)=0 mm
Y co-ordinate                     Y(13)=900 mm
X co-ordinate                     X(14)=0 mm
Y co-ordinate                     Y(14)=600 mm
X co-ordinate                     X(15)=500 mm
Y co-ordinate                     Y(15)=600 mm
X co-ordinate                     X(16)=500 mm
Y co-ordinate                     Y(16)=0 mm

Item No. 2

Second moment of area             IXX2=13.681E9 mm⁴
Area of item                      A2=369053 mm²
Centroid from X-X axis            YC2=296.39 mm
Item weighting                    iw2=0.10145
Depth of composite section        di=900 mm
Initial force per strand          if=146 kN
Modulus of elasticity             Es=200 kN/mm²

Cross-section 1

Distance from mid-span of beam    csd(1)=2.25 m
Number of layers of strand        nls(1)=4
Height above soffit               ds(1)=60 mm
Number of strands                 ns(1)=10
Height above soffit               ds(2)=100 mm
Number of strands                 ns(2)=8
Height above soffit               ds(3)=140 mm
Number of strands                 ns(3)=3
Height above soffit               ds(4)=600 mm
Number of strands                 ns(4)=2

Short Term Losses

Relaxation at transfer ( % )      str=1.5

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.37</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>258.45</td>
<td>7.6966</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>308.82</td>
<td>9.1966</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force                     3358 kN
Force at transfer  
\[ Pt = P_0 - loft = 3049.2 \text{ kN} \]

### Stresses at Transfer N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-4.7481</td>
<td>19.167</td>
<td>14.336</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.9728</td>
<td>-3.33</td>
<td>-1.8547</td>
</tr>
<tr>
<td>Total</td>
<td>-0.77531</td>
<td>15.837</td>
<td>12.481</td>
</tr>
</tbody>
</table>

#### Beam Concrete Stresses at Transfer

**Stress Limitations**

**Top of beam:**
The flexural tensile stress of 0.77531 N/mm² at transfer is less than the permissible value of 1 N/mm² (Clause 6.3.2.4(b)(1)).

**Bottom of beam:**
Compressive stress 15.837 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=21 N/mm².

**Long Term Losses**

Relaxation after transfer ( % )  
\[ slr(1)=1 \]
Shrinkage per unit length  
\[ sul(1)=0.3E-3 \]

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.37</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>258.45</td>
<td>7.6966</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>308.82</td>
<td>9.1966</td>
</tr>
<tr>
<td>Relaxation</td>
<td>33.58</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>154.56</td>
<td>4.6027</td>
</tr>
<tr>
<td>Creep</td>
<td>337.64</td>
<td>10.055</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>525.78</td>
<td>15.658</td>
</tr>
<tr>
<td>All Losses</td>
<td>834.6</td>
<td>24.854</td>
</tr>
</tbody>
</table>

**Losses - Long term**

Initial force  
3358 kN  

Final force  
\[ Pf = P_0 - loft = 2523.4 \text{ kN} \]
Unit weight of in-situ concrete \( wc=24 \text{ kN/m}^3 \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.9294</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.9728</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.8564</td>
</tr>
<tr>
<td>Total</td>
<td>2.8998</td>
</tr>
</tbody>
</table>

Composite section

Height from beam soffit to bottom of in-situ concrete \( h_{sis}=600 \text{ mm} \)
Perc'age of shrinkage remaining \( p_s=45\% \)
Percentage \( p_{cr}=45\% \)
Percentage \( s_tT=60\% \)
The net moment is \( M_c=F*e/1E3-M_b=-292.06 \text{ kNm} \)

Stress at top of in-situ concrete:
\[
ft_{isds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(d_i-Yc)/Ic)*mr*1E3
\]
\[
=-1.3192 \text{ N/mm}\(^2\)
\]

Stress at bottom of in-situ concrete:
\[
fb_{isds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(h_{sis}-Yc)/Ic)*mr*1E3
\]
\[
=0.54414 \text{ N/mm}\(^2\)
\]

Stress at top of precast concrete beam:
\[
ft_{pbds}=(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3=1.1609 \text{ N/mm}\(^2\)
\]

Stress at bottom of precast concrete beam:
\[
fb_{pbds}=(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3=-0.64917 \text{ N/mm}\(^2\)
\]

Combination 1 loading

Applied bending moment \( c_{lb}(1)=400 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-1.3192</td>
</tr>
<tr>
<td>Combination 1</td>
<td>4.1418</td>
</tr>
<tr>
<td>Total</td>
<td>2.8226</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses
Combination 2 to 5 loading

Applied bending moment \( c2b(1)=500 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Dif. shrinkage Combination 2-5</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>-1.3192</td>
</tr>
<tr>
<td></td>
<td>4.602</td>
</tr>
<tr>
<td>Total</td>
<td>3.2828</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 3.2828 N/mm\(^2\) is less than the maximum allowable stress of 0.5*fcuc=20 N/mm\(^2\).

Top of beam:
Compressive stress 6.2237 N/mm\(^2\) is less than the maximum allowable stress of 0.4*fcu=21 N/mm\(^2\).

Bottom of in-situ concrete:
Compressive stress 1.956 N/mm\(^2\) is less than the maximum allowable stress of 0.5*fcuc=20 N/mm\(^2\).

Bottom of beam:
Compressive stress 3.9596 N/mm\(^2\) is less than the maximum allowable stress of 0.4*fcu=21 N/mm\(^2\).
Location: Ex3 - 8 metre span TYE4 beam with in-situ top slab

Precast prestressed concrete TYE beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam                      span=8 m

Beam TYE 4 section properties

Depth of beam                     d=550 mm
Cross-sectional area              Ac=316654 mm²
Centroid above soffit             Yb=246.79 mm
Section modulus top               Zt=26.86E6 mm³
Section modulus bottom            Zb=33E6 mm³
Unit weight of beam               uwb=24.5 kN/m³

Concrete strengths

Strength at transfer              fci=37.5 N/mm²
Characteristic strength           fcu=45 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 37.5 N/mm² is:
Eci=30.25 kN/mm²
Modulus of elasticity for concrete strength 45 N/mm² is:
Modulus Ecb=32.5 kN/mm²

Composite section properties

Char strength (in-situ concrete)  fcuc=30 N/mm²
Modulus of elasticity for concrete strength 30 N/mm² is:
Ecc=28 kN/mm²
Transfered second moment of area  Ic=Ixx+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi
-Yc)^2)=20.016E9 mm⁴
Depth of composite section        di=680 mm
Nominal diameter                  nomd=12 mm
Nominal tensile strength          nomt=1600 N/mm²
Nominal steel area                noma=90 mm²
Characteristic breaking load      cbl=160 kN
Initial force per strand          if=100 kN
Modulus of elasticity Es=200 kN/mm²

Cross-section 1

Distance from mid-span of beam    csd(1)=0.5 m
Number of layers of strand       nls(1)=3
Height above soffit              ds(1)=60 mm
Number of strands                ns(1)=6
Height above soffit              ds(2)=100 mm
Number of strands                ns(2)=8
SCALE 5.48 Office 1007 Proforma 116

Height above soffit \( ds(3) = 180 \text{ mm} \)
Number of strands \( ns(3) = 3 \)
Bending moment from temp.loads \( tlb(1) = 100 \text{ kNm} \)
Bending moment from other loads \( olb(1) = 150 \text{ kNm} \)

**Short Term Losses**

Relaxation at transfer ( % ) \( \text{str}=1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>17</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>98.808</td>
<td>5.8122</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>115.81</td>
<td>6.8122</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \( 1700 \text{ kN} \)
Force at transfer \( Pt=P0-loft=1584.2 \text{ kN} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.6549</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>2.2746</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>3.7231</td>
</tr>
<tr>
<td>Other Loads</td>
<td>5.5846</td>
</tr>
<tr>
<td>Total</td>
<td>7.9274</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress \( 7.9274 \text{ N/mm}\(^2\) \) is less than allowable stresses of \( 0.5 \times fci = 18.75 \text{ N/mm}\(^2\) \) and \( 0.4 \times fcu = 18 \text{ N/mm}\(^2\) \).

Bottom of beam:
Compressive stress \( 2.6226 \text{ N/mm}\(^2\) \) is less than allowable stresses of \( 0.5 \times fci = 18.75 \text{ N/mm}\(^2\) \) and \( 0.4 \times fcu = 18 \text{ N/mm}\(^2\) \).

**Long Term Losses**

Relaxation after transfer ( % ) \( \text{slr}(1)=0.8 \)
Shrinkage per unit length \( \text{sul}(1)=0.3E-3 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>17</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>98.808</td>
<td>5.8122</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>115.81</td>
<td>6.8122</td>
</tr>
<tr>
<td>Relaxation</td>
<td>13.6</td>
<td>0.8</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>91.8</td>
<td>5.4</td>
</tr>
<tr>
<td>Creep</td>
<td>84.439</td>
<td>4.967</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>189.84</td>
<td>11.167</td>
</tr>
<tr>
<td>All Losses</td>
<td>305.65</td>
<td>17.979</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force: 1700 kN
Final force: Pf = P0 - loftl = 1394.4 kN
Unit-weight of in-situ concrete: wc = 24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.2169</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>2.2746</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>1.0281</td>
</tr>
<tr>
<td>Total</td>
<td>0.085822</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete: hsis = 520 mm
Percentage of shrinkage remaining: ps = 50 %
Percentage: pcr = 50
Percentage: stT = 45 %
The net moment is: Mc = F*e/1E3 - Mb = -127.43 kNm

Stress at top of in-situ concrete:
\[ f_{tisds} = -\frac{F}{(A_i*mr)} - F/(A_i*mr+Ac) - Mc*1E3* (di-Yc)/Ic)*mr*1E3 \]
\[ \approx -1.1985 \text{ N/mm}^2 \]

Stress at bottom of in-situ concrete:
\[ f_{bisds} = -\frac{F}{(A_i*mr)} - F/(A_i*mr+Ac) - Mc*1E3* (hsis-Yc)/Ic)*mr*1E3 \]
\[ \approx -0.32097 \text{ N/mm}^2 \]

Stress at top of precast concrete beam:
Stress at bottom of precast concrete beam:
\[
fb_{pbds} = \frac{F}{(Ai*mr+Ac)} - \frac{Mc*1E3*Yc}{Ic} - \frac{Mb*1E3*Yb}{Ixx} \times 1E3 = -0.94254 \text{ N/mm}^2
\]

Combination 1 loading

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>0.085822</td>
<td>0</td>
<td>7.9176</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-1.1985</td>
<td>1.9741</td>
<td>-0.32097</td>
<td>-0.94254</td>
</tr>
<tr>
<td>Combination 1</td>
<td>5.6986</td>
<td>4.0165</td>
<td>2.9438</td>
<td>-5.2312</td>
</tr>
<tr>
<td>Total</td>
<td>4.5</td>
<td>6.0764</td>
<td>2.6229</td>
<td>1.7439</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>0.085822</td>
<td>0</td>
<td>7.9176</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2-5</td>
<td>-1.1985</td>
<td>1.9741</td>
<td>-0.32097</td>
<td>-0.94254</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>7.1232</td>
<td>5.0206</td>
<td>3.6798</td>
<td>-8.7186</td>
</tr>
<tr>
<td>Total</td>
<td>5.9247</td>
<td>7.0805</td>
<td>3.3588</td>
<td>-1.7436</td>
</tr>
</tbody>
</table>

Combination 3 loading. The stresses from the proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 5.9247 N/mm\(^2\) is less than the maximum allowable stress of 0.5*f\(c_{uc}\)=15 N/mm\(^2\).
Top of beam:
Compressive stress 7.0805 N/mm\(^2\) is less than the maximum allowable stress of 0.4*f\(c_u\)=18 N/mm\(^2\).
Bottom of in-situ concrete:
Compressive stress 3.3588 N/mm\(^2\) is less than the maximum allowable stress of 0.5*f\(c_{uc}\)=15 N/mm\(^2\).
Bottom of beam:

Tensile stress 1.7436 N/mm$^2$ is less than the maximum allowable tensile stress of 3.0187 N/mm$^2$. 
Location: Ex1- 16 metre span TYE10 beam with in-situ top slab

Precast prestressed concrete TYE beams with in-situ top slab


Beam span under consideration span=16 m

Beam TYE 10 section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=850 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=480040 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=400.57 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>2t=69.86E6 mm³</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>2b=78.4E6 mm³</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=24 kN/m³</td>
</tr>
</tbody>
</table>

Concrete compressive strengths

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic cylinder strength</td>
<td>fck=32 N/mm² (transfer)</td>
</tr>
<tr>
<td>Characteristic cylinder strength</td>
<td>fck=40 N/mm²</td>
</tr>
</tbody>
</table>

Concrete moduli (short-term)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete modulus of elasticity</td>
<td>Ecmi=33 kN/mm² (transfer)</td>
</tr>
<tr>
<td>Concrete modulus of elasticity</td>
<td>Ecmb=35 kN/mm²</td>
</tr>
</tbody>
</table>

Composite section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char strength (in-situ concrete)</td>
<td>fckc=32 N/mm²</td>
</tr>
<tr>
<td>Short-term modulus of elasticity</td>
<td>Ecmc=33 kN/mm²</td>
</tr>
<tr>
<td>Second moment of area</td>
<td>Ic=Iyy+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi-Yc)^2)=60.133E9 mm⁴</td>
</tr>
<tr>
<td>Depth of composite section</td>
<td>di=980 mm</td>
</tr>
<tr>
<td>Modulus of elasticity (strand)</td>
<td>Ep=195 kN/mm²</td>
</tr>
</tbody>
</table>

Cross-section 1

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance from mid-span of beam</td>
<td>csd(1)=0 m</td>
</tr>
<tr>
<td>Number of layers of strand</td>
<td>nls(1)=4</td>
</tr>
<tr>
<td>Height above soffit</td>
<td>ds(1)=60 mm</td>
</tr>
<tr>
<td>Number of strands</td>
<td>ns(1)=11</td>
</tr>
<tr>
<td>Height above soffit</td>
<td>ds(2)=100 mm</td>
</tr>
<tr>
<td>Number of strands</td>
<td>ns(2)=13</td>
</tr>
<tr>
<td>Height above soffit</td>
<td>ds(3)=140 mm</td>
</tr>
<tr>
<td>Number of strands</td>
<td>ns(3)=7</td>
</tr>
<tr>
<td>Height above soffit</td>
<td>ds(4)=800 mm</td>
</tr>
<tr>
<td>Number of strands</td>
<td>ns(4)=2</td>
</tr>
</tbody>
</table>

Short Term Losses (BS EN 1992-1-1)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation at transfer ( % )</td>
<td>str=1</td>
</tr>
</tbody>
</table>
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.59</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>289.78</td>
<td>7.1393</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>330.37</td>
<td>8.1393</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 4059 kN
Force at transfer Pt=P0-loft=3728.6 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-6.2661</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.276</td>
</tr>
<tr>
<td>Total</td>
<td>-0.99005</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam:
The flexural tensile stress of 0.99005 N/mm² at transfer is less than the permissible value of 1 N/mm² (as was permitted in BS5400-4).
Bottom of beam:
Compressive stress 15.573 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer (% ) slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(=,to) crf=2
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.59</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>289.78</td>
<td>7.1393</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>330.37</td>
<td>8.1393</td>
</tr>
<tr>
<td>Relaxation</td>
<td>81.18</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>179.54</td>
<td>4.4232</td>
</tr>
<tr>
<td>Creep</td>
<td>465.78</td>
<td>11.475</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>726.5</td>
<td>17.898</td>
</tr>
<tr>
<td>All Losses</td>
<td>1056.9</td>
<td>26.038</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 4059 kN
Final force $P_f = P_0 - l_0t_1 = 3002.1$ kN
Unit-weight of in-situ concrete $w_c = 24$ kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-5.0452</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.276</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>1.5858</td>
</tr>
<tr>
<td>Total</td>
<td>1.8166</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete $h_{sis} = 820$ mm
Percentage of shrinkage remaining $p_s = 25$%
Percentage $p_c r = 25$
Percentage $s T = 67$%
The net moment is $M_c = F * e / 1E3 - M_b = -149.44$ kNm

Stress at top of in-situ concrete:
$$f_{tisds} = -(F/(A_i * mr) - F/(A_i * mr + A_c) - M_c * 1E3 * (di - Y_c)/I_c) * mr * 1E3 = -1.4506 \text{ N/mm}^2$$

Stress at bottom of in-situ concrete:
$$f_{bisds} = -(F/(A_i * mr) - F/(A_i * mr + A_c) - M_c * 1E3 * (h_{sis} - Y_c)/I_c) * mr * 1E3 = -1.053 \text{ N/mm}^2$$

Stress at top of precast concrete beam:
ftpbds=(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)*1E3
=1.7186 N/mm²

Stress at bottom of precast concrete beam:
fbpbds=(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)*1E3
=-0.81018 N/mm²

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment  clb(1)=900 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Dif. shrinkage Combination 1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>-1.4506</td>
</tr>
<tr>
<td></td>
<td>8.4582</td>
</tr>
<tr>
<td>Total</td>
<td>7.0076</td>
</tr>
</tbody>
</table>

**Loading Combination 1 - Stresses**

**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment  c2b(1)=1200 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Dif. shrinkage Combination 2</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>-1.4506</td>
</tr>
<tr>
<td></td>
<td>9.2271</td>
</tr>
<tr>
<td>Total</td>
<td>7.7765</td>
</tr>
</tbody>
</table>

**Loading Combination 2 - Stresses**

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 7.7765 N/mm² is less than the maximum allowable stress of 0.6*fckc=19.2 N/mm².

Top of beam:
Compressive stress 10.168 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².
Bottom of in-situ concrete:
Compressive stress 4.9812 N/mm$^2$ is less than the maximum allowable stress of 0.6*fck=19.2 N/mm$^2$.

Bottom of beam:
Tensile stress 0.93104 N/mm$^2$ is less than the maximum allowable tensile stress of 3.0263 N/mm$^2$. 
**Location:** Ex2 - 12.5 metre span TYE6 beam with in-situ top slab

**Precast prestressed concrete TYE beams with in-situ top slab**


**Beam span under consideration**  
span=12.5 m

**Beam TYE 6 section properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=650 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=369053 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=296.39 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>Zt=38.69E6 mm³</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>Zb=46.16E6 mm³</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=24.5 kN/m³</td>
</tr>
</tbody>
</table>

**Concrete compressive strengths**

| Characteristic cylinder strength      | fcki=32 N/mm² (transfer) |
| Characteristic cylinder strength      | fck=42 N/mm² |

**Concrete moduli (short-term)**

| Concrete modulus of elasticity        | Ecmi=34 kN/mm² (transfer) |
| Concrete modulus of elasticity        | Ecmb=36 kN/mm² |

**Composite section properties**

| Char strength (in-situ concrete)     | fckc=32 N/mm² |
| Short-term modulus of elasticity for in-situ concrete | Ecmc=33 kN/mm² |

<table>
<thead>
<tr>
<th>X co-ordinate</th>
<th>Y co-ordinate</th>
</tr>
</thead>
<tbody>
<tr>
<td>X(1)=500 mm</td>
<td>Y(1)=0 mm</td>
</tr>
<tr>
<td>X(2)=1250 mm</td>
<td>Y(2)=0 mm</td>
</tr>
<tr>
<td>X(3)=1245 mm</td>
<td>Y(3)=135 mm</td>
</tr>
<tr>
<td>X(4)=1017.5 mm</td>
<td>Y(4)=240 mm</td>
</tr>
<tr>
<td>X(5)=967.5 mm</td>
<td>Y(5)=320 mm</td>
</tr>
<tr>
<td>X(6)=1023 mm</td>
<td>Y(6)=600 mm</td>
</tr>
<tr>
<td>X(7)=983 mm</td>
<td>Y(7)=600 mm</td>
</tr>
<tr>
<td>X(8)=983 mm</td>
<td>Y(8)=620 mm</td>
</tr>
<tr>
<td>X(9)=1375 mm</td>
<td>Y(9)=620 mm</td>
</tr>
<tr>
<td>X(10)=1375 mm</td>
<td>Y(10)=780 mm</td>
</tr>
<tr>
<td>X(11)=500 mm</td>
<td>Y(11)=500 mm</td>
</tr>
</tbody>
</table>
Y co-ordinate                     Y(11)=780 mm
X co-ordinate                     X(12)=500 mm
Y co-ordinate                     Y(12)=880 mm
X co-ordinate                     X(13)=0 mm
Y co-ordinate                     Y(13)=900 mm
X co-ordinate                     X(14)=0 mm
Y co-ordinate                     Y(14)=600 mm
X co-ordinate                     X(15)=500 mm
Y co-ordinate                     Y(15)=600 mm
X co-ordinate                     X(16)=500 mm
Y co-ordinate                     Y(16)=0 mm

Depth of composite section        di=900 mm
Area of in-situ concrete          Ai=270871 mm$^2$
Centroid of in-situ concrete      Yi=726.25 mm
Modulus of elasticity (strand)    Ep=195 kN/mm$^2$

Cross-section 1

Distance from mid-span of beam    csd(1)=2.25 m
Number of layers of strand        nls(1)=4
Height above soffit               ds(1)=60 mm
Number of strands                 ns(1)=10
Height above soffit               ds(2)=100 mm
Number of strands                 ns(2)=8
Height above soffit               ds(3)=140 mm
Number of strands                 ns(3)=3
Height above soffit               ds(4)=600 mm
Number of strands                 ns(4)=2

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % )       str=1.5

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.37</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>214.83</td>
<td>6.3977</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>265.2</td>
<td>7.8977</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force                     3358 kN
Force at transfer                 Pt=P0-loft=3092.8 kN
### Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
The flexural tensile stress of 0.84323 N/mm² at transfer is less than the permissible value of 1 N/mm² (as was permitted in BS5400-4).

Bottom of beam:
Compressive stress 16.111 N/mm² is less than allowable stresses of 0.7*fck=22.4 N/mm² and 0.6*fck=25.2 N/mm².

### Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer( % ) slr(1)=1
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(∗,to) crf=2

### Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.37</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>214.83</td>
<td>6.3977</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>265.2</td>
<td>7.8977</td>
</tr>
<tr>
<td>Relaxation</td>
<td>33.58</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>150.7</td>
<td>4.4877</td>
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<tr>
<td>Creep</td>
<td>383.61</td>
<td>11.424</td>
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<tr>
<td>Long-term Total</td>
<td>567.89</td>
<td>16.911</td>
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<tr>
<td>All Losses</td>
<td>833.09</td>
<td>24.809</td>
</tr>
</tbody>
</table>

### Losses - Long term

Initial force 3358 kN
Final force Pf=P0-lofstl=2524.9 kN
Unit-weight of in-situ concrete wc=24 kN/m³
### Composite section

Height from beam soffit to bottom of in-situ concrete: \( h_{sis} = 600 \text{ mm} \)

Perc'age of shrinkage remaining: \( p_s = 45 \% \)

Percentage: \( p_{cr} = 45 \% \)

Percentage: \( st_T = 60 \% \)

The net moment is: \( M_c = F \times e / 1E3 - M_b = -372.73 \text{ kNm} \)

Stress at top of in-situ concrete:

\[
ft_{isds} = -\left( \frac{F}{A_i \times mr} - \frac{F}{A_i \times mr + Ac} - Mc \times 1E3 \times (d - Y_c)/I_c \right) \times mr \times 1E3
\]

\[
-1.7412 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:

\[
fb_{isds} = -\left( \frac{F}{A_i \times mr} - \frac{F}{A_i \times mr + Ac} - Mc \times 1E3 \times (hsis - Y_c)/I_c \right) \times mr \times 1E3
\]

\[
0.78969 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:

\[
ft_{pbds} = \left( \frac{F}{A_i \times mr + Ac} + Mc \times 1E3 \times (d - Y_c)/I_c + Mb \times 1E3 \times Y_t/I_{yy} \right) \times 1E3
\]

\[
1.3875 \text{ N/mm}^2
\]

Stress at bottom of precast concrete beam:

\[
fb_{pbds} = \left( \frac{F}{A_i \times mr + Ac} - Mc \times 1E3 \times Y_c/I_c - Mb \times 1E3 \times Y_b/I_{yy} \right) \times 1E3
\]

\[
-0.79134 \text{ N/mm}^2
\]

### Loading Combination 1

(no tensile stress permitted in precast beam)

Characteristic bending moment: \( clb(1) = 400 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading Combination 1</th>
<th>Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-1.7412</td>
</tr>
<tr>
<td></td>
<td>4.2947</td>
</tr>
<tr>
<td>Total</td>
<td>2.5535</td>
</tr>
</tbody>
</table>
### Loading Combination 2

(tensile stress permitted (no visible cracking))

Characteristic bending moment  \( c_{2b(1)} = 500 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2</th>
<th>Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>2.8975</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>-1.7412</td>
<td>1.3875</td>
</tr>
<tr>
<td></td>
<td>4.7719</td>
<td>1.9426</td>
</tr>
<tr>
<td>Total</td>
<td>3.0306</td>
<td>6.2276</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 3.0306 N/mm² is less than the maximum allowable stress of 0.6*\( f_{ckc} \)=19.2 N/mm².

Top of beam:
Compressive stress 6.2276 N/mm² is less than the maximum allowable stress of 0.6*\( f_{ck} \)=25.2 N/mm².

Bottom of in-situ concrete:
Compressive stress 2.1665 N/mm² is less than the maximum allowable stress of 0.6*\( f_{ckc} \)=19.2 N/mm².

Bottom of beam:
Compressive stress 3.9427 N/mm² is less than the maximum allowable stress of 0.6*\( f_{ck} \)=25.2 N/mm².
Location: Ex3 - 8 metre span TYE4 beam with in-situ top slab

Precast prestressed concrete TYE beams with in-situ top slab


Beam span under consideration \( \text{span}=8 \, \text{m} \)

**Beam TYE 4 section properties**

- **Depth of beam** \( d=550 \, \text{mm} \)
- **Cross-sectional area** \( A_c=316654 \, \text{mm}^2 \)
- **Centroid above soffit** \( Y_b=246.79 \, \text{mm} \)
- **Section modulus top** \( Z_t=26.86 \times 10^6 \, \text{mm}^3 \)
- **Section modulus bottom** \( Z_b=33 \times 10^6 \, \text{mm}^3 \)
- **Unit weight of beam** \( u_w b=24.5 \, \text{kN/m}^3 \)

**Concrete compressive strengths**

- **Characteristic cylinder strength** \( f_{ck_i}=30 \, \text{N/mm}^2 \) (transfer)
- **Characteristic cylinder strength** \( f_{ck}=36 \, \text{N/mm}^2 \)

**Concrete moduli (short-term)**

- **Concrete modulus of elasticity** \( E_{cmi}=33 \, \text{kN/mm}^2 \) (transfer)
- **Concrete modulus of elasticity** \( E_{cmb}=34 \, \text{kN/mm}^2 \)

**Composite section properties**

- **Char strength (in-situ concrete)** \( f_{ckc}=24 \, \text{N/mm}^2 \)
- **Short-term modulus of elasticity for in-situ concrete** \( E_{cmc}=31 \, \text{kN/mm}^2 \)
- **Transf'med second moment of area** \( I_c=I_y y+A_c*(Y_b-Y_c)^2+mr*(I_i+A_i*(Y_i-Y_c)^2)=20.507 \times 10^9 \, \text{mm}^4 \)
- **Depth of composite section** \( d_i=680 \, \text{mm} \)
- **Nominal diameter** \( \text{nomd}=12 \, \text{mm} \)
- **Nominal tensile strength** \( n_o m_t=1600 \, \text{N/mm}^2 \)
- **Nominal steel area** \( n_o m_a=90 \, \text{mm}^2 \)
- **Char breaking load (strand)** \( f_{pk}=160 \, \text{kN} \)
- **Initial force per strand** \( i_{pf}=100 \, \text{kN} \)
- **Modulus of elasticity (strand)** \( E_p=195 \, \text{kN/mm}^2 \)

**Cross-section 1**

- **Distance from mid-span of beam** \( c_s d(1)=0.5 \, \text{m} \)
- **Number of layers of strand** \( n_{ls}(1)=3 \)
- **Height above soffit** \( d_s(1)=60 \, \text{mm} \)
- **Number of strands** \( n_s(1)=6 \)
- **Height above soffit** \( d_s(2)=100 \, \text{mm} \)
- **Number of strands** \( n_s(2)=8 \)
- **Height above soffit** \( d_s(3)=180 \, \text{mm} \)
- **Number of strands** \( n_s(3)=3 \)
- **Bending moment from temp.loads** \( t_l b(1)=100 \, \text{kNm} \)
Bending moment from other loads \( olb(1) = 150 \text{ kNm} \)

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer ( % ) \( \text{str}=1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>17</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>83.907</td>
<td>4.9357</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>100.91</td>
<td>5.9357</td>
</tr>
</tbody>
</table>

Losses at Transfer

- Initial force: 1700 kN
- Force at transfer: \( P_t = P_0 - loft = 1599.1 \text{ kN} \)

**Stresses at Transfer N/mm\(^2\)**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-3.6892</td>
<td>12.163</td>
<td>9.2808</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>2.2746</td>
<td>-1.8513</td>
<td>-1.1012</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>3.7231</td>
<td>-3.0303</td>
<td>-1.8024</td>
</tr>
<tr>
<td>Other Loads</td>
<td>5.5846</td>
<td>-4.5455</td>
<td>-2.7036</td>
</tr>
<tr>
<td>Total</td>
<td>7.893</td>
<td>2.7359</td>
<td>3.6736</td>
</tr>
</tbody>
</table>

**Beam Concrete Stresses at Transfer**

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

- Top of beam:
  - Compressive stress 7.893 N/mm\(^2\) is less than allowable stresses of 0.7*fcki=21 N/mm\(^2\) and 0.6*fck=21.6 N/mm\(^2\).
- Bottom of beam:
  - Compressive stress 2.7359 N/mm\(^2\) is less than allowable stresses of 0.7*fcki=21 N/mm\(^2\) and 0.6*fck=21.6 N/mm\(^2\).

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

- Relaxation after transfer ( % ) \( \text{slr}(1) = 0.8 \)
- Shrinkage per unit length \( \text{sul}(1) = 0.3 \text{E-3} \)
- Creep coefficient \( \varphi(t_o) \) \( \text{crf} = 2 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>17</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>83.907</td>
<td>4.9357</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>100.91</td>
<td>5.9357</td>
</tr>
<tr>
<td>Relaxation</td>
<td>13.6</td>
<td>0.8</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>89.505</td>
<td>5.265</td>
</tr>
<tr>
<td>Creep</td>
<td>96.104</td>
<td>5.6532</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>199.21</td>
<td>11.718</td>
</tr>
<tr>
<td>All Losses</td>
<td>300.12</td>
<td>17.654</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 1700 kN
Final force Pf=P0-loftl=1399.9 kN
Unit-weight of in-situ concrete wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.2297</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>2.2746</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>1.0281</td>
</tr>
<tr>
<td>Total</td>
<td>0.073062</td>
</tr>
</tbody>
</table>

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=520 mm
Perc'age of shrinkage remaining ps=50 %
Percentage pcr=50
Percentage stT=45 %
The net moment is Mc=F*e/1E3-Mb=-167.95 kNm
Stress at top of in-situ concrete:
ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3
=-1.4431 N/mm²
Stress at bottom of in-situ concrete:
fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
=-0.24828 N/mm²
Stress at top of precast concrete beam:
Stress at bottom of precast concrete beam:

\[
\text{ftpbd} = \frac{F}{(A_i \cdot mr + A_c)} + \frac{M_c \cdot 1E3 \cdot (d - Y_c)}{I_c} + \frac{M_b \cdot 1E3 \cdot Y_t}{I_{yy}} \cdot 1E3
\]

\[
= 2.2317 \text{ N/mm}^2
\]

Loading Combination 1

(no tensile stress permitted in precast beam)

Characteristic bending moment \(c_{lb(1)} = 300 \text{ kNm}\)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of in-situ</td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-1.4431</td>
</tr>
<tr>
<td>Total</td>
<td>4.3687</td>
</tr>
</tbody>
</table>

Loading Combination 2

(tensile stress permitted (no visible cracking))

Characteristic bending moment \(c_{2b(1)} = 500 \text{ kNm}\)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of in-situ</td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>-1.4431</td>
</tr>
<tr>
<td>Total</td>
<td>5.8216</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 5.8216 N/mm² is less than the maximum allowable stress of 0.6*fckc=14.4 N/mm².

Top of beam:
Compressive stress 7.1028 N/mm² is less than the maximum allowable stress of 0.6*fck=21.6 N/mm².
Bottom of in-situ concrete:
Compressive stress 3.4595 N/mm² is less than the maximum allowable stress of 0.6*fckc=14.4 N/mm².
Bottom of beam:
Tensile stress 1.7224 N/mm² is less than the maximum allowable tensile stress of 2.9399 N/mm².
Location: Ex1 - 13.5 metre span TYE7 beam with solid infill

Precast prestressed concrete TYE beams with in-fill concrete

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=13.5 m

Beam TYE 7 section properties

- Depth of beam d=700 mm
- Cross-sectional area Ac=395966 mm²
- Centroid above soffit Yb=321.9 mm
- Section modulus top Zt=45.52E6 mm³
- Section modulus bottom Zb=53.49E6 mm³
- Unit weight of beam uwb=25 kN/m³

Concrete strengths

- Strength at transfer fci=40 N/mm²
- Characteristic strength fcu=50 N/mm²

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 40 N/mm² is: Eci=31 kN/mm²
- Modulus of elasticity for concrete strength 50 N/mm² is: Ecb=34 kN/mm²

Composite section properties

- Char strength (in-situ concrete) fcuc=40 N/mm²
- Modulus of elasticity for concrete strength 40 N/mm² is: Ecc=31 kN/mm²

Composite section is rectangular in shape.

![Composite section diagram]

- Breadth of rectangular section b=765 mm
- Depth of rectangular section di=875 mm
- Second moment of area Ic=Ixx+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi-Yc)^2)=42.708E9 mm⁴
Initial force per strand \( if = 116 \text{ kN} \)
Modulus of elasticity \( Es = 200 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( csd(1) = 0 \text{ m} \)
Number of layers of strand \( nls(1) = 4 \)
Height above soffit \( ds(1) = 60 \text{ mm} \)
Number of strands \( ns(1) = 11 \)
Height above soffit \( ds(2) = 100 \text{ mm} \)
Number of strands \( ns(2) = 9 \)
Height above soffit \( ds(3) = 140 \text{ mm} \)
Number of strands \( ns(3) = 5 \)
Height above soffit \( ds(4) = 650 \text{ mm} \)
Number of strands \( ns(4) = 2 \)

**Short Term Losses**

Relaxation at transfer ( % ) \( str = 1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.32</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>232.22</td>
<td>7.4146</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>263.54</td>
<td>8.4146</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 3132 kN
Force at transfer \( Pt = P0 - loft = 2868.5 \text{ kN} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.7267</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.9521</td>
</tr>
<tr>
<td>Total</td>
<td>0.22544</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 0.22544 N/mm² is less than allowable stresses of 0.5*fcit=20 N/mm² and 0.4*fct=20 N/mm².
Bottom of beam:
Compressive stress 13.22 N/mm² is less than allowable stresses
of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Long Term Losses

Relaxation after transfer( % )    slr(1)=1.5
Shrinkage per unit length         sul(1)=0.3E-3

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.32</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>232.22</td>
<td>7.4146</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>263.54</td>
<td>8.4146</td>
</tr>
<tr>
<td>Relaxation</td>
<td>46.98</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>150.66</td>
<td>4.8103</td>
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<tr>
<td>Creep</td>
<td>259.67</td>
<td>8.2908</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>457.31</td>
<td>14.601</td>
</tr>
<tr>
<td>All Losses</td>
<td>720.85</td>
<td>23.016</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force         3132 kN
Final force           Pf=P0-loftl=2411.1 kN

Unit-weight of in-situ concrete  wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.9731</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.9521</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.4193</td>
</tr>
<tr>
<td>Total</td>
<td>4.3983</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=135 mm
**Combination 1 loading**

Applied bending moment  \( c_{1b}(1) = 320 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 1</td>
<td>3.6879</td>
</tr>
<tr>
<td>Total</td>
<td>3.6879</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 loading**

Applied bending moment  \( c_{2b}(1) = 400 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>4.0976</td>
</tr>
<tr>
<td>Total</td>
<td>4.0976</td>
</tr>
</tbody>
</table>

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 4.0976 N/mm\(^2\) is less than the maximum allowable stress of 0.5*fcuc=20 N/mm\(^2\).

Top of beam:
Compressive stress 6.8569 N/mm\(^2\) is less than the maximum allowable stress of 0.4*fcu=20 N/mm\(^2\).
Bottom of in-situ concrete:
Tensile stress 2.8332 N/mm\(^2\) is less than the maximum allowable tensile stress of 4.4 N/mm\(^2\) from Table 32.
Bottom of beam:
Compressive stress 3.4312 N/mm\(^2\) is less than the maximum allowable stress of 0.4*fcu=20 N/mm\(^2\).
Location: Ex2 - 13.5 metre span TYE7 edge beam with solid infill

Precast prestressed concrete TYE beams with in-fill concrete

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=13.5 m

Beam TYE 7 section properties

Depth of beam d=700 mm
Cross-sectional area Ac=395966 mm²
Centroid above soffit Yb=321.9 mm
Section modulus top Zt=45.52E6 mm³
Section modulus bottom Zb=53.49E6 mm³
Unit weight of beam uwb=25 kN/m³

Concrete strengths

Strength at transfer fc=40 N/mm²
Characteristic strength fcu=50 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 40 N/mm² is: Eci=31 kN/mm²
Modulus of elasticity for concrete strength 50 N/mm² is: Ecb=34 kN/mm²

Composite section properties

Char strength (in-situ concrete) fcuc=30 N/mm²
Modulus of elasticity for concrete strength 30 N/mm² is: Ecc=28 kN/mm²

Item No. 1

Item weighting iw1=0.82353
X co-ordinate X(1)=0 mm
Y co-ordinate Y(1)=0 mm
X co-ordinate X(2)=765 mm
Y co-ordinate Y(2)=0 mm
X co-ordinate X(3)=765 mm
Y co-ordinate Y(3)=250 mm
X co-ordinate X(4)=915 mm
Y co-ordinate Y(4)=350 mm
X co-ordinate X(5)=915 mm
Y co-ordinate Y(5)=875 mm
X co-ordinate X(6)=415 mm
Y co-ordinate Y(6)=875 mm
X co-ordinate X(7)=415 mm
Y co-ordinate Y(7)=775 mm
X co-ordinate X(8)=0 mm
Y co-ordinate  \( Y(8) = 775 \text{ mm} \)
X co-ordinate  \( X(9) = 0 \text{ mm} \)
Y co-ordinate  \( Y(9) = 0 \text{ mm} \)

**Item No. 2**

- Second moment of area  \( I_{XX2} = 17.218 \times 10^9 \text{ mm}^4 \)
- Area of item  \( A_2 = 395966 \text{ mm}^2 \)
- Centroid from X-X axis  \( Y_C2 = 321.9 \text{ mm} \)
- Item weighting  \( i_{w2} = 0.17647 \)
- Depth of composite section  \( d_i = 775 \text{ mm} \)
- Nominal diameter  \( n_{omd} = 12 \text{ mm} \)
- Nominal tensile strength  \( n_{omt} = 1700 \text{ N/mm}^2 \)
- Nominal steel area  \( n_{oma} = 100 \text{ mm}^2 \)
- Characteristic breaking load  \( c_{bl} = 150 \text{ kN} \)
- Initial force per strand  \( i_f = 100 \text{ kN} \)
- Modulus of elasticity  \( E_s = 200 \text{ kN/mm}^2 \)

**Cross-section 1**

- Distance from mid-span of beam  \( c_{sd}(1) = 2 \text{ m} \)
- Number of layers of strand  \( n_{ls}(1) = 4 \)
- Height above soffit  \( d_s(1) = 60 \text{ mm} \)
- Number of strands  \( n_s(1) = 11 \)
- Height above soffit  \( d_s(2) = 100 \text{ mm} \)
- Number of strands  \( n_s(2) = 13 \)
- Height above soffit  \( d_s(3) = 140 \text{ mm} \)
- Number of strands  \( n_s(3) = 8 \)
- Height above soffit  \( d_s(4) = 650 \text{ mm} \)
- Number of strands  \( n_s(4) = 2 \)
- Bending moment from temp.loads  \( t_{lb}(1) = 80 \text{ kNm} \)
- Bending moment from other loads  \( o_{lb}(1) = 50 \text{ kNm} \)

**Short Term Losses**

Relaxation at transfer ( % )  \( \text{str} = 1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>34</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>346.32</td>
<td>10.186</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>380.32</td>
<td>11.186</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force  \( 3400 \text{ kN} \)
Force at transfer  \( P_t = P_0 - \text{loft} = 3019.7 \text{ kN} \)
## Stress Limitations

**Top of beam:**
Compressive stress 2.1954 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

**Bottom of beam:**
Compressive stress 12.25 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

## Long Term Losses

Relaxation after transfer ( % ) \( slr(1)=1.5 \)
Shrinkage per unit length \( sul(1)=0.3E-3 \)

## Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>34</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>346.32</td>
<td>10.186</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>380.32</td>
<td>11.186</td>
</tr>
<tr>
<td>Relaxation</td>
<td>51</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>204</td>
<td>6</td>
</tr>
<tr>
<td>Creep</td>
<td>374.41</td>
<td>11.012</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>629.41</td>
<td>18.512</td>
</tr>
<tr>
<td>All Losses</td>
<td>1009.7</td>
<td>29.698</td>
</tr>
</tbody>
</table>

**Losses - Long term**

Initial force 3400 kN
Final force \( Pf=P0-loftl=2390.3 \) kN

Unit-weight of in-situ concrete \( wc=25 \) kN/m³
## Composite section

Height from beam soffit to bottom of in-situ concrete: $h_{sis}=135$ mm  
Percentage of shrinkage remaining: $p_s=40\%$  
Percentage: $p_{cr}=40 \%$  
Percentage: $s_{TT}=50\%$

Apply moment to beam to straighten it as follows:

The net moment is $M_c=F\cdot e/1E3-M_b=-141.24$ kNm

Stress at top of in-situ concrete:

$$f_{tisds}=-(F/(A_i\cdot mr)-F/(A_i\cdot mr+Ac)+Mc*1E3*(di-Yc)/Ic)*mr*1E3$$

Stress at bottom of in-situ concrete:

$$f_{bisds}=-(F/(A_i\cdot mr)-F/(A_i\cdot mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3$$

Stress at top of precast concrete beam:

$$f_{tpbds}=(F/(A_i\cdot mr+Ac)-Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3$$

Stress at bottom of precast concrete beam:

$$f_{pbpds}=(F/(A_i\cdot mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3=-0.61187 \text{ N/mm}^2$$

## Combination 1 loading

Applied bending moment $c_{lb(1)}=550$ kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage</td>
<td>-0.45825</td>
</tr>
<tr>
<td>Combination 1</td>
<td>4.677</td>
</tr>
<tr>
<td>Total</td>
<td>4.2187</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses
Combination 2 to 5 loading

Applied bending moment \( c2b(1) = 625 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam</td>
<td>0</td>
<td>4.0494</td>
<td>0</td>
<td>7.7284</td>
</tr>
<tr>
<td>and in-situ Dif. shrinkage</td>
<td>-0.45825</td>
<td>0.41178</td>
<td>1.5333</td>
<td>-0.61187</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>4.8718</td>
<td>4.6617</td>
<td>-3.9412</td>
<td>-7.0432</td>
</tr>
<tr>
<td>Total</td>
<td>4.4136</td>
<td>9.1229</td>
<td>-2.4079</td>
<td>0.073314</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 4.4136 N/mm² is less than the maximum allowable stress of 0.5*\( f_{cuc} = 15 \) N/mm².

Top of beam:
Compressive stress 9.1229 N/mm² is less than the maximum allowable stress of 0.4*\( f_{cu} = 20 \) N/mm².

Bottom of in-situ concrete:
Tensile stress 2.4079 N/mm² is less than the maximum allowable tensile stress of 3.6 N/mm² from Table 32.

Bottom of beam:
Compressive stress 0.073314 N/mm² is less than the maximum allowable stress of 0.4*\( f_{cu} = 20 \) N/mm².
**Location:** Ex3 - 15.0 metre span TYE9 beam

**Precast prestressed concrete TYE beams with in-fill concrete**

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam \( \text{span}=15 \text{ m} \)

**Beam TYE 9 section properties**

- **Depth of beam** \( d=800 \text{ mm} \)
- **Cross-sectional area** \( Ac=451581 \text{ mm}^2 \)
- **Centroid above soffit** \( Yb=374.03 \text{ mm} \)
- **Section modulus top** \( Zt=61.14E6 \text{ mm}^3 \)
- **Section modulus bottom** \( Zb=69.62E6 \text{ mm}^3 \)
- **Unit weight of beam** \( uwb=25 \text{ kN/m}^3 \)

**Concrete strengths**

- **Strength at transfer** \( fci=35 \text{ N/mm}^2 \)
- **Characteristic strength** \( fcu=40 \text{ N/mm}^2 \)

**Concrete moduli**

Modulus of elasticity at transfer for concrete strength 35 N/mm\(^2\) is: \( Eci=29.5 \text{ kN/mm}^2 \)

Modulus of elasticity for concrete strength 40 N/mm\(^2\) is: \( Ecb=31 \text{ kN/mm}^2 \)

**Composite section properties**

Char strength (in-situ concrete) \( fcuc=35 \text{ N/mm}^2 \)

Modulus of elasticity for concrete strength 35 N/mm\(^2\) is: \( Ecc=29.5 \text{ kN/mm}^2 \)

Composite section is rectangular in shape.

\[ \begin{align*}
\text{TYE 9} & \quad \text{beam} \\
\text{In-situ} & \quad \text{concrete} \\
\text{breadth of rectangular section} & \quad b=765 \text{ mm} \\
\text{depth of rectangular section} & \quad di=875 \text{ mm} \\
\text{second moment of area} & \quad Ic=Ixx+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi-Yc)^2)=42.708E9 \text{ mm}^4
\end{align*} \]
Initial force per strand $i_f = 130$ kN
Modulus of elasticity $E_s = 200$ kN/mm$^2$

**Cross-section 1**

Distance from mid-span of beam $c_{sd}(1) = 6$ m
Number of layers of strand $n_{ls}(1) = 6$
Height above soffit $d_s(1) = 60$ mm
Number of strands $n_s(1) = 9$
Height above soffit $d_s(2) = 100$ mm
Number of strands $n_s(2) = 9$
Height above soffit $d_s(3) = 140$ mm
Number of strands $n_s(3) = 5$
Height above soffit $d_s(4) = 650$ mm
Number of strands $n_s(4) = 2$
Height above soffit $d_s(5) = 700$ mm
Number of strands $n_s(5) = 2$
Height above soffit $d_s(6) = 750$ mm
Number of strands $n_s(6) = 2$

**Short Term Losses**

Relaxation at transfer ( % ) $s_{tr} = 1$

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>37.7</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>230.56</td>
<td>6.1156</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>268.26</td>
<td>7.1156</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force $3770$ kN
Force at transfer $P_t = P_0 - loft = 3501.7$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.1478</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>1.8699</td>
</tr>
<tr>
<td>Total</td>
<td>0.72202</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress $0.72202$ N/mm$^2$ is less than allowable stresses
of $0.5\times f_{ci}=17.5 \text{ N/mm}^2$ and $0.4\times f_{cu}=16 \text{ N/mm}^2$.

Bottom of beam:
Compressive stress $13.929 \text{ N/mm}^2$ is less than allowable stresses
of $0.5\times f_{ci}=17.5 \text{ N/mm}^2$ and $0.4\times f_{cu}=16 \text{ N/mm}^2$.

**Long Term Losses**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>37.7</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>230.56</td>
<td>6.1156</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>268.26</td>
<td>7.1156</td>
</tr>
<tr>
<td>Relaxation</td>
<td>75.4</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>174</td>
<td>4.6154</td>
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<tr>
<td>Creep</td>
<td>360.2</td>
<td>9.5543</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>609.6</td>
<td>16.17</td>
</tr>
<tr>
<td>All Losses</td>
<td>877.85</td>
<td>23.285</td>
</tr>
</tbody>
</table>

Losses - Long term

*Initial force* 3770 kN
*Final force* $P_f=P_0-loftl=2892.1 \text{ kN}$

Unit-weight of in-situ concrete $w_c=25 \text{ kN/m}^3$

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.94802</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>1.8699</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>0.90182</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>1.8237</strong></td>
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</tbody>
</table>

Beam concrete stresses - Long term

**Composite section**

Height from beam soffit to bottom of in-situ concrete $h_{sis}=135 \text{ mm}$
**Combination 1 loading**

Applied bending moment \( c_{lb}(1)=200 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 1</td>
<td>2.3049</td>
</tr>
<tr>
<td>Total</td>
<td>2.3049</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 loading**

Applied bending moment \( c_{lb}(1)=250 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>2.561</td>
</tr>
<tr>
<td>Total</td>
<td>2.561</td>
</tr>
</tbody>
</table>

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 2.561 N/mm² is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm².

Top of beam:
Compressive stress 3.9456 N/mm² is less than the maximum allowable stress of 0.4*fcu=16 N/mm².

Bottom of in-situ concrete:
Tensile stress 1.7708 N/mm² is less than the maximum allowable tensile stress of 4 N/mm² from Table 32.

Bottom of beam:
Compressive stress 7.8657 N/mm² is less than the maximum allowable stress of 0.4*fcu=16 N/mm².
Location: Ex1 - 13.5 metre span TYE7 beam with solid infill

Precast prestressed concrete TYE beams with in-fill concrete


Beam span under consideration\( \text{span}=13.5 \text{ m} \)

**Beam TYE 7 section properties**

- Depth of beam \( d=700 \text{ mm} \)
- Cross-sectional area \( A_c=395966 \text{ mm}^2 \)
- Centroid above soffit \( y_b=321.9 \text{ mm} \)
- Section modulus top \( Z_t=45.52E6 \text{ mm}^3 \)
- Section modulus bottom \( Z_b=53.49E6 \text{ mm}^3 \)
- Unit weight of beam \( u_w=25 \text{ kN/m}^3 \)

**Concrete compressive strengths**

- Characteristic cylinder strength \( f_{cki}=32 \text{ N/mm}^2 \) (transfer)
- Characteristic cylinder strength \( f_{ck}=40 \text{ N/mm}^2 \)

**Concrete moduli (short-term)**

- Concrete modulus of elasticity \( E_{cmi}=33 \text{ kN/mm}^2 \) (transfer)
- Concrete modulus of elasticity \( E_{cmb}=35 \text{ kN/mm}^2 \)

**Composite section properties**

- Char strength (in-situ concrete) \( f_{ckc}=32 \text{ N/mm}^2 \)
- Short-term modulus of elasticity for in-situ concrete \( E_{cmc}=33 \text{ kN/mm}^2 \)
- Composite section is rectangular in shape.

![Composite Section Diagram]

- Breadth of rectangular section \( b=765 \text{ mm} \)
- Depth of rectangular section \( d_i=875 \text{ mm} \)
- Second moment of area \( I_c=I_{yy}+A_c(y_b-Y_c)^2+m_r(I_{ii}+A_i(y_i-Y_c)^2)=42.708E9 \text{ mm}^4 \)
- Modulus of elasticity (strand) \( E_p=195 \text{ kN/mm}^2 \)
Cross-section 1

Distance from mid-span of beam \( csd(1)=0 \) m
Number of layers of strand \( nls(1)=4 \)
Height above soffit \( ds(1)=60 \) mm
Number of strands \( ns(1)=11 \)
Height above soffit \( ds(2)=100 \) mm
Number of strands \( ns(2)=9 \)
Height above soffit \( ds(3)=140 \) mm
Number of strands \( ns(3)=5 \)
Height above soffit \( ds(4)=650 \) mm
Number of strands \( ns(4)=2 \)

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % ) \( str=1 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.32</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>199.04</td>
<td>6.3551</td>
</tr>
<tr>
<td>Total</td>
<td>230.36</td>
<td>7.3551</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \( 3132 \) kN
Force at transfer \( P_t=P_0-loft=2901.6 \) kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.7813</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.9521</td>
</tr>
<tr>
<td>Total</td>
<td>0.17076</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam:
Compressive stress 0.17076 N/mm\(^2\) is less than allowable stresses of 0.7*fck=22.4 N/mm\(^2\) and 0.6*fck=24 N/mm\(^2\).

Bottom of beam:
Compressive stress 13.421 N/mm\(^2\) is less than allowable stresses of 0.7*fck=22.4 N/mm\(^2\) and 0.6*fck=24 N/mm\(^2\).
Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer(%)  slr(1)=1.5
Shrinkage per unit length   sul(1)=0.3E-3
Creep coefficient ϕ(=,to)    crf=2

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.32</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>199.04</td>
<td>6.3551</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>230.36</td>
<td>7.3551</td>
</tr>
<tr>
<td>Relaxation</td>
<td>46.98</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>146.89</td>
<td>4.6901</td>
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<tr>
<td>Creep</td>
<td>305.69</td>
<td>9.7603</td>
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<tr>
<td>Long-term Total</td>
<td>499.57</td>
<td>15.95</td>
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<tr>
<td>All Losses</td>
<td>729.93</td>
<td>23.306</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 3132 kN
Final force Pf=P0-loft1=2402.1 kN

Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.9581</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.9521</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.4193</td>
</tr>
<tr>
<td>Total</td>
<td>4.4133</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=135 mm
Loading Combination 1

(no tensile stress permitted in precast beam)
Characteristic bending moment \( c_{lb}(1) = 320 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 1</td>
<td>3.6879</td>
</tr>
<tr>
<td>Total</td>
<td>3.6879</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses

Loading Combination 2

(tensile stress permitted (no visible cracking))
Characteristic bending moment \( c_{lb}(1) = 400 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 2</td>
<td>4.0976</td>
</tr>
<tr>
<td>Total</td>
<td>4.0976</td>
</tr>
</tbody>
</table>

Loading Combination 2 - Stresses

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 4.0976 N/mm\(^2\) is less than the maximum allowable stress of \( 0.6f_{ckc} = 19.2 \text{ N/mm}^2 \).

Top of beam:
Compressive stress 6.8719 N/mm\(^2\) is less than the maximum allowable stress of \( 0.6f_{ck} = 24 \text{ N/mm}^2 \).

Bottom of in-situ concrete:
Tensile stress 2.8332 N/mm\(^2\) is less than the maximum allowable tensile stress of \( 3.0231 \text{ N/mm}^2 \) (EN 1992-1-1 expression 3.23).

Bottom of beam:
Compressive stress 3.3761 N/mm\(^2\) is less than the maximum allowable stress of \( 0.6f_{ck} = 24 \text{ N/mm}^2 \).
Location: Ex2 - 13.5 metre span TYE7 edge beam with solid infill

Precast prestressed concrete TYE beams with in-fill concrete


Beam span under consideration  \( \text{span}=13.5 \text{ m} \)

**Beam TYE 7 section properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>( d=700 \text{ mm} )</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>( A_c=395966 \text{ mm}^2 )</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>( y_b=321.9 \text{ mm} )</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>( Z_t=45.52E6 \text{ mm}^3 )</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>( Z_b=53.49E6 \text{ mm}^3 )</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>( u_w=b=25 \text{ kN/m}^3 )</td>
</tr>
</tbody>
</table>

**Concrete compressive strengths**

- *Characteristic cylinder strength* \( f_{ck,i}=32 \text{ N/mm}^2 \) (transfer)
- *Characteristic cylinder strength* \( f_{ck}=40 \text{ N/mm}^2 \)

**Concrete moduli (short-term)**

- *Concrete modulus of elasticity* \( E_{cm,i}=33 \text{ kN/mm}^2 \) (transfer)
- *Concrete modulus of elasticity* \( E_{cm,b}=35 \text{ kN/mm}^2 \)

**Composite section properties**

- *Char strength (in-situ concrete)* \( f_{ckc}=24 \text{ N/mm}^2 \)
- *Short-term modulus of elasticity for in-situ concrete* \( E_{cmc}=31 \text{ kN/mm}^2 \)

**Item No. 1**

<table>
<thead>
<tr>
<th>Item weighting</th>
<th>( iw_1=0.88571 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>X co-ordinate</td>
<td>( X(1)=0 \text{ mm} )</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>( Y(1)=0 \text{ mm} )</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>( X(2)=765 \text{ mm} )</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>( Y(2)=0 \text{ mm} )</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>( X(3)=765 \text{ mm} )</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>( Y(3)=250 \text{ mm} )</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>( X(4)=915 \text{ mm} )</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>( Y(4)=350 \text{ mm} )</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>( X(5)=915 \text{ mm} )</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>( Y(5)=875 \text{ mm} )</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>( X(6)=415 \text{ mm} )</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>( Y(6)=875 \text{ mm} )</td>
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<tr>
<td>X co-ordinate</td>
<td>( X(7)=415 \text{ mm} )</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>( Y(7)=775 \text{ mm} )</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>( X(8)=0 \text{ mm} )</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>( Y(8)=775 \text{ mm} )</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>( X(9)=0 \text{ mm} )</td>
</tr>
</tbody>
</table>
Y co-ordinate

Y(9)=0 mm

**Item No. 2**

Second moment of area IXX2=17.218E9 mm^4
Area of item A2=395966 mm^2
Centroid from X-X axis YC2=321.9 mm
Item weighting iw2=0.17647
Depth of composite section di=775 mm
Nominal diameter nomd=12 mm
Nominal tensile strength nomt=1700 N/mm^2
Nominal steel area noma=100 mm^2
Char breaking load (per strand) fpk=150 kN
Initial force (per strand) ipf=100 kN
Modulus of elasticity (strand) Ep=195 kN/mm^2

**Cross-section 1**

Distance from mid-span of beam csd(1)=2 m
Number of layers of strand nls(1)=4
Height above soffit ds(1)=60 mm
Number of strands ns(1)=11
Height above soffit ds(2)=100 mm
Number of strands ns(2)=13
Height above soffit ds(3)=140 mm
Number of strands ns(3)=8
Height above soffit ds(4)=650 mm
Number of strands ns(4)=2
Bending moment from temp.loads tlb(1)=80 kNm
Bending moment from other loads olb(1)=50 kNm

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer ( % ) str=1

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force (kN)</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>34</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>289.88</td>
<td>8.526</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>323.88</td>
<td>9.526</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 3400 kN
Force at transfer Pt=P0-loft=3076.1 kN
Reinforced & prestressed concrete: BS5400 & Eurocode

TYE beam with in-situ infill

Date: 02/12/19

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---

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-5.2734</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.5173</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>1.7567</td>
</tr>
<tr>
<td>Other Loads</td>
<td>1.098</td>
</tr>
<tr>
<td>Total</td>
<td>2.0986</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
Compressive stress 2.0986 N/mm² is less than allowable stresses
of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

Bottom of beam:
Compressive stress 12.596 N/mm² is less than allowable stresses
of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer ( % )  slr(1)=1.5
Shrinkage per unit length        sul(1)=0.3E-3
Creep coefficient φ(ω,to)        crf=2

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>34</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>289.88</td>
<td>8.526</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>323.88</td>
<td>9.526</td>
</tr>
<tr>
<td>Relaxation</td>
<td>51</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>198.9</td>
<td>5.85</td>
</tr>
<tr>
<td>Creep</td>
<td>438</td>
<td>12.882</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>687.9</td>
<td>20.232</td>
</tr>
<tr>
<td>All Losses</td>
<td>1011.8</td>
<td>29.758</td>
</tr>
</tbody>
</table>

**Losses - Long term**

Initial force 3400 kN
Final force Pf=P0-loftl=2388.2 kN

Unit-weight of in-situ concrete wc=25 kN/m³
## Composite section

Height from beam soffit to bottom of in-situ concrete: $h_{sis} = 135$ mm

Percentage of shrinkage remaining: $p_s = 40\%$

Percentage of shrinkage remaining: $p_{cr} = 40$

Percentage of shrinkage remaining: $s_T = 50\%$

The net moment is: $M_c = F \times e / 1E3 - M_b = -189.54$ kNm

Stress at top of in-situ concrete:

$$f_{tisds} = -\left(\frac{F}{(A_i \times mr)} - \frac{F}{(A_i \times mr + Ac)} - Mc \times 1E3 \times (d_i - Yc) / Ic \right) \times mr \times 1E3 = -0.49353 \text{ N/mm}^2$$

Stress at bottom of in-situ concrete:

$$f_{bisds} = -\left(\frac{F}{(A_i \times mr)} - \frac{F}{(A_i \times mr + Ac)} - Mc \times 1E3 \times (h_{sis} - Yc) / Ic \right) \times mr \times 1E3 = 2.1981 \text{ N/mm}^2$$

Stress at top of precast concrete beam:

$$f_{tpbds} = \left(\frac{F}{(A_i \times mr + Ac)} + Mc \times 1E3 \times (d - Yc) / Ic + Mb \times 1E3 \times Yt / Iyy \right) \times 1E3 = 0.27525 \text{ N/mm}^2$$

Stress at bottom of precast concrete beam:

$$f_{bpbds} = \left(\frac{F}{(A_i \times mr + Ac)} - Mc \times 1E3 \times Yc / Ic - Mb \times 1E3 \times Yb / Iyy \right) \times 1E3 = -0.82932 \text{ N/mm}^2$$

### Loading Combination 1

(no tensile stress permitted in precast beam)

Characteristic bending moment: $c_{lb(1)} = 550$ kNm

## Final Stresses N/mm$^2$

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
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<tr>
<td>Prestress</td>
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<tr>
<td>Self weight of Beam</td>
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<td>In-situ concrete</td>
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<td><strong>Total</strong></td>
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## Loading Combination 1 - Stresses

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<tr>
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<td></td>
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<td><strong>Total</strong></td>
<td><strong>4.2065</strong></td>
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</table>
# Loading Combination 2

(tensile stress permitted (no visible cracking))

Characteristic bending moment  \( c_{2b(1)} = 625 \, \text{kN} \cdot \text{m} \)

<table>
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<tr>
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<th>Stresses N/mm²</th>
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</thead>
<tbody>
<tr>
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<td>Top of Beam</td>
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<tr>
<td>Prestress, beam and in-situ</td>
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<tr>
<td>Dif. shrinkage Combination 2</td>
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<td>4.8959</td>
<td>4.3534</td>
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<tr>
<td>Total</td>
<td>4.4024</td>
<td>8.6815</td>
</tr>
</tbody>
</table>

**NOTE:** The stresses from temperature differences should be added to the total stresses.

**Top of in-situ concrete:**
Compressive stress 4.4024 N/mm² is less than the maximum allowable stress of 0.6*fckc=14.4 N/mm².

**Top of beam:**
Compressive stress 8.6815 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

**Bottom of in-situ concrete:**
Tensile stress 1.7813 N/mm² is less than the maximum allowable tensile stress of 2.4956 N/mm² (EN 1992-1-1 expression 3.23).

**Bottom of beam:**
Compressive stress 0.27982 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².
Location: Ex3 - 15.0 metre span TYE9 beam

Precast prestressed concrete TYE beams with in-fill concrete


Beam span under consideration span=15 m

Beam TYE 9 section properties

- Depth of beam: $d=800$ mm
- Cross-sectional area: $A_c=451581$ mm$^2$
- Centroid above soffit: $Y_b=374.03$ mm
- Section modulus top: $Z_t=61.14 \times 10^6$ mm$^3$
- Section modulus bottom: $Z_b=69.62 \times 10^6$ mm$^3$
- Unit weight of beam: $u_{wb}=25$ kN/m$^3$

Concrete compressive strengths

- Characteristic cylinder strength (transfer): $f_{ck}=32$ N/mm$^2$
- Characteristic cylinder strength: $f_{cki}=28$ N/mm$^2$

Concrete moduli (short-term)

- Concrete modulus of elasticity (transfer): $E_{cm}=32$ kN/mm$^2$
- Concrete modulus of elasticity: $E_{cmb}=33$ kN/mm$^2$

Composite section properties

- Char strength (in-situ concrete): $f_{ckc}=28$ N/mm$^2$
- Short-term modulus of elasticity for in-situ concrete: $E_{cmc}=32$ kN/mm$^2$

Composite section is rectangular in shape.

Breadth of rectangular section: $b=765$ mm
Depth of rectangular section: $d_i=875$ mm
Second moment of area: $I_c=I_{yy}+A_c(Y_b-Y_c)^2+mr^2(I_{ii}+A_i(Y_i-Y_c)^2)=42.708 \times 10^9$ mm$^4$
Modulus of elasticity (strand): $E_p=195$ kN/mm$^2$
Cross-section 1

Distance from mid-span of beam  csd(1)=6 m  
Number of layers of strand  nls(1)=6  
Height above soffit  ds(1)=60 mm  
Number of strands  ns(1)=9  
Height above soffit  ds(2)=100 mm  
Number of strands  ns(2)=9  
Height above soffit  ds(3)=140 mm  
Number of strands  ns(3)=5  
Height above soffit  ds(4)=650 mm  
Number of strands  ns(4)=2  
Height above soffit  ds(5)=700 mm  
Number of strands  ns(5)=2  
Height above soffit  ds(6)=750 mm  
Number of strands  ns(6)=2

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % )  str=1

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>37.7</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>196.33</td>
<td>5.2077</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>234.03</td>
<td>6.2077</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force  3770 kN
Force at transfer  Pt=P0-loft=3536 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.1591</td>
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<tr>
<td>Self weight of Beam</td>
<td>1.8699</td>
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<tr>
<td>Total</td>
<td>0.7108</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations ( BS EN 1992-1-1, Clause 5.10.2.2 )

Top of beam:
Compressive stress 0.7108 N/mm² is less than allowable stresses of 0.7*fck=19.6 N/mm² and 0.6*fck=19.2 N/mm².
Bottom of beam:
Compressive stress 14.082 N/mm² is less than allowable stresses of 0.7*fcki=19.6 N/mm² and 0.6*fck=19.2 N/mm².

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer( % ) slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(∞,to) crf=2

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>37.7</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>196.33</td>
<td>5.2077</td>
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<tr>
<td>Transfer Total</td>
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<td>6.2077</td>
</tr>
<tr>
<td>Relaxation</td>
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<tr>
<td>Shrinkage</td>
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<tr>
<td>Creep</td>
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<td>All Losses</td>
<td>865.98</td>
<td>22.97</td>
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</table>

Losses - Long term

Initial force 3770 kN
Final force Pf=P0-loftl=2904 kN

Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
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<td>Top of Beam</td>
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<tr>
<td>Prestress</td>
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<td>Self weight of Beam</td>
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<td>In-situ concrete</td>
<td>0.90182</td>
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<tr>
<td>Total</td>
<td>1.8198</td>
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</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=135 mm
**Loading Combination 1**

(no tensile stress permitted in precast beam)  
Characteristic bending moment  \( c_{1b}(1)=200 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
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<tr>
<td>Prestress, beam and in-situ Combination 1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>2.3049</td>
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<tr>
<td>Total</td>
<td>2.3049</td>
</tr>
</tbody>
</table>

**Loading Combination 1 - Stresses**

**Loading Combination 2**

(tensile stress permitted (no visible cracking))  
Characteristic bending moment  \( c_{2b}(1)=250 \text{ kNm} \)

<table>
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<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
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</thead>
<tbody>
<tr>
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<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Combination 2</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>2.561</td>
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<tr>
<td>Total</td>
<td>2.561</td>
</tr>
</tbody>
</table>

**Loading Combination 2 - Stresses**

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:  
Compressive stress 2.561 N/mm² is less than the maximum allowable stress of 0.6*fckc=16.8 N/mm².  
Top of beam:  
Compressive stress 3.9418 N/mm² is less than the maximum allowable stress of 0.6*fck=19.2 N/mm².  
Bottom of in-situ concrete:  
Tensile stress 1.7708 N/mm² is less than the maximum allowable tensile stress of 2.7656 N/mm² (EN 1992-1-1 expression 3.23).  
Bottom of beam:  
Compressive stress 7.9185 N/mm² is less than the maximum allowable stress of 0.6*fck=19.2 N/mm².
Location: Ex1 - 34 metre span SY beam

Precast prestressed concrete SY beams with in-situ top slab

Calculations for the stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=34 m

Beam SY 4 section properties

Depth of beam d=1800 mm
Cross-sectional area Ac=644992 mm²
Centroid above soffit Yb=752.98 mm
Section modulus top Zt=199.16E6 mm⁴
Section modulus bottom Zb=276.91E6 mm⁴
Unit weight of beam uwb=24 kN/m³

Concrete strengths

Strength at transfer fci=40 N/mm²
Characteristic strength fcu=50 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 40 N/mm² is:
Eci=31 kN/mm²
Modulus of elasticity for concrete strength 50 N/mm² is:
Ecb=34 kN/mm²

Composite section properties

Char strength (in-situ concrete) fcuc=40 N/mm²
Modulus of elasticity for concrete strength 40 N/mm² is:
Ecc=31 kN/mm²
Sample output for SCALE Proforma 138. (ans=1)
Reinforced & prestressed concrete: BS5400 & Eurocode
SY beams with insitu top slab
Made by: IFB
Date: 02/12/19
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Second moment of area:
\[ I_c = I_{xx} + A_c (Y_b - Y_c)^2 + m_r (I_i + A_i (Y_i - Y_c)^2) = 441.35E9 \text{ mm}^4 \]

Depth of composite section \[ d_i = 1960 \text{ mm} \]
Initial force \[ i_f = 174 \text{ kN} \]
Modulus of elasticity \[ E_s = 200 \text{ kN/mm}^2 \]

**Cross-section 1**

Distance from mid-span of beam \[ c_{sd}(1) = 0 \text{ m} \]
Number of layers of strand \[ n_{ls}(1) = 4 \]
Height above soffit \[ d_s(1) = 60 \text{ mm} \]
Number of strands \[ n_s(1) = 11 \]
Height above soffit \[ d_s(2) = 110 \text{ mm} \]
Number of strands \[ n_s(2) = 12 \]
Height above soffit \[ d_s(3) = 160 \text{ mm} \]
Number of strands \[ n_s(3) = 12 \]
Height above soffit \[ d_s(4) = 210 \text{ mm} \]
Number of strands \[ n_s(4) = 12 \]

**Short Term Losses**

Relaxation at transfer ( % ) \[ s_{tr} = 1 \]

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>81.78</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
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<td>14.073</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>1232.6</td>
<td>15.073</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \[ 8178 \text{ kN} \]
Force at transfer \[ P_t = P_0 - \text{loft} = 6945.4 \text{ kN} \]
**Stresses at Transfer N/mm²**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-10.729</td>
<td>26.228</td>
<td>23.424</td>
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<tr>
<td>Self weight of Beam</td>
<td>11.232</td>
<td>-8.0778</td>
<td>-6.6125</td>
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<tr>
<td>Total</td>
<td>0.5033</td>
<td>18.15</td>
<td>16.811</td>
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</table>

**Stress Limitations**

Top of beam:
Compressive stress 0.5033 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Bottom of beam:
Compressive stress 18.15 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

**Long Term Losses**

Relaxation after transfer (% )  slr(1)=2
Shrinkage per unit length      sul(1)=0.3E-3

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
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<tbody>
<tr>
<td>Relaxation</td>
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<tr>
<td>Elastic Shortening</td>
<td>1150.9</td>
<td>14.073</td>
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<td>15.073</td>
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<tr>
<td>Relaxation</td>
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<tr>
<td>Shrinkage</td>
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<td>Creep</td>
<td>1244.8</td>
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<td>Long-term Total</td>
<td>1800.3</td>
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<tr>
<td>All Losses</td>
<td>3033</td>
<td>37.087</td>
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</tbody>
</table>

**Losses - Long term**

Initial force          8178 kN
Final force            Pf=P0-loftl=5145 kN

Unit-weight of in-situ concrete wc=24 kN/m³
Reinforced & prestressed concrete: BS5400 & Eurocode

Made by: IFB

SY beams with insitu top slab

Date: 02/12/19

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<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
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<tbody>
<tr>
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<tr>
<td>Prestress</td>
<td>-7.9479</td>
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<tr>
<td>Self weight of Beam</td>
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<tr>
<td>In-situ concrete</td>
<td>4.6079</td>
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<tr>
<td>Total</td>
<td>7.8923</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

- Height from beam soffit to bottom of in-situ concrete: $h_{sis}=1765$ mm
- Perc'age of shrinkage remaining: $ps=25\%$
- Percentage: $pcr=25$
- Percentage: $stT=67\%$
- The net moment is $Mc=F*e/1E3-Mb=-222.6$ kNm

Stress at top of in-situ concrete:
$$f_{tisds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(d-Yc)/Ic)*mr*1E3$$
$$=-0.23467 \text{ N/mm}^2$$

Stress at bottom of in-situ concrete:
$$f_{bisds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3$$
$$=-0.13632 \text{ N/mm}^2$$

Stress at top of precast concrete beam:
$$f_{tpbds}=F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3$$
$$=0.35464 \text{ N/mm}^2$$

Stress at bottom of precast concrete beam:
$$f_{pbds}=F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3$$
$$=-0.12332 \text{ N/mm}^2$$

**Combination 1 loading**

Applied bending moment: $clb(1)=3000$ kNm

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<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
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</thead>
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</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-0.23467</td>
</tr>
<tr>
<td>Total</td>
<td>6.3715</td>
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</table>

Combination 1 Loading - Stresses
Combination 2 to 5 loading

Applied bending moment $c2b(1) = 3500$ kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Prestress, beam and in-situ Dif. shrinkage</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ Dif. shrinkage</td>
<td>Prestress, beam and in-situ Dif. shrinkage</td>
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<tr>
<td>Combination 2-5</td>
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<td>-0.13632</td>
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<tr>
<td>7.0065</td>
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<td>5.4601</td>
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<tr>
<td>Total</td>
<td>Prestress, beam and in-situ Dif. shrinkage</td>
<td>6.7718</td>
<td>13.985</td>
<td>5.3238</td>
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</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 6.7718 N/mm² is less than the maximum allowable stress of $0.5 \times fcuc = 20$ N/mm².

Top of beam:
Compressive stress 13.985 N/mm² is less than the maximum allowable stress of $0.4 \times fcu = 20$ N/mm².

Bottom of in-situ concrete:
Compressive stress 5.3238 N/mm² is less than the maximum allowable stress of $0.5 \times fcuc = 20$ N/mm².

Bottom of beam:
Tensile stress 0.62222 N/mm² is less than the maximum allowable tensile stress of 3.182 N/mm².
Location: Ex2 - 24.5 metre span SY2 beam

Precast prestressed concrete SY beams with in-situ top slab

Calculations for the stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam: span=24.5 m

Beam SY 2 section properties

- Depth of beam: d=1600 mm
- Cross-sectional area: Ac=580992 mm²
- Centroid above soffit: Yb=650.06 mm
- Section modulus top: Zt=153.97E6 mm³
- Section modulus bottom: Zb=224.96E6 mm³
- Unit weight of beam: uwb=24.5 kN/m³

Concrete strengths

- Strength at transfer: fci=35 N/mm²
- Characteristic strength: fcu=45 N/mm²

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 35 N/mm² is: Eci=29.5 kN/mm²
- Modulus of elasticity for concrete strength 45 N/mm² is: Ecb=32.5 kN/mm²

Composite section properties

- Char strength (in-situ concrete): fcuc=35 N/mm²
- Modulus of elasticity for concrete strength 35 N/mm² is: Ecc=29.5 kN/mm²

Item No. 1

- X co-ordinate: X(1)=500 mm
- Y co-ordinate: Y(1)=0 mm
- X co-ordinate: X(2)=1250 mm
- Y co-ordinate: Y(2)=0 mm
- X co-ordinate: X(3)=1245 mm
- Y co-ordinate: Y(3)=252 mm
- X co-ordinate: X(4)=975 mm
- Y co-ordinate: Y(4)=429 mm
- X co-ordinate: X(5)=1035 mm
- Y co-ordinate: Y(5)=1550 mm
- X co-ordinate: X(6)=995 mm
- Y co-ordinate: Y(6)=1550 mm
- X co-ordinate: X(7)=995 mm
- Y co-ordinate: Y(7)=1570 mm
- X co-ordinate: X(8)=1800 mm
- Y co-ordinate: Y(8)=1570 mm
X co-ordinate                     X(9)=1800 mm
Y co-ordinate                     Y(9)=1900 mm
X co-ordinate                     X(10)=1300 mm
Y co-ordinate                     Y(10)=1900 mm
X co-ordinate                     X(11)=1300 mm
Y co-ordinate                     Y(11)=1800 mm
X co-ordinate                     X(12)=0 mm
Y co-ordinate                     Y(12)=1800 mm
X co-ordinate                     X(13)=0 mm
Y co-ordinate                     Y(13)=1570 mm
X co-ordinate                     X(14)=755 mm
Y co-ordinate                     Y(14)=1570 mm
X co-ordinate                     X(15)=755 mm
Y co-ordinate                     Y(15)=1550 mm
X co-ordinate                     X(16)=715 mm
Y co-ordinate                     Y(16)=1550 mm
X co-ordinate                     X(17)=775 mm
Y co-ordinate                     Y(17)=429 mm
X co-ordinate                     X(18)=505 mm
Y co-ordinate                     Y(18)=252 mm
X co-ordinate                     X(19)=500 mm
Y co-ordinate                     Y(19)=0 mm

Depth of composite section        di=1900 mm
Area of in-situ concrete          Ai=450198 mm²
Centroid of in-situ concrete      Yi=1713.4 mm
Initial force                     if=90 kN
Modulus of elasticity             Es=200 kN/mm²

Cross-section 1

Distance from mid-span of beam    csd(1)=5.6 m
Number of layers of strand       nls(1)=4
Height above soffit              ds(1)=60 mm
Number of strands                ns(1)=8
Height above soffit              ds(2)=110 mm
Number of strands                ns(2)=8
Height above soffit              ds(3)=160 mm
Number of strands                ns(3)=10
Height above soffit              ds(4)=596 mm
Number of strands                ns(4)=2

Short Term Losses

Relaxation at transfer ( % )      str=0.9
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22.68</td>
<td>0.9</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>122.41</td>
<td>4.8576</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>145.09</td>
<td>5.7576</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 2520 kN
Force at transfer $P_t = P_0 - \delta f = 2374.9$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Top of Beam</td>
<td>Bottom of Beam</td>
<td>Centroid of Strands</td>
</tr>
<tr>
<td>Prestress</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>1.8346</td>
<td>5.6295</td>
<td>5.2778</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 1.8346 N/mm² is less than allowable stresses of 0.5$f_{ci}$=17.5 N/mm² and 0.4$f_{cu}$=18 N/mm².

Bottom of beam:
Compressive stress 5.6295 N/mm² is less than allowable stresses of 0.5$f_{ci}$=17.5 N/mm² and 0.4$f_{cu}$=18 N/mm².

**Long Term Losses**

Relaxation after transfer( % ) $slr(1)=0.6$
Shrinkage per unit length $sul(1)=0.3E-3$
Sample output for SCALE Proforma 138. (ans=2)  
Reinforced prestressed concrete: BS5400 & Eurocode  
Made by: IFB  
SY beams with insitu top slab  
Date: 02/12/19  
Copyright 1986-2019 Fitzroy Systems Ltd.  
Ref No: SC138 BS

## Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22.68</td>
<td>0.9</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>122.41</td>
<td>4.8576</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>145.09</td>
<td>5.7576</td>
</tr>
<tr>
<td>Relaxation</td>
<td>15.12</td>
<td>0.6</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>126</td>
<td>5</td>
</tr>
<tr>
<td>Creep</td>
<td>121.6</td>
<td>4.8254</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>262.72</td>
<td>10.425</td>
</tr>
<tr>
<td>All Losses</td>
<td>407.81</td>
<td>16.183</td>
</tr>
</tbody>
</table>

### Losses - Long term

- **Initial force**: 2520 kN
- **Final force**: Pf=P0-loftl=2112.2 kN

**Unit-weight of in-situ concrete**: wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
<td>Bottom of Beam</td>
<td>Centroid of Strands</td>
<td></td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.2491</td>
<td>8.3467</td>
<td>7.272</td>
<td></td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.4879</td>
<td>-3.7554</td>
<td>-2.8988</td>
<td></td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>4.1656</td>
<td>-2.8506</td>
<td>-2.2004</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>6.4044</td>
<td>1.7407</td>
<td>2.1729</td>
<td></td>
</tr>
</tbody>
</table>

### Composite section

- Height from beam soffit to bottom of in-situ concrete: hsis=1570 mm
- Percentage of shrinkage remaining: ps=45 %
- Percentage: pcr=45
- Percentage: stT=50 %
- The net moment is: Mc=F*e/1E3-Mb=93.037 kNm

**Stress at top of in-situ concrete**:

\[
\text{ftisds} = \frac{(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3}{0.49975} \text{ N/mm}^2
\]

**Stress at bottom of in-situ concrete**:

\[
\text{fbisds} = \frac{(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3}{0.4289} \text{ N/mm}^2
\]

**Stress at top of precast concrete beam**:  

---

SCALE 5.48  
Office 1007  
Proforma 138
ftpbds=\frac{F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3}{1E3}=-1.7861 \text{ N/mm}^2

Stress at bottom of precast concrete beam:
\[ fbpbds=\frac{F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3}{1E3}=0.62227 \text{ N/mm}^2 \]

Combination 1 loading
Applied bending moment \( c_{lb}(1)=600 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage</td>
<td>0.49975</td>
</tr>
<tr>
<td>Combination 1</td>
<td>1.2692</td>
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<tr>
<td>Total</td>
<td>1.769</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

Combination 2 to 5 loading
Applied bending moment \( c_{2b}(1)=1000 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage</td>
<td>0.49975</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>1.8132</td>
</tr>
<tr>
<td>Total</td>
<td>2.3129</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 2.3129 N/mm² is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm².
Top of beam:
Compressive stress 5.7392 N/mm² is less than the maximum allowable stress of 0.4*fcu=18 N/mm².
Bottom of in-situ concrete:
Compressive stress 1.4805 N/mm² is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm².
Bottom of beam:

Tensile stress 0.20857 N/mm² is less than the maximum allowable tensile stress of 3.0187 N/mm².
Calculations for the stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam: span=40 m

Beam SY 6 section properties

- Depth of beam: d=2000 mm
- Cross-sectional area: Ac=708992 mm²
- Centroid above soffit: Yb=855.36 mm
- Section modulus top: Zt=247.88E6 mm⁴
- Section modulus bottom: Zb=331.64E6 mm⁴
- Unit weight of beam: uwb=25 kN/m³

Concrete strengths

- Strength at transfer: fci=35 N/mm²
- Characteristic strength: fcu=40 N/mm²

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 35 N/mm² is: Eci=29.5 kN/mm²
- Modulus of elasticity for concrete strength 40 N/mm² is: Ecb=31 kN/mm²

Composite section properties

- Char strength (in-situ concrete): fcuc=35 N/mm²
- Modulus of elasticity for concrete strength 35 N/mm² is: Ecc=29.5 kN/mm²
Reinforced & prestressed concrete: BS5400 & Eurocode

Made by: IFB
SY beams with insitu top slab
Date: 02/12/19

Copyright 1986-2019 Fitzroy Systems Ltd.
Ref No: SC138 BS

Area 1: 240
Area 2: 190

In-situ concrete

\[ \text{Area} 2 = 240 \times (50 - \text{dpf}) \text{ mm}^2 \]

Second moment of area:
\[ I_c = I_{xx} + A_c \times (Y_b - Y_c)^2 + m_r \times (I_i + A_i \times (Y_i - Y_c)^2) = 544.6 \times 10^9 \text{ mm}^4 \]

Depth of composite section:
\[ d_i = 2155 \text{ mm} \]

Initial force:
\[ i_f = 150 \text{ kN} \]

Modulus of elasticity:
\[ E_s = 200 \text{ kN/mm}^2 \]

Cross-section 1

Distance from mid-span of beam:
\[ c_{sd(1)} = 9.4 \text{ m} \]

Number of layers of strand:
\[ n_{ls(1)} = 4 \]

Height above soffit:
\[ d_{s(1)} = 55 \text{ mm} \]

Number of strands:
\[ n_{s(1)} = 10 \]

WARNING:
Strand location is non-standard.

Height above soffit:
\[ d_{s(2)} = 110 \text{ mm} \]

Number of strands:
\[ n_{s(2)} = 13 \]

WARNING:
Number of strands exceeds maximum for this location.

Height above soffit:
\[ d_{s(3)} = 165 \text{ mm} \]

Number of strands:
\[ n_{s(3)} = 8 \]

WARNING:
Strand location is non-standard.

Height above soffit:
\[ d_{s(4)} = 600 \text{ mm} \]

WARNING:
Height of strand above soffit exceeds maximum 596.

Number of strands:
\[ n_{s(4)} = 2 \]

WARNING:
Strand location is non-standard.

Bending moment from temp. loads:
\[ t_{lb(1)} = 423 \text{ kNm} \]

Bending moment from other loads:
\[ o_{lb(1)} = 367 \text{ kNm} \]
Short Term Losses

Relaxation at transfer ( % ) \(\text{str}=0.5\)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>24.75</td>
<td>0.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>534.35</td>
<td>10.795</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>559.1</td>
<td>11.295</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force          4950 kN
Force at transfer      \(\text{Pt}=\text{P0}-\text{loft}=4390.9\) kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-6.5458</td>
<td>15.713</td>
<td>14.195</td>
<td></td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>11.144</td>
<td>-8.3279</td>
<td>-7.0003</td>
<td></td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>1.7068</td>
<td>-1.2755</td>
<td>-1.0721</td>
<td></td>
</tr>
<tr>
<td>Other Loads</td>
<td>1.4809</td>
<td>-1.1066</td>
<td>-0.9302</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>7.7864</td>
<td>5.0026</td>
<td>5.1924</td>
<td></td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
Compressive stress 7.7864 N/mm² is less than allowable stresses of 0.5*fci=17.5 N/mm² and 0.4*fcu=16 N/mm².

Bottom of beam:
Compressive stress 5.0026 N/mm² is less than allowable stresses of 0.5*fci=17.5 N/mm² and 0.4*fcu=16 N/mm².

Long Term Losses

Relaxation after transfer( % ) \(\text{slr}(1)=0.4\)
Shrinkage per unit length \(\text{sul}(1)=0.3\times10^{-3}\)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>24.75</td>
<td>0.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>534.35</td>
<td>10.795</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>559.1</td>
<td>11.295</td>
</tr>
<tr>
<td>Relaxation</td>
<td>19.8</td>
<td>0.4</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>297</td>
<td>6</td>
</tr>
<tr>
<td>Creep</td>
<td>340.22</td>
<td>6.8731</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>657.02</td>
<td>13.273</td>
</tr>
<tr>
<td>All Losses</td>
<td>1216.1</td>
<td>24.568</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 4950 kN
Final force Pf=P0-loftl=3733.9 kN

Unit-weight of in-situ concrete wc=23.6 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-5.5663</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>11.144</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.5405</td>
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<tr>
<td>Total</td>
<td>9.1185</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=1965 mm
Percentage of shrinkage remaining ps=40 %
Percentage pcr=40
Percentage stT=67 %
The net moment is Mc=F*e/1E3-Mb=239.02 kNm

Stress at top of in-situ concrete:
ftisds=-F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3
=1.2482 N/mm²

Stress at bottom of in-situ concrete:
fbisds=-F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
=1.1648 N/mm²

Stress at top of precast concrete beam:
Sample output for SCALE Proforma 138. (ans=3)  Page: 5
Reinforced & prestressed concrete: BS5400 & Eurocode  Made by: IFB
SY beams with in situ top slab  Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.  Ref No: SC138 BS

\[
ftpbd = \left( \frac{F}{(A_i \cdot mr + A_c)} \right) + \left( \frac{Mc}{Ic} \cdot \frac{(d-Y_c)}{} + Mb \cdot 1E3 \cdot \frac{Y_t}{I_{xx}} \right) \cdot 1E3
\]

Stress at bottom of precast concrete beam:
\[
fbpbds = \left( \frac{F}{(A_i \cdot mr + A_c)} \right) - \left( \frac{Mc}{Ic} \cdot \frac{(d-Y_c)}{} + Mb \cdot 1E3 \cdot \frac{Y_b}{I_{xx}} \right) \cdot 1E3 = 0.64352 \text{ N/mm}^2
\]

**Combination 1 loading**

Applied bending moment  \( c_{lb}(1) = 1900 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>1.2482</td>
</tr>
<tr>
<td>Combination 1</td>
<td>3.6564</td>
</tr>
<tr>
<td>Total</td>
<td>4.9045</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

**WARNING:**
Bottom of beam:
Tensile stress 1.0134 N/mm\(^2\) exceeds maximum allowable tensile stress of 0 N/mm\(^2\).

**Combination 2 to 5 loading**

Applied bending moment  \( c_{2b}(1) = 2200 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>1.2482</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>4.022</td>
</tr>
<tr>
<td>Total</td>
<td>5.2702</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 5.2702 N/mm\(^2\) is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm\(^2\).
Top of beam:
Compressive stress $10.703 \text{ N/mm}^2$ is less than the maximum allowable stress of $0.4 \times f_{cu}=16 \text{ N/mm}^2$.

Bottom of in-situ concrete:
Compressive stress $4.4192 \text{ N/mm}^2$ is less than the maximum allowable stress of $0.5 \times f_{cuc}=17.5 \text{ N/mm}^2$.

Bottom of beam:
Tensile stress $1.652 \text{ N/mm}^2$ is less than the maximum allowable tensile stress of $2.846 \text{ N/mm}^2$. 
Location: Ex1 - 34 metre span SY beam

Precast prestressed concrete SY beams with in-situ top slab


Beam span under consideration span=34 m

Beam SY 4 section properties

Depth of beam d=1800 mm
Cross-sectional area Ac=644992 mm²
Centroid above soffit Yb=752.98 mm
Section modulus top Zt=199.16E6 mm⁵
Section modulus bottom Zb=276.91E6 mm⁵
Unit weight of beam uwb=24 kN/m³

Concrete compressive strengths

Characteristic cylinder strength fck=40 N/mm²
Characteristic cylinder strength fck=32 N/mm² (transfer)

Concrete moduli (short-term)

Concrete modulus of elasticity Ecmb=35 kN/mm²
Concrete modulus of elasticity Ecmi=33 kN/mm² (transfer)

Composite section properties

Char strength (in-situ concrete) fckc=32 N/mm²
Short-term modulus of elasticity for in-situ concrete Ecmc=33 kN/mm²
Second moment of area:
\[ I_c = I_{yy} + A_c(Y_b - Y_c)^2 + m_r(I_i + A_i(Y_l - Y_c)^2) = 441.35 \times 10^9 \text{ mm}^4 \]

Depth of composite section \( d_i = 1960 \text{ mm} \)
Modulus of elasticity (strand) \( E_p = 195 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( c_{sd}(1) = 0 \text{ m} \)
Number of layers of strand \( n_{ls}(1) = 4 \)
Height above soffit \( d_{s}(1) = 60 \text{ mm} \)
Number of strands \( n_s(1) = 11 \)
Height above soffit \( d_{s}(2) = 110 \text{ mm} \)
Number of strands \( n_s(2) = 12 \)
Height above soffit \( d_{s}(3) = 160 \text{ mm} \)
Number of strands \( n_s(3) = 12 \)
Height above soffit \( d_{s}(4) = 210 \text{ mm} \)
Number of strands \( n_s(4) = 12 \)

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer ( % ) \( \text{str}=1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force (kN)</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>81.78</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>932.65</td>
<td>11.404</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>1014.4</td>
<td>12.404</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 8178 kN
Force at transfer \( P_t = P_0 - \text{loft} = 7163.6 \text{ kN} \)
### Beam Concrete Stresses at Transfer

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
<td>Bottom of Beam</td>
<td>Centroid of Strands</td>
</tr>
<tr>
<td>Prestress</td>
<td>-11.066</td>
<td>27.052</td>
<td>24.159</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>11.232</td>
<td>-8.0778</td>
<td>-6.6125</td>
</tr>
<tr>
<td>Total</td>
<td>0.16623</td>
<td>18.974</td>
<td>17.547</td>
</tr>
</tbody>
</table>

#### Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

**Top of beam:**
Compressive stress 0.16623 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Bottom of beam:**
Compressive stress 18.974 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

#### Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>81.78</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>932.65</td>
<td>11.404</td>
</tr>
<tr>
<td>Total</td>
<td>1014.4</td>
<td>12.404</td>
</tr>
<tr>
<td>Relaxation</td>
<td>163.56</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>382.18</td>
<td>4.6733</td>
</tr>
<tr>
<td>Creep</td>
<td>1582.8</td>
<td>19.355</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>2128.6</td>
<td>26.028</td>
</tr>
<tr>
<td>All Losses</td>
<td>3143</td>
<td>38.432</td>
</tr>
</tbody>
</table>

#### Summary

**Losses - Long term**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial force</td>
<td>8178 kN</td>
<td></td>
</tr>
<tr>
<td>Final force</td>
<td>Pf=P₀-loft₁=5035 kN</td>
<td></td>
</tr>
<tr>
<td>Unit-weight of in-situ concrete</td>
<td>wc=24 kN/m³</td>
<td></td>
</tr>
</tbody>
</table>
Sample output for SCALE Proforma 138. (ans=1) Page: 4
Reinforced & prestressed concrete: BS5400 & Eurocode Made by: IFB
SY beams with insitu top slab Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd. Ref No: SC138 EC

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-7.7779</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>11.232</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>4.6079</td>
</tr>
<tr>
<td>Total</td>
<td>8.0622</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

**Composite section**

Height from beam soffit to bottom of in-situ concrete: hs=1765 mm
Percentage of shrinkage remaining: p_s=25%
Percentage: p_c=25%
Percentage: s_T=67%
The net moment is: Mc=F*e/1E3-Mb=-454.45 kNm

Stress at top of in-situ concrete:
ftisds=-\(\frac{F}{(Ai*mr)}-\frac{F}{(Ai*mr+Ac)}-Mc*1E3*(di-Yc)/Ic)*mr*1E3
\=-0.065071 \text{ N/mm}^2

Stress at bottom of in-situ concrete:
fbisds=-\(\frac{F}{(Ai*mr)}-\frac{F}{(Ai*mr+Ac)}-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
\=0.13572 \text{ N/mm}^2

Stress at top of precast concrete beam:
ftpbsd=(\frac{F}{(Ai*mr+Ac)}+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)*1E3
\=-0.057565 \text{ N/mm}^2

Stress at bottom of precast concrete beam:
fbpbds=(\frac{F}{(Ai*mr+Ac)}-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)*1E3
\=0.018331 \text{ N/mm}^2

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment: cl_b(1)=3000 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-0.065071</td>
</tr>
<tr>
<td>Total</td>
<td>6.5411</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses
Loading Combination 2

(tensile stress permitted (no visible cracking))
Characteristic bending moment \( c_{2b(1)} = 3500 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam</td>
<td>0</td>
</tr>
<tr>
<td>and in-situ Dif. shrinkage Combination 2</td>
<td>-0.065071</td>
</tr>
<tr>
<td></td>
<td>7.0065</td>
</tr>
<tr>
<td>Total</td>
<td>6.9414</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 6.9414 N/mm² is less than the maximum allowable stress of 0.6*\( f_{ckc} = 19.2 \text{ N/mm}^2 \).

Top of beam:
Compressive stress 13.742 N/mm² is less than the maximum allowable stress of 0.6*\( f_{ck} = 24 \text{ N/mm}^2 \).

Bottom of in-situ concrete:
Compressive stress 5.5958 N/mm² is less than the maximum allowable stress of 0.6*\( f_{ckc} = 19.2 \text{ N/mm}^2 \).

Bottom of beam:
Tensile stress 0.8961 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
**Location: Ex2 - 24.5 metre span SY2 beam**

**Precast prestressed concrete SY beams with in-situ top slab**


Beam span under consideration \( \text{span}=24.5 \text{ m} \)

**Beam SY 2 section properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>( d=1600 \text{ mm} )</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>( A_c=580992 \text{ mm}^2 )</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>( Y_b=650.06 \text{ mm} )</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>( Z_t=153.97 \text{E6 mm}^3 )</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>( Z_b=224.96 \text{E6 mm}^3 )</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>( u_wb=24.5 \text{ kN/m}^3 )</td>
</tr>
</tbody>
</table>

**Concrete compressive strengths**

<table>
<thead>
<tr>
<th>Strength</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic cylinder strength</td>
<td>( f_{ck1}=28 \text{ N/mm}^2 ) (transfer)</td>
</tr>
<tr>
<td>Characteristic cylinder strength</td>
<td>( f_{ck}=36 \text{ N/mm}^2 )</td>
</tr>
</tbody>
</table>

**Concrete moduli (short-term)**

<table>
<thead>
<tr>
<th>Modulus</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete modulus of elasticity</td>
<td>( E_{cm1}=32 \text{ kN/mm}^2 ) (transfer)</td>
</tr>
<tr>
<td>Concrete modulus of elasticity</td>
<td>( E_{cm2}=34 \text{ kN/mm}^2 )</td>
</tr>
</tbody>
</table>

**Composite section properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char strength (in-situ concrete)</td>
<td>( f_{ckc}=35 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Short-term modulus of elasticity</td>
<td>( E_{cmc}=34 \text{ kN/mm}^2 )</td>
</tr>
</tbody>
</table>

**Item No. 1**

<table>
<thead>
<tr>
<th>X co-ordinate</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>X(1)</td>
<td>500 mm</td>
</tr>
<tr>
<td>Y(1)</td>
<td>0 mm</td>
</tr>
<tr>
<td>X(2)</td>
<td>1250 mm</td>
</tr>
<tr>
<td>Y(2)</td>
<td>0 mm</td>
</tr>
<tr>
<td>X(3)</td>
<td>1245 mm</td>
</tr>
<tr>
<td>Y(3)</td>
<td>252 mm</td>
</tr>
<tr>
<td>X(4)</td>
<td>975 mm</td>
</tr>
<tr>
<td>Y(4)</td>
<td>429 mm</td>
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<tr>
<td>X(5)</td>
<td>1035 mm</td>
</tr>
<tr>
<td>Y(5)</td>
<td>1550 mm</td>
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<tr>
<td>X(6)</td>
<td>995 mm</td>
</tr>
<tr>
<td>Y(6)</td>
<td>1550 mm</td>
</tr>
<tr>
<td>X(7)</td>
<td>995 mm</td>
</tr>
<tr>
<td>Y(7)</td>
<td>1570 mm</td>
</tr>
<tr>
<td>X(8)</td>
<td>1800 mm</td>
</tr>
<tr>
<td>Y(8)</td>
<td>1570 mm</td>
</tr>
<tr>
<td>X(9)</td>
<td>1800 mm</td>
</tr>
<tr>
<td>Y(9)</td>
<td>1900 mm</td>
</tr>
</tbody>
</table>
X co-ordinate                  X(10)=1300 mm
Y co-ordinate                  Y(10)=1900 mm
X co-ordinate                  X(11)=1300 mm
Y co-ordinate                  Y(11)=1800 mm
X co-ordinate                  X(12)=0 mm
Y co-ordinate                  Y(12)=1800 mm
X co-ordinate                  X(13)=0 mm
Y co-ordinate                  Y(13)=1570 mm
X co-ordinate                  X(14)=755 mm
Y co-ordinate                  Y(14)=1570 mm
X co-ordinate                  X(15)=755 mm
Y co-ordinate                  Y(15)=1550 mm
X co-ordinate                  X(16)=715 mm
Y co-ordinate                  Y(16)=1550 mm
X co-ordinate                  X(17)=775 mm
Y co-ordinate                  Y(17)=429 mm
X co-ordinate                  X(18)=505 mm
Y co-ordinate                  Y(18)=252 mm
X co-ordinate                  X(19)=500 mm
Y co-ordinate                  Y(19)=0 mm

Depth of composite section     di=1900 mm
Area of in-situ concrete       Ai=450198 mm²
Centroid of in-situ concrete   Yi=1713.4 mm
Modulus of elasticity (strand) Ep=195 kN/mm²

Cross-section 1

Distance from mid-span of beam csd(1)=5.6 m
Number of layers of strand    nls(1)=4
Height above soffit           ds(1)=60 mm
Number of strands             ns(1)=8
Height above soffit           ds(2)=110 mm
Number of strands             ns(2)=8
Height above soffit           ds(3)=160 mm
Number of strands             ns(3)=10
Height above soffit           ds(4)=596 mm
Number of strands             ns(4)=2

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % ) str=0.9

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22.68</td>
<td>0.9</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>105.38</td>
<td>4.1819</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>128.06</td>
<td>5.0819</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force      2520 kN
Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
Compressive stress 1.8084 N/mm² is less than allowable stresses of 0.7*fck=19.6 N/mm² and 0.6*fck=21.6 N/mm².

Bottom of beam:
Compressive stress 5.6968 N/mm² is less than allowable stresses of 0.7*fck=19.6 N/mm² and 0.6*fck=21.6 N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer(%) slr(1)=0.6
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(∞,to) crf=2

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22.68</td>
<td>0.9</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>105.38</td>
<td>4.1819</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>128.06</td>
<td>5.0819</td>
</tr>
<tr>
<td>Relaxation</td>
<td>15.12</td>
<td>0.6</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>122.85</td>
<td>4.875</td>
</tr>
<tr>
<td>Creep</td>
<td>128.54</td>
<td>5.101</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>266.51</td>
<td>10.576</td>
</tr>
<tr>
<td>All Losses</td>
<td>394.58</td>
<td>15.658</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2520 kN
Final force Pf=P0-loft=2125.4 kN
Unit-weight of in-situ concrete wc=24 kN/m³
### Composite section

**Height from beam soffit to bottom of in-situ concrete:** \( h_{sis} = 1570 \text{ mm} \)

**Percentage of shrinkage remaining:** \( p_s = 45 \% \)

**Percentage:** \( p_c = 45 \% \)

**Percentage:** \( s_{T} = 50 \% \)

The net moment is

\[
M_c = \frac{F}{e} - M_b = 29.445 \text{ kNm}
\]

**Stress at top of in-situ concrete:**

\[
ft_{is} = \frac{F}{A_i \cdot mr} - \frac{F}{A_i \cdot mr + Ac} - M_c \cdot 1E3 \cdot \frac{d - Y_c}{I_c} \cdot 1E3
\]

\[
= 0.56887 \text{ N/mm}^2
\]

**Stress at bottom of in-situ concrete:**

\[
f_{b_{is}} = \frac{F}{A_i \cdot mr} - \frac{F}{A_i \cdot mr + Ac} - M_c \cdot 1E3 \cdot \frac{h_{sis} - Y_c}{I_c} \cdot 1E3
\]

\[
= 0.54644 \text{ N/mm}^2
\]

**Stress at top of precast concrete beam:**

\[
f_{tp} = \frac{F}{A_i \cdot mr + Ac} + M_c \cdot 1E3 \cdot \frac{d - Y_c}{I_c} + M_b \cdot 1E3 \cdot \frac{Y_t}{I_{yy}} \cdot 1E3
\]

\[
= -2.1606 \text{ N/mm}^2
\]

**Stress at bottom of precast concrete beam:**

\[
f_{bp} = \frac{F}{A_i \cdot mr + Ac} - M_c \cdot 1E3 \cdot \frac{Y_c}{I_c} - M_b \cdot 1E3 \cdot \frac{Y_b}{I_{yy}} \cdot 1E3
\]

\[
= 0.75265 \text{ N/mm}^2
\]

### Loading Combination 1

(no tensile stress permitted in precast beam)

**Characteristic bending moment:** \( c_{lb(1)} = 600 \text{ kNm} \)

---

### Table: Loading Combination 1 Stresses N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Prestress, beam and in-situ</th>
<th>Prestress, beam and in-situ</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-3.2695</td>
<td>5.4879</td>
<td>4.1656</td>
<td></td>
<td>8.399</td>
<td>-3.7554</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.4879</td>
<td>8.399</td>
<td>-3.7554</td>
<td>-2.8988</td>
<td>4.1656</td>
<td>-2.8506</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>4.1656</td>
<td>-2.8506</td>
<td>-2.8988</td>
<td>-2.2004</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>6.384</td>
<td>1.793</td>
<td>2.2185</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses
Loading Combination 2

(tensile stress permitted (no visible cracking))

Characteristic bending moment \(c_{2b}(1)=1000 \text{ kNm}\)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>0.56887</td>
</tr>
<tr>
<td></td>
<td>1.8132</td>
</tr>
<tr>
<td>Total</td>
<td>2.382</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 2.382 N/mm² is less than the maximum allowable stress of 0.6*\(f_{ckc}\)=21 N/mm².

Top of beam:
Compressive stress 5.3443 N/mm² is less than the maximum allowable stress of 0.6*\(f_{ck}\)=21.6 N/mm².

Bottom of in-situ concrete:
Compressive stress 1.5981 N/mm² is less than the maximum allowable stress of 0.6*\(f_{ckc}\)=21 N/mm².

Bottom of beam:
Tensile stress 0.025901 N/mm² is less than the maximum allowable tensile stress of 2.9399 N/mm².
Location: Ex3 - 40.0 metre span SY6 beam

Precast prestressed concrete SY beams with in-situ top slab


Beam span under consideration  span=40 m

Beam SY 6 section properties

- Depth of beam  d=2000 mm
- Cross-sectional area  Ac=708992 mm²
- Centroid above soffit  Yb=855.36 mm
- Section modulus top  Zt=247.88E6 mm³
- Section modulus bottom  Zb=331.64E6 mm³
- Unit weight of beam  uwb=25 kN/m³

Concrete compressive strengths

- Characteristic cylinder strength  fck=40 N/mm²
- Concrete modulus of elasticity  Ecmb=35 kN/mm²

Concrete moduli (short-term)

- Concrete modulus of elasticity  Ecmi=34 kN/mm²

Composite section properties

- Char strength (in-situ concrete)  fckc=35 N/mm²
- Short-term modulus of elasticity for in-situ concrete  Ecmc=34 kN/mm²
Second moment of area:

\[ I_c = I_{yy} + A_c (Y_b - Y_c)^2 + m_r (I_i + A_i (Y_i - Y_c)^2) = 544.6 \times 10^9 \text{ mm}^4 \]

Depth of composite section: \( d_i = 2155 \text{ mm} \)

Modulus of elasticity (strand): \( E_p = 195 \text{ kN/mm}^2 \)

### Cross-section 1

- Distance from mid-span of beam: \( c_s d(1) = 9.4 \text{ m} \)
- Number of layers of strand: \( n_{ls}(1) = 4 \)
- Height above soffit: \( d_s(1) = 55 \text{ mm} \)
- Number of strands: \( n_s(1) = 10 \)

**WARNING:**

Strand location is non-standard.

- Height above soffit: \( d_s(2) = 110 \text{ mm} \)
- Number of strands: \( n_s(2) = 13 \)

**WARNING:**

Number of strands exceeds maximum for this location.

- Height above soffit: \( d_s(3) = 165 \text{ mm} \)
- Number of strands: \( n_s(3) = 8 \)

**WARNING:**

Strand location is non-standard.

- Height above soffit: \( d_s(4) = 600 \text{ mm} \)

**WARNING:**

Height of strand above soffit exceeds maximum 596.

- Number of strands: \( n_s(4) = 2 \)

**WARNING:**

Strand location is non-standard.

- Bending moment from temp.loads: \( t_{lb}(1) = 423 \text{ kNm} \)
- Bending moment from other loads: \( o_{lb}(1) = 367 \text{ kNm} \)

### Short Term Losses (BS EN 1992-1-1)

- Relaxation at transfer ( % ): \( s_{tr} = 0.5 \)
**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>24.75</td>
<td>0.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>414.03</td>
<td>8.3643</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>438.78</td>
<td>8.8643</td>
</tr>
</tbody>
</table>

**Losses at Transfer**

Initial force 4950 kN  
Force at transfer $P_t = P_0 - \text{loft} = 4511.2$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-6.7251</td>
<td>16.143</td>
<td>14.584</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>11.144</td>
<td>-8.3279</td>
<td>-7.0003</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>1.7068</td>
<td>-1.2755</td>
<td>-1.0721</td>
</tr>
<tr>
<td>Other Loads</td>
<td>1.4809</td>
<td>-1.1066</td>
<td>-0.9302</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>7.607</strong></td>
<td><strong>5.4331</strong></td>
<td><strong>5.5814</strong></td>
</tr>
</tbody>
</table>

**Beam Concrete Stresses at Transfer**

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

- **Top of beam:**
  Compressive stress $7.607$ N/mm² is less than allowable stresses of $0.7\times f_{ck,i} = 24.5$ N/mm² and $0.6\times f_{ck} = 24$ N/mm².
- **Bottom of beam:**
  Compressive stress $5.4331$ N/mm² is less than allowable stresses of $0.7\times f_{ck,i} = 24.5$ N/mm² and $0.6\times f_{ck} = 24$ N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

- Relaxation after transfer (%): $slr(1) = 0.4$
- Shrinkage per unit length: $sul(1) = 0.3E-3$
- Creep coefficient $\varphi(=,to)$: $crf = 2$
### Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>24.75</td>
<td>0.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>414.03</td>
<td>8.3643</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>438.78</td>
<td>8.8643</td>
</tr>
<tr>
<td>Relaxation</td>
<td>19.8</td>
<td>0.4</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>289.58</td>
<td>5.85</td>
</tr>
<tr>
<td>Creep</td>
<td>366.99</td>
<td>7.4139</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>676.36</td>
<td>13.664</td>
</tr>
<tr>
<td>All Losses</td>
<td>1115.1</td>
<td>22.528</td>
</tr>
</tbody>
</table>

**Losses - Long term**

- **Initial force**: 4950 kN
- **Final force**: Pf=P0-loftl=3834.9 kN
- **Unit-weight of in-situ concrete**: wc=23.6 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
<td>Bottom of Beam</td>
<td>Centroid of Strands</td>
</tr>
<tr>
<td>Prestress</td>
<td>-5.7168</td>
<td>13.723</td>
<td>12.397</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>11.144</td>
<td>-8.3279</td>
<td>-7.0003</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.5405</td>
<td>-2.6457</td>
<td>-2.2239</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>8.968</td>
<td>2.7492</td>
<td>3.1732</td>
</tr>
</tbody>
</table>

**Beam concrete stresses - Long term**

- **Height from beam soffit to bottom of in-situ concrete**: hsis=1965 mm
- **Percentage of shrinkage remaining**: ps=40 %
- **Percentage**: pcr=40
- **Percentage**: stT=67 %
- **The net moment is**: Mc=F*e/1E3-Mb=181.7 kNm

**Stress at top of in-situ concrete**:

\[
ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3 =1.4448 N/mm²
\]

**Stress at bottom of in-situ concrete**:

\[
fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3 =1.3814 N/mm²
\]

**Stress at top of precast concrete beam**: 

---

**SCALE 5.48**

**Office 1007**

**Proforma 138**
Stress at bottom of precast concrete beam:

\[
\text{fbpbds} = \frac{F}{(A_i m_r + A_c) - M_c 1E3 \cdot \left(Y_c / I_c - M_b 1E3 \cdot Y_t / I_{yy}\right)} 1E3 = 0.7531 \text{ N/mm}^2
\]

**Loading Combination 1**

(no tensile stress permitted in precast beam)
Characteristic bending moment \( c_1b(1) = 1900 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>1.4448</td>
</tr>
<tr>
<td></td>
<td>3.6564</td>
</tr>
<tr>
<td>Total</td>
<td>5.1011</td>
</tr>
</tbody>
</table>

**Loading Combination 1 - Stresses**

**WARNING:**
Bottom of beam:
Tensile stress 0.5425 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

**Loading Combination 2**

(tensile stress permitted (no visible cracking))
Characteristic bending moment \( c_2b(1) = 2200 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>1.4448</td>
</tr>
<tr>
<td></td>
<td>4.022</td>
</tr>
<tr>
<td>Total</td>
<td>5.4668</td>
</tr>
</tbody>
</table>

**Loading Combination 2 - Stresses**

**NOTE:** The stresses from temperature differences should be added to the total stresses.
Top of in-situ concrete:
Compressive stress 5.4668 N/mm² is less than the maximum allowable stress of 0.6*fckc=21 N/mm².

Top of beam:
Compressive stress 10.243 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

Bottom of in-situ concrete:
Compressive stress 4.6359 N/mm² is less than the maximum allowable stress of 0.6*fckc=21 N/mm².

Bottom of beam:
Tensile stress 1.1812 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
Location: Ex1 - 20 metre span YE beam

Precast prestressed concrete YE beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=20 m

Beam YE 4 section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=1000 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=577926 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=463.07 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>Zt=95.2E6 mm³</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>Zb=110.33E6 mm³</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=24 kN/m³</td>
</tr>
</tbody>
</table>

Concrete strengths

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength at transfer fci</td>
<td>fci=40 N/mm²</td>
</tr>
<tr>
<td>Characteristic strength fcu</td>
<td>fcu=50 N/mm²</td>
</tr>
</tbody>
</table>

Concrete moduli

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity at transfer for concrete strength 40 N/mm² is:</td>
<td>Eci=31 kN/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity for concrete strength 50 N/mm² is:</td>
<td>Ecc=34 kN/mm²</td>
</tr>
</tbody>
</table>

Composite section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char strength (in-situ concrete) fcuc</td>
<td>fcuc=40 N/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity for concrete strength 40 N/mm² is:</td>
<td>Ecc=31 kN/mm²</td>
</tr>
</tbody>
</table>
Area 1 = \frac{477}{160} = 0.66 m^2

\text{Second moment of area:}
\begin{align*}
I_c &= I_{xx} + A_c(Y_b - Y_c)^2 + m_r(I_i + A_i(Y_i - Y_c)^2) \\
&= 91.223 \times 10^9 \text{ mm}^4
\end{align*}

Depth of composite section: \( d_i = 1125 \text{ mm} \)

Initial force: \( I_f = 174 \text{ kN} \)

Modulus of elasticity: \( E_s = 200 \text{ kN/mm}^2 \)

\textbf{Cross-section 1}

Distance from mid-span of beam: \( csd(1) = 0 \text{ m} \)

Number of layers of strand: \( n_{ls}(1) = 4 \)

Height above soffit: \( ds(1) = 60 \text{ mm} \)

Number of strands: \( n_s(1) = 11 \)

Height above soffit: \( ds(2) = 110 \text{ mm} \)

Number of strands: \( n_s(2) = 14 \)

Height above soffit: \( ds(3) = 800 \text{ mm} \)

Number of strands: \( n_s(3) = 2 \)

Height above soffit: \( ds(4) = 900 \text{ mm} \)

Number of strands: \( n_s(4) = 2 \)

\textbf{Short Term Losses}

Relaxation at transfer (%): \( str = 1 \)

\textbf{Summary}

\begin{tabular}{|c|c|c|}
\hline
Loss & Loss of Force & % Loss \\
& kN & \\
\hline
Relaxation & 50.46 & 1 \\
Elastic Shortening & 410.13 & 8.1278 \\
Transfer Total & 460.59 & 9.1278 \\
\hline
\end{tabular}

Losses at Transfer

Initial force: \( 5046 \text{ kN} \)

Force at transfer: \( P_t = P_0 - loft = 4585.4 \text{ kN} \)
<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress Self weight of Beam</td>
<td>-5.0754</td>
</tr>
<tr>
<td></td>
<td>7.2884</td>
</tr>
<tr>
<td>Total</td>
<td>2.213</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 2.213 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Bottom of beam:
Compressive stress 12.869 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

**Long Term Losses**

Relaxation after transfer( % ) slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation Elastic Shortening</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>410.13</td>
<td>8.1278</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>460.59</td>
<td>9.1278</td>
</tr>
<tr>
<td>Relaxation Shrinkage Creep</td>
<td>100.92</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>241.86</td>
<td>4.7931</td>
</tr>
<tr>
<td></td>
<td>418.36</td>
<td>8.2908</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>761.14</td>
<td>15.084</td>
</tr>
<tr>
<td>All Losses</td>
<td>1221.7</td>
<td>24.212</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 5046 kN
Final force Pf=P0-loftl=3824.3 kN

Unit-weight of in-situ concrete wc=24 kN/m³
### Final Stresses N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-4.2329</td>
<td>15.975</td>
<td>12.073</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.2884</td>
<td>-6.2858</td>
<td>-3.6646</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>1.8073</td>
<td>-1.5587</td>
<td>-0.90868</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>4.8627</td>
<td>8.1304</td>
<td>7.4994</td>
</tr>
</tbody>
</table>

**Beam concrete stresses - Long term**

**Composite section**

- Height from beam soffit to bottom of in-situ concrete: \( h_{sis} = 965 \text{ mm} \)
- Perc'age of shrinkage remaining: \( p_s = 25\% \)
- Percentage: \( p_{cr} = 25\% \)
- Percentage: \( s_t = 67\% \)

The net moment is: \( M_c = F \cdot e / 1E3 - M_b = -31.103 \text{ kNm} \)

- Stress at top of in-situ concrete:
  \[
  f_{tisds} = -(F/(Ai*mr) - F/(Ai*mr+Ac) - Mc*1E3*(d_i-Y_c)/I_c)*mr*1E3
  \]
  
  \[
  = -0.8043 \text{ N/mm}^2
  \]

- Stress at bottom of in-situ concrete:
  \[
  f_{bisds} = -(F/(Ai*mr) - F/(Ai*mr+Ac) - Mc*1E3*(h_{sis}-Y_c)/I_c)*mr*1E3
  \]
  
  \[
  = -0.74975 \text{ N/mm}^2
  \]

- Stress at top of precast concrete beam:
  \[
  f_{tpbds} = (F/(Ai*mr+Ac) + Mc*1E3*(d-Y_c)/I_c+M_b*1E3*Y_t/I_{xx})*1E3
  \]
  
  \[
  = 0.88597 \text{ N/mm}^2
  \]

- Stress at bottom of precast concrete beam:
  \[
  f_{bpbds} = (F/(Ai*mr+Ac) - Mc*1E3*Y_c/I_c-M_b*1E3*Y_b/I_{xx})*1E3
  \]
  
  \[
  = -0.4041 \text{ N/mm}^2
  \]

**Combination 1 loading**

- Applied bending moment: \( c_{lb(1)} = 1000 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage</td>
<td>-0.8043</td>
</tr>
<tr>
<td>Combination 1</td>
<td>7.1673</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>6.363</td>
</tr>
</tbody>
</table>

**Combination 1 Loading - Stresses**
Combination 2 to 5 loading

Applied bending moment \( c2b(1) = 1500 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>4.8627</td>
<td>0</td>
<td>8.1304</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.8043</td>
<td>0.88597</td>
<td>-0.74975</td>
<td>-0.4041</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>8.9592</td>
<td>6.9038</td>
<td>6.3283</td>
<td>-9.5394</td>
</tr>
<tr>
<td>Total</td>
<td>8.1549</td>
<td>12.652</td>
<td>5.5785</td>
<td>-1.8131</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 8.1549 N/mm\(^2\) is less than the maximum allowable stress of 0.5*fcuc=20 N/mm\(^2\).

Top of beam:
Compressive stress 12.652 N/mm\(^2\) is less than the maximum allowable stress of 0.4*fcu=20 N/mm\(^2\).

Bottom of in-situ concrete:
Compressive stress 5.5785 N/mm\(^2\) is less than the maximum allowable stress of 0.5*fcuc=20 N/mm\(^2\).

Bottom of beam:
Tensile stress 1.8131 N/mm\(^2\) is less than the maximum allowable tensile stress of 3.182 N/mm\(^2\).
Location: Ex2 - 29.5 metre span YE8 beam

Precast prestressed concrete YE beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=29.5 m

Beam YE 8 section properties

Depth of beam d=1400 mm
Cross-sectional area Ac=815050 mm²
Centroid above soffit Yb=676.56 mm
Section modulus top Zt=200.07E6 mm³
Section modulus bottom Zb=213.93E6 mm³
Unit weight of beam uwb=24.5 kN/m³

Concrete strengths

Strength at transfer fci=40 N/mm²
Characteristic strength fcu=52.5 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 40 N/mm² is:
Eci=31 kN/mm²
Modulus of elasticity for concrete strength 52.5 N/mm² is:
Ecb=34.5 kN/mm²

Composite section properties

Char strength (in-situ concrete) fcu=40 N/mm²
Modulus of elasticity for concrete strength 40 N/mm² is:
Ecc=31 kN/mm²

Item No. 1

Item weighting iw1=0.89855
X co-ordinate X(1)=500 mm
Y co-ordinate Y(1)=0 mm
X co-ordinate X(2)=1250 mm
Y co-ordinate Y(2)=0 mm
X co-ordinate X(3)=1250 mm
Y co-ordinate Y(3)=202 mm
X co-ordinate X(4)=975 mm
Y co-ordinate Y(4)=379 mm
X co-ordinate X(5)=1115 mm
Y co-ordinate Y(5)=1350 mm
X co-ordinate X(6)=1615 mm
Y co-ordinate Y(6)=1350 mm
X co-ordinate X(7)=1615 mm
Y co-ordinate Y(7)=1550 mm
X co-ordinate X(8)=500 mm
Y co-ordinate Y(8)=1550 mm
X co-ordinate X(9)=500 mm
Y co-ordinate Y(9)=1650 mm
X co-ordinate X(10)=0 mm
Y co-ordinate Y(10)=1650 mm
X co-ordinate X(11)=0 mm
Y co-ordinate Y(11)=1350 mm
X co-ordinate X(12)=500 mm
Y co-ordinate Y(12)=1350 mm
X co-ordinate X(13)=500 mm
Y co-ordinate Y(13)=0 mm

**Item No. 2**

Second moment of area IXX2=144.74E9 mm^4
Area of item A2=815050 mm^2
Centroid from X-X axis YC2=676.56 mm
Item weighting iw2=0.10145
Depth of composite section di=1650 mm
Initial force if=200 kN
Modulus of elasticity Es=200 kN/mm^2

**Cross-section 1**

Distance from mid-span of beam csd(1)=9.75 m
Number of layers of strand nls(1)=4
Height above soffit ds(1)=60 mm
Number of strands ns(1)=10
Height above soffit ds(2)=110 mm
Number of strands ns(2)=12
Height above soffit ds(3)=210 mm
Number of strands ns(3)=8
Height above soffit ds(4)=1200 mm
Number of strands ns(4)=2

**Short Term Losses**

Relaxation at transfer ( % ) str=1.5

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>96</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>618.34</td>
<td>9.6615</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>714.34</td>
<td>11.162</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 6400 kN
Force at transfer Pt=P0-loft=5685.7 kN
<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-6.9226</td>
<td>19.974</td>
<td>16.372</td>
<td></td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.1134</td>
<td>-5.7172</td>
<td>-4.1328</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>-0.80927</td>
<td>14.256</td>
<td>12.239</td>
<td></td>
</tr>
</tbody>
</table>

### Stress Limitations

**Top of beam:**
The flexural tensile stress of 0.80927 N/mm² at transfer is less than the permissible value of 1 N/mm². Clause 6.3.2.4(b)(1).

**Bottom of beam:**
Compressive stress 14.256 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=21 N/mm².

### Long Term Losses

Relaxation after transfer ( % ) \(slr(1)=1\)
Shrinkage per unit length \(sul(1)=0.3E-3\)

### Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>96</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>618.34</td>
<td>9.6615</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>714.34</td>
<td>11.162</td>
</tr>
<tr>
<td>Relaxation</td>
<td>64</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>316.8</td>
<td>4.95</td>
</tr>
<tr>
<td>Creep</td>
<td>641.83</td>
<td>10.029</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>1022.6</td>
<td>15.979</td>
</tr>
<tr>
<td>All Losses</td>
<td>1737</td>
<td>27.14</td>
</tr>
</tbody>
</table>

**Losses - Long term**

Initial force 6400 kN
Final force \(Pf=P0-loftl=4663\) kN
Unit-weight of in-situ concrete \(wc=24\) kN/m³
### Composite section

Height from beam soffit to bottom of in-situ concrete: \( h_{sis}=1350 \text{ mm} \)

Percentage of shrinkage remaining: \( ps=45 \% \)

Percentage: \( pcr=45 \% \)

Percentage: \( stT=60 \% \)

The net moment is: \( M_c=F*e/1E3-M_b=-797.1 \text{ kNm} \)

Stress at top of in-situ concrete:
\[
ft_{isds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3
=0.78066 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
fb_{isds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
=0.043479 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:
\[
ft_{pbsd}=F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3
=0.67508 \text{ N/mm}^2
\]

Stress at bottom of precast concrete beam:
\[
fb_{pbsd}=F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3=-0.34487 \text{ N/mm}^2
\]

### Combination 1 loading

Applied bending moment: \( clb(1)=1750 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
<td>2.9859</td>
<td>0</td>
<td>8.2792</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.78066</td>
<td>0.67508</td>
<td>-0.043479</td>
<td>-0.34487</td>
</tr>
<tr>
<td>Combination 1</td>
<td>4.4095</td>
<td>3.2777</td>
<td>2.6523</td>
<td>5.3866</td>
</tr>
<tr>
<td>Total</td>
<td>3.6289</td>
<td>6.9387</td>
<td>2.6088</td>
<td>2.5477</td>
</tr>
</tbody>
</table>

### Final Stresses N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>16.381</td>
<td>13.427</td>
<td></td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>-5.7172</td>
<td>-4.1328</td>
<td></td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>-2.3848</td>
<td>-1.7239</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>8.2792</td>
<td>7.5703</td>
<td></td>
</tr>
</tbody>
</table>
## Combination 2 to 5 loading

Applied bending moment \( c2b(1) = 2100 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>2.9859</td>
<td>0</td>
<td>8.2792</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.78066</td>
<td>0.67508</td>
<td>-0.043479</td>
<td>-0.34487</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>4.8737</td>
<td>3.6227</td>
<td>2.9315</td>
<td>-6.4639</td>
</tr>
<tr>
<td>Total</td>
<td>4.093</td>
<td>7.2837</td>
<td>2.888</td>
<td>1.4704</td>
</tr>
</tbody>
</table>

### Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 4.093 N/mm² is less than the maximum allowable stress of \( 0.5 \times f_{c\text{uc}} = 20 \text{ N/mm}^2 \).

Top of beam:
Compressive stress 7.2837 N/mm² is less than the maximum allowable stress of \( 0.4 \times f_{cu} = 21 \text{ N/mm}^2 \).

Bottom of in-situ concrete:
Compressive stress 2.888 N/mm² is less than the maximum allowable stress of \( 0.5 \times f_{c\text{uc}} = 20 \text{ N/mm}^2 \).

Bottom of beam:
Compressive stress 1.4704 N/mm² is less than the maximum allowable stress of \( 0.4 \times f_{cu} = 21 \text{ N/mm}^2 \).
Location: Ex3 - 15 metre span YE1 beam

Precast prestressed concrete YE beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=15 m

Beam YE 1 section properties

Depth of beam d=700 mm
Cross-sectional area Ac=414814 mm²
Centroid above soffit Yb=313.39 mm
Section modulus top Zt=43.53E6 mm⁴
Section modulus bottom Zb=53.7E6 mm⁴
Unit weight of beam uwb=24.5 kN/m³

Concrete strengths

Strength at transfer fci=37.5 N/mm²
Characteristic strength fcu=45 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 37.5 N/mm² is:
Eci=30.25 kN/mm²
Modulus of elasticity for concrete strength 45 N/mm² is:
Ecb=32.5 kN/mm²

Composite section properties

Char strength (in-situ concrete) fcuc=30 N/mm²
Modulus of elasticity for concrete strength 30 N/mm² is:
Ecc=28 kN/mm²
Transf'med second moment of area:

\[ I_c = I_{xx} + A_c (Y_b - Y_c)^2 + m_r (I_i + A_i (Y_i - Y_c)^2) = 36.848 \times 10^9 \text{ mm}^4 \]

Depth of composite section \( d_i = 810 \text{ mm} \)
Nominal diameter \( \text{nomd} = 12 \text{ mm} \)
Nominal tensile strength \( \text{nomt} = 1600 \text{ N/mm}^2 \)
Nominal steel area \( \text{noma} = 90 \text{ mm}^2 \)
Characteristic breaking load \( \text{cbl} = 160 \text{ kN} \)
Initial force per strand \( \text{if} = 100 \text{ kN} \)
Modulus of elasticity \( \text{Es} = 200 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( \text{csd}(1) = 2.4 \text{ m} \)
Number of layers of strand \( \text{nls}(1) = 3 \)
Height above soffit \( \text{ds}(1) = 50 \text{ mm} \)
Number of strands \( \text{ns}(1) = 8 \)

**WARNING:**

**Strand location is non-standard.**

Height above soffit \( \text{ds}(2) = 100 \text{ mm} \)
Number of strands \( \text{ns}(2) = 12 \)

**WARNING:**

**Strand location is non-standard.**

Height above soffit \( \text{ds}(3) = 250 \text{ mm} \)
Number of strands \( \text{ns}(3) = 6 \)

**WARNING:**

**Strand location is non-standard.**

Bending moment from temp.loads \( \text{tlb}(1) = 166 \text{ kNm} \)
Bending moment from other loads \( \text{olb}(1) = 191 \text{ kNm} \)

**Short Term Losses**

Relaxation at transfer ( % ) \( \text{str} = 1 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>26</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>185.21</td>
<td>7.1233</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>211.21</td>
<td>8.1233</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 2600 kN
Force at transfer Pt=P0-loft=2388.8 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.8962</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.894</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>3.8135</td>
</tr>
<tr>
<td>Other Loads</td>
<td>4.3878</td>
</tr>
<tr>
<td>Total</td>
<td>9.199</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 9.199 N/mm² is less than allowable stresses of 0.5*fci=18.75 N/mm² and 0.4*fcu=18 N/mm².

Bottom of beam:
Compressive stress 2.9699 N/mm² is less than allowable stresses of 0.5*fci=18.75 N/mm² and 0.4*fcu=18 N/mm².

**Long Term Losses**

Relaxation after transfer( % ) slr(1)=0.8
Shrinkage per unit length sul(1)=0.3E-3
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>26</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>185.21</td>
<td>7.1233</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>211.21</td>
<td>8.1233</td>
</tr>
<tr>
<td>Relaxation</td>
<td>20.8</td>
<td>0.8</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>140.4</td>
<td>5.4</td>
</tr>
<tr>
<td>Creep</td>
<td>142.48</td>
<td>5.4799</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>303.68</td>
<td>11.68</td>
</tr>
<tr>
<td>All Losses</td>
<td>514.88</td>
<td>19.803</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2600 kN
Final force Pf=P0-loftl=2085.1 kN

Unit-weight of in-situ concrete wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.2738</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.894</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.3681</td>
</tr>
<tr>
<td>Total</td>
<td>3.9883</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=660 mm
Perc'age of shrinkage remaining ps=50 %
Percentage pcr=50
Percentage stT=45 %
The net moment is Mc=F*e/1E3-Mb=-135.38 kNm

Stress at top of in-situ concrete:
\[ f_{tisds} = \left( \frac{F}{A_i \cdot mr} - \frac{F}{A_i \cdot mr + Ac} - Mc \cdot 1E3 \cdot (di - Yc) / Ic \right) \cdot mr \cdot 1E3 \]
\[ = -0.12934 \text{ N/mm}^2 \]

Stress at bottom of in-situ concrete:
\[ f_{bisds} = \left( \frac{F}{A_i \cdot mr} - \frac{F}{A_i \cdot mr + Ac} - Mc \cdot 1E3 \cdot (hsis - Yc) / Ic \right) \cdot mr \cdot 1E3 \]
\[ = 0.34545 \text{ N/mm}^2 \]

Stress at top of precast concrete beam:
Stress at bottom of precast concrete beam:

\[
\text{fbpbds} = \frac{F}{(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx} \times 1E3 = 0.06801 \text{ N/mm}^2
\]

\[
	ext{ftpbds} = \frac{F}{(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx} \times 1E3 = -0.16662 \text{ N/mm}^2
\]

**Combination 1 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.12934</td>
</tr>
<tr>
<td>Combination 1</td>
<td>5.4018</td>
</tr>
<tr>
<td>Total</td>
<td>5.2724</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

**Combination 2 to 5 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.12934</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>6.3021</td>
</tr>
<tr>
<td>Total</td>
<td>6.1727</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 6.1727 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².

Top of beam:
Compressive stress 9.047 N/mm² is less than the maximum allowable stress of 0.4*fcu=18 N/mm².

Bottom of in-situ concrete:
Compressive stress 4.1926 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².
Bottom of beam:

Tensile stress 2.1361 N/mm² is less than the maximum allowable tensile stress of 3.0187 N/mm².
Location: Ex1 - 20 metre span YE beam

Precast prestressed concrete YE beams with in-situ top slab


Beam span under consideration span=20 m

Beam YE 4 section properties

- Depth of beam d=1000 mm
- Cross-sectional area Ac=577926 mm²
- Centroid above soffit Yb=463.07 mm
- Section modulus top Zt=95.2E6 mm⁴
- Section modulus bottom Zb=110.33E6 mm⁴
- Unit weight of beam uwb=24 kN/m³

Concrete compressive strengths

- Characteristic cylinder strength fcki=32 N/mm² (transfer)
- Characteristic cylinder strength fck=40 N/mm²

Concrete moduli (short-term)

- Concrete modulus of elasticity Ecmi=33 kN/mm² (transfer)
- Concrete modulus of elasticity Ecmb=35 kN/mm²

Composite section properties

- Char strength (in-situ concrete) fckc=32 N/mm²
- Short-term modulus of elasticity for in-situ concrete Ecmc=33 kN/mm²
Second moment of area:
\[ I_c = I_{yy} + A_c (Y_b - Y_c)^2 + m_r (I_i + A_i (Y_i - Y_c)^2) = 91.223 \times 10^9 \text{ mm}^4 \]

Depth of composite section \( d_i = 1125 \text{ mm} \)

Modulus of elasticity (strand) \( E_p = 195 \text{ kN/mm}^2 \)

**Cross-section 1**

- Distance from mid-span of beam \( c_{sd}(1) = 0 \text{ m} \)
- Number of layers of strand \( n_{ls}(1) = 4 \)
- Height above soffit \( d_s(1) = 60 \text{ mm} \)
- Number of strands \( n_s(1) = 11 \)
- Height above soffit \( d_s(2) = 110 \text{ mm} \)
- Number of strands \( n_s(2) = 14 \)
- Height above soffit \( d_s(3) = 800 \text{ mm} \)
- Number of strands \( n_s(3) = 2 \)
- Height above soffit \( d_s(4) = 900 \text{ mm} \)
- Number of strands \( n_s(4) = 2 \)

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer ( % ) \( \text{str}=1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force ( \text{kN} )</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>349.37</td>
<td>6.9237</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>399.83</td>
<td>7.9237</td>
</tr>
</tbody>
</table>

Losses at Transfer

- Initial force \( 5046 \text{ kN} \)
- Force at transfer \( P_t = F_0 - \text{loft} = 4646.2 \text{ kN} \)
### Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

**Top of beam:**
Compressive stress 2.1457 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Bottom of beam:**
Compressive stress 13.122 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>349.37</td>
<td>6.9237</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>399.83</td>
<td>7.9237</td>
</tr>
<tr>
<td>Relaxation</td>
<td>100.92</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>235.81</td>
<td>4.6733</td>
</tr>
<tr>
<td>Creep</td>
<td>494.21</td>
<td>9.794</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>830.94</td>
<td>16.467</td>
</tr>
<tr>
<td>All Losses</td>
<td>1230.8</td>
<td>24.391</td>
</tr>
</tbody>
</table>

**Initial force**
5046 kN

**Final force**
Pf=P0-loftl=3815.2 kN

**Unit-weight of in-situ concrete**
w=24 kN/m³

---

---
### Composite section

Height from beam soffit to bottom of in-situ concrete \( h_{sis} = 965 \text{ mm} \)

Percentage of shrinkage remaining \( p_s = 25 \% \)

Percentage \( p_c = 25 \% \)

Percentage \( p_{stT} = 67 \% \)

The net moment is \( M_c = F*\epsilon/1E3 - M_b = -79.319 \text{ kNm} \)

Stress at top of in-situ concrete:
\[
f_{tisds} = \frac{F}{(A_i * \epsilon)} - \frac{F}{(A_i * \epsilon + A_c)} - M_c * 1E3 * (d_i - Y_c) / I_c * \epsilon * 1E3
\]
\[
= -0.76325 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
f_{bisds} = \frac{F}{(A_i * \epsilon)} - \frac{F}{(A_i * \epsilon + A_c)} - M_c * 1E3 * (h_{sis} - Y_c) / I_c * \epsilon * 1E3
\]
\[
= -0.62412 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:
\[
f_{tpbds} = \frac{F}{(A_i * \epsilon + A_c)} + M_c * 1E3 * (d - Y_c) / I_c + M_b * 1E3 * Y_t / I_{yy} * 1E3
\]
\[
= 0.7972 \text{ N/mm}^2
\]

Stress at bottom of precast concrete beam:
\[
f_{bpbds} = \frac{F}{(A_i * \epsilon + A_c)} - M_c * 1E3 * Y_c / I_c - M_b * 1E3 * Y_b / I_{yy} * 1E3
\]
\[
= -0.36425 \text{ N/mm}^2
\]

### Loading Combination 1

(no tensile stress permitted in precast beam)

Characteristic bending moment \( c_{lb(1)} = 1000 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-0.76325</td>
</tr>
<tr>
<td>Total</td>
<td>6.4041</td>
</tr>
</tbody>
</table>

---

Sample output for SCALE Proforma 139. (ans=1)           Page: 4
Reinforced & prestressed concrete: BS5400 & Eurocode   Made by: IFB
YE beams with insitu top slab                           Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.              Ref No: SC139 EC
**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment  \( c_2b(1) = 1500 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>-0.76325</td>
</tr>
<tr>
<td></td>
<td>8.9592</td>
</tr>
<tr>
<td>Total</td>
<td>8.1959</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 8.1959 N/mm\(^2\) is less than the maximum allowable stress of 0.6\(f_{ckc}\)=19.2 N/mm\(^2\).

Top of beam:
Compressive stress 12.574 N/mm\(^2\) is less than the maximum allowable stress of 0.6\(f_{ck}\)=24 N/mm\(^2\).

Bottom of in-situ concrete:
Compressive stress 5.7041 N/mm\(^2\) is less than the maximum allowable stress of 0.6\(f_{ckc}\)=19.2 N/mm\(^2\).

Bottom of beam:
Tensile stress 1.811 N/mm\(^2\) is less than the maximum allowable tensile stress of 3.0263 N/mm\(^2\).
Location: Ex2 - 29.5 metre span YE8 beam

Precast prestressed concrete YE beams with in-situ top slab


Beam span under consideration span=29.5 m

Beam YE 8 section properties

Depth of beam d=1400 mm
Cross-sectional area Ac=815050 mm²
Centroid above soffit Yb=676.56 mm
Section modulus top Zt=200.07E6 mm³
Section modulus bottom Zb=213.93E6 mm³
Unit weight of beam uwb=24.5 kN/m³

Concrete compressive strengths

Characteristic cylinder strength fcki=32 N/mm² (transfer)
Characteristic cylinder strength fck=42 N/mm²

Concrete moduli (short-term)

Concrete modulus of elasticity Ecmi=31 kN/mm² (transfer)
Concrete modulus of elasticity Ecmb=36 kN/mm²

Composite section properties

Char strength (in-situ concrete) fckc=32 N/mm²
Short-term modulus of elasticity for in-situ concrete Ecmc=33 kN/mm²

Item No. 1

X co-ordinate Y co-ordinate X(1)=500 mm Y(1)=0 mm
X co-ordinate Y co-ordinate X(2)=1250 mm Y(2)=0 mm
X co-ordinate Y co-ordinate X(3)=1250 mm Y(3)=202 mm
X co-ordinate Y co-ordinate X(4)=975 mm Y(4)=379 mm
X co-ordinate Y co-ordinate X(5)=1115 mm Y(5)=1350 mm
X co-ordinate Y co-ordinate X(6)=1615 mm Y(6)=1350 mm
X co-ordinate Y co-ordinate X(7)=1615 mm Y(7)=1550 mm
X co-ordinate Y co-ordinate X(8)=500 mm Y(8)=1550 mm
X co-ordinate Y co-ordinate X(9)=500 mm Y(9)=1650 mm
X co-ordinate
Y co-ordinate
X co-ordinate
Y co-ordinate
X co-ordinate
Y co-ordinate
X co-ordinate
Y co-ordinate
Depth of composite section
Area of in-situ concrete
Centroid of in-situ concrete
Modulus of elasticity (strand)

Cross-section 1

Distance from mid-span of beam
Number of layers of strand
Height above soffit
Number of strands
Height above soffit
Number of strands
Height above soffit
Number of strands
Height above soffit
Number of strands

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer (%)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force (kN)</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>96</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>550.26</td>
<td>8.5978</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>646.26</td>
<td>10.098</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 6400 kN
Force at transfer $P_t = P_0 - loft = 5753.7$ kN
Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam:
The flexural tensile stress of 0.89217 N/mm² at transfer is less than the permissible value of 1 N/mm² (as was permitted in BS5400-4).

Bottom of beam:
Compressive stress 14.496 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=25.2 N/mm².

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer( % ) \( slr(1)=1 \)
Shrinkage per unit length \( sul(1)=0.3E-3 \)
Creep coefficient \( \varphi(\infty,\tau_0) \) \( crf=2 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>96</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>550.26</td>
<td>8.5978</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>646.26</td>
<td>10.098</td>
</tr>
<tr>
<td>Relaxation</td>
<td>64</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>308.88</td>
<td>4.8263</td>
</tr>
<tr>
<td>Creep</td>
<td>714.47</td>
<td>11.164</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>1087.3</td>
<td>16.99</td>
</tr>
<tr>
<td>All Losses</td>
<td>1733.6</td>
<td>27.088</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force \( 6400 \text{ kN} \)
Final force \( Pf=P_0-lofl=4666.4 \text{ kN} \)

Unit-weight of in-situ concrete \( wc=24 \text{ kN/m}^3 \)
## Composite section

Height from beam soffit to bottom of in-situ concrete: \( h_{sis} = 1350 \text{ mm} \)
Percentage of shrinkage remaining: \( ps = 45 \% \)
Percentage: \( pcr = 45 \) %
Percentage: \( stT = 60 \% \)
The net moment is: \( Mc = 1003.1 \text{ kNm} \)

### Beam concrete stresses - Long term

**Stress at top of in-situ concrete:**
\[
ft_{is} = \frac{F}{(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3} = -0.97236 \text{ N/mm}^2
\]

**Stress at bottom of in-situ concrete:**
\[
fb_{is} = \frac{F}{(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3} = 0.020124 \text{ N/mm}^2
\]

**Stress at top of precast concrete beam:**
\[
ft_{pb} = \frac{F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)*1E3} = 0.76651 \text{ N/mm}^2
\]

**Stress at bottom of precast concrete beam:**
\[
fb_{pb} = \frac{F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)*1E3}{1E3} = -0.39599 \text{ N/mm}^2
\]

### Loading Combination 1

(no tensile stress permitted in precast beam)

**Characteristic bending moment** \( clb(1) = 1750 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.97236</td>
</tr>
<tr>
<td>Combination 1</td>
<td>4.608</td>
</tr>
<tr>
<td>Total</td>
<td>3.6356</td>
</tr>
</tbody>
</table>

---

**Final Stresses N/mm²**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-5.6816</td>
<td>16.393</td>
<td>13.437</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.1134</td>
<td>-5.7172</td>
<td>-4.1328</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.55</td>
<td>-2.3848</td>
<td>-1.7239</td>
</tr>
<tr>
<td>Total</td>
<td>2.9818</td>
<td>8.291</td>
<td>7.58</td>
</tr>
</tbody>
</table>
Loading Combination 2

(tensile stress permitted (no visible cracking))
Characteristic bending moment \( c_{2b}(1) = 2100 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>-0.97236</td>
</tr>
<tr>
<td></td>
<td>5.093</td>
</tr>
<tr>
<td>Total</td>
<td>4.1207</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 4.1207 N/mm\(^2\) is less than the maximum allowable stress of 0.6\( *f_{ckc}=19.2 \text{ N/mm}^2\).

Top of beam:
Compressive stress 7.1098 N/mm\(^2\) is less than the maximum allowable stress of 0.6\( *f_{ck}=25.2 \text{ N/mm}^2\).

Bottom of in-situ concrete:
Compressive stress 3.0354 N/mm\(^2\) is less than the maximum allowable stress of 0.6\( *f_{ckc}=19.2 \text{ N/mm}^2\).

Bottom of beam:
Compressive stress 1.5602 N/mm\(^2\) is less than the maximum allowable stress of 0.6\( *f_{ck}=25.2 \text{ N/mm}^2\).
Precast prestressed concrete YE beams with in-situ top slab


Beam span under consideration  span=15 m

Beam YE 1 section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=700 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=414814 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=313.39 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>Zt=43.53E6 mm³</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>Zb=53.7E6 mm³</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=24.5 kN/m³</td>
</tr>
</tbody>
</table>

Concrete compressive strengths

- Characteristic cylinder strength fcki=30 N/mm² (transfer)
- Characteristic cylinder strength fck=36 N/mm²

Concrete moduli (short-term)

- Concrete modulus of elasticity Ecmi=33 kN/mm² (transfer)
- Concrete modulus of elasticity Ecmb=34 kN/mm²

Composite section properties

- Char strength (in-situ concrete) fckc=24 N/mm²
- Short-term modulus of elasticity for in-situ concrete Ecmc=31 kN/mm²
Transf'med second moment of area:
\[ I_c = I_{yy} + A_c (Y_b - Y_c)^2 + m_r (I_i + A_i (Y_i - Y_c)^2) = 37.702E9 \text{ mm}^4 \]

Depth of composite section \( d_i = 810 \text{ mm} \)
Nominal diameter \( \text{nomd} = 12 \text{ mm} \)
Nominal tensile strength \( \text{nomt} = 1600 \text{ N/mm}^2 \)
Nominal steel area \( \text{noma} = 90 \text{ mm}^2 \)
Char breaking load (strand) \( f_{pk} = 160 \text{ kN} \)
Initial force per strand \( \text{ipf} = 100 \text{ kN} \)
Modulus of elasticity (strand) \( E_p = 195 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( c_{sd(1)} = 2.4 \text{ m} \)
Number of layers of strand \( n_{ls(1)} = 3 \)
Height above soffit \( d_{s(1)} = 50 \text{ mm} \)
Number of strands \( n_{s(1)} = 8 \)

**WARNING:**
Strand location is non-standard.

Height above soffit \( d_{s(2)} = 100 \text{ mm} \)
Number of strands \( n_{s(2)} = 12 \)

**WARNING:**
Strand location is non-standard.

Height above soffit \( d_{s(3)} = 250 \text{ mm} \)
Number of strands \( n_{s(3)} = 6 \)

**WARNING:**
Strand location is non-standard.

Bending moment from temp.loads \( t_{lb(1)} = 166 \text{ kNm} \)
Bending moment from other loads \( o_{lb(1)} = 191 \text{ kNm} \)

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer ( % ) \( \text{str} = 1 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>26</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>155.53</td>
<td>5.9818</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>181.53</td>
<td>6.9818</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 2600 kN
Force at transfer $P_t = P_0 - \text{loft} = 2418.5$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.957</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.894</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>3.8135</td>
</tr>
<tr>
<td>Other Loads</td>
<td>4.3878</td>
</tr>
<tr>
<td>Total</td>
<td>9.1382</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam:
Compressive stress 9.1382 N/mm² is less than allowable stresses of $0.7*f_{ck,i}=21$ N/mm² and $0.6*f_{ck}=21.6$ N/mm².

Bottom of beam:
Compressive stress 3.1488 N/mm² is less than allowable stresses of $0.7*f_{ck,i}=21$ N/mm² and $0.6*f_{ck}=21.6$ N/mm².

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer (%) $\text{slr}(1)=0.8$
Shrinkage per unit length $\text{sul}(1)=0.3E-3$
Creep coefficient $\phi(\omega, to)$ $\text{crf}=2$
## Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>26</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>155.53</td>
<td>5.9818</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>181.53</td>
<td>6.9818</td>
</tr>
<tr>
<td>Relaxation</td>
<td>20.8</td>
<td>0.8</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>136.89</td>
<td>5.265</td>
</tr>
<tr>
<td>Creep</td>
<td>163.31</td>
<td>6.281</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>321</td>
<td>12.346</td>
</tr>
<tr>
<td>All Losses</td>
<td>502.52</td>
<td>19.328</td>
</tr>
</tbody>
</table>

**Losses - Long term**

- **Initial force**: 2600 kN
- **Final force**: Pf=P0-loftl=2097.5 kN

**Unit-weight of in-situ concrete**: wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.2991</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.894</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.3681</td>
</tr>
<tr>
<td>Total</td>
<td>3.963</td>
</tr>
</tbody>
</table>

**Beam concrete stresses - Long term**

**Height from beam soffit to bottom of in-situ concrete**: hsis=660 mm

**Percentage of shrinkage remaining**: ps=50 %

**Percentage of creep**: pcr=50

**Percentage of stress**: stT=45 %

The net moment is: \( Mc=F*e/1E3-M_b=-188.03 \) kNm

Stress at top of in-situ concrete:
\[
ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3
=-0.12258 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
=0.5595 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:
ftpbds = (F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)*1E3
= -0.37088 N/mm²

Stress at bottom of precast concrete beam:
fbpbds = (F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)*1E3
= 0.15735 N/mm²

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment  c1b(1)=400 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-0.12258</td>
</tr>
<tr>
<td>Total</td>
<td>5.396</td>
</tr>
</tbody>
</table>

**Loading Combination 1 - Stresses**

**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment  c2b(1)=700 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>-0.12258</td>
</tr>
<tr>
<td>Total</td>
<td>6.3158</td>
</tr>
</tbody>
</table>

**Loading Combination 2 - Stresses**

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 6.3158 N/mm² is less than the maximum allowable stress of 0.6*fckc=14.4 N/mm².

Top of beam:
Compressive stress 8.6112 N/mm² is less than the maximum allowable stress of 0.6*fck=21.6 N/mm².
Bottom of in-situ concrete:
Compressive stress 4.4586 N/mm² is less than the maximum allowable stress of 0.6*fckc=14.4 N/mm².

Bottom of beam:
Tensile stress 1.8775 N/mm² is less than the maximum allowable tensile stress of 2.9399 N/mm².
**Location:** Ex1 - 13.5 metre span T beam

**Precast prestressed concrete inverted T beams with in-fill concrete**

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam  
\[ \text{span} = 13.5 \text{ m} \]

**Beam T 5 section properties**

- Depth of beam  
  \[ d = 615 \text{ mm} \]
- Cross-sectional area  
  \[ A_c = 130560 \text{ mm}^2 \]
- Centroid above soffit  
  \[ Y_b = 244 \text{ mm} \]
- Section modulus top  
  \[ Z_t = 14.3 \times 10^6 \text{ mm}^3 \]
- Section modulus bottom  
  \[ Z_b = 21.78 \times 10^6 \text{ mm}^3 \]
- Unit weight of beam  
  \[ u_b = 25 \text{ kN/m}^3 \]

**Concrete strengths**

- Strength at transfer  
  \[ f_{ci} = 40 \text{ N/mm}^2 \]
- Characteristic strength  
  \[ f_{cu} = 50 \text{ N/mm}^2 \]

**Concrete moduli**

- Modulus of elasticity at transfer for concrete strength 40 N/mm² is:  
  \[ E_{ci} = 31 \text{ kN/mm}^2 \]
- Modulus of elasticity for concrete strength 50 N/mm² is:  
  \[ E_{cb} = 34 \text{ kN/mm}^2 \]

**Composite section properties**

- Char strength (in-situ concrete)  
  \[ f_{cuc} = 30 \text{ N/mm}^2 \]
- Modulus of elasticity for concrete strength 30 N/mm² is:  
  \[ E_{cc} = 28 \text{ kN/mm}^2 \]

Composite section is rectangular in shape.

- Breadth of rectangular section  
  \[ b = 508 \text{ mm} \]
- Depth of rectangular section  
  \[ d_i = 690 \text{ mm} \]
- Transf’med second moment of area  
  \[ I_c = I_{xx} + A_c (Y_b - Y_c)^2 + m_r (I_i + A_i (Y_i - Y_c)^2) = 12.608 \times 10^9 \text{ mm}^4 \]
Initial force per strand \( if = 116 \, \text{kN} \)
Modulus of elasticity \( Es = 200 \, \text{kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( csd(1) = 0 \, \text{m} \)
Number of layers of strand \( nls(1) = 4 \)
Height above soffit \( ds(1) = 50 \, \text{mm} \)
Number of strands \( ns(1) = 7 \)
Height above soffit \( ds(2) = 90 \, \text{mm} \)
Number of strands \( ns(2) = 6 \)
Height above soffit \( ds(3) = 540 \, \text{mm} \)
Number of strands \( ns(3) = 1 \)
Height above soffit \( ds(4) = 580 \, \text{mm} \)
Number of strands \( ns(4) = 1 \)

**Short Term Losses**

Relaxation at transfer ( % ) \( str = 1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force ( \text{kN} )</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>17.4</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>154.04</td>
<td>8.8531</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>171.44</td>
<td>9.8531</td>
</tr>
</tbody>
</table>

Losses at Transfer

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer ( \text{N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>Top of Beam: 5.191</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>-0.031274</td>
</tr>
<tr>
<td>Total</td>
<td>5.1598</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 5.1598 \( \text{N/mm}^2 \) is less than allowable stresses of \( 0.5\times fci = 20 \, \text{N/mm}^2 \) and \( 0.4\times fcu = 20 \, \text{N/mm}^2 \).
Bottom of beam:
Compressive stress 16.522 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

**Long Term Losses**

Relaxation after transfer ( % ) slr(1)=1.5
Shrinkage per unit length sul(1)=0.3E-3

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>17.4</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>154.04</td>
<td>8.8531</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>171.44</td>
<td>9.8531</td>
</tr>
<tr>
<td>Relaxation</td>
<td>26.1</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>83.7</td>
<td>4.8103</td>
</tr>
<tr>
<td>Creep</td>
<td>210.6</td>
<td>12.104</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>320.4</td>
<td>18.414</td>
</tr>
<tr>
<td>All Losses</td>
<td>491.85</td>
<td>28.267</td>
</tr>
</tbody>
</table>

**Losses - Long term**

Initial force 1740 kN
Final force Pf=P0-loftl=1248.2 kN

Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.024885</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.191</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>8.7456</td>
</tr>
<tr>
<td>Total</td>
<td>13.912</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

**Composite section**

Height from beam soffit to bottom of in-situ concrete hsis=100 mm
Perc'age of shrinkage remaining \( ps = 25 \% \)
Percentage \( pcr = 25 \%
Percentage \( stT = 50 \% \)

Apply moment to beam to straighten it as follows:
The net moment is \( Mc = F*e/1E3 - Mb = 2.9973 \text{ kNm} \)

Stress at top of in-situ concrete:
\[
ftisds = - \frac{F}{(Ai*mr)} - \frac{F}{(Ai*mr+Ac)} - Mc*1E3*(di-Yc)/Ic)*mr*1E3 \\
= 0.48626 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
fbisds = -\frac{F}{(Ai*mr)} - \frac{F}{(Ai*mr+Ac)} - Mc*1E3*(hsis-Yc)/Ic)*mr*1E3 \\
= 0.37075 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:
\[
ftpbds = \frac{F}{(Ai*mr+Ac)} + Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3 \\
= -1.8775 \text{ N/mm}^2
\]

Stress at bottom of precast concrete beam:
\[
fbpbds = \frac{F}{(Ai*mr+Ac)} - Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3 = 0.03264 \text{ N/mm}^2
\]

### Combination 1 loading

**Applied bending moment** \( c1b(1) = 204 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm(^2)</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>13.912</td>
<td>0</td>
<td>6.698</td>
<td></td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>0.48626</td>
<td>-1.8775</td>
<td>0.37075</td>
<td>0.03264</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.5253</td>
<td>5.2816</td>
<td>-3.1651</td>
<td>-5.4613</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>6.0115</td>
<td>17.316</td>
<td>-2.7943</td>
<td>1.2693</td>
<td></td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

### Combination 2 to 5 loading

**Applied bending moment** \( c2b(1) = 260 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm(^2)</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>13.912</td>
<td>0</td>
<td>6.698</td>
<td></td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2-5</td>
<td>0.48626</td>
<td>-1.8775</td>
<td>0.37075</td>
<td>0.03264</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.9857</td>
<td>5.7218</td>
<td>-4.0339</td>
<td>-6.9605</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>6.472</td>
<td>17.756</td>
<td>-3.6632</td>
<td>-0.22986</td>
<td></td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.
Top of in-situ concrete:
Compressive stress 6.472 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².

Top of beam:
Compressive stress 17.756 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².

WARNING:
Bottom of in-situ concrete:
Tensile stress 3.6632 N/mm² exceeds maximum allowable tensile stress of 3.6 N/mm² from Table 32.

Bottom of beam:
Tensile stress 0.22986 N/mm² is less than the maximum allowable tensile stress of 3.182 N/mm².
Location: Ex2 - 13.5 metre span edge beam

Precast prestressed concrete inverted T beams with in-fill concrete

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=13.5 m

Beam T 5 section properties

Depth of beam d=615 mm
Cross-sectional area Ac=130560 mm²
Centroid above soffit Yb=244 mm
Section modulus top Zt=14.3E6 mm³
Section modulus bottom Zb=21.78E6 mm³
Unit weight of beam uwb=25 kN/m³

Concrete strengths

Strength at transfer fci=40 N/mm²
Characteristic strength fcu=50 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 40 N/mm² is: Eci=31 kN/mm²
Modulus of elasticity for concrete strength 50 N/mm² is: Ecb=34 kN/mm²

Composite section properties

Char strength (in-situ concrete) fcuc=30 N/mm²
Modulus of elasticity for concrete strength 30 N/mm² is: Ecc=28 kN/mm²

Item No. 1

Item weighting iw1=0.82353
X co-ordinate X(1)=0 mm
Y co-ordinate Y(1)=0 mm
X co-ordinate X(2)=508 mm
Y co-ordinate Y(2)=0 mm
X co-ordinate X(3)=508 mm
Y co-ordinate Y(3)=250 mm
X co-ordinate X(4)=650 mm
Y co-ordinate Y(4)=350 mm
X co-ordinate X(5)=650 mm
Y co-ordinate Y(5)=790 mm
X co-ordinate X(6)=150 mm
Y co-ordinate Y(6)=790 mm
X co-ordinate X(7)=150 mm
Y co-ordinate Y(7)=690 mm
X co-ordinate X(8)=0 mm
Y co-ordinate         Y(8)=690 mm
X co-ordinate         X(9)=0 mm
Y co-ordinate         Y(9)=0 mm

Item No. 2

Second moment of area  IXX2=5.3143E9 mm^4
Area of item           A2=130560 mm^2
Centroid from X-X axis YC2=244 mm
Item weighting         iw2=0.17647
Depth of composite section di=790 mm
Nominal diameter       nomd=12 mm
Nominal tensile strength nfmt=1700 N/mm^2
Nominal steel area     noma=100 mm^2
Characteristic breaking load cbl=150 kN
Initial force per strand if=100 kN
Modulus of elasticity  Es=200 kN/mm^2

Cross-section 1

Distance from mid-span of beam csd(1)=2 m
Number of layers of strand nls(1)=4
Height above soffit ds(1)=49 mm
Number of strands ns(1)=10

WARNING:
Strand location is non-standard.

Height above soffit ds(2)=85 mm
Number of strands ns(2)=8

WARNING:
Strand location is non-standard.

Height above soffit ds(3)=520 mm
Number of strands ns(3)=2

WARNING:
Strand location is non-standard.

Height above soffit ds(4)=580 mm
Number of strands ns(4)=2

WARNING:
Number of strands exceeds maximum for this location.

Bending moment from temp.loads tlb(1)=80 kNm
Bending moment from other loads olb(1)=50 kNm

Short Term Losses

Relaxation at transfer ( % ) str=1
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>284.76</td>
<td>12.943</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>306.76</td>
<td>13.943</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force: 2200 kN
Force at transfer: Pt=P0-loft=1893.2 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stress at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>2.4975</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.7353</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>5.5849</td>
</tr>
<tr>
<td>Other Loads</td>
<td>3.4906</td>
</tr>
<tr>
<td>Total</td>
<td>16.308</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
Compressive stress 16.308 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Bottom of beam:
Compressive stress 13.312 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Long Term Losses

Relaxation after transfer( % ) slr(1)=1.5
Shrinkage per unit length      sul(1)=0.3E-3
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>284.76</td>
<td>12.943</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>306.76</td>
<td>13.943</td>
</tr>
<tr>
<td>Relaxation</td>
<td>33</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>132</td>
<td>6</td>
</tr>
<tr>
<td>Creep</td>
<td>370.41</td>
<td>16.837</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>535.41</td>
<td>24.337</td>
</tr>
<tr>
<td>All Losses</td>
<td>842.16</td>
<td>38.28</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2200 kN
Final force Pf=P0-loftl=1357.8 kN

Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>1.7912</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.7353</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>11.8</td>
</tr>
<tr>
<td>Total</td>
<td>18.326</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=100 mm
Percentage of shrinkage remaining ps=40 %
Percentage pcr=40
Percentage stT=50 %

Apply moment to beam to straighten it as follows:

The net moment is Mc=F*e/1E3-Mb=-73.022 kNm

Stress at top of in-situ concrete:
ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3
=0.61842 N/mm²

Stress at bottom of in-situ concrete:
fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
=2.7339 N/mm²

Stress at top of precast concrete beam:
ftpbds = (F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3
Stress at bottom of precast concrete beam:

fbpdbd = (F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3=-0.21965 N/mm²

**Combination 1 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>18.326</td>
<td>0</td>
<td>5.1871</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.61842</td>
<td>-9.6815</td>
<td>2.7339</td>
<td>-0.21965</td>
</tr>
<tr>
<td>Combination 1</td>
<td>9.8996</td>
<td>6.6678</td>
<td>-6.8588</td>
<td>-11.133</td>
</tr>
<tr>
<td>Total</td>
<td>10.518</td>
<td>15.313</td>
<td>-4.125</td>
<td>-6.1651</td>
</tr>
</tbody>
</table>

**Combination 1 Loading - Stresses**

**WARNING:**

Bottom of beam:
Tensile stress 6.1651 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

**Combination 2 to 5 loading**

Applied bending moment c2b(1)=625 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>18.326</td>
<td>0</td>
<td>5.1871</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.61842</td>
<td>-9.6815</td>
<td>2.7339</td>
<td>-0.21965</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>10.312</td>
<td>6.9456</td>
<td>-7.7941</td>
<td>-12.651</td>
</tr>
<tr>
<td>Total</td>
<td>10.931</td>
<td>15.591</td>
<td>-5.0603</td>
<td>-7.6832</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 Loading - Stresses**

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 10.931 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².

Top of beam:
Compressive stress 15.591 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².
WARNING:
Bottom of in-situ concrete:
Tensile stress 5.0603 N/mm² exceeds maximum allowable
tensile stress of 3.6 N/mm² from Table 32.

WARNING:
Bottom of beam:
Tensile stress 7.6832 N/mm² exceeds maximum allowable
tensile stress of 3.182 N/mm².
Precast prestressed concrete inverted T beams with in-fill concrete

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam
span=17 m

Beam T 9 section properties

Depth of beam
d=775 mm
Cross-sectional area
Ac=163360 mm²
Centroid above soffit
Yb=334 mm
Section modulus top
Zt=24.32E6 mm⁴
Section modulus bottom
Zb=32.06E6 mm⁴
Unit weight of beam
uwb=25 kN/m³

Concrete strengths

Strength at transfer
fci=35 N/mm²
Characteristic strength
fcu=40 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 35 N/mm² is:
Eci=29.5 kN/mm²
Modulus of elasticity for concrete strength 40 N/mm² is:
Ecb=31 kN/mm²

Composite section properties

Char strength (in-situ concrete)
f cuc=35 N/mm²
Modulus of elasticity for concrete strength 35 N/mm² is:
Ecc=29.5 kN/mm²
Composite section is rectangular in shape.

Breadth of rectangular section
b=550 mm
Depth of rectangular section
di=850 mm
Second moment of area
Ic=Ixx+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi-Yc)^2)=28.147E9 mm⁶
Sample output for SCALE Proforma 140. (ans=3)  
Reinforced&prestressed concrete: BS5400 & Eurocode  
Inverted T beam  
Date: 02/12/19  
Made by: IFB  
Ref No: SC140 BS

Initial force per strand \( if = 200 \text{ kN} \)
Modulus of elasticity \( E_s = 200 \text{ kN/mm}^2 \)

Cross-section 1

- Distance from mid-span of beam \( csd(1) = 6 \text{ m} \)
- Number of layers of strand \( nls(1) = 6 \)
- Height above soffit \( ds(1) = 50 \text{ mm} \)
- Number of strands \( ns(1) = 4 \)
- Height above soffit \( ds(2) = 90 \text{ mm} \)
- Number of strands \( ns(2) = 3 \)
- Height above soffit \( ds(3) = 260 \text{ mm} \)
- Number of strands \( ns(3) = 1 \)
- Height above soffit \( ds(4) = 500 \text{ mm} \)
- Number of strands \( ns(4) = 1 \)
- Height above soffit \( ds(5) = 620 \text{ mm} \)
- Number of strands \( ns(5) = 1 \)
- Height above soffit \( ds(6) = 740 \text{ mm} \)
- Number of strands \( ns(6) = 1 \)

Short Term Losses

Relaxation at transfer ( % ) \( str = 1 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>254.58</td>
<td>11.572</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>276.58</td>
<td>12.572</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 2200 kN
Force at transfer \( Pt = P_0 - loft = 1923.4 \text{ kN} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>3.9679</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.0485</td>
</tr>
<tr>
<td>Total</td>
<td>7.0165</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer
**Stress Limitations**

Top of beam:
Compressive stress 7.0165 N/mm$^2$ is less than allowable stresses of 0.5$f_{ci}$=17.5 N/mm$^2$ and 0.4$f_{cu}$=16 N/mm$^2$.

Bottom of beam:
Compressive stress 15.377 N/mm$^2$ is less than allowable stresses of 0.5$f_{ci}$=17.5 N/mm$^2$ and 0.4$f_{cu}$=16 N/mm$^2$.

**Long Term Losses**

Relaxation after transfer( % ) slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>254.58</td>
<td>11.572</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>276.58</td>
<td>12.572</td>
</tr>
<tr>
<td>Relaxation</td>
<td>44</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>147.18</td>
<td>6.69</td>
</tr>
<tr>
<td>Creep</td>
<td>400.43</td>
<td>18.202</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>591.61</td>
<td>26.892</td>
</tr>
<tr>
<td>All Losses</td>
<td>868.19</td>
<td>39.463</td>
</tr>
</tbody>
</table>

**Losses - Long term**

Initial force 2200 kN
Final force $P_f=P_0-loftl=1331.8$ kN

Unit-weight of in-situ concrete $w_c=25$ kN/m$^3$

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>2.7475</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.0485</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>5.6757</td>
</tr>
<tr>
<td>Total</td>
<td>11.472</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term
Composite section

Height from beam soffit to bottom of in-situ concrete \( h_{\text{sis}} = 100 \text{ mm} \)

### Combination 1 loading

Applied bending moment \( c_{1b}(1) = 200 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Combination 1</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>3.3973</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

### Combination 2 to 5 loading

Applied bending moment \( c_{2b}(1) = 250 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Combination 2-5</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>3.7748</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 3.7748 N/mm\(^2\) is less than the maximum allowable stress of 0.5*\( f_{\text{uc}} \)=17.5 N/mm\(^2\).

Top of beam:
Compressive stress 14.58 N/mm\(^2\) is less than the maximum allowable stress of 0.4*\( f_{\text{cu}} \)=16 N/mm\(^2\).

Bottom of in-situ concrete:
Tensile stress 2.8866 N/mm\(^2\) is less than the maximum allowable tensile stress of 4 N/mm\(^2\) from Table 32.
Bottom of beam:
Compressive stress 1.8641 N/mm² is less than the maximum allowable stress of 0.4*fcu=16 N/mm².
Location: Ex1 - 17.0 metre span T9 beam

Precast prestressed concrete inverted T beams with in-fill concrete


Beam span under consideration \( \text{span}=17 \text{ m} \)

**Beam T 9 section properties**

- Depth of beam \( d=775 \text{ mm} \)
- Cross-sectional area \( A_c=163360 \text{ mm}^2 \)
- Centroid above soffit \( Y_b=334 \text{ mm} \)
- Section modulus top \( Z_t=24.32E6 \text{ mm}^3 \)
- Section modulus bottom \( Z_b=32.06E6 \text{ mm}^3 \)
- Unit weight of beam \( u_w=25 \text{ kN/m}^3 \)

**Concrete compressive strengths**

- Characteristic cylinder strength \( f_{ck}=28 \text{ N/mm}^2 \) (transfer)
- Characteristic cylinder strength \( f_{ck}=32 \text{ N/mm}^2 \)

**Concrete moduli (short-term)**

- Concrete modulus of elasticity \( E_{cmi}=32 \text{ kN/mm}^2 \) (transfer)
- Concrete modulus of elasticity \( E_{cmb}=33 \text{ kN/mm}^2 \)

**Composite section properties**

- Char strength (in-situ concrete) \( f_{ckc}=32 \text{ N/mm}^2 \)
- Short-term modulus of elasticity for in-situ concrete \( E_{cmc}=33 \text{ kN/mm}^2 \)

Composite section is rectangular in shape.

- Breadth of rectangular section \( b=550 \text{ mm} \)
- Depth of rectangular section \( d_i=850 \text{ mm} \)
- Second moment of area \( I_c=I_{yy}+A_c(Y_b-Y_c)^2+mr(I_{ii}+A_i(Y_i-Y_c)^2)=28.147E9 \text{ mm}^4 \)
- Modulus of elasticity (strand) \( E_p=195 \text{ kN/mm}^2 \)
Cross-section 1

Distance from mid-span of beam          csd(1)=6 m
Number of layers of strand             nls(1)=6
Height above soffit                    ds(1)=50 mm
Number of strands                      ns(1)=4
Height above soffit                    ds(2)=90 mm
Number of strands                      ns(2)=3
Height above soffit                    ds(3)=260 mm
Number of strands                      ns(3)=1
Height above soffit                    ds(4)=500 mm
Number of strands                      ns(4)=1
Height above soffit                    ds(5)=620 mm
Number of strands                      ns(5)=1
Height above soffit                    ds(6)=740 mm
Number of strands                      ns(6)=1

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % ) str=1

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>207.07</td>
<td>9.4121</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>229.07</td>
<td>10.412</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force                     2200 kN
Force at transfer                 Pt=P0-loft=1970.9 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>4.0659</td>
<td>18.123</td>
<td>13.852</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.0485</td>
<td>-2.3089</td>
<td>-0.68122</td>
</tr>
<tr>
<td>Total</td>
<td>7.1145</td>
<td>15.814</td>
<td>13.171</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations ( BS EN 1992-1-1, Clause 5.10.2.2 )

Top of beam:
Compressive stress 7.1145 N/mm² is less than allowable stresses
of 0.7*fckl=19.6 N/mm² and 0.6*fck=19.2 N/mm².
Bottom of beam:
Compressive stress 15.814 N/mm² is less than allowable stresses of 0.7*fcki=19.6 N/mm² and 0.6*fck=19.2 N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer( % ) s<sub>lr</sub>(1)=2
Shrinkage per unit length s<sub>ul</sub>(1)=0.3E-3
Creep coefficient φ(∞,to) crf=2

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>207.07</td>
<td>9.4121</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>229.07</td>
<td>10.412</td>
</tr>
<tr>
<td>Relaxation</td>
<td>44</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>143.5</td>
<td>6.5228</td>
</tr>
<tr>
<td>Creep</td>
<td>453.58</td>
<td>20.617</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>641.08</td>
<td>29.14</td>
</tr>
<tr>
<td>All Losses</td>
<td>870.15</td>
<td>39.552</td>
</tr>
</tbody>
</table>

**Losses - Long term**

Initial force 2200 kN
Final force Pf=P₀-loftl=1329.9 kN

Unit-weight of in-situ concrete wc=25 kN/m³

**Loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
<td>Bottom of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>2.7434</td>
<td>12.228</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.0485</td>
<td>-2.3089</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>5.6757</td>
<td>-4.2986</td>
</tr>
<tr>
<td>Total</td>
<td>11.468</td>
<td>5.6208</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

**Composite section**

Height from beam soffit to bottom of in-situ concrete h<sub>sis</sub>=100 mm
Loading Combination 1

(no tensile stress permitted in precast beam)
Characteristic bending moment \( c_{1b}(1) = 200 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 1</td>
<td>3.3973</td>
</tr>
<tr>
<td>Total</td>
<td>3.3973</td>
</tr>
</tbody>
</table>


Loading Combination 2

(tensile stress permitted (no visible cracking))
Characteristic bending moment \( c_{2b}(1) = 250 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 2</td>
<td>3.7748</td>
</tr>
<tr>
<td>Total</td>
<td>3.7748</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 3.7748 N/mm\(^2\) is less than the maximum allowable stress of 0.6*fckc=19.2 N/mm\(^2\).

Top of beam:
Compressive stress 14.576 N/mm\(^2\) is less than the maximum allowable stress of 0.6*fck=19.2 N/mm\(^2\).

Bottom of in-situ concrete:
Tensile stress 2.8866 N/mm\(^2\) is less than the maximum allowable tensile stress of 3.0231 N/mm\(^2\) (EN 1992-1-1 expression 3.23)

Bottom of beam:
Compressive stress 1.846 N/mm\(^2\) is less than the maximum allowable stress of 0.6*fck=19.2 N/mm\(^2\).
Location: Ex2 - 13.5 metre span edge beam

Precast prestressed concrete inverted T beams with in-fill concrete


Beam span under consideration  span=13.5 m

Beam T 5 section properties

Depth of beam  d=615 mm
Cross-sectional area  Ac=130560 mm²
Centroid above soffit  Yb=244 mm
Section modulus top  Zt=14.3E6 mm³
Section modulus bottom  Zb=21.78E6 mm³
Unit weight of beam  uwb=25 kN/m³

Concrete compressive strengths

Characteristic cylinder strength  fck=40 N/mm²
Characteristic cylinder strength  fcki=32 N/mm² (transfer)

Concrete moduli (short-term)

Concrete modulus of elasticity  Ecmb=35 kN/mm²
Concrete modulus of elasticity  Ecmi=33 kN/mm² (transfer)

Composite section properties

Char strength (in-situ concrete)  fckc=24 N/mm²
Short-term modulus of elasticity for in-situ concrete  Ecmc=31 kN/mm²

Item No. 1

Item weighting  iw1=0.88571
X co-ordinate  X(1)=0 mm
Y co-ordinate  Y(1)=0 mm
X co-ordinate  X(2)=508 mm
Y co-ordinate  Y(2)=0 mm
X co-ordinate  X(3)=508 mm
Y co-ordinate  Y(3)=250 mm
X co-ordinate  X(4)=650 mm
Y co-ordinate  Y(4)=350 mm
X co-ordinate  X(5)=650 mm
Y co-ordinate  Y(5)=790 mm
X co-ordinate  X(6)=150 mm
Y co-ordinate  Y(6)=790 mm
X co-ordinate  X(7)=150 mm
Y co-ordinate  Y(7)=690 mm
X co-ordinate  X(8)=0 mm
Y co-ordinate  Y(8)=690 mm
X co-ordinate  X(9)=0 mm
Y co-ordinate \( Y(9) = 0 \text{ mm} \)

**Item No. 2**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Second moment of area</td>
<td>( I_{XX2} = 5.3143 \times 10^9 \text{ mm}^4 )</td>
</tr>
<tr>
<td>Area of item</td>
<td>( A_2 = 130560 \text{ mm}^2 )</td>
</tr>
<tr>
<td>Centroid from X-X axis</td>
<td>( Y_{C2} = 244 \text{ mm} )</td>
</tr>
<tr>
<td>Item weighting</td>
<td>( i_{w2} = 0.11429 )</td>
</tr>
<tr>
<td>Depth of composite section</td>
<td>( d_i = 790 \text{ mm} )</td>
</tr>
<tr>
<td>Nominal diameter</td>
<td>( n_{omd} = 12 \text{ mm} )</td>
</tr>
<tr>
<td>Nominal tensile strength</td>
<td>( n_{omt} = 1700 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Nominal steel area</td>
<td>( n_{oma} = 100 \text{ mm}^2 )</td>
</tr>
<tr>
<td>Char breaking load (strand)</td>
<td>( f_{pk} = 150 \text{ kN} )</td>
</tr>
<tr>
<td>Initial force per strand</td>
<td>( i_{pf} = 100 \text{ kN} )</td>
</tr>
<tr>
<td>Modulus of elasticity (strand)</td>
<td>( E_p = 195 \text{ kN/mm}^2 )</td>
</tr>
</tbody>
</table>

**Cross-section 1**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance from mid-span of beam</td>
<td>( c_{sd}(1) = 2 \text{ m} )</td>
</tr>
<tr>
<td>Number of layers of strand</td>
<td>( n_{ls}(1) = 4 )</td>
</tr>
<tr>
<td>Height above soffit</td>
<td>( d_s(1) = 49 \text{ mm} )</td>
</tr>
<tr>
<td>Number of strands</td>
<td>( n_s(1) = 10 )</td>
</tr>
</tbody>
</table>

**WARNING:**

Strand location is non-standard.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height above soffit</td>
<td>( d_s(2) = 85 \text{ mm} )</td>
</tr>
<tr>
<td>Number of strands</td>
<td>( n_s(2) = 8 )</td>
</tr>
</tbody>
</table>

**WARNING:**

Strand location is non-standard.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height above soffit</td>
<td>( d_s(3) = 520 \text{ mm} )</td>
</tr>
<tr>
<td>Number of strands</td>
<td>( n_s(3) = 2 )</td>
</tr>
</tbody>
</table>

**WARNING:**

Strand location is non-standard.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height above soffit</td>
<td>( d_s(4) = 580 \text{ mm} )</td>
</tr>
<tr>
<td>Number of strands</td>
<td>( n_s(4) = 2 )</td>
</tr>
</tbody>
</table>

**WARNING:**

Number of strands exceeds maximum for this location.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending moment from temp.loads</td>
<td>( t_{lb}(1) = 80 \text{ kNm} )</td>
</tr>
<tr>
<td>Bending moment from other loads</td>
<td>( o_{lb}(1) = 50 \text{ kNm} )</td>
</tr>
</tbody>
</table>

**Short Term Losses (BS EN 1992-1-1)**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation at transfer (%)</td>
<td>( s_{tr} = 1 )</td>
</tr>
</tbody>
</table>
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>232.92</td>
<td>10.587</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>254.92</td>
<td>11.587</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 2200 kN
Force at transfer $P_t = P_0 - \Delta f = 1945.1$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>2.5659</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.7353</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>5.5849</td>
</tr>
<tr>
<td>Other Loads</td>
<td>3.4906</td>
</tr>
<tr>
<td>Total</td>
<td>16.377</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam:
Compressive stress 16.377 N/mm² is less than allowable stresses
of $0.7f_{ck,i} = 22.4$ N/mm² and $0.6f_{ck} = 24$ N/mm².

Bottom of beam:
Compressive stress 13.925 N/mm² is less than allowable stresses
of $0.7f_{ck,i} = 22.4$ N/mm² and $0.6f_{ck} = 24$ N/mm².

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer (%) $slr(1) = 1.5$
Shrinkage per unit length $sul(1) = 0.3E-3$
Creep coefficient $\phi(\omega, to) = crf = 2$
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>232.92</td>
<td>10.587</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>254.92</td>
<td>11.587</td>
</tr>
<tr>
<td>Relaxation</td>
<td>33</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>128.7</td>
<td>5.85</td>
</tr>
<tr>
<td>Creep</td>
<td>452.92</td>
<td>20.587</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>614.62</td>
<td>27.937</td>
</tr>
<tr>
<td>All Losses</td>
<td>869.53</td>
<td>39.524</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2200 kN
Final force Pf=P0-loftl=1330.5 kN

Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>1.7551</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.7353</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>11.8</td>
</tr>
<tr>
<td>Total</td>
<td>18.29</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Height from beam soffit to bottom of in-situ concrete hsis=100 mm
Perc'age of shrinkage remaining ps=40 %
Percentage pcr=40
Percentage stT=50 %
The net moment is Mc=F*e/1E3-Mb=-98.179 kNm

Stress at top of in-situ concrete:
ftisds=(-F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3
=0.63313 N/mm²

Stress at bottom of in-situ concrete:
fbisds=(-F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
=3.566 N/mm²

Stress at top of precast concrete beam:
ftpbds=(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)*1E3  
=-11.705 N/mm²

Stress at bottom of precast concrete beam:  
fbpbds=(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)*1E3=-0.50805 N/mm²

Loading Combination 1

(no tensile stress permitted in precast beam)

Characteristic bending moment  c1b(1)=550 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.63313</td>
</tr>
<tr>
<td>Combination 1</td>
<td>10.115</td>
</tr>
<tr>
<td>Total</td>
<td>10.748</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses

WARNING:
Bottom of in-situ concrete:
Tensile stress 3.5924 N/mm² exceeds maximum allowable tensile stress of 2.4956 N/mm² (EN 1992-1-1 expression 3.23)

WARNING:
Bottom of beam:
Tensile stress 6.4151 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

Loading Combination 2

(tensile stress permitted (no visible cracking))

Characteristic bending moment  c2b(1)=625 kNm
### Loading Combination 2 - Stresses

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>18.29</td>
<td>0</td>
<td>4.8634</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>0.63313</td>
<td>-11.705</td>
<td>3.566</td>
<td>-0.50805</td>
</tr>
<tr>
<td>Total</td>
<td>11.169</td>
<td>13.135</td>
<td>-4.5685</td>
<td>-7.8838</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 11.169 N/mm² is less than the maximum allowable stress of 0.6*fckc=14.4 N/mm².

Top of beam:
Compressive stress 13.135 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

WARNING:
Bottom of in-situ concrete:
Tensile stress 4.5685 N/mm² exceeds maximum allowable tensile stress of 2.4956 N/mm² (EN 1992-1-1 expression 3.23)

WARNING:
Bottom of beam:
Tensile stress 7.8838 N/mm² exceeds maximum allowable tensile stress of 3.0263 N/mm².
Location: Ex3 - 13.5 metre span T beam

Precast prestressed concrete inverted T beams with in-fill concrete


Beam span under consideration  span=13.5 m

Beam T 5 section properties

- Depth of beam  d=615 mm
- Cross-sectional area  Ac=130560 mm²
- Centroid above soffit  Yb=244 mm
- Section modulus top  Zt=14.3E6 mm⁴
- Section modulus bottom  Zb=21.78E6 mm⁴
- Unit weight of beam  uwb=25 kN/m³

Concrete compressive strengths

- Characteristic cylinder strength  fcki=32 N/mm² (transfer)
- Characteristic cylinder strength  fck=40 N/mm²

Concrete moduli (short-term)

- Concrete modulus of elasticity  Ecmi=33 kN/mm² (transfer)
- Concrete modulus of elasticity  Ecmb=35 kN/mm²

Composite section properties

- Char strength (in-situ concrete)  fckc=25 N/mm²
- Short-term modulus of elasticity for in-situ concrete  Ecmc=31 kN/mm²

Composite section is rectangular in shape.

Breadth of rectangular section  b=508 mm
Depth of rectangular section  di=690 mm
Transf'med second moment of area  Ic=Iyy+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi-Yc)^2)=13.07E9 mm⁶
Modulus of elasticity (strand)  Ep=195 kN/mm²
Cross-section 1

Distance from mid-span of beam $c_{sd}(1)=0\ m$
Number of layers of strand $n_{ls}(1)=4$
Height above soffit $d_{s}(1)=50\ mm$
Number of strands $n_{s}(1)=7$
Height above soffit $d_{s}(2)=90\ mm$
Number of strands $n_{s}(2)=6$
Height above soffit $d_{s}(3)=540\ mm$
Number of strands $n_{s}(3)=1$
Height above soffit $d_{s}(4)=580\ mm$
Number of strands $n_{s}(4)=1$

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer (% ) $str=1$

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>17.4</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>130.41</td>
<td>7.4948</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>147.81</td>
<td>8.4948</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force $1740\ kN$
Force at transfer $P_t=P_0-loft=1592.2\ kN$

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.031745</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.191</td>
</tr>
<tr>
<td>Total</td>
<td>5.1593</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam:
Compressive stress $5.1593\ N/mm²$ is less than allowable stresses of $0.7*f_{ck}=22.4\ N/mm²$ and $0.6*f_{ck}=24\ N/mm²$. 
Bottom of beam:
Compressive stress 16.822 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Long Term Losses ( BS EN 1992-1-1, Clause 5.10.6(1) )**

Relaxation after transfer( % )  slr(1)=1.5
Shrinkage per unit length     sul(1)=0.3E-3
Creep coefficient φ(∞,to)    crf=2

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>17.4</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>130.41</td>
<td>7.4948</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>147.81</td>
<td>8.4948</td>
</tr>
<tr>
<td>Relaxation</td>
<td>26.1</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>81.608</td>
<td>4.6901</td>
</tr>
<tr>
<td>Creep</td>
<td>248.68</td>
<td>14.292</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>356.39</td>
<td>20.482</td>
</tr>
<tr>
<td>All Losses</td>
<td>504.2</td>
<td>28.977</td>
</tr>
</tbody>
</table>

**Losses - Long term**

Initial force          1740 kN
Final force            Pf=P₀-loftl=1235.8 kN
Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.024639</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.191</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>8.7456</td>
</tr>
<tr>
<td>Total</td>
<td>13.912</td>
</tr>
</tbody>
</table>

**Composite section**

Height from beam soffit to bottom of in-situ concrete hsis=100 mm
Perc'age of shrinkage remaining \( ps = 25 \% \)
Percentage \( pcr = 25 \% \)
Percentage \( stT = 50 \% \)

The net moment is
\[
Mc = F*e/1E3 - Mb = -5.541 \text{ kNm}
\]

Stress at top of in-situ concrete:
\[
ftisds = -(F/(Ai*mr) - F/(Ai*mr+Ac) - Mc*1E3*(di-Yc)/Ic)*mr*1E3
= 0.63216 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
fbisds = -(F/(Ai*mr) - F/(Ai*mr+Ac) - Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
= 0.8537 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:
\[
ftpbds = (F/(Ai*mr+Ac) + Mc*1E3*(d-Yc)/Ic + Mb*1E3*Yt/Iyy)*1E3
= -2.9026 \text{ N/mm}^2
\]

Stress at bottom of precast concrete beam:
\[
fbpbds = (F/(Ai*mr+Ac) - Mc*1E3*Yc/Ic - Mb*1E3*Yb/Iyy)*1E3
= -0.15537 \text{ N/mm}^2
\]

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment \( c1b(1) = 204 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>0.63216</td>
</tr>
<tr>
<td>Total</td>
<td>6.3185</td>
</tr>
</tbody>
</table>

**Loading Combination 1 - Stresses**

**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment \( c2b(1) = 260 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>0.63216</td>
</tr>
<tr>
<td>Total</td>
<td>6.7924</td>
</tr>
</tbody>
</table>

**Loading Combination 2 - Stresses**
NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 6.7924 N/mm² is less than the maximum allowable stress of 0.6*fckc=15 N/mm².

Top of beam:
Compressive stress 16.472 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

WARNING:
Bottom of in-situ concrete:
Tensile stress 3.3814 N/mm² exceeds maximum allowable tensile stress of 2.5901 N/mm² (EN 1992-1-1 expression 3.23)

Bottom of beam:
Tensile stress 0.38529 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
**Location:** Ex1 - 20 metre span M beam

**Precast prestressed concrete M beams with in-situ top slab**

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

<table>
<thead>
<tr>
<th>Span of beam</th>
<th>span=20 m</th>
</tr>
</thead>
</table>

**Beam M 6 section properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=1040 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=387050 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=409 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>Zt=75.39E6 mm³</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>Zb=116.23E6 mm³</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=24 kN/m³</td>
</tr>
</tbody>
</table>

**Concrete strengths**

<table>
<thead>
<tr>
<th>Strength</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength at transfer</td>
<td>fci=40 N/mm²</td>
</tr>
<tr>
<td>Characteristic strength</td>
<td>fcu=50 N/mm²</td>
</tr>
</tbody>
</table>

**Concrete moduli**

<table>
<thead>
<tr>
<th>Modulus of elasticity</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus at transfer for concrete strength 40 N/mm² is:</td>
<td>Eci=31 kN/mm²</td>
</tr>
<tr>
<td>Modulus for concrete strength 50 N/mm² is:</td>
<td>Ecb=34 kN/mm²</td>
</tr>
</tbody>
</table>

**Composite section properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char strength (in-situ concrete)</td>
<td>fcuc=40 N/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity for concrete strength 40 N/mm² is:</td>
<td>Ecc=31 kN/mm²</td>
</tr>
</tbody>
</table>
Second moment of area:
\[ I_c = I_{xx} + A_c(Y_b - Y_c)^2 + m_r(I_i + A_i(Y_i - Y_c)^2) = 114.4E9 \text{ mm}^4 \]

Depth of composite section \( d_i = 1165 \text{ mm} \)
Initial force per strand \( i_f = 174 \text{ kN} \)
Modulus of elasticity \( E_s = 200 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( c_{sd(1)} = 0 \text{ m} \)
Number of layers of strand \( n_{ls(1)} = 4 \)
Height above soffit \( d_{s(1)} = 60 \text{ mm} \)
Number of strands \( n_{s(1)} = 11 \)
Height above soffit \( d_{s(2)} = 110 \text{ mm} \)
Number of strands \( n_{s(2)} = 14 \)
Height above soffit \( d_{s(3)} = 880 \text{ mm} \)
Number of strands \( n_{s(3)} = 2 \)
Height above soffit \( d_{s(4)} = 930 \text{ mm} \)
Number of strands \( n_{s(4)} = 2 \)

**Short Term Losses**

Relaxation at transfer ( % ) \( s_{tr} = 1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force (kN)</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>454.25</td>
<td>9.0021</td>
</tr>
<tr>
<td><strong>Transfer Total</strong></td>
<td><strong>504.71</strong></td>
<td><strong>10.002</strong></td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \( 5046 \text{ kN} \)
Force at transfer \( P_t = P_0 - loft = 4541.3 \text{ kN} \)
### Stress Limitations

**Top of beam:**
Compressive stress 5.3414 N/mm² is less than allowable stresses of $0.5 \times f_{ci} = 20$ N/mm² and $0.4 \times f_{cu} = 20$ N/mm².

**Bottom of beam:**
Compressive stress 15.876 N/mm² is less than allowable stresses of $0.5 \times f_{ci} = 20$ N/mm² and $0.4 \times f_{cu} = 20$ N/mm².

### Long Term Losses

Relaxation after transfer (% ) $slr(1) = 2$
Shrinkage per unit length $sul(1) = 0.3E^{-3}$

### Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>454.25</td>
<td>9.0021</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>504.71</td>
<td>10.002</td>
</tr>
<tr>
<td>Relaxation</td>
<td>100.92</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>241.86</td>
<td>4.7931</td>
</tr>
<tr>
<td>Creep</td>
<td>586.78</td>
<td>11.629</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>929.56</td>
<td>18.422</td>
</tr>
<tr>
<td>All Losses</td>
<td>1434.3</td>
<td>28.424</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 5046 kN
Final force $P_f = P_0 - lofl = 3611.7$ kN
Unit-weight of in-situ concrete $w_c = 24$ kN/m³
**Loading** | **Final Stresses N/mm²**
---|---
| | Top of Beam | Bottom of Beam | Centroid of Strands |
| Prestress | -0.65509 | 15.804 | 12.628 |
| Self weight of Beam | 6.165 | -3.996 | -2.0352 |
| In-situ concrete | 3.6555 | -2.3694 | -1.2068 |
| **Total** | 9.1655 | 9.439 | 9.3862 |

**Beam concrete stresses - Long term**

**Composite section**

Height from beam soffit to bottom of in-situ concrete: $h_{sis}=1005$ mm

Per centage of shrinkage remaining: $p_s=25\%$

Percentage: $p_c r=25$

Percentage: $s T=67\%$

The net moment is: $M_c=F*e/1E3-M_b=-81.196$ kNm

Stress at top of in-situ concrete:

$$f_{tisds}=-\frac{F}{(A_1*mr)}-\frac{F}{(A_1*mr+Ac)}-Mc*1E3*(di-Yc)/Ic)*mr*1E3$$

$$=-0.026723 \text{ N/mm}^2$$

Stress at bottom of in-situ concrete:

$$f_{bisds}=-\frac{F}{(A_1*mr)}-\frac{F}{(A_1*mr+Ac)}-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3$$

$$=0.086836 \text{ N/mm}^2$$

Stress at top of precast concrete beam:

$$f_{tpbsd}=(F/(A_1*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3$$

$$=-0.070168 \text{ N/mm}^2$$

Stress at bottom of precast concrete beam:

$$f_{bpbsd}=(F/(A_1*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3=0.018091 \text{ N/mm}^2$$

**Combination 1 loading**

Applied bending moment: $c_{lb(1)}=1446$ kNm

**Combination 1 Loading - Stresses**
**Combination 2 to 5 loading**

Applied bending moment \( c2b(1)=1907 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>9.1655</td>
<td>0</td>
<td>9.439</td>
</tr>
<tr>
<td>Diff. shrinkage Combination 2-5</td>
<td>-0.026723</td>
<td>-0.070168</td>
<td>0.086836</td>
<td>0.018091</td>
</tr>
<tr>
<td></td>
<td>8.3898</td>
<td>6.3061</td>
<td>5.7227</td>
<td>-11.03</td>
</tr>
<tr>
<td>Total</td>
<td>8.363</td>
<td>15.401</td>
<td>5.8095</td>
<td>-1.5729</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 8.363 N/mm\(^2\) is less than the maximum allowable stress of 0.5\(\cdot\)fcuc=20 N/mm\(^2\).

Top of beam:
Compressive stress 15.401 N/mm\(^2\) is less than the maximum allowable stress of 0.4\(\cdot\)fcu=20 N/mm\(^2\).

Bottom of in-situ concrete:
Compressive stress 5.8095 N/mm\(^2\) is less than the maximum allowable stress of 0.5\(\cdot\)fcuc=20 N/mm\(^2\).

Bottom of beam:
Tensile stress 1.5729 N/mm\(^2\) is less than the maximum allowable tensile stress of 3.182 N/mm\(^2\).
Location: Ex2 - 25 metre span M beam

Precast prestressed concrete M beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam  

Beam M 9 section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=1280 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=425450 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=512 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>Zt=108.09E6 mm³</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>Zb=161.96E6 mm³</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=24.25 kN/m³</td>
</tr>
</tbody>
</table>

Concrete strengths

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength at transfer</td>
<td>fci=40 N/mm²</td>
</tr>
<tr>
<td>Characteristic strength</td>
<td>fcu=45 N/mm²</td>
</tr>
</tbody>
</table>

Concrete moduli

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity at transfer for concrete strength 40 N/mm² is:</td>
<td>Eci=31 kN/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity for concrete strength 45 N/mm² is:</td>
<td>Ecb=32.5 kN/mm²</td>
</tr>
</tbody>
</table>

Composite section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char strength (in-situ concrete)</td>
<td>fcuc=30 N/mm²</td>
</tr>
</tbody>
</table>

Modulus of elasticity for concrete strength 30 N/mm² is:  
Ecc=28 kN/mm²

Item No. 1

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Item weighting</td>
<td>iw1=0.86154</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(1)=15 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(1)=0 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(2)=985 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(2)=0 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(3)=985 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(3)=950 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(4)=1150 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(4)=1060 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(5)=1150 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(5)=1505 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(6)=650 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(6)=1505 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(7)=650 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(7)=1405 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(8)=0 mm</td>
</tr>
</tbody>
</table>
Y co-ordinate                     Y(8)=1405 mm
X co-ordinate                     X(9)=0 mm
Y co-ordinate                     Y(9)=1245 mm
X co-ordinate                     X(10)=350 mm
Y co-ordinate                     Y(10)=1245 mm
X co-ordinate                     X(11)=350 mm
Y co-ordinate                     Y(11)=1230 mm
X co-ordinate                     X(12)=300 mm
Y co-ordinate                     Y(12)=1230 mm
X co-ordinate                     X(13)=300 mm
Y co-ordinate                     Y(13)=1050 mm
X co-ordinate                     X(14)=420 mm
Y co-ordinate                     Y(14)=970 mm
X co-ordinate                     X(15)=420 mm
Y co-ordinate                     Y(15)=290 mm
X co-ordinate                     X(16)=340 mm
Y co-ordinate                     Y(16)=210 mm
X co-ordinate                     X(17)=15 mm
Y co-ordinate                     Y(17)=160 mm
X co-ordinate                     X(18)=15 mm
Y co-ordinate                     Y(18)=0 mm

Item No. 2

Second moment of area             IXX2=82.924E9 mm^4
Area of item                      A2=425450 mm^2
Centroid from X-X axis            YC2=512 mm
Item weighting                    iw2=0.13846
Depth of composite section        di=1505 mm
Initial force per strand          if=125 kN
Modulus of elasticity             Es=200 kN/mm^2

Cross-section 1

Distance from mid-span of beam     csd(1)=3.5 m
Number of layers of strand        nls(1)=5
Height above soffit               ds(1)=60 mm
Number of strands                 ns(1)=13
Height above soffit               ds(2)=110 mm
Number of strands                 ns(2)=12
Height above soffit               ds(3)=630 mm
Number of strands                 ns(3)=2
Height above soffit               ds(4)=1080 mm
Number of strands                 ns(4)=2
Height above soffit               ds(5)=1230 mm
Number of strands                 ns(5)=2

Short Term Losses

Relaxation at transfer ( % )     str=1.1
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>42.625 kN</td>
<td>1.1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>240.06 kN</td>
<td>6.1952</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>282.69 kN</td>
<td>7.2952</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 3875 kN
Force at transfer $P_t = P_0 - \text{loft} = 3592.3$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.026432</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.8798</td>
</tr>
<tr>
<td>Total</td>
<td>6.8534</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 6.8534 N/mm$^2$ is less than allowable stresses of $0.5\times f_{ci}=20$ N/mm$^2$ and $0.4\times f_{cu}=18$ N/mm$^2$.

Bottom of beam:
Compressive stress 9.5037 N/mm$^2$ is less than allowable stresses of $0.5\times f_{ci}=20$ N/mm$^2$ and $0.4\times f_{cu}=18$ N/mm$^2$.

**Long Term Losses**

Relaxation after transfer (%) $slr(1)=1.2$
Shrinkage per unit length $sul(1)=0.3E-3$
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>42.625 kN</td>
<td>1.1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>240.06 kN</td>
<td>6.1952</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>282.69 kN</td>
<td>7.2952</td>
</tr>
<tr>
<td>Relaxation</td>
<td>46.5 kN</td>
<td>1.2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>186 kN</td>
<td>4.8</td>
</tr>
<tr>
<td>Creep</td>
<td>266.97 kN</td>
<td>6.8895</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>499.47 kN</td>
<td>12.889</td>
</tr>
<tr>
<td>All Losses</td>
<td>782.15 kN</td>
<td>20.185</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 3875 kN
Final force Pf=P0-loftl=3092.8 kN

Unit-weight of in-situ concrete wc=24.6 kN/m²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.022757</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.8798</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>10.813</td>
</tr>
<tr>
<td>Total</td>
<td>17.67</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=160 mm
Percentage of shrinkage remaining ps=40 %
Percentage pcr=40
Percentage stT=50 %
The net moment is Mc=F*e/1E3-Mb=300.14 kNm

Stress at top of in-situ concrete:
\[ ftisds=\left(-\frac{F}{(Ai*mr)}\right)-\left(\frac{F}{(Ai*mr+Ac)}\right)\cdot Mc \cdot 1E3 \cdot 1E3 \cdot (di-Yc)/Ic \cdot mr \cdot 1E3 =1.8739 \text{ N/mm}^2 \]

Stress at bottom of in-situ concrete:
\[ fbisds=\left(-\frac{F}{(Ai*mr)}\right)-\left(\frac{F}{(Ai*mr+Ac)}\right)\cdot Mc \cdot 1E3 \cdot (hsis-Yc)/Ic \cdot mr \cdot 1E3 =0.25425 \text{ N/mm}^2 \]

Stress at top of precast concrete beam:
ftpbds = \(\frac{F}{(A_i \times mr + Ac)} + \frac{Mc \times 1E3 \times (d - Yc)}{Ic} + \frac{Mb \times 1E3 \times Yt}{Ixx}\) * 1E3

\(= -6.1033\) N/mm²

Stress at bottom of precast concrete beam:
\(f_fbpbds = \left(\frac{F}{(A_i \times mr + Ac)} - \frac{Mc \times 1E3 \times Yc}{Ic} - \frac{Mb \times 1E3 \times Yb}{Ixx}\right) \times 1E3 = 0.985\) N/mm²

### Combination 1 loading

Applied bending moment \(c_{lb(1)} = 400\) kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>1.8739</td>
</tr>
<tr>
<td>Combination 1</td>
<td>1.3504</td>
</tr>
<tr>
<td>Total</td>
<td>3.2242</td>
</tr>
</tbody>
</table>

**Combination 1 Loading - Stresses**

**WARNING:**
Bottom of in-situ concrete is in tension. Crack widths should be calculated using Clause 5.8.8.2.

**WARNING:**
Bottom of beam:
Tensile stress 0.089056 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

### Combination 2 to 5 loading

Applied bending moment \(c_{2b(1)} = 525\) kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>1.8739</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>1.5754</td>
</tr>
<tr>
<td>Total</td>
<td>3.4493</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 Loading - Stresses**

Combination 3 loading. The stresses from the proforma for temperature differences should be added to the total stresses.
Top of in-situ concrete:
Compressive stress 3.4493 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².
Top of beam:
Compressive stress 12.845 N/mm² is less than the maximum allowable stress of 0.4*fcu=18 N/mm².

**WARNING:**
Bottom of in-situ concrete is in tension. Crack widths should be calculated using Clause 5.8.8.2.

Bottom of beam:
Tensile stress 0.52974 N/mm² is less than the maximum allowable tensile stress of 3.0187 N/mm².
**Location:** Ex3 - 16 metre span M2 beam

**Precast prestressed concrete M beams with in-situ top slab**

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam: span=16 m

**Beam M 2 section properties**

- Depth of beam: $d=720$ mm
- Cross-sectional area: $A_c=316650$ mm$^2$
- Centroid above soffit: $y_b=265$ mm
- Section modulus top: $Z_t=35.64E6$ mm$^3$
- Section modulus bottom: $Z_b=61.04E6$ mm$^3$
- Unit weight of beam: $u_{wb}=24$ kN/m$^3$

**Concrete strengths**

- Strength at transfer: $f_{ci}=35$ N/mm$^2$
- Characteristic strength: $f_{cu}=45$ N/mm$^2$

**Concrete moduli**

- Modulus of elasticity at transfer for concrete strength 35 N/mm$^2$ is: $E_{ci}=29.5$ kN/mm$^2$
- Modulus of elasticity for concrete strength 45 N/mm$^2$ is: $E_{cb}=32.5$ kN/mm$^2$

**Composite section properties**

- Char strength (in-situ concrete): $f_{cuc}=32.5$ N/mm$^2$
- Modulus of elasticity for concrete strength 32.5 N/mm$^2$ is: $E_{cc}=28.75$ kN/mm$^2$
Transf'med second moment of area:
\[ I_c = I_{xx} + A_c \cdot (Y_b - Y_c)^2 + m_r \cdot (I_i + A_i \cdot (Y_i - Y_c)^2) = 52.396 \times 10^9 \text{ mm}^4 \]

Depth of composite section \( d_i = 890 \text{ mm} \)
Nominal diameter \( \text{nomd} = 12 \text{ mm} \)
Nominal tensile strength \( \text{nomt} = 1600 \text{ N/mm}^2 \)
Nominal steel area \( \text{noma} = 90 \text{ mm}^2 \)
Characteristic breaking load \( \text{cbl} = 160 \text{ kN} \)
Initial force per strand \( \text{if} = 100 \text{ kN} \)
Modulus of elasticity \( \text{Es} = 200 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( \text{csd}(1) = 4.1 \text{ m} \)
Number of layers of strand \( \text{nls}(1) = 3 \)
Height above soffit \( \text{ds}(1) = 65 \text{ mm} \)
Number of strands \( \text{ns}(1) = 13 \)

**WARNING:**
Strand location is non-standard.

Height above soffit \( \text{ds}(2) = 100 \text{ mm} \)
Number of strands \( \text{ns}(2) = 10 \)

**WARNING:**
Strand location is non-standard.

Height above soffit \( \text{ds}(3) = 620 \text{ mm} \)
Number of strands \( \text{ns}(3) = 2 \)

**WARNING:**
Strand location is non-standard.

Bending moment from temp.loads \( \text{tlb}(1) = 100 \text{ kNm} \)
Bending moment from other loads \( \text{olb}(1) = 108 \text{ kNm} \)

**Short Term Losses**

Relaxation at transfer ( % ) \( \text{str} = 0.5 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>12.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>166.87</td>
<td>6.6747</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>179.37</td>
<td>7.1747</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 2500 kN
Force at transfer \( P_t=P_0+\text{loft}=2320.6 \) kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-1.9145</td>
<td>12.712</td>
<td>10.205</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.0438</td>
<td>-2.9376</td>
<td>-1.5697</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>2.8129</td>
<td>-1.6383</td>
<td>-0.87539</td>
</tr>
<tr>
<td>Other Loads</td>
<td>3.0379</td>
<td>-1.7693</td>
<td>-0.94542</td>
</tr>
<tr>
<td>Total</td>
<td>8.9802</td>
<td>6.3669</td>
<td>6.8147</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
Compressive stress 8.9802 N/mm\(^2\) is less than allowable stresses of 0.5*\(f_{ci}\)=17.5 N/mm\(^2\) and 0.4*\(f_{cu}\)=18 N/mm\(^2\).

Bottom of beam:
Compressive stress 6.3669 N/mm\(^2\) is less than allowable stresses of 0.5*\(f_{ci}\)=17.5 N/mm\(^2\) and 0.4*\(f_{cu}\)=18 N/mm\(^2\).

Long Term Losses

Relaxation after transfer( % ) \( \text{slr}(1)=0.7 \)
Shrinkage per unit length \( \text{sul}(1)=0.3E-3 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>12.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>166.87</td>
<td>6.6747</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>179.37</td>
<td>7.1747</td>
</tr>
<tr>
<td>Relaxation</td>
<td>17.5</td>
<td>0.7</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>135</td>
<td>5.4</td>
</tr>
<tr>
<td>Creep</td>
<td>189.84</td>
<td>7.5935</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>342.34</td>
<td>13.693</td>
</tr>
<tr>
<td>All Losses</td>
<td>521.7</td>
<td>20.868</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force                     2500 kN
Final force                       Pf=P0-loftl=1978.3 kN
Unit-weight of in-situ concrete   wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.632</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.0438</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.8388</td>
</tr>
<tr>
<td>Total</td>
<td>7.2506</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Height from beam soffit to bottom of in-situ concrete     hsis=690 mm
Perc'age of shrinkage remaining                           ps=50 %
Percentage                                               pcr=50
Percentage                                               stT=45 %
The net moment is                                         Mc=F*e/1E3-Mb=-211.81 kNm
Stress at top of in-situ concrete:
    ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3
           =0.24377 N/mm²
Stress at bottom of in-situ concrete:
    fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
           =0.95897 N/mm²
Stress at top of precast concrete beam:
ftpbds=(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3
=-2.4847 N/mm²

Stress at bottom of precast concrete beam:

\[
fbpbds=(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3=0.73651 N/mm²
\]

**Combination 1 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>7.2506</td>
<td>0</td>
<td>5.6634</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.24377</td>
<td>-2.4847</td>
<td>0.95897</td>
<td>0.73651</td>
</tr>
<tr>
<td>Combination 1</td>
<td>4.8748</td>
<td>3.2395</td>
<td>2.5111</td>
<td>-5.4682</td>
</tr>
<tr>
<td>Total</td>
<td>5.1186</td>
<td>8.0054</td>
<td>3.4701</td>
<td>0.93167</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>7.2506</td>
<td>0</td>
<td>5.6634</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.24377</td>
<td>-2.4847</td>
<td>0.95897</td>
<td>0.73651</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>5.223</td>
<td>3.4709</td>
<td>2.6905</td>
<td>-6.8353</td>
</tr>
<tr>
<td>Total</td>
<td>5.4668</td>
<td>8.2368</td>
<td>3.6495</td>
<td>-0.43539</td>
</tr>
</tbody>
</table>

**Combination 3 loading**. The stresses from the proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 5.4668 N/mm² is less than the maximum allowable stress of 0.5*fcuc=16.25 N/mm².

Top of beam:
Compressive stress 8.2368 N/mm² is less than the maximum allowable stress of 0.4*fcu=18 N/mm².

Bottom of in-situ concrete:
Compressive stress 3.6495 N/mm² is less than the maximum allowable stress of 0.5*fcuc=16.25 N/mm².

Bottom of beam:
Tensile stress 0.43539 N/mm² is less than the maximum allowable
tensile stress of 3.0187 N/mm².
Location: Ex1 - 20 metre span M beam

Precast prestressed concrete M beams with in-situ top slab


Beam span under consideration \( \text{span}=20 \text{ m} \)

**Beam M 6 section properties**

- Depth of beam \( d=1040 \text{ mm} \)
- Cross-sectional area \( A_c=387050 \text{ mm}^2 \)
- Centroid above soffit \( Y_b=409 \text{ mm} \)
- Section modulus top \( Z_t=75.39E6 \text{ mm}^3 \)
- Section modulus bottom \( Z_b=116.23E6 \text{ mm}^3 \)
- Unit weight of beam \( u_{wb}=24 \text{ kN/m}^3 \)

**Concrete compressive strengths**

- Characteristic cylinder strength \( f_{cki}=32 \text{ N/mm}^2 \) (transfer)
- Characteristic cylinder strength \( f_{ck}=40 \text{ N/mm}^2 \)

**Concrete moduli (short-term)**

- Concrete modulus of elasticity \( E_{cmi}=33 \text{ kN/mm}^2 \) (transfer)
- Concrete modulus of elasticity \( E_{cmb}=35 \text{ kN/mm}^2 \)

**Composite section properties**

- Char strength (in-situ concrete) \( f_{ckc}=40 \text{ N/mm}^2 \)
- Short-term modulus of elasticity for in-situ concrete \( E_{cmc}=35 \text{ kN/mm}^2 \)
In-situ concrete

Area 1

Area 2=300*(50-dpf) mm^2

Second moment of area

\[ I_c = I_{yy} + A_c (Y_b - Y_c)^2 + m_r (I_i + A_i (Y_i - Y_c)^2) = 114.4E9 \text{ mm}^4 \]

Depth of composite section

\[ d_i = 1165 \text{ mm} \]

Modulus of elasticity (strand)

\[ E_p = 195 \text{ kN/mm}^2 \]

**Cross-section 1**

Distance from mid-span of beam

\[ c_{sd}(1) = 0 \text{ m} \]

Number of layers of strand

\[ n_{ls}(1) = 4 \]

Height above soffit

\[ d_s(1) = 60 \text{ mm} \]

Number of strands

\[ n_s(1) = 11 \]

Height above soffit

\[ d_s(2) = 110 \text{ mm} \]

Number of strands

\[ n_s(2) = 14 \]

Height above soffit

\[ d_s(3) = 880 \text{ mm} \]

Number of strands

\[ n_s(3) = 2 \]

Height above soffit

\[ d_s(4) = 930 \text{ mm} \]

Number of strands

\[ n_s(4) = 2 \]

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer ( % )

\[ \text{str} = 1 \]

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>384.06</td>
<td>7.6112</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>434.52</td>
<td>8.6112</td>
</tr>
</tbody>
</table>

Initial force

\[ 5046 \text{ kN} \]

Prestress force at transfer

\[ P_t = P_0 -\text{loft} = 4611.5 \text{ kN} \]
<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.83642</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.165</td>
</tr>
<tr>
<td>Total</td>
<td>5.3286</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
Compressive stress 5.3286 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

Bottom of beam:
Compressive stress 16.183 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer(%) slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(=,to) crf=2

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>384.06</td>
<td>7.6112</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>434.52</td>
<td>8.6112</td>
</tr>
<tr>
<td>Relaxation</td>
<td>100.92</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>235.81</td>
<td>4.6733</td>
</tr>
<tr>
<td>Creep</td>
<td>687.98</td>
<td>13.634</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>1024.7</td>
<td>20.308</td>
</tr>
<tr>
<td>All Losses</td>
<td>1459.2</td>
<td>28.919</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 5046 kN
Final prestress force Pf=P₀-loftl=3586.8 kN
Unit-weight of in-situ concrete wc=24 kN/m³
Beam concrete stresses - Long term

Composite section

Height from beam soffit to
bottom of in-situ concrete  hsis=1005 mm
Perc'age of shrinkage remaining  ps=25 %
Percentage  pcr=25
Percentage  stT=67 %
The net moment is  Mc=F*e/1E3-Mb=-166.22 kNm
Stress at top of in-situ concrete:
\[ ftisds = \frac{F}{(A_i*mr)}-\frac{F}{(A_i*mr+Ac)}-\frac{Mc*1E3*(di-y_c)}{I_c} \times mr*1E3 \]
\[ = 0.09964 \text{ N/mm}^2 \]
Stress at bottom of in-situ concrete:
\[ fbisds = \frac{F}{(A_i*mr)}-\frac{F}{(A_i*mr+Ac)}-\frac{Mc*1E3*(hsis-y_c)}{I_c} \times mr*1E3 \]
\[ = 0.33212 \text{ N/mm}^2 \]
Stress at top of precast concrete beam:
\[ ftpbds = \frac{F}{(A_i*mr+Ac)} + \frac{Mc*1E3*(d-y_c)}{I_c} + Mb*1E3*Y_t/I_{yy} \times 1E3 \]
\[ = -0.55436 \text{ N/mm}^2 \]
Stress at bottom of precast concrete beam:
\[ fbpbds = \frac{F}{(A_i*mr+Ac)} - \frac{Mc*1E3*Y_c}{I_c} - Mb*1E3*Y_b/I_{yy} \times 1E3 \]
\[ = 0.15241 \text{ N/mm}^2 \]

Loading Combination 1

(no tensile stress permitted in precast beam)
Characteristic bending moment  \( c_1b(1)=1200 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 - Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Dif. shrinkage Combination 1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.09964</td>
</tr>
<tr>
<td></td>
<td>9.846</td>
</tr>
<tr>
<td>Total</td>
<td>9.9456</td>
</tr>
</tbody>
</table>
Loading Combination 2

(tensile stress permitted (no visible cracking))
Characteristic bending moment \( c_2b(1)=2075 \) kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage Combination 2</td>
<td>0.09964</td>
</tr>
<tr>
<td>Total</td>
<td>9.2285</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 9.2285 N/mm² is less than the maximum allowable stress of 0.6*fckc=24 N/mm².

Top of beam:
Compressive stress 15.477 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

Bottom of in-situ concrete:
Compressive stress 6.5589 N/mm² is less than the maximum allowable stress of 0.6*fckc=24 N/mm².

Bottom of beam:
Tensile stress 2.5195 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
Location: Ex2 - 25 metre span M beam

Precast prestressed concrete M beams with in-situ top slab


Beam span under consideration \( \text{span} = 25 \text{ m} \)

Beam M 9 section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>( d = 1280 \text{ mm} )</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>( A_c = 425450 \text{ mm}^2 )</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>( Y_b = 512 \text{ mm} )</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>( Z_t = 108.09 \times 10^6 \text{ mm}^3 )</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>( Z_b = 161.96 \times 10^6 \text{ mm}^3 )</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>( u_{wb} = 24.25 \text{ kN/m}^3 )</td>
</tr>
</tbody>
</table>

Concrete compressive strengths

<table>
<thead>
<tr>
<th>Strength</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic cylinder strength</td>
<td>( f_{ck} = 32 \text{ N/mm}^2 ) (transfer)</td>
</tr>
<tr>
<td>Characteristic cylinder strength</td>
<td>( f_{c} = 36 \text{ N/mm}^2 )</td>
</tr>
</tbody>
</table>

Concrete moduli (short-term)

<table>
<thead>
<tr>
<th>Modulus</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete modulus of elasticity</td>
<td>( E_{c,mi} = 31 \text{ kN/mm}^2 ) (transfer)</td>
</tr>
<tr>
<td>Concrete modulus of elasticity</td>
<td>( E_{c,mb} = 32.5 \text{ kN/mm}^2 )</td>
</tr>
</tbody>
</table>

Composite section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char strength (in-situ concrete)</td>
<td>( f_{ck} = 24 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Short-term modulus of elasticity for in-situ concrete</td>
<td>( E_{cm} = 28 \text{ kN/mm}^2 )</td>
</tr>
</tbody>
</table>

Item No. 1

<table>
<thead>
<tr>
<th>Item</th>
<th>Weighting</th>
<th>X co-ordinate</th>
<th>Y co-ordinate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>iw1=0.86154</td>
<td>X(1)=15 mm</td>
<td>Y(1)=0 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X(2)=985 mm</td>
<td>Y(2)=0 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X(3)=985 mm</td>
<td>Y(3)=950 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X(4)=1150 mm</td>
<td>Y(4)=1060 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X(5)=1150 mm</td>
<td>Y(5)=1505 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X(6)=650 mm</td>
<td>Y(6)=1505 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X(7)=650 mm</td>
<td>Y(7)=1405 mm</td>
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<tr>
<td></td>
<td></td>
<td>X(8)=0 mm</td>
<td>Y(8)=1405 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X(9)=0 mm</td>
<td>Y(9)=1405 mm</td>
</tr>
</tbody>
</table>
Y co-ordinate $Y(9)=1245$ mm
X co-ordinate $X(10)=350$ mm
Y co-ordinate $Y(10)=1245$ mm
X co-ordinate $X(11)=350$ mm
Y co-ordinate $Y(11)=1230$ mm
X co-ordinate $X(12)=300$ mm
Y co-ordinate $Y(12)=1230$ mm
X co-ordinate $X(13)=300$ mm
Y co-ordinate $Y(13)=1050$ mm
X co-ordinate $X(14)=420$ mm
Y co-ordinate $Y(14)=970$ mm
X co-ordinate $X(15)=420$ mm
Y co-ordinate $Y(15)=290$ mm
X co-ordinate $X(16)=340$ mm
Y co-ordinate $Y(16)=210$ mm
X co-ordinate $X(17)=15$ mm
Y co-ordinate $Y(17)=160$ mm
X co-ordinate $X(18)=15$ mm
Y co-ordinate $Y(18)=0$ mm

**Item No. 2**

Second moment of area $\text{IXX}_2=82.924E9$ mm$^4$
Area of item $A_2=425450$ mm$^2$
Centroid from X-X axis $\text{YC}_2=512$ mm
Item weighting $\text{iw}_2=0.13846$
Depth of composite section $d_i=1505$ mm
Modulus of elasticity (strand) $E_p=200$ kN/mm$^2$

**Cross-section 1**

Distance from mid-span of beam $c_{sd}(1)=3.5$ m
Number of layers of strand $n_{ls}(1)=5$
Height above soffit $d_{s}(1)=60$ mm
Number of strands $n_s(1)=13$
Height above soffit $d_{s}(2)=110$ mm
Number of strands $n_s(2)=12$
Height above soffit $d_{s}(3)=630$ mm
Number of strands $n_s(3)=2$
Height above soffit $d_{s}(4)=1080$ mm
Number of strands $n_s(4)=2$
Height above soffit $d_{s}(5)=1230$ mm
Number of strands $n_s(5)=2$

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer (%) $\text{str}=1.1$
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>42.625 kN</td>
<td>1.1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>225.91 kN</td>
<td>5.83</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>268.54 kN</td>
<td>6.93</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 3875 kN
Prestress force at transfer $P_t = P_0 - \Delta P = 3606.5$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of Beam</td>
<td>Bottom of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.026536</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.8798</td>
</tr>
<tr>
<td>Total</td>
<td>6.8533</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
Compressive stress 6.8533 N/mm$^2$ is less than allowable stresses of $0.7 f_{ck_i} = 22.4$ N/mm$^2$ and $0.6 f_{ck} = 21.6$ N/mm$^2$.

Bottom of beam:
Compressive stress 9.5592 N/mm$^2$ is less than allowable stresses of $0.7 f_{ck_i} = 22.4$ N/mm$^2$ and $0.6 f_{ck} = 21.6$ N/mm$^2$.

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer (%): $slr(1) = 1.2$
Shrinkage per unit length: $sul(1) = 0.3E-3$
Creep coefficient $\phi(\infty, to)$: $crf = 2$
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>42.625 kN</td>
<td>1.1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>225.91 kN</td>
<td>5.83</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>268.54 kN</td>
<td>6.93</td>
</tr>
<tr>
<td>Relaxation</td>
<td>46.5</td>
<td>1.2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>186</td>
<td>4.8</td>
</tr>
<tr>
<td>Creep</td>
<td>343.96</td>
<td>8.8763</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>576.46</td>
<td>14.876</td>
</tr>
<tr>
<td>All Losses</td>
<td>844.99</td>
<td>21.806</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force         3875 kN
Final prestress force Pf=P0-loftl=3030 kN

Unit-weight of in-situ concrete wc=24.6 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
<td>Bottom of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.022295</td>
<td>11.885</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.8798</td>
<td>-4.5865</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>10.813</td>
<td>-7.2085</td>
</tr>
<tr>
<td>Total</td>
<td>17.67</td>
<td>0.08965</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete   hsis=160 mm
Perc'age of shrinkage remaining                        ps=40 %
Percentage                                             pcr=40
Percentage                                             stT=50 %
The net moment is                                     Mc=F*e/1E3-Mb=318.6 kNm

Stress at top of in-situ concrete:
\[ f_{tisds} = \frac{-F}{(Ai*mr)-(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)mr*1E3}{2.4533 \text{ N/mm}^2} \]

Stress at bottom of in-situ concrete:
\[ f_{bisds} = \frac{-F}{(Ai*mr)-(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)mr*1E3}{0.7341 \text{ N/mm}^2} \]

Stress at top of precast concrete beam:
Stress at bottom of precast concrete beam:
\[ f_{bpbd s} = \frac{F}{(A_i * m + A_c)} - \frac{M_c * 10^3 (d - Y_c)}{I_c} - \frac{M_b * 10^3 * Y_b}{I_{yy}} * 10^3 = 1.1362 \text{ N/mm}^2 \]

**Loading Combination 1**

(no tensile stress permitted in precast beam)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage Combination 1</td>
<td>2.4533</td>
</tr>
<tr>
<td>Total</td>
<td>3.8037</td>
</tr>
</tbody>
</table>

**Loading Combination 1 - Stresses**

**WARNING:**
Bottom of in-situ concrete is in tension. Crack widths should be calculated to BS EN 1992-2.

**WARNING:**
Bottom of beam:
Tensile stress 0.18435 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

**Loading Combination 2**

(tensile stress permitted (no visible cracking))

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage Combination 2</td>
<td>2.4533</td>
</tr>
<tr>
<td>Total</td>
<td>4.0287</td>
</tr>
</tbody>
</table>

**Loading Combination 2 - Stresses**
NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 4.0287 N/mm$^2$ is less than the maximum allowable stress of $0.6 \times f_{ckc} = 14.4$ N/mm$^2$.

Top of beam:
Compressive stress 10.536 N/mm$^2$ is less than the maximum allowable stress of $0.6 \times f_{ck} = 21.6$ N/mm$^2$.

WARNING:
Bottom of in-situ concrete is in tension. Crack widths should be calculated to BS EN 1992-2.

Bottom of beam:
Tensile stress 0.62503 N/mm$^2$ is less than the maximum allowable tensile stress of 2.8102 N/mm$^2$. 
Location: Ex3 - 16 metre span M2 beam

Precast prestressed concrete M beams with in-situ top slab


Beam span under consideration span=16 m

Beam M 2 section properties

- Depth of beam d=720 mm
- Cross-sectional area Ac=316650 mm²
- Centroid above soffit Yb=265 mm
- Section modulus top Zt=35.64E6 mm³
- Section modulus bottom Zb=61.04E6 mm³
- Unit weight of beam uwb=24 kN/m³

Concrete compressive strengths

- Characteristic cylinder strength fcki=28 N/mm² (transfer)
- Characteristic cylinder strength fck=36 N/mm²

Concrete moduli (short-term)

- Concrete modulus of elasticity Ecmi=29.5 kN/mm² (transfer)
- Concrete modulus of elasticity Ecmb=32.5 kN/mm²

Composite section properties

- Char strength (in-situ concrete) fckc=24 N/mm²
- Short-term modulus of elasticity for in-situ concrete Ecmc=28.75 kN/mm²
Transf'med second moment of area \[ I_c = I_{yy} + A_c (Y_b - Y_c)^2 + m_r (I_i + A_i (Y_i - Y_c)^2) = 52.396E9 \, \text{mm}^4 \]

Depth of composite section \[ d_i = 890 \, \text{mm} \]
Nominal diameter \[ n_{omd} = 12 \, \text{mm} \]
Nominal tensile strength \[ n_{omt} = 1600 \, \text{N/mm}^2 \]
Nominal steel area \[ n_{oma} = 90 \, \text{mm}^2 \]
Char breaking load (strand) \[ f_{pk} = 160 \, \text{kN} \]
Initial force per strand \[ i_{pf} = 100 \, \text{kN} \]
Modulus of elasticity (strand) \[ E_p = 200 \, \text{kN/mm}^2 \]

**Cross-section 1**

Distance from mid-span of beam \[ c_{sd(1)} = 4.1 \, \text{m} \]
Number of layers of strand \[ n_{ls(1)} = 3 \]
Height above soffit \[ d_{s(1)} = 65 \, \text{mm} \]
Number of strands \[ n_{s(1)} = 13 \]

**WARNING:**
Strand location is non-standard.

Height above soffit \[ d_{s(2)} = 100 \, \text{mm} \]
Number of strands \[ n_{s(2)} = 10 \]

**WARNING:**
Strand location is non-standard.

Height above soffit \[ d_{s(3)} = 620 \, \text{mm} \]
Number of strands \[ n_{s(3)} = 2 \]

**WARNING:**
Strand location is non-standard.

Bending moment from temp.loads \[ t_{lb(1)} = 100 \, \text{kNm} \]
Bending moment from other loads \[ o_{lb(1)} = 108 \, \text{kNm} \]

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer ( % ) \[ s_{tr} = 0.5 \]
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>12.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>156.38</td>
<td>6.2551</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>168.88</td>
<td>6.7551</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force: 2500 kN
Prestress force at transfer: $P_t = P_0 - loft = 2331.1$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.9231</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.0438</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>2.8129</td>
</tr>
<tr>
<td>Other Loads</td>
<td>3.0379</td>
</tr>
<tr>
<td>Total</td>
<td>8.9715</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
Compressive stress 8.9715 N/mm² is less than allowable stresses of $0.7 \times f_{ck,i} = 19.6$ N/mm² and $0.6 \times f_{ck} = 21.6$ N/mm².

Bottom of beam:
Compressive stress 6.4243 N/mm² is less than allowable stresses of $0.7 \times f_{ck,i} = 19.6$ N/mm² and $0.6 \times f_{ck} = 21.6$ N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer (%): $slr(1) = 0.7$
Shrinkage per unit length: $sul(1) = 0.3 \times 10^{-3}$
Creep coefficient $\phi(\infty, to)$: $crf = 2$
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>12.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>156.38</td>
<td>6.2551</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>168.88</td>
<td>6.7551</td>
</tr>
<tr>
<td>Relaxation</td>
<td>17.5</td>
<td>0.7</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>135</td>
<td>5.4</td>
</tr>
<tr>
<td>Creep</td>
<td>214.24</td>
<td>8.5694</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>366.74</td>
<td>14.669</td>
</tr>
<tr>
<td>All Losses</td>
<td>535.61</td>
<td>21.424</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force: 2500 kN
Final prestress force: Pf=P0-loftl=1964.4 kN

Unit-weight of in-situ concrete: wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.6206</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.0438</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.8388</td>
</tr>
<tr>
<td>Total</td>
<td>7.2621</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete: hsis=690 mm
Perc'age of shrinkage remaining: ps=50 %
Percentage: pcr=50
Percentage: stT=45 %
The net moment is: Mc=F*e/1E3-Mb=-254.66 kNm

Stress at top of in-situ concrete:
\[ ftisds = -\left( \frac{F}{(Ai*mr)} - \frac{F}{(Ai*mr+Ac)} - Mc*1E3*(di-Yc)/Ic \right)*mr*1E3 \]
\[ = 0.28915 \text{ N/mm}^2 \]

Stress at bottom of in-situ concrete:
\[ fbisds = -\left( \frac{F}{(Ai*mr)} - \frac{F}{(Ai*mr+Ac)} - Mc*1E3*(hsis-Yc)/Ic \right)*mr*1E3 \]
\[ = 1.149 \text{ N/mm}^2 \]

Stress at top of precast concrete beam:
Stress at bottom of precast concrete beam:
\[ \text{fbpbds} = \left( \frac{F}{(A_i \cdot mr + Ac)} - \frac{Mc}{Ic} \cdot 10^3 \cdot \frac{Yc}{Ic} - \frac{Mb}{Iyy} \cdot 10^3 \cdot Yb \right) \cdot 10^3 = 0.8803 \text{ N/mm}^2 \]

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment \( c_{1b}(1) = 600 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>0.28915</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>4.8748</td>
</tr>
<tr>
<td>Total</td>
<td>5.1639</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses

**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment \( c_{2b}(1) = 750 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>0.28915</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>5.223</td>
</tr>
<tr>
<td>Total</td>
<td>5.5121</td>
</tr>
</tbody>
</table>

Loading Combination 2 - Stresses

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 5.5121 N/mm² is less than the maximum allowable stress of 0.6*fck=14.4 N/mm².

Top of beam:
Compressive stress 7.7627 N/mm² is less than the maximum allowable stress of 0.6*fck=21.6 N/mm².
Bottom of in-situ concrete:
Compressive stress 3.8396 N/mm² is less than the maximum allowable stress of 0.6*fckc=14.4 N/mm².
Bottom of beam:
Tensile stress 0.36779 N/mm² is less than the maximum allowable tensile stress of 2.8102 N/mm².
Location: Ex1 - 20 metre span UM beam

Precast prestressed concrete UM beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=20 m

Beam UM 6 section properties

- Depth of beam: d=1040 mm
- Cross-sectional area: \( A_c = 446150 \text{ mm}^2 \)
- Centroid above soffit: \( y_b = 387.7 \text{ mm} \)
- Section modulus top: \( Z_{t} = 65.86 \times 10^6 \text{ mm}^3 \)
- Section modulus bottom: \( Z_{b} = 111.63 \times 10^6 \text{ mm}^3 \)
- Unit weight of beam: \( u_{wb} = 25 \text{ kN/m}^3 \)

Concrete strengths

- Strength at transfer: \( f_{ci} = 40 \text{ N/mm}^2 \)
- Characteristic strength: \( f_{cu} = 50 \text{ N/mm}^2 \)

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 40 N/mm\(^2\) is: \( E_{ci} = 31 \text{ kN/mm}^2 \)
- Modulus of elasticity for concrete strength 50 N/mm\(^2\) is: \( E_{cb} = 34 \text{ kN/mm}^2 \)

Composite section properties

- Char strength of in-situ concrete \( f_{cu,c} = 40 \text{ N/mm}^2 \)
- Modulus of elasticity for concrete strength 40 N/mm\(^2\) is: \( E_{cc} = 31 \text{ kN/mm}^2 \)

Breadth of slab \( b = 1500 \text{ mm} \)
Depth of slab \( d_{is} = 160 \text{ mm} \)
Depth of permanent formwork \( d_{pf} = 15 \text{ mm} \)

\[
\begin{align*}
\text{Area 1} &= 100 \\
\text{Area 2} &= 100 \times (50 - d_{pf}) \text{ mm}^2
\end{align*}
\]

Second moment of area
\[
I_c = I_{xx} + A_c (Y_b - Y_c)^2 + m_r (I_i + A_i (Y_i - Y_c)^2) = 118.59 \times 10^9 \text{ mm}^4
\]

Depth of composite section \( d_i = 1165 \text{ mm} \)
Initial force per strand \( i_f = 174 \text{ kN} \)
Modulus of elasticity \( E_s = 200 \text{ kN/mm}^2 \)

Cross-section 1

- Distance from mid-span of beam \( c_{sd (1)} = 0 \text{ m} \)
- Number of layers of strand \( n_{ls (1)} = 4 \)
- Height above soffit \( d_{s (1)} = 60 \text{ mm} \)
- Number of strands \( n_{s (1)} = 14 \)
- Height above soffit \( d_{s (2)} = 110 \text{ mm} \)
- Number of strands \( n_{s (2)} = 12 \)
- Height above soffit \( d_{s (3)} = 700 \text{ mm} \)
- Number of strands \( n_{s (3)} = 2 \)
- Height above soffit \( d_{s (4)} = 940 \text{ mm} \)
- Number of strands \( n_{s (4)} = 2 \)

Short Term Losses

Relaxation at transfer (\% ) \( s_{tr} = 1 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>52.2</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>448.43</td>
<td>8.5906</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>500.63</td>
<td>9.5906</td>
</tr>
</tbody>
</table>

Losses at Transfer

- Initial force \( 5220 \text{ kN} \)
- Force at transfer \( P_t = P_0 - loft = 4719.4 \text{ kN} \)
Stresses at Transfer N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-4.1009</td>
<td>19.303</td>
<td>15.222</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>8.4055</td>
<td>-4.9959</td>
<td>-2.6592</td>
</tr>
<tr>
<td>Total</td>
<td>4.3045</td>
<td>14.307</td>
<td>12.563</td>
</tr>
</tbody>
</table>

Stress Limitations

Top of beam:
Compressive stress 4.3045 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².
Bottom of beam:
Compressive stress 14.307 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Long Term Losses

Relaxation after transfer( % ) slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>52.2</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>448.43</td>
<td>8.5906</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>500.63</td>
<td>9.5906</td>
</tr>
<tr>
<td>Relaxation</td>
<td>104.4</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>250.2</td>
<td>4.7931</td>
</tr>
<tr>
<td>Creep</td>
<td>521.27</td>
<td>9.986</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>875.87</td>
<td>16.779</td>
</tr>
<tr>
<td>All Losses</td>
<td>1376.5</td>
<td>26.37</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 5220 kN
Final force Pf=P0-loftl=3843.5 kN
Unit-weight of in-situ concrete wc=24 kN/m³
<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.3398</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>8.4055</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>4.2141</td>
</tr>
<tr>
<td>Total</td>
<td>9.2797</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete: $h_{sis} = 1005$ mm
Percentage of shrinkage remaining: $ps = 25\%$
Percentage: $pcr = 25$
Percentage: $stT = 67\%$
The net moment is: $M_c = F_e/E - M_b = -15.561$ kNm

Stress at top of in-situ concrete:
\[
ft_{isds} = \frac{F}{(Ai*mr)} - \frac{F}{(Ai*mr+Ac)} - M_c*1E3*(d_i-Y_c)/I_c)*mr*1E3
\]
\[-0.010076 \text{ N/mm}²\]

Stress at bottom of in-situ concrete:
\[
fb_{isds} = \frac{F}{(Ai*mr)} - \frac{F}{(Ai*mr+Ac)} - M_c*1E3*(h_{sis}-Y_c)/I_c)*mr*1E3
\]
\[0.010919 \text{ N/mm}²\]

Stress at top of precast concrete beam:
\[
ft_{pbds} = \frac{F}{(Ai*mr+Ac)} + M_c*1E3*(d-Y_c)/I_c + M_b*1E3*Y_t/I_{xx})*1E3
\]
\[0.43493E-3 \text{ N/mm}²\]

Stress at bottom of precast concrete beam:
\[
fb_{pbds} = \frac{F}{(Ai*mr+Ac)} - M_c*1E3*Y_c/I_c - M_b*1E3*Y_b/I_{xx})*1E3
\]
\[-0.40407E-3 \text{ N/mm}²\]

Combination 1 loading

Applied bending moment: \[c_{lb(1)} = 1450 \text{ kNm}\]

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-0.010076</td>
</tr>
<tr>
<td>Total</td>
<td>9.0538</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses
Combination 2 to 5 loading

Applied bending moment \( c2b(1) = 2100 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ Dif. shrinkage Combination 2-5</td>
<td>0</td>
<td>9.2797</td>
<td>0</td>
<td>8.2196</td>
</tr>
<tr>
<td>9.5171</td>
<td>0.43493E-3</td>
<td>0.010919</td>
<td>-0.40407E-3</td>
<td>-11.113</td>
</tr>
<tr>
<td>Total</td>
<td>9.507</td>
<td>16.584</td>
<td>6.6946</td>
<td>-2.8941</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 9.507 N/mm² is less than the maximum allowable stress of 0.5*fcu=20 N/mm².

Top of beam:
Compressive stress 16.584 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².

Bottom of in-situ concrete:
Compressive stress 6.6946 N/mm² is less than the maximum allowable stress of 0.5*fcu=20 N/mm².

Bottom of beam:
Tensile stress 2.8941 N/mm² is less than the maximum allowable tensile stress of 3.182 N/mm².
Precast prestressed concrete UM beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=16 m

Beam UM 3 section properties

Depth of beam d=800 mm
Cross-sectional area Ac=374150 mm²
Centroid above soffit Yb=287.9 mm
Section modulus top Zt=38.36E6 mm⁴
Section modulus bottom Zb=68.82E6 mm⁴
Unit weight of beam uwb=24.6 kN/m³

Concrete strengths

Strength at transfer fci=45 N/mm²
Characteristic strength fcu=50 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 45 N/mm² is:
Eci=32.5 kN/mm²
Modulus of elasticity for concrete strength 50 N/mm² is:
Ecb=34 kN/mm²

Composite section properties

Char strength of in-situ concrete fcuc=35 N/mm²
Modulus of elasticity for concrete strength 35 N/mm² is:
Ecc=29.5 kN/mm²

Item No. 1

Item weighting iw1=0.86765
X co-ordinate X(1)=0 mm
Y co-ordinate Y(1)=770 mm
X co-ordinate X(2)=300 mm
Y co-ordinate Y(2)=770 mm
X co-ordinate X(3)=300 mm
Y co-ordinate Y(3)=800 mm
X co-ordinate X(4)=400 mm
Y co-ordinate Y(4)=800 mm
X co-ordinate X(5)=400 mm
Y co-ordinate Y(5)=770 mm
X co-ordinate X(6)=920 mm
Y co-ordinate Y(6)=770 mm
X co-ordinate X(7)=920 mm
Y co-ordinate Y(7)=800 mm
X co-ordinate X(8)=1020 mm

Location: Ex2 – 16 metre span UM3 beam
Y co-ordinate Y(8)=800 mm
X co-ordinate X(9)=1020 mm
Y co-ordinate Y(9)=750 mm
X co-ordinate X(10)=1045 mm
Y co-ordinate Y(10)=750 mm
X co-ordinate X(11)=1045 mm
Y co-ordinate Y(11)=500 mm
X co-ordinate X(12)=1500 mm
Y co-ordinate Y(12)=700 mm
X co-ordinate X(13)=1500 mm
Y co-ordinate Y(13)=1070 mm
X co-ordinate X(14)=1000 mm
Y co-ordinate Y(14)=1070 mm
X co-ordinate X(15)=1000 mm
Y co-ordinate Y(15)=970 mm
X co-ordinate X(16)=0 mm
Y co-ordinate Y(16)=970 mm
X co-ordinate X(17)=0 mm
Y co-ordinate Y(17)=770 mm

Second moment of area IXX2=19.813E9 mm⁴
Area of item A2=374150 mm²
Centroid from X-X axis YC2=287.9 mm
Item weighting iw2=1
Depth of composite section di=1070 mm
Initial force per strand if=90 kN
Modulus of elasticity Es=200 kN/mm²

Cross-section 1

Distance from mid-span of beam csd(1)=2.6 m
Number of layers of strand nls(1)=4
Height above soffit ds(1)=60 mm
Number of strands ns(1)=12
Height above soffit ds(2)=110 mm
Number of strands ns(2)=14
Height above soffit ds(3)=600 mm
Number of strands ns(3)=2
Height above soffit ds(4)=675 mm
Number of strands ns(4)=2

Short Term Losses

Relaxation at transfer ( % ) str=1.2
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>32.4</td>
<td>1.2</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>129.06</td>
<td>4.7799</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>161.46</td>
<td>5.9799</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 2700 kN
Force at transfer $Pt=P0-loft=2538.5$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.5851</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.8085</td>
</tr>
<tr>
<td>Total</td>
<td>5.2234</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
Compressive stress 5.2234 N/mm² is less than allowable stresses of 0.5*fci=22.5 N/mm² and 0.4*fcu=20 N/mm².

Bottom of beam:
Compressive stress 7.6627 N/mm² is less than allowable stresses of 0.5*fci=22.5 N/mm² and 0.4*fcu=20 N/mm².

Long Term Losses

Relaxation after transfer( % ) slr(1)=1
Shrinkage per unit length sul(1)=0.3E-3
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>32.4</td>
<td>1.2</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>129.06</td>
<td>4.7799</td>
</tr>
<tr>
<td>Total</td>
<td>161.46</td>
<td>5.9799</td>
</tr>
<tr>
<td>Relaxation</td>
<td>27</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>135</td>
<td>5</td>
</tr>
<tr>
<td>Creep</td>
<td>154.95</td>
<td>5.739</td>
</tr>
<tr>
<td>Total</td>
<td>316.95</td>
<td>11.739</td>
</tr>
<tr>
<td>All Losses</td>
<td>478.41</td>
<td>17.719</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2700 kN
Final force Pf=P0-loftl=2221.6 kN
Unit-weight of in-situ concrete wc=23.5 kN/m²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.3872</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.8085</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>7.3332</td>
</tr>
<tr>
<td>Total</td>
<td>12.755</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=500 mm
Perc'age of shrinkage remaining ps=45 %
Percentage pcr=45
Percentage stT=60 %
The net moment is Mc=F*e/1E3-Mb=-102.53 kNm

Stress at top of in-situ concrete:
\[ f_{tisds} = -\frac{F}{(A_i*mr)} - \frac{F}{(A_i*mr+Ac)} - \frac{Mc*1E3*(di-Yc)/Ic)}{mr*1E3} \]
\[ = 0.55782 \ N/mm² \]

Stress at bottom of in-situ concrete:
\[ f_{bisds} = -\frac{F}{(A_i*mr)} - \frac{F}{(A_i*mr+Ac)} - \frac{Mc*1E3*(hsis-Yc)/Ic)}{mr*1E3} \]
\[ = 1.1652 \ N/mm² \]

Stress at top of precast concrete beam:
Reinforced & prestressed concrete: BS5400 & Eurocode

UM beam

Date: 02/12/19

Copyright 1986-2019 Fitzroy Systems Ltd.

Reinforced and prestressed concrete: BS5400 & Eurocode

Made by: IFB

Copyright 1986-2019 Fitzroy Systems Ltd.

Ref No: SC142 BS

ft pb ds = (F/(Ai * mr + Ac) + Mc * 1E3 * (d - Yc)/Ic + Mb * 1E3 * Yt/Ixx) * 1E3

Stress at bottom of precast concrete beam:

fb pb ds = (F/(Ai * mr + Ac) - Mc * 1E3 * Yc/Ic - Mb * 1E3 * Yb/Ixx) * 1E3 = 1.7626 N/mm²

**Combination 1 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>12.755</td>
<td>0</td>
<td>2.1053</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.55782</td>
<td>-5.6068</td>
<td>1.1652</td>
<td>1.7626</td>
</tr>
<tr>
<td>Combination 1</td>
<td>2.8729</td>
<td>1.5323</td>
<td>-0.35032</td>
<td>-3.3984</td>
</tr>
<tr>
<td>Total</td>
<td>3.4307</td>
<td>8.68</td>
<td>0.81488</td>
<td>0.4695</td>
</tr>
</tbody>
</table>

**Combination 1 Loading - Stresses**

**Combination 2 to 5 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>12.755</td>
<td>0</td>
<td>2.1053</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.55782</td>
<td>-5.6068</td>
<td>1.1652</td>
<td>1.7626</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>4.1788</td>
<td>2.2288</td>
<td>-0.56052</td>
<td>-5.4374</td>
</tr>
<tr>
<td>Total</td>
<td>4.7366</td>
<td>9.3766</td>
<td>0.60469</td>
<td>-1.5695</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 Loading - Stresses**

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
- Compressive stress 4.7366 N/mm² is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm².
- Tensile stress 1.5695 N/mm² is less than the maximum allowable stress of 3.182 N/mm².

Top of beam:
- Compressive stress 9.3766 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².
- Tensile stress 1.5695 N/mm² is less than the maximum allowable stress of 3.182 N/mm².

Bottom of in-situ concrete:
- Compressive stress 0.60469 N/mm² is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm².
- Tensile stress 1.5695 N/mm² is less than the maximum allowable stress of 3.182 N/mm².
Location: Ex3 - 14 metre span UM2 beam

Precast prestressed concrete UM beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam \( \text{span}=14 \text{ m} \)

**Beam UM 2 section properties**

- Depth of beam \( d=720 \text{ mm} \)
- Cross-sectional area \( A_c=350150 \text{ mm}^2 \)
- Centroid above soffit \( y_b=256.6 \text{ mm} \)
- Section modulus top \( Z_t=30.8E6 \text{ mm}^3 \)
- Section modulus bottom \( Z_b=56.17E6 \text{ mm}^3 \)
- Unit weight of beam \( u_{wb}=24 \text{ kN/m}^3 \)

**Concrete strengths**

- Strength at transfer \( f_{ci}=35 \text{ N/mm}^2 \)
- Characteristic strength \( f_{cu}=40 \text{ N/mm}^2 \)

**Concrete moduli**

- Modulus of elasticity at transfer for concrete strength 35 N/mm\(^2\) is: \( E_{ci}=29.5 \text{ kN/mm}^2 \)
- Modulus of elasticity for concrete strength 40 N/mm\(^2\) is: \( E_{cb}=31 \text{ kN/mm}^2 \)

**Composite section properties**

- Char strength of in-situ concrete \( f_{cu,c}=30 \text{ N/mm}^2 \)
- Modulus of elasticity for concrete strength 30 N/mm\(^2\) is: \( E_{cc}=28 \text{ kN/mm}^2 \)

Breadth of slab \( b=1500 \text{ mm} \)
Depth of slab \( d_{is} = 200 \text{ mm} \)
Depth of permanent formwork \( d_{pf} = 20 \text{ mm} \)

\[ \text{Area 1} = 100 \times (50 - d_{pf}) \text{ mm}^2 \]

Second moment of area \( I_c = I_{xx} + A_c(Y_b - Y_c)^2 + m_r(I_i + A_i(Y_i - Y_c)^2) = 61.134 \times 10^9 \text{ mm}^4 \)

Depth of composite section \( d_i = 890 \text{ mm} \)
Nominal diameter \( \text{nomd} = 11 \text{ mm} \)
Nominal tensile strength \( \text{nomt} = 1625 \text{ N/mm}^2 \)
Nominal steel area \( \text{noma} = 90 \text{ mm}^2 \)
Characteristic breaking load \( c_{bl} = 140 \text{ kN} \)
Initial force per strand \( i_{f} = 80 \text{ kN} \)
Modulus of elasticity \( E_s = 200 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( c_{sd}(1) = 4.1 \text{ m} \)
Number of layers of strand \( n_{ls}(1) = 3 \)
Height above soffit \( d_{s}(1) = 55 \text{ mm} \)
Number of strands \( n_{s}(1) = 10 \)

**WARNING:**

Strand location is non-standard.

Height above soffit \( d_{s}(2) = 100 \text{ mm} \)
Number of strands \( n_{s}(2) = 12 \)

**WARNING:**

Strand location is non-standard.

Height above soffit \( d_{s}(3) = 650 \text{ mm} \)
Number of strands \( n_{s}(3) = 2 \)

**WARNING:**

Strand location is too high.

Bending moment from temp.loads \( t_{lb}(1) = 246 \text{ kNm} \)
Bending moment from other loads \( o_{lb}(1) = 155 \text{ kNm} \)

**Short Term Losses**

Relaxation at transfer ( % ) \( \text{str} = 1 \)
**Summary**

<table>
<thead>
<tr>
<th></th>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td></td>
<td>19.2</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td></td>
<td>111.89</td>
<td>5.8277</td>
</tr>
<tr>
<td>Transfer Total</td>
<td></td>
<td>131.09</td>
<td>6.8277</td>
</tr>
</tbody>
</table>

**Losses at Transfer**

Initial force 1920 kN

Force at transfer $P_t=P_0-loft=1788.9$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-2.3402</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.3486</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>7.9092</td>
</tr>
<tr>
<td>Other Loads</td>
<td>4.9834</td>
</tr>
<tr>
<td>Total</td>
<td>14.901</td>
</tr>
</tbody>
</table>

**Beam Concrete Stresses at Transfer**

**Stress Limitations**

Top of beam:
Compressive stress 14.901 N/mm² is less than allowable stresses of $0.5*f_{ci}=17.5$ N/mm² and $0.4*f_{cu}=16$ N/mm².
Bottom of beam:
The flexural tensile stress of 0.31318 N/mm² at transfer is less than the permissible value of 1 N/mm². Clause 6.3.2.4(b)(1).

**Long Term Losses**

Relaxation after transfer( % ) $slr(1)=1$

Shrinkage per unit length $sul(1)=0.3E-3$
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>19.2</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>111.89</td>
<td>5.8277</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>131.09</td>
<td>6.8277</td>
</tr>
<tr>
<td>Relaxation</td>
<td>19.2</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>129.6</td>
<td>6.75</td>
</tr>
<tr>
<td>Creep</td>
<td>108.6</td>
<td>5.6564</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>257.4</td>
<td>13.406</td>
</tr>
<tr>
<td>All Losses</td>
<td>388.49</td>
<td>20.234</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force: 1920 kN  
Final force: Pf=P0-lofl=1531.5 kN
Unit-weight of in-situ concrete: wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-2.0035</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.3486</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.6513</td>
</tr>
<tr>
<td>Total</td>
<td>5.9964</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

<table>
<thead>
<tr>
<th>Composite section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height from beam soffit to bottom of in-situ concrete: hsis=690 mm</td>
</tr>
<tr>
<td>Perc'age of shrinkage remaining: ps=30%</td>
</tr>
<tr>
<td>Percentage: pcr=30</td>
</tr>
<tr>
<td>Percentage: stT=65%</td>
</tr>
<tr>
<td>The net moment is: Mc=F*e/1E3-Mb=65.131 kNm</td>
</tr>
<tr>
<td>Stress at top of in-situ concrete: fhisds=-(F/(Ai<em>mr)-F/(Ai</em>mr+Ac)-Mc<em>1E3</em>(di-Yc)/Ic)<em>mr</em>1E3 =0.11008 N/mm²</td>
</tr>
<tr>
<td>Stress at bottom of in-situ concrete: fhisds=-(F/(Ai<em>mr)-F/(Ai</em>mr+Ac)-Mc<em>1E3</em>(hsis-Yc)/Ic)<em>mr</em>1E3 =-0.103 N/mm²</td>
</tr>
<tr>
<td>Stress at top of precast concrete beam:</td>
</tr>
</tbody>
</table>
ftpbds=(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3
=-0.064493 N/mm²

Stress at bottom of precast concrete beam:

\[ f_{pbds} = (F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3 = 0.028685 \text{ N/mm}^2 \]

**Combination 1 loading**

Applied bending moment \( c_{lb(1)} = 410 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.11008</td>
</tr>
<tr>
<td>Combination 1</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>3.0764</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

**Combination 2 to 5 loading**

Applied bending moment \( c_{2b(1)} = 512 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.11008</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>3.2592</td>
</tr>
<tr>
<td>Total</td>
<td>3.3693</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 3.3693 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².

Top of beam:
Compressive stress 7.7674 N/mm² is less than the maximum allowable stress of 0.4*fcu=16 N/mm².

Bottom of in-situ concrete:
Compressive stress 1.4812 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².

Bottom of beam:
Tensile stress 0.69049 N/mm² is less than the maximum allowable tensile stress of 2.846 N/mm².
Location: Ex1 - 20 metre span UM beam

Precast prestressed concrete UM beams with in-situ top slab


Beam span under consideration span=20 m

Beam UM 6 section properties

Depth of beam d=1040 mm
Cross-sectional area Ac=446150 mm$^2$
Centroid above soffit Yb=387.7 mm
Section modulus top Zt=65.86E6 mm$^3$
Section modulus bottom Zb=111.63E6 mm$^3$
Unit weight of beam uwb=25 kN/m$^3$

Concrete compressive strengths

Characteristic cylinder strength fcki=32 N/mm$^2$ (transfer)
Characteristic cylinder strength fck=40 N/mm$^2$

Concrete moduli (short-term)

Concrete modulus of elasticity Ecmi=33 kN/mm$^2$ (transfer)
Concrete modulus of elasticity Ecmb=35 kN/mm$^2$

Composite section properties

Char strength (in-situ concrete) fckc=32 N/mm$^2$
Short-term modulus of elasticity for in-situ concrete Ecmc=33 kN/mm$^2$

Breadth of slab b=1500 mm
Depth of slab dis=160 mm
Depth of permanent formwork dpf=15 mm
Cross-section 1

Distance from mid-span of beam \(csd(1)=0\) m
Number of layers of strand \(nls(1)=4\)
Height above soffit \(ds(1)=60\) mm
Number of strands \(ns(1)=14\)
Height above soffit \(ds(2)=110\) mm
Number of strands \(ns(2)=12\)
Height above soffit \(ds(3)=700\) mm
Number of strands \(ns(3)=2\)
Height above soffit \(ds(4)=940\) mm
Number of strands \(ns(4)=2\)

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer (\%) \(str=1\)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>52.2</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>380.48</td>
<td>7.2889</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>432.68</td>
<td>8.2889</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \(5220\) kN
Force at transfer \(Pt=P0-loft=4787.3\) kN
Beam Concrete Stresses at Transfer

Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam:
Compressive stress 4.2455 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

Bottom of beam:
Compressive stress 14.585 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer( % ) slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(∞,to) crf=2

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>52.2</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>380.48</td>
<td>7.2889</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>432.68</td>
<td>8.2889</td>
</tr>
<tr>
<td>Relaxation</td>
<td>104.4</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>243.95</td>
<td>4.6733</td>
</tr>
<tr>
<td>Creep</td>
<td>599.08</td>
<td>11.477</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>947.42</td>
<td>18.15</td>
</tr>
<tr>
<td>All Losses</td>
<td>1380.1</td>
<td>26.439</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 5220 kN
Final force Pf=P0-loftl=3839.9 kN
Unit-weight of in-situ concrete wc=24 kN/m³
<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.3367</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>8.4055</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>4.2141</td>
</tr>
<tr>
<td>Total</td>
<td>9.2829</td>
</tr>
</tbody>
</table>

**Beam concrete stresses - Long term**

**Composite section**

Height from beam soffit to bottom of in-situ concrete: \( h_{sis} = 1005 \text{ mm} \)
Percentage of shrinkage remaining: \( p_s = 25 \% \)
Percentage of shrinkage remaining: \( p_{cr} = 25 \% \)
Percentage of shrinkage remaining: \( p_{stT} = 67 \% \)
The net moment is: \( M_c = F*e/1E3 - M_b = -76.365 \text{ kNm} \)

Stress at top of in-situ concrete:
\[
ft_{isds} = -\frac{F}{(Ai*mr)} - \frac{F}{(Ai*mr+Ac)} - Mc*1E3*(d-Yc)/Ic)*mr*1E3
\]
\[
= 0.069729 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
fb_{isds} = -\frac{F}{(Ai*mr)} - \frac{F}{(Ai*mr+Ac)} - Mc*1E3*(h_{sis}-Yc)/Ic)*mr*1E3
\]
\[
= 0.17276 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:
\[
ft_{pbds} = \frac{F}{(Ai*mr+Ac)} + Mc*1E3*(d-Yc)/Ic + Mb*1E3*Yt/Iyy)*1E3
\]
\[
= -0.35273 \text{ N/mm}^2
\]

Stress at bottom of precast concrete beam:
\[
fb_{pbds} = \frac{F}{(Ai*mr+Ac)} - Mc*1E3*Yc/Ic - Mb*1E3*Yb/Iyy)*1E3 = 0.1097 \text{ N/mm}^2
\]

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment: \( c_{lb(1)} = 1450 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>0.069729</td>
</tr>
<tr>
<td>Total</td>
<td>9.1336</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses
Loading Combination 2

(tensile stress permitted (no visible cracking))
Characteristic bending moment \[ c_{2b(1)} = 2100 \text{ kNm} \]

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
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<tr>
<td>Prestress, beam and in-situ</td>
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</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>0.069729</td>
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<tr>
<td></td>
<td>9.5171</td>
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<tr>
<td>Total</td>
<td>9.5868</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 9.5868 N/mm² is less than the maximum allowable stress of 0.6*f_{ckc} = 19.2 N/mm².

Top of beam:
Compressive stress 16.234 N/mm² is less than the maximum allowable stress of 0.6*f_{ck} = 24 N/mm².

Bottom of in-situ concrete:
Compressive stress 6.8565 N/mm² is less than the maximum allowable stress of 0.6*f_{ckc} = 19.2 N/mm².

Bottom of beam:
Tensile stress 2.7988 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
Location: Ex2 - 16 metre span UM3 beam

Precast prestressed concrete UM beams with in-situ top slab


Beam span under consideration span=16 m

Beam UM 3 section properties

- Depth of beam $d=800$ mm
- Cross-sectional area $A_c=374150$ mm$^2$
- Centroid above soffit $Y_b=287.9$ mm
- Section modulus top $Z_t=38.36E6$ mm$^3$
- Section modulus bottom $Z_b=68.82E6$ mm$^3$
- Unit weight of beam $u_{wb}=24.6$ kN/m$^3$

Concrete compressive strengths

- Characteristic cylinder strength $f_{ck}=35$ N/mm$^2$ (transfer)
- Characteristic cylinder strength $f_{ck}=40$ N/mm$^2$

Concrete moduli (short-term)

- Concrete modulus of elasticity $E_{cm}=34$ kN/mm$^2$ (transfer)
- Concrete modulus of elasticity $E_{cm}=35$ kN/mm$^2$

Composite section properties

- Char strength (in-situ concrete) $f_{ckc}=28$ N/mm$^2$
- Short-term modulus of elasticity for in-situ concrete $E_{cmc}=32$ kN/mm$^2$

Item No. 1

- Item weighting $iw_1=0.91429$
- X co-ordinate $X(1)=0$ mm
- Y co-ordinate $Y(1)=770$ mm
- X co-ordinate $X(2)=300$ mm
- Y co-ordinate $Y(2)=770$ mm
- X co-ordinate $X(3)=300$ mm
- Y co-ordinate $Y(3)=800$ mm
- X co-ordinate $X(4)=400$ mm
- Y co-ordinate $Y(4)=800$ mm
- X co-ordinate $X(5)=400$ mm
- Y co-ordinate $Y(5)=770$ mm
- X co-ordinate $X(6)=920$ mm
- Y co-ordinate $Y(6)=770$ mm
- X co-ordinate $X(7)=920$ mm
- Y co-ordinate $Y(7)=800$ mm
- X co-ordinate $X(8)=1020$ mm
- Y co-ordinate $Y(8)=800$ mm
- X co-ordinate $X(9)=1020$ mm
Y co-ordinate                     Y(9)=750 mm
X co-ordinate                     X(10)=1045 mm
Y co-ordinate                     Y(10)=750 mm
X co-ordinate                     X(11)=1045 mm
Y co-ordinate                     Y(11)=500 mm
X co-ordinate                     X(12)=1500 mm
Y co-ordinate                     Y(12)=700 mm
X co-ordinate                     X(13)=1500 mm
Y co-ordinate                     Y(13)=1070 mm
X co-ordinate                     X(14)=1000 mm
Y co-ordinate                     Y(14)=1070 mm
X co-ordinate                     X(15)=1000 mm
Y co-ordinate                     Y(15)=970 mm
X co-ordinate                     X(16)=0 mm
Y co-ordinate                     Y(16)=970 mm
X co-ordinate                     X(17)=0 mm
Y co-ordinate                     Y(17)=770 mm

Second moment of area             IXX2=19.813E9 mm^4
Area of item                      A2=374150 mm^2
Centroid from X-X axis            YC2=287.9 mm
Item weighting                    iw2=1
Depth of composite section        di=1070 mm
Modulus of elasticity (strand)    Ep=195 kN/mm^2

Cross-section 1

Distance from mid-span of beam    csd(1)=2.6 m
Number of layers of strand       nls(1)=4
Height above soffit              ds(1)=60 mm
Number of strands                ns(1)=12
Height above soffit              ds(2)=110 mm
Number of strands                ns(2)=14
Height above soffit              ds(3)=600 mm
Number of strands                ns(3)=2
Height above soffit              ds(4)=675 mm
Number of strands                ns(4)=2

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % )     str=1.2

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>32.4</td>
<td>1.2</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>115.09</td>
<td>4.2626</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>147.49</td>
<td>5.4626</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force           2700 kN
Beam Concrete Stresses at Transfer

Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam:
Compressive stress 5.2147 N/mm² is less than allowable stresses of 0.7*fcki=24.5 N/mm² and 0.6*fck=24 N/mm².

Bottom of beam:
Compressive stress 7.7259 N/mm² is less than allowable stresses of 0.7*fcki=24.5 N/mm² and 0.6*fck=24 N/mm².

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer( % ) slr(1)=1
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient ϕ(∞,to) crf=2

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>32.4</td>
<td>1.2</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>115.09</td>
<td>4.2626</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>147.49</td>
<td>5.4626</td>
</tr>
<tr>
<td>Relaxation</td>
<td>27</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>131.63</td>
<td>4.875</td>
</tr>
<tr>
<td>Creep</td>
<td>181.08</td>
<td>6.7067</td>
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<tr>
<td>Long-term Total</td>
<td>339.71</td>
<td>12.582</td>
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<tr>
<td>All Losses</td>
<td>487.2</td>
<td>18.044</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2700 kN
Final force Pf=P0-loftl=2212.8 kN
Unit-weight of in-situ concrete wc=23.5 kN/m³
### Final Stresses N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.3817</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.8085</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>7.3332</td>
</tr>
<tr>
<td>Total</td>
<td>12.76</td>
</tr>
</tbody>
</table>

**Beam concrete stresses - Long term**

**Composite section**

- Height from beam soffit to bottom of in-situ concrete: \( h_{sis} = 500 \text{ mm} \)
- Percentage of shrinkage remaining: \( ps = 45 \% \)
- Percentage: \( pcr = 45 \% \)
- Percentage: \( stT = 60 \% \)

The net moment is: \( Mc = F*e/1E3 - Mb = -184.96 \text{ kNm} \)

**Stress at top of in-situ concrete:**

\[
ftisds = -\left(\frac{F}{Ai*mr} - \frac{F}{Ai*mr+Ac} - Mc*1E3*(d-Yc)/Ic\right)*mr*1E3
= 0.60673 \text{ N/mm}^2
\]

**Stress at bottom of in-situ concrete:**

\[
fbisds = -\left(\frac{F}{Ai*mr} - \frac{F}{Ai*mr+Ac} - Mc*1E3*(h_{sis}-Yc)/Ic\right)*mr*1E3
= 1.7369 \text{ N/mm}^2
\]

**Stress at top of precast concrete beam:**

\[
ftpbds = \left(\frac{F}{Ai*mr+Ac} + Mc*1E3*(d-Yc)/Ic + Mb*1E3*Yt/Iyy\right)*1E3
= -7.2888 \text{ N/mm}^2
\]

**Stress at bottom of precast concrete beam:**

\[
fbpbds = \left(\frac{F}{Ai*mr+Ac} - Mc*1E3*Yc/Ic - Mb*1E3*Yb/Iyy\right)*1E3 = 2.2717 \text{ N/mm}^2
\]

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment \( clb(1) = 500 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>0.60673</td>
</tr>
<tr>
<td>Total</td>
<td>3.5262</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses

---

**Sample output for SCALE Proforma 142. (ans=2)**

Reinforced & prestressed concrete: BS5400 & Eurocode

Made by: IFB

UM beam

Date: 02/12/19

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Loading Combination 2

(tensile stress permitted (no visible cracking))
Characteristic bending moment \( c_2b(1) = 800 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.60673</td>
</tr>
<tr>
<td>Combination 2</td>
<td>4.2465</td>
</tr>
<tr>
<td>Total</td>
<td>4.8532</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 4.8532 N/mm\(^2\) is less than the maximum allowable stress of \( 0.6 \times f_{ckc} = 16.8 \) N/mm\(^2\).

Top of beam:
Compressive stress 7.5833 N/mm\(^2\) is less than the maximum allowable stress of \( 0.6 \times f_{ck} = 24 \) N/mm\(^2\).

Bottom of in-situ concrete:
Compressive stress 1.0953 N/mm\(^2\) is less than the maximum allowable stress of \( 0.6 \times f_{ckc} = 16.8 \) N/mm\(^2\).

Bottom of beam:
Tensile stress 1.0541 N/mm\(^2\) is less than the maximum allowable tensile stress of 3.0263 N/mm\(^2\).
Location: Ex3 - 14 metre span UM2 beam

Precast prestressed concrete UM beams with in-situ top slab


Beam span under consideration span=14 m

Beam UM 2 section properties

Depth of beam d=720 mm
Cross-sectional area Ac=350150 mm²
Centroid above soffit Yb=256.6 mm
Section modulus top Zt=30.8E6 mm³
Section modulus bottom Zb=56.17E6 mm³
Unit weight of beam uwb=24 kN/m³

Concrete compressive strengths

Characteristic cylinder strength fcki=28 N/mm² (transfer)
Characteristic cylinder strength fck=32 N/mm²

Concrete moduli (short-term)

Concrete modulus of elasticity Ecmi=32 kN/mm² (transfer)
Concrete modulus of elasticity Ecmb=33 kN/mm²

Composite section properties

Char strength (in-situ concrete) fckc=25 N/mm²
Short-term modulus of elasticity for in-situ concrete Ecmc=31 kN/mm²

Breadth of slab b=1500 mm
Depth of slab dis=200 mm
Depth of permanent formwork dpf=20 mm
Area 1

In-situ concrete

Area2=100*(50-dpf) mm²

Second moment of area

Ic=Iyy+Ac*(Yb-Yc)^2+mr*(Ii+Ai*(Yi-Yc)^2)=61.134E9 mm⁶

Depth of composite section
di=890 mm

Nominal diameter
nomd=11 mm

Nominal tensile strength
nomt=1625 N/mm²

Nominal steel area
noma=90 mm²

Characteristic breaking load
fpk=140 kN

Initial force per strand
ipf=80 kN

Modulus of elasticity (strand)
Ep=195 kN/mm²

Cross-section 1

Distance from mid-span of beam
csd(1)=4.1 m

Number of layers of strand
nls(1)=3

Height above soffit
ds(1)=55 mm

Number of strands
ns(1)=10

WARNING:
Strand location is non-standard.

Height above soffit
ds(2)=100 mm

Number of strands
ns(2)=12

WARNING:
Strand location is non-standard.

Height above soffit
ds(3)=650 mm

Number of strands
ns(3)=2

WARNING:
Strand location is too high.

Bending moment from temp.loads
tlb(1)=246 kNm

Bending moment from other loads
olb(1)=155 kNm

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % )
str=1
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>19.2</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>95.517</td>
<td>4.9749</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>114.72</td>
<td>5.9749</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 1920 kN
Force at transfer $P_t = P_0 - \text{loft} = 1805.3$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-2.3616</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.3486</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>7.9092</td>
</tr>
<tr>
<td>Other Loads</td>
<td>4.9834</td>
</tr>
<tr>
<td>Total</td>
<td>14.88</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
Compressive stress 14.88 N/mm² is less than allowable stresses of $0.7\times f_{cki} = 19.6$ N/mm² and $0.6\times f_{ck} = 19.2$ N/mm².

Bottom of beam:
The flexural tensile stress of 0.22866 N/mm² at transfer is less than the permissible value of 1 N/mm² (as was permitted in BS5400-4).

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer ( % ) $slr(1) = 1$
Shrinkage per unit length $sul(1) = 0.3E-3$
Creep coefficient $\phi(\infty, to)$ $crf = 2.5$
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>19.2</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>95.517</td>
<td>4.9749</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>114.72</td>
<td>5.9749</td>
</tr>
<tr>
<td>Relaxation</td>
<td>19.2</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>126.36</td>
<td>6.5813</td>
</tr>
<tr>
<td>Creep</td>
<td>148.33</td>
<td>7.7256</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>293.89</td>
<td>15.307</td>
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<tr>
<td>All Losses</td>
<td>408.61</td>
<td>21.282</td>
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</tbody>
</table>

Losses - Long term

Initial force 1920 kN
Final force Pf=P0-loftl=1511.4 kN
Unit-weight of in-situ concrete wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-1.9772</td>
<td>7.8014</td>
<td>6.0754</td>
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<tr>
<td>Self weight of Beam</td>
<td>4.3486</td>
<td>-2.408</td>
<td>-1.2154</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.6513</td>
<td>-2.0218</td>
<td>-1.0205</td>
</tr>
<tr>
<td>Total</td>
<td>6.0227</td>
<td>3.3716</td>
<td>3.8395</td>
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</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=690 mm
Percentage of shrinkage remaining ps=30 %
Percentage pcr=30
Percentage stT=65 %
The net moment is Mc=F*e/1E3-Mb=8.7193 kNm
Stress at top of in-situ concrete:
\[ f_{tisds} = -\frac{F}{(A_i*mr)} - F/(A_i*mr+Ac) - Mc*1E3*(di-Yc)/Ic)*mr*1E3 = 0.16494 N/mm² \]
Stress at bottom of in-situ concrete:
\[ f_{bisds} = -\frac{F}{(A_i*mr)} - F/(A_i*mr+Ac) - Mc*1E3*(hsis-Yc)/Ic)*mr*1E3 = 0.13641 N/mm² \]
Stress at top of precast concrete beam:
ftpbds\(=\frac{F}{(A_i mr + A_c) + Mc*1E3*(d-Y_c)/I_c + Mb*1E3*Y_t/I_{yy}}\) *1E3
\(-0.89452\ N/mm^2\)

Stress at bottom of precast concrete beam:
\(\text{fbpbds} = \frac{F}{(A_i mr + A_c) - Mc*1E3*Y_c/I_c - Mb*1E3*Y_b/I_{yy}}\) *1E3
\(=0.29843\ N/mm^2\)

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment \(c_{lb}(1)=410\ kNm\)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Dif. shrinkage Combination 1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.16494</td>
</tr>
<tr>
<td></td>
<td>2.9664</td>
</tr>
<tr>
<td>Total</td>
<td>3.1313</td>
</tr>
</tbody>
</table>

**Loading Combination 1 - Stresses**

**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment \(c_{2b}(1)=512\ kNm\)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Dif. shrinkage Combination 2</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.16494</td>
</tr>
<tr>
<td></td>
<td>3.2592</td>
</tr>
<tr>
<td>Total</td>
<td>3.4241</td>
</tr>
</tbody>
</table>

**Loading Combination 2 - Stresses**

**NOTE:** The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete: Compressive stress 3.4241 N/mm² is less than the maximum allowable stress of 0.6*fckc=15 N/mm².

Top of beam: Compressive stress 6.9636 N/mm² is less than the maximum allowable stress of 0.6*fck=19.2 N/mm².
Bottom of in-situ concrete:
Compressive stress 1.7206 N/mm² is less than the maximum allowable
stress of 0.6*fckc=15 N/mm².

Bottom of beam:
Tensile stress 0.52457 N/mm² is less than the maximum allowable
tensile stress of 2.8534 N/mm².
Location: Ex1 - 20 metre span Y beam

Precast prestressed concrete Y beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=20 m

Beam Y 4 section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=1000 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=410880 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=399.3 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>Zt=63.63E6 mm³</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>Zb=95.72E6 mm³</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=24 kN/m³</td>
</tr>
</tbody>
</table>

Concrete strengths

<table>
<thead>
<tr>
<th>Strength</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength at transfer</td>
<td>fc1=40 N/mm²</td>
</tr>
<tr>
<td>Characteristic strength</td>
<td>fcu=50 N/mm²</td>
</tr>
</tbody>
</table>

Concrete moduli

<table>
<thead>
<tr>
<th>Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity at transfer for concrete strength 40 N/mm² is:</td>
<td>Eci=31 kN/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity for concrete strength 50 N/mm² is:</td>
<td>Ecb=34 kN/mm²</td>
</tr>
</tbody>
</table>

Composite section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char strength of in-situ concrete</td>
<td>fcuc=40 N/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity for concrete strength 40 N/mm² is:</td>
<td>Ecc=31 kN/mm²</td>
</tr>
</tbody>
</table>
Second moment of area:
Depth of composite section \( d_i = 1125 \text{ mm} \)
Initial force \( f_i = 174 \text{ kN} \)
Modulus of elasticity \( E_s = 200 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( c_{sd}(1) = 0 \text{ m} \)
Number of layers of strand \( n_{ls}(1) = 4 \)
Height above soffit \( d_s(1) = 60 \text{ mm} \)
Number of strands \( n_s(1) = 11 \)
Height above soffit \( d_s(2) = 110 \text{ mm} \)
Number of strands \( n_s(2) = 14 \)
Height above soffit \( d_s(3) = 800 \text{ mm} \)
Number of strands \( n_s(3) = 2 \)
Height above soffit \( d_s(4) = 900 \text{ mm} \)
Number of strands \( n_s(4) = 2 \)

**Short Term Losses**

Relaxation at transfer ( % ) \( s_{tr} = 1 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46 kN</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>460.71 kN</td>
<td>9.1302</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>511.17 kN</td>
<td>10.13</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force: 5046 kN
Force at transfer: Pt=P₀-loft=4534.8 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.6591</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.7491</td>
</tr>
<tr>
<td>Total</td>
<td>4.09</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
Compressive stress 4.09 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Bottom of beam:
Compressive stress 15.655 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Long Term Losses

Relaxation after transfer( % ) slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>460.71</td>
<td>9.1302</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>511.17</td>
<td>10.13</td>
</tr>
<tr>
<td>Relaxation</td>
<td>100.92</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>241.86</td>
<td>4.7931</td>
</tr>
<tr>
<td>Creep</td>
<td>564.59</td>
<td>11.189</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>907.37</td>
<td>17.982</td>
</tr>
<tr>
<td>All Losses</td>
<td>1418.5</td>
<td>28.112</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 5046 kN
Final force \( Pf=P_0-loftl=3627.5 \) kN
Unit-weight of in-situ concrete \( wc=24 \) kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-2.9269</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.7491</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.8294</td>
</tr>
<tr>
<td>Total</td>
<td>7.6516</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to
bottom of in-situ concrete \( h_{sis}=965 \) mm
Perc'age of shrinkage remaining \( ps=25 \) %
Percentage \( pcr=25 \)
Percentage \( stT=67 \) %
The net moment is \( Mc=F*e/1E3-Mb=-69.607 \) kNm

Stress at top of in-situ concrete:
\[
ftisds=-\frac{F}{(Ai*mr)}-\frac{F}{(Ai*mr+Ac)}-Mc*1E3*(di-Yc)/Ic)*mr*1E3
\]
\( =-0.27908 \) N/mm²

Stress at bottom of in-situ concrete:
\[
fbisds=-\frac{F}{(Ai*mr)}-\frac{F}{(Ai*mr+Ac)}-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
\]
\( =-0.14795 \) N/mm²

Stress at top of precast concrete beam:
Sample output for SCALE Proforma 143. (ans=1)  
Reinforced&prestressed concrete: BS5400 & Eurocode  
Made by: IFB  
Y beam  
Date: 02/12/19  
Ref No: SC143 BS

ftpbd=(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3
=0.41544 N/mm²

Stress at bottom of precast concrete beam:
fbpdbd=(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3=-0.1443 N/mm²

Combination 1 loading

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam</td>
<td>0</td>
</tr>
<tr>
<td>Prestress,beam and in-situ</td>
<td>-0.27908</td>
</tr>
<tr>
<td>Combination 1</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>11.417</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

Combination 2 to 5 loading

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam</td>
<td>0</td>
</tr>
<tr>
<td>Prestress,beam and in-situ</td>
<td>-0.27908</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>12.021</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>11.742</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 11.742 N/mm² is less than the maximum allowable stress of 0.5*fcuc=20 N/mm².

Top of beam:
Compressive stress 17.366 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².

Bottom of in-situ concrete:
Compressive stress 8.3882 N/mm² is less than the maximum allowable stress of 0.5*fcuc=20 N/mm².
Bottom of beam:
Tensile stress 3.0173 N/mm² is less than the maximum allowable
tensile stress of 3.182 N/mm².
Location: Ex2 - 29.5 metre span Y8 beam

Precast prestressed concrete Y beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam: span=29.5 m

Beam Y 8 section properties

- Depth of beam: d=1400 mm
- Cross-sectional area: Ac=585580 mm²
- Centroid above soffit: Yb=637.8 mm
- Section modulus top: Zt=156.12E6 mm³
- Section modulus bottom: Zb=186.56E6 mm³
- Unit weight of beam: uwb=24.5 kN/m³

Concrete strengths

- Strength at transfer: fci=40 N/mm²
- Characteristic strength: fcu=52.5 N/mm²

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 40 N/mm² is: Eci=31 kN/mm²
- Modulus of elasticity for concrete strength 52.5 N/mm² is: Ecb=34.5 kN/mm²

Composite section properties

- Char strength of in-situ concrete fcuc=40 N/mm²
- Modulus of elasticity for concrete strength 40 N/mm² is: Ecc=31 kN/mm²

Item No. 1

- Item weighting: iw1=0.89855
- X co-ordinate: X(1)=100 mm
- Y co-ordinate: Y(1)=0 mm
- X co-ordinate: X(2)=850 mm
- Y co-ordinate: Y(2)=0 mm
- X co-ordinate: X(3)=850 mm
- Y co-ordinate: Y(3)=202 mm
- X co-ordinate: X(4)=1100 mm
- Y co-ordinate: Y(4)=1300 mm
- X co-ordinate: X(5)=1350 mm
- Y co-ordinate: Y(5)=1300 mm
- X co-ordinate: X(6)=1350 mm
- Y co-ordinate: Y(6)=1700 mm
- X co-ordinate: X(7)=850 mm
- Y co-ordinate: Y(7)=1700 mm
- X co-ordinate: X(8)=850 mm
Y co-ordinate         Y(8)=1600 mm
X co-ordinate         X(9)=0 mm
Y co-ordinate         Y(9)=1600 mm
X co-ordinate         X(10)=0 mm
Y co-ordinate         Y(10)=1375 mm
X co-ordinate         X(11)=275 mm
Y co-ordinate         Y(11)=1375 mm
X co-ordinate         X(12)=275 mm
Y co-ordinate         Y(12)=1350 mm
X co-ordinate         X(13)=235 mm
Y co-ordinate         Y(13)=1350 mm
X co-ordinate         X(14)=375 mm
Y co-ordinate         Y(14)=379 mm
X co-ordinate         X(15)=100 mm
Y co-ordinate         Y(15)=202 mm
X co-ordinate         X(16)=100 mm
Y co-ordinate         Y(16)=0 mm

**Item No. 2**

Second moment of area         IXX2=118.99E9 mm
Area of item                    A2=585580 mm²
Centroid from X-X axis         YC2=637.8 mm
Item weighting                  iw2=0.10145
Depth of composite section      di=1700 mm
Initial force                   if=200 kN
Modulus of elasticity          Es=200 kN/mm²

**Cross-section 1**

Distance from mid-span of beam  csd(1)=9.75 m
Number of layers of strand      nls(1)=4
Height above soffit            ds(1)=60 mm
Number of strands               ns(1)=10
Height above soffit            ds(2)=110 mm
Number of strands               ns(2)=12
Height above soffit            ds(3)=210 mm
Number of strands               ns(3)=8
Height above soffit            ds(4)=1200 mm
Number of strands               ns(4)=2

**Short Term Losses**

Relaxation at transfer ( % )    str=1.5
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>96</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>732.67</td>
<td>11.448</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>828.67</td>
<td>12.948</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 6400 kN
Force at transfer $P_t = P_0 - \delta = 5571.3$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-6.5562</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.6289</td>
</tr>
<tr>
<td>Total</td>
<td>-0.92729</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
The flexural tensile stress of 0.92729 N/mm² at transfer is less than the permissible value of 1 N/mm². Clause 6.3.2.4(b)(1).

Bottom of beam:
Compressive stress 18.252 N/mm² is less than allowable stresses of $0.5 \times f_{ci} = 20$ N/mm² and $0.4 \times f_{cu} = 21$ N/mm².

Long Term Losses

Relaxation after transfer ( % ) $slr(1) = 1$
Shrinkage per unit length $sul(1) = 0.3 \times 10^{-3}$
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>96</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>732.67</td>
<td>11.448</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>828.67</td>
<td>12.948</td>
</tr>
<tr>
<td>Relaxation</td>
<td>64</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>316.8</td>
<td>4.95</td>
</tr>
<tr>
<td>Creep</td>
<td>941.55</td>
<td>14.712</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>1322.3</td>
<td>20.662</td>
</tr>
<tr>
<td>All Losses</td>
<td>2151</td>
<td>33.61</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force: 6400 kN
Final force: Pf=P0-loftl=4249 kN
Unit-weight of in-situ concrete: wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-5.0001</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.6289</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>7.0284</td>
</tr>
<tr>
<td>Total</td>
<td>7.6572</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete: hsis=202 mm
Percentage of shrinkage remaining: ps=45 %
Percentage: pcr=45
Percentage: stT=60 %
The net moment is: Mc=F*e/1E3-Mb=-758.35 kNm

Stress at top of in-situ concrete:
\[ f_{tisds} = \left( \frac{F}{A_i m r} - F/(A_i m r + A_c) - M_c 1E3 \times (d_i - Y_c)/I_c \right) \times m r \times 1E3\]
\[ = -0.10982 \text{ N/mm}^2 \]

Stress at bottom of in-situ concrete:
\[ f_{bisds} = -\left( \frac{F}{A_i m r} - F/(A_i m r + A_c) - M_c 1E3 \times (h_{sis} - Y_c)/I_c \right) \times m r \times 1E3\]
\[ = 3.1235 \text{ N/mm}^2 \]

Stress at top of precast concrete beam:
ftpbds=(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3
=-2.2853 N/mm²
Stress at bottom of precast concrete beam:

\[ fbpbds=(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3=-0.50847 \text{ N/mm}² \]

### Combination 1 loading

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>7.6572</td>
<td>0</td>
<td>6.9203</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-0.10982</td>
<td>-2.2853</td>
<td>3.1235</td>
<td>-0.50847</td>
</tr>
<tr>
<td></td>
<td>1.993</td>
<td>1.3628</td>
<td>-1.5369</td>
<td>-2.1903</td>
</tr>
<tr>
<td>Total</td>
<td>1.8832</td>
<td>6.7347</td>
<td>1.5866</td>
<td>4.2216</td>
</tr>
</tbody>
</table>

### Combination 2 to 5 loading

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>7.6572</td>
<td>0</td>
<td>6.9203</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2-5</td>
<td>-0.10982</td>
<td>-2.2853</td>
<td>3.1235</td>
<td>-0.50847</td>
</tr>
<tr>
<td></td>
<td>2.4359</td>
<td>1.6656</td>
<td>-2.2541</td>
<td>-3.2124</td>
</tr>
<tr>
<td>Total</td>
<td>2.3261</td>
<td>7.0375</td>
<td>0.86942</td>
<td>3.1995</td>
</tr>
</tbody>
</table>

### Combination 3 loading

Top of in-situ concrete:
Compressive stress 2.3261 N/mm² is less than the maximum allowable stress of 0.5*fcuc=20 N/mm².
Top of beam:
Compressive stress 7.0375 N/mm² is less than the maximum allowable stress of 0.4*fcu=21 N/mm².
Bottom of in-situ concrete:
Compressive stress 0.86942 N/mm² is less than the maximum allowable stress of 0.5*fcuc=20 N/mm².
Bottom of beam:
Compressive stress 3.1995 N/mm² is less than the maximum allowable stress of 0.4*fcu=21 N/mm².
Location: Ex3 – 15 metre span Y1 beam

Precast prestressed concrete Y beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=15 m

Beam Y 1 section properties

- Depth of beam d=700 mm
- Cross-sectional area Ac=310140 mm²
- Centroid above soffit Yb=255 mm
- Section modulus top Zt=24.88E6 mm³
- Section modulus bottom Zb=43.42E6 mm³
- Unit weight of beam uwb=24.5 kN/m³

Concrete strengths

- Strength at transfer fci=37.5 N/mm²
- Characteristic strength fcu=45 N/mm²

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 37.5 N/mm² is: Eci=30.25 kN/mm²
- Modulus of elasticity for concrete strength 45 N/mm² is: Ecb=32.5 kN/mm²

Composite section properties

- Char strength of in-situ concrete fcuc=30 N/mm²
- Modulus of elasticity for concrete strength 30 N/mm² is: Ecc=28 kN/mm²
Transf'med second moment of area:
Depth of composite section \( di = 810 \text{ mm} \)
Nominal diameter \( \text{nomd} = 12 \text{ mm} \)
Nominal tensile strength \( \text{nomt} = 1600 \text{ N/mm}^2 \)
Nominal steel area \( \text{noma} = 90 \text{ mm}^2 \)
Characteristic breaking load \( \text{cbl} = 160 \text{ kN} \)
Initial force per strand \( \text{if} = 100 \text{ kN} \)
Modulus of elasticity \( \text{Es} = 200 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( \text{csd}(1) = 2.4 \text{ m} \)
Number of layers of strand \( \text{nls}(1) = 3 \)
Height above soffit \( \text{ds}(1) = 50 \text{ mm} \)
Number of strands \( \text{ns}(1) = 8 \)

**WARNING:**
**Strand location is non-standard.**

Height above soffit \( \text{ds}(2) = 100 \text{ mm} \)
Number of strands \( \text{ns}(2) = 12 \)

**WARNING:**
**Strand location is non-standard.**
Height above soffit \( ds(3) = 250 \text{ mm} \)
Number of strands \( ns(3) = 6 \)

**WARNING:**

Strand location is non-standard.

Bending moment from temp.loads \( tlb(1) = 166 \text{ kNm} \)
Bending moment from other loads \( olb(1) = 191 \text{ kNm} \)

**Short Term Losses**

Relaxation at transfer ( % ) \( str = 1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>26</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>194.7</td>
<td>7.4885</td>
</tr>
<tr>
<td><strong>Transfer Total</strong></td>
<td>220.7</td>
<td>8.4885</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 2600 kN
Force at transfer \( Pt = P0 - loft = 2379.3 \text{ kN} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-5.3115</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.7096</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>6.6717</td>
</tr>
<tr>
<td>Other Loads</td>
<td>7.6765</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>16.746</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 16.746 N/mm\(^2\) is less than allowable stresses of 0.5*\( fci = 18.75 \text{ N/mm}^2 \) and 0.4*\( fcu = 18 \text{ N/mm}^2 \).

Bottom of beam:
Compressive stress 2.4716 N/mm\(^2\) is less than allowable stresses of 0.5*\( fci = 18.75 \text{ N/mm}^2 \) and 0.4*\( fcu = 18 \text{ N/mm}^2 \).
Long Term Losses

Relaxation after transfer (\(\%\)) \(slr(1)=0.8\)
Shrinkage per unit length \(sul(1)=0.3E-3\)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>26</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>194.7</td>
<td>7.4885</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>220.7</td>
<td>8.4885</td>
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<tr>
<td>Relaxation</td>
<td>20.8</td>
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<tr>
<td>Shrinkage</td>
<td>140.4</td>
<td>5.4</td>
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<tr>
<td>Creep</td>
<td>166.26</td>
<td>6.3946</td>
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<tr>
<td>Long-term Total</td>
<td>327.46</td>
<td>12.595</td>
</tr>
<tr>
<td>All Losses</td>
<td>548.16</td>
<td>21.083</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2600 kN
Final force \(P_f=P_0-loftl=2051.8\) kN
Unit-weight of in-situ concrete \(w_c=24\) kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.5805</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.7096</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>4.373</td>
</tr>
<tr>
<td>Total</td>
<td>7.502</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete \(h_{sis}=660\) mm
Percentage of shrinkage remaining \(p_s=50\) %
Percentage \(p_c=50\)
Percentage \(s_{T}=45\) %
The net moment is \(M_c=F*E/1E3-M_b=-156.01\) kNm
Stress at top of in-situ concrete:
\[
ft_{lds}=\left(\frac{F}{(A_i*mr)}-\frac{F}{(A_i*mr+Ac)}-Mc*1E3*(di-Yc)/Ic\right)*mr*1E3
\]
\[=0.28208\] N/mm²
Stress at bottom of in-situ concrete:
fbisds = \left(\frac{-F}{(Ai*mr) - F/(Ai*mr+Ac)} - \frac{Mc*1E3*(hsis-Yc)/Ic)*mr*1E3}{1E3}\right)
\equiv 0.85199 \text{ N/mm}^2

Stress at top of precast concrete beam:
ftpbd = \left(\frac{F}{(Ai*mr+Ac)} + \frac{Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3}{1E3}\right)
\equiv -2.2156 \text{ N/mm}^2

Stress at bottom of precast concrete beam:
fbpdb = \left(\frac{F}{(Ai*mr+Ac)} - \frac{Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3}{1E3}\right)
\equiv 0.76152 \text{ N/mm}^2

**Combination 1 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
<td>7.502</td>
<td>0</td>
<td>6.108</td>
</tr>
<tr>
<td>Combination 1</td>
<td>0.28208</td>
<td>-2.2156</td>
<td>0.85199</td>
<td>0.76152</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>5.7636</td>
<td>4.8243</td>
<td>3.5718</td>
<td>-4.6988</td>
</tr>
<tr>
<td>Total</td>
<td>6.0457</td>
<td>10.111</td>
<td>4.4238</td>
<td>2.1707</td>
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**Combination 1 Loading - Stresses**

**Combination 2 to 5 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
<td>7.502</td>
<td>0</td>
<td>6.108</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>0.28208</td>
<td>-2.2156</td>
<td>0.85199</td>
<td>0.76152</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>6.7242</td>
<td>5.6283</td>
<td>4.1671</td>
<td>-8.223</td>
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<tr>
<td>Total</td>
<td>7.0063</td>
<td>10.915</td>
<td>5.0191</td>
<td>-1.3534</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 Loading - Stresses**

**Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.**

Top of in-situ concrete:
Compressive stress 7.0063 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².

Top of beam:
Compressive stress 10.915 N/mm² is less than the maximum allowable stress of 0.4*fcu=18 N/mm².
Bottom of in-situ concrete:
Compressive stress 5.0191 N/mm² is less than the maximum allowable stress of 0.5\( \times \)fcuc=15 N/mm².

Bottom of beam:
Tensile stress 1.3534 N/mm² is less than the maximum allowable tensile stress of 3.0187 N/mm².
Location: Ex1 - 20 metre span Y beam

Precast prestressed concrete Y beams with in-situ top slab


Beam span under consideration span=20 m

Beam Y 4 section properties

- Depth of beam: d=1000 mm
- Cross-sectional area: Ac=410880 mm²
- Centroid above soffit: Yb=399.3 mm
- Section modulus top: Zt=63.63E6 mm³
- Section modulus bottom: Zb=95.72E6 mm³
- Unit weight of beam: uwb=24 kN/m³

Concrete compressive strengths

- Characteristic cylinder strength: fcki=32 N/mm² (transfer)
- Characteristic cylinder strength: fck=40 N/mm²

Concrete moduli (short-term)

- Concrete modulus of elasticity: Ecmi=33 kN/mm² (transfer)
- Concrete modulus of elasticity: Ecmb=35 kN/mm²

Composite section properties

- Char strength (in-situ concrete): fckc=33 N/mm²
- Short-term modulus of elasticity for in-situ concrete: Ecmc=34 kN/mm²
Sample output for SCALE Proforma 143. (ans=1)

Reinforced & prestressed concrete: BS5400 & Eurocode

Made by: IFB

Date: 02/12/19

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Reinforced & prestressed concrete: BS5400 & Eurocode

Made by: IFB

Date: 02/12/19

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Y beam

Reinforced & prestressed concrete: BS5400 & Eurocode

Made by: IFB

Date: 02/12/19

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ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ

1000

³uling

160

15

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1000

³uling

160

15

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Second moment of area:

Depth of composite section di=1125 mm

Modulus of elasticity (strand) Ep=195 kN/mm²

Cross-section 1

Distance from mid-span of beam csd(1)=0 m

Number of layers of strand nls(1)=4

Height above soffit ds(1)=60 mm

Number of strands ns(1)=11

Height above soffit ds(2)=110 mm

Number of strands ns(2)=14

Height above soffit ds(3)=800 mm

Number of strands ns(3)=2

Height above soffit ds(4)=900 mm

Number of strands ns(4)=2

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % ) str=1

Loss of force (expression 5.46) lfst=lfst'/(1+comp1*comp2)=389.1 kN

Force after losses P2=P1-lfst=4606.4 kN

Elastic deform. losses ( % ) ste=lfst*100/P0=7.7111 %

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>389.1</td>
<td>7.7111</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>439.56</td>
<td>8.7111</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 5046 kN

Force at transfer Pt=P0-lfst=4606.4 kN
### Beam Concrete Stresses at Transfer

#### Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

**Top of beam:**
Compressive stress 4.0322 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Bottom of beam:**
Compressive stress 15.983 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

#### Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

**Relaxation after transfer (%)**  
\[ slr(1) = 2 \]

**Shrinkage per unit length**  
\[ sul(1) = 0.3E-3 \]

**Creep coefficient \( \varphi(=\text{to}) \)**  
\[ crf = 2 \]

#### Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>389.1</td>
<td>7.7111</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>439.56</td>
<td>8.7111</td>
</tr>
<tr>
<td>Relaxation</td>
<td>100.92</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>235.81</td>
<td>4.6733</td>
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<tr>
<td>Creep</td>
<td>661.57</td>
<td>13.111</td>
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<tr>
<td>Long-term Total</td>
<td>998.31</td>
<td>19.784</td>
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<tr>
<td>All Losses</td>
<td>1437.9</td>
<td>28.495</td>
</tr>
</tbody>
</table>

**Losses - Long term**

- **Initial force**: 5046 kN
- **Final force**: \( Pf = P_0 - \text{loftl} = 3608.1 \) kN
- **Unit-weight of in-situ concrete**: \( wc = 24 \) kN/m³
Reinforced & prestressed concrete: BS5400 & Eurocode

Made by: IFB
Date: 02/12/19

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Ref No: SC143 EC

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
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<tr>
<td>Prestress</td>
<td>-2.9114</td>
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<tr>
<td>Self weight of Beam</td>
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<tr>
<td>In-situ concrete</td>
<td>2.8294</td>
</tr>
<tr>
<td>Total</td>
<td>7.6672</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete: hsis=965 mm
Perc’age of shrinkage remaining: ps=25 %
Percentage: pcr=25
Percentage: stT=67 %
The net moment is: Mc=F*e/1E3-Mb=-129.72 kNm

Stress at top of in-situ concrete:
\[ ftisds = \frac{-F}{(Ai*mr) - F/(Ai*mr+Ac) - Mc*1E3*(di-Yc)/Ic)*mr*1E3 } \]
\[ = -0.17953 \text{ N/mm}^2 \]

Stress at bottom of in-situ concrete:
\[ fbisds = \frac{-F}{(Ai*mr) - F/(Ai*mr+Ac) - Mc*1E3*(hsis-Yc)/Ic)*mr*1E3 } \]
\[ = 0.064842 \text{ N/mm}^2 \]

Stress at top of precast concrete beam:
\[ ftpbsd = \frac{(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)*1E3 } \]
\[ = 0.12802 \text{ N/mm}^2 \]

Stress at bottom of precast concrete beam:
\[ fbpbd = \frac{(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)*1E3 } \]
\[ = -0.046384 \text{ N/mm}^2 \]

Loading Combination 1

(no tensile stress permitted in precast beam)
Characteristic bending moment: clb(1)=1400 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-0.17953</td>
</tr>
<tr>
<td>Total</td>
<td>11.517</td>
</tr>
</tbody>
</table>

Loading combination 1 Loading - Stresses
Loading Combination 2

(tensile stress permitted (no visible cracking))
Characteristic bending moment  \( c_{2b(1)} = 1850 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage Combination 2</td>
<td>-0.17953</td>
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<tr>
<td></td>
<td>12.021</td>
</tr>
<tr>
<td>Total</td>
<td>11.842</td>
</tr>
</tbody>
</table>

Loading combination 2 - Stresses

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 11.842 N/mm² is less than the maximum allowable stress of 0.6*fckc=19.8 N/mm².

Top of beam:
Compressive stress 17.094 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

Bottom of in-situ concrete:
Compressive stress 8.601 N/mm² is less than the maximum allowable stress of 0.6*fckc=19.8 N/mm².

Bottom of beam:
Tensile stress 3.0081 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
Location: Ex2 - 29.5 metre span Y8 beam

Precast prestressed concrete Y beams with in-situ top slab


Beam span under consideration \( \text{span}=29.5 \ m \)

**Beam Y 8 section properties**

- Depth of beam \( d=1400 \ mm \)
- Cross-sectional area \( A_c=585580 \ mm^2 \)
- Centroid above soffit \( Y_b=637.8 \ mm \)
- Section modulus top \( Z_t=156.12E6 \ mm^3 \)
- Section modulus bottom \( Z_b=186.56E6 \ mm^3 \)
- Unit weight of beam \( u_{wb}=24.5 \ kN/m^3 \)

**Concrete compressive strengths**

- Characteristic cylinder strength \( f_{ck} = 32 \ N/mm^2 \) (transfer)
- Characteristic cylinder strength \( f_{ck} = 42 \ N/mm^2 \)

**Concrete moduli (short-term)**

- Concrete modulus of elasticity \( E_{cmi} = 34 \ kN/mm^2 \) (transfer)
- Concrete modulus of elasticity \( E_{cmb} = 36 \ kN/mm^2 \)

**Composite section properties**

- Char strength (in-situ concrete) \( f_{ckc} = 32 \ N/mm^2 \)
- Short-term modulus of elasticity for in-situ concrete \( E_{cmc} = 33 \ kN/mm^2 \)

**Item No. 1**

<table>
<thead>
<tr>
<th>X co-ordinate</th>
<th>Y co-ordinate</th>
</tr>
</thead>
<tbody>
<tr>
<td>X(1)=100 mm</td>
<td>Y(1)=0 mm</td>
</tr>
<tr>
<td>X(2)=850 mm</td>
<td>Y(2)=0 mm</td>
</tr>
<tr>
<td>X(3)=850 mm</td>
<td>Y(3)=202 mm</td>
</tr>
<tr>
<td>X(4)=1100 mm</td>
<td>Y(4)=1300 mm</td>
</tr>
<tr>
<td>X(5)=1350 mm</td>
<td>Y(5)=1300 mm</td>
</tr>
<tr>
<td>X(6)=1350 mm</td>
<td>Y(6)=1700 mm</td>
</tr>
<tr>
<td>X(7)=850 mm</td>
<td>Y(7)=1700 mm</td>
</tr>
<tr>
<td>X(8)=850 mm</td>
<td>Y(8)=1600 mm</td>
</tr>
<tr>
<td>X(9)=0 mm</td>
<td>Y(9)=1600 mm</td>
</tr>
</tbody>
</table>
Reinforced & prestressed concrete: BS5400 & Eurocode

Y beam

Date: 02/12/19

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Ref No: SC143 EC

X co-ordinate                     X(10)=0 mm
Y co-ordinate                     Y(10)=1375 mm
X co-ordinate                     X(11)=275 mm
Y co-ordinate                     Y(11)=1375 mm
X co-ordinate                     X(12)=275 mm
Y co-ordinate                     Y(12)=1350 mm
X co-ordinate                     X(13)=235 mm
Y co-ordinate                     Y(13)=1350 mm
X co-ordinate                     X(14)=375 mm
Y co-ordinate                     Y(14)=379 mm
X co-ordinate                     X(15)=100 mm
Y co-ordinate                     Y(15)=202 mm
X co-ordinate                     X(16)=100 mm
Y co-ordinate                     Y(16)=0 mm

Depth of composite section        di=1700 mm
Area of in-situ concrete          Ai=746403 mm²
Centroid of in-situ concrete      Yi=1170.1 mm
Modulus of elasticity (strand)    Ep=195 kN/mm²

Cross-section 1

Distance from mid-span of beam    csd(1)=9.75 m
Number of layers of strand        nls(1)=4
Height above soffit               ds(1)=60 mm
Number of strands                 ns(1)=10
Height above soffit               ds(2)=110 mm
Number of strands                 ns(2)=12
Height above soffit               ds(3)=210 mm
Number of strands                 ns(3)=8
Height above soffit               ds(4)=1200 mm
Number of strands                 ns(4)=2

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % )      str=1.5
Loss of force (expression 5.46)   lfst=lfst'/(1+comp1*comp2)=590.33 kN
Force after losses                P2=P1-lfst=5713.7 kN
Elastic deform. losses ( % )      ste=lfst*100/P0=9.2239 %

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>96</td>
<td>1.5</td>
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<tr>
<td>Elastic Shortening</td>
<td>590.33</td>
<td>9.2239</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>686.33</td>
<td>10.724</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 6400 kN
Force at transfer Pt=P0-loft=5713.7 kN
### Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

**Top of beam:**

**WARNING:**
The flexural tensile stress 1.0948 N/mm² at transfer in the concrete due to prestress and co-existent dead and temporary loads during erection should not exceed 1 N/mm² (as was permitted in BS5400-4).

**Bottom of beam:**
Compressive stress 18.838 N/mm² is less than allowable stresses of $0.7*f_{cki}=22.4$ N/mm² and $0.6*f_{ck}=25.2$ N/mm².

### Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer( % ) $slr(1)=1$
Shrinkage per unit length $sul(1)=0.3E-3$
Creep coefficient $\varphi(\infty, to)$ $crf=2$

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force (kN)</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>96</td>
<td>1.5</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>590.33</td>
<td>9.2239</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>686.33</td>
<td>10.724</td>
</tr>
<tr>
<td>Relaxation</td>
<td>64</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>308.88</td>
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</tr>
<tr>
<td>Creep</td>
<td>1138.7</td>
<td>17.793</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>1511.6</td>
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</tr>
<tr>
<td>All Losses</td>
<td>2197.9</td>
<td>34.343</td>
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</tbody>
</table>

### Losses - Long term

Initial force 6400 kN
Final force \( Pf = P_0 \cdot \text{loft} = 4202.1 \ \text{kN} \)

Unit-weight of in-situ concrete \( wc = 24 \ \text{kN/m}^3 \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.9449</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.6289</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>7.0284</td>
</tr>
<tr>
<td>Total</td>
<td>7.7124</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

**Composite section**

- Height from beam soffit to bottom of in-situ concrete: \( hs_{is} = 202 \ \text{mm} \)
- Percentage of shrinkage remaining: \( ps = 45 \% \)
- Percentage: \( pr_c = 45 \% \)
- Percentage: \( st_T = 60 \% \)
- The net moment is: \( Mc = F \cdot e / 1E3 - Mb = -1024.7 \ \text{kNm} \)

Stress at top of in-situ concrete:
\[
ft_{isds} = \left( \frac{F}{Ai \cdot mr} - \frac{F}{Ai \cdot mr + Ac} - Mc \cdot 1E3 \cdot (d_Y - Yc) / Ic \right) \cdot mr \cdot 1E3
\]
\[
= -0.23412 \ \text{N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
fb_{isds} = \left( \frac{F}{Ai \cdot mr} - \frac{F}{Ai \cdot mr + Ac} - Mc \cdot 1E3 \cdot (hs_{is} - Yc) / Ic \right) \cdot mr \cdot 1E3
\]
\[
= 4.3851 \ \text{N/mm}^2
\]

Stress at top of precast concrete beam:
\[
ft_{pbds} = \left( \frac{F}{Ai \cdot mr + Ac} + Mc \cdot 1E3 \cdot (d_Y - Yc) / Ic + Mb \cdot 1E3 \cdot Yt / Iyy \right) \cdot 1E3
\]
\[
= -2.9706 \ \text{N/mm}^2
\]

Stress at bottom of precast concrete beam:
\[
fb_{pbds} = \left( \frac{F}{Ai \cdot mr + Ac} - Mc \cdot 1E3 \cdot Yc / Ic - Mb \cdot 1E3 \cdot Yb / Iyy \right) \cdot 1E3
\]
\[
= -0.79191 \ \text{N/mm}^2
\]

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment \( clb(1) = 750 \ \text{kNm} \)
### Loading Combination 1 Stresses N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>7.7124</td>
<td>0</td>
<td>6.727</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.23412</td>
<td>-2.9706</td>
<td>4.3851</td>
<td>-0.79191</td>
</tr>
<tr>
<td>Combination 1</td>
<td>2.069</td>
<td>1.2565</td>
<td>-1.6567</td>
<td>-2.1126</td>
</tr>
<tr>
<td>Total</td>
<td>1.8349</td>
<td>5.9983</td>
<td>2.7283</td>
<td>3.8224</td>
</tr>
</tbody>
</table>

### Loading Combination 2 - Stresses

(tensile stress permitted (no visible cracking))

**Characteristic bending moment** \( c2b(1)=1100 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>7.7124</td>
<td>0</td>
<td>6.727</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.23412</td>
<td>-2.9706</td>
<td>4.3851</td>
<td>-0.79191</td>
</tr>
<tr>
<td>Combination 2</td>
<td>2.5287</td>
<td>1.5357</td>
<td>-2.4299</td>
<td>-3.0986</td>
</tr>
<tr>
<td>Total</td>
<td>2.2946</td>
<td>6.2776</td>
<td>1.9552</td>
<td>2.8365</td>
</tr>
</tbody>
</table>

**NOTE:** The stresses from temperature differences should be added to the total stresses.

**Top of in-situ concrete:**
Compressive stress 2.2946 N/mm² is less than the maximum allowable stress of 0.6\(*fckc=19.2\) N/mm².

**Top of beam:**
Compressive stress 6.2776 N/mm² is less than the maximum allowable stress of 0.6\(*fck=25.2\) N/mm².

**Bottom of in-situ concrete:**
Compressive stress 1.9552 N/mm² is less than the maximum allowable stress of 0.6\(*fckc=19.2\) N/mm².

**Bottom of beam:**
Compressive stress 2.8365 N/mm² is less than the maximum allowable stress of 0.6\(*fck=25.2\) N/mm².
Location: Ex3 - 15 metre span Y1 beam

Precast prestressed concrete Y beams with in-situ top slab


Beam span under consideration span=15 m

Beam Y 1 section properties

- Depth of beam d=700 mm
- Cross-sectional area Ac=310140 mm²
- Centroid above soffit Yb=255 mm
- Section modulus top Zt=24.88E6 mm⁴
- Section modulus bottom Zb=43.42E6 mm⁴
- Unit weight of beam uwb=24.5 kN/m³

Concrete compressive strengths

- Characteristic cylinder strength fck=35 N/mm² (transfer)
- Characteristic cylinder strength fcki=30 N/mm² (transfer)

Concrete moduli (short-term)

- Concrete modulus of elasticity Ecmb=34 kN/mm² (transfer)
- Concrete modulus of elasticity Ecmi=33 kN/mm² (transfer)

Composite section properties

- Char strength (in-situ concrete) fckc=25 N/mm²
- Short-term modulus of elasticity for in-situ concrete Ecmc=31 kN/mm²
Second moment of area:
Depth of composite section        $d_i = 810$ mm
Nominal diameter                  $n_{omd} = 12$ mm
Nominal tensile strength          $n_{omt} = 1600$ N/mm²
Nominal steel area                $n_{oma} = 90$ mm²
Characteristic breaking load      $f_{pk} = 160$ kN
Initial force per strand          $i_{pf} = 100$ kN
Modulus of elasticity (strand)    $E_p = 195$ kN/mm²

**Cross-section 1**

Distance from mid-span of beam    $c_{sd}(1) = 2.4$ m
Number of layers of strand        $n_{ls}(1) = 3$
Height above soffit               $d_s(1) = 50$ mm
Number of strands                 $n_s(1) = 8$

**WARNING:**
Strand location is non-standard.

Height above soffit               $d_s(2) = 100$ mm
Number of strands                 $n_s(2) = 12$

**WARNING:**
Strand location is non-standard.

Height above soffit               $d_s(3) = 250$ mm
Number of strands                 $n_s(3) = 6$

**WARNING:**
Strand location is non-standard.

Bending moment from temp.loads    $t_{lb}(1) = 166$ kNm
Bending moment from other loads   $o_{lb}(1) = 191$ kNm

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer ( % )      $s_{tr} = 1$
Loss of force (expression 5.46)   $l_{fst} = l_{fst}' / (1 + \text{compl} \times \text{comp}^2) = 162.99$ kN
Force after losses                $P_2 = P_1 - l_{fst} = 2411$ kN
Elastic deform. losses ( % )      $s_{te} = l_{fst} \times 100 / P_0 = 6.269$ %
## Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>26</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>162.99</td>
<td>6.269</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>188.99</td>
<td>7.269</td>
</tr>
</tbody>
</table>

### Losses at Transfer

Initial force: 2600 kN  
Force at transfer: $P_t = P_0 - \text{loft} = 2411$ kN

### Stresses at Transfer N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-5.3822</td>
<td>15.313</td>
<td>11.788</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.7096</td>
<td>-4.4178</td>
<td>-2.3522</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>6.6717</td>
<td>-3.8231</td>
<td>-2.0355</td>
</tr>
<tr>
<td>Other Loads</td>
<td>7.6765</td>
<td>-4.3989</td>
<td>-2.3421</td>
</tr>
<tr>
<td>Total</td>
<td>16.676</td>
<td>2.673</td>
<td>5.0581</td>
</tr>
</tbody>
</table>

### Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

**Top of beam:**
Compressive stress 16.676 N/mm² is less than allowable stresses of $0.7f_{cki}=21$ N/mm² and $0.6f_{ck}=21$ N/mm².

**Bottom of beam:**
Compressive stress 2.673 N/mm² is less than allowable stresses of $0.7f_{cki}=21$ N/mm² and $0.6f_{ck}=21$ N/mm².

### Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer (%): $\text{slr}(1)=0.8$  
Shrinkage per unit length: $\text{sul}(1)=0.3E-3$  
Creep coefficient $\phi(\infty, to)$: crf=2
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>26</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>162.99</td>
<td>6.269</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>188.99</td>
<td>7.269</td>
</tr>
<tr>
<td>Relaxation</td>
<td>20.8</td>
<td>0.8</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>136.89</td>
<td>5.265</td>
</tr>
<tr>
<td>Creep</td>
<td>190.4</td>
<td>7.3231</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>348.09</td>
<td>13.388</td>
</tr>
<tr>
<td>All Losses</td>
<td>537.08</td>
<td>20.657</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2600 kN
Final force $P_f=P_0-lofl=2062.9$ kN
Unit-weight of in-situ concrete $w_c=24$ kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.6052</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.7096</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>4.373</td>
</tr>
<tr>
<td>Total</td>
<td>7.4773</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete $h_{sis}=660$ mm
Percentage of shrinkage remaining $p_s=50$ %
Percentage $p_{cr}=50$
Percentage $s_{T}=45$ %
The net moment is $M_c=F*e/1E3-M_b=-209.35$ kNm

Stress at top of in-situ concrete:
\[
ft_{isds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3 =0.27142 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
fb_{isds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3 =1.1007 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:
ftpbds = (F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy) *1E3
-2.6593 N/mm²

Stress at bottom of precast concrete beam:

fbpbds = (F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy) *1E3
=0.91119 N/mm²

### Loading Combination 1

(no tensile stress permitted in precast beam)

Characteristic bending moment  \( c_{lb(1)} = 400 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage</td>
<td>0.27142</td>
</tr>
<tr>
<td>Combination 1</td>
<td>5.991</td>
</tr>
<tr>
<td>Total</td>
<td>6.2625</td>
</tr>
</tbody>
</table>

### Loading combination 1 Loading - Stresses

### Loading Combination 2

(tensile stress permitted (no visible cracking))

Characteristic bending moment  \( c_{lb(2)} = 700 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Diff. shrinkage</td>
<td>0.27142</td>
</tr>
<tr>
<td>Combination 2</td>
<td>6.9895</td>
</tr>
<tr>
<td>Total</td>
<td>7.261</td>
</tr>
</tbody>
</table>

### Loading combination 2 - Stresses

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 7.261 N/mm² is less than the maximum allowable stress of 0.6*fckc=15 N/mm².

Top of beam:
Compressive stress 9.7741 N/mm² is less than the maximum allowable stress of 0.6*fck=21 N/mm².

Bottom of in-situ concrete:
Compressive stress 5.3173 N/mm² is less than the maximum allowable stress of 0.6*fckc=15 N/mm².
Bottom of beam:
Tensile stress 0.89473 N/mm² is less than the maximum allowable tensile stress of 2.9399 N/mm².
Location: Ex1 - 20 metre span U beam

Precast prestressed concrete U beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=20 m

**Beam U 7 section properties**

- Depth of beam d=1100 mm
- Cross-sectional area Ac=577217 mm²
- Centroid above soffit Yb=491.7 mm
- Section modulus top Zt=121.93E6 mm⁸
- Section modulus bottom Zb=150.83E6 mm⁸
- Unit weight of beam uwb=24 kN/m³

**Concrete strengths**

- Strength at transfer fci=40 N/mm²
- Characteristic strength fcu=50 N/mm²

**Concrete moduli**

- Modulus of elasticity at transfer for concrete strength 40 N/mm² is: Eci=31 kN/mm²
- Modulus of elasticity for concrete strength 50 N/mm² is: Ecb=34 kN/mm²

**Composite section properties**

- Char strength (in-situ concrete) fcuc=40 N/mm²
- Modulus of elasticity for concrete strength 40 N/mm² is: Ecc=31 kN/mm²
Second moment of area:
Depth of composite section \( d_i = 1245 \text{ mm} \)
Initial force per strand \( i_f = 174 \text{ kN} \)
Modulus of elasticity \( E_s = 200 \text{ kN/mm}^2 \)

Cross-section 1

Distance from mid-span of beam \( c_{sd}(1) = 0 \text{ m} \)
Number of layers of strand \( n_{ls}(1) = 4 \)
Height above soffit \( d_s(1) = 60 \text{ mm} \)
Number of strands \( n_s(1) = 14 \)
Height above soffit \( d_s(2) = 110 \text{ mm} \)
Number of strands \( n_s(2) = 12 \)
Height above soffit \( d_s(3) = 700 \text{ mm} \)
Number of strands \( n_s(3) = 4 \)
Height above soffit \( d_s(4) = 1000 \text{ mm} \)
Number of strands \( n_s(4) = 4 \)

Short Term Losses

Relaxation at transfer ( % ) \( str = 1 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>59.16</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>434.74</td>
<td>7.3485</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>493.9</td>
<td>8.3485</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \( 5916 \text{ kN} \)
Force at transfer \( P_t = P_0 - loft = 5422.1 \text{ kN} \)
### Beam Concrete Stresses at Transfer

#### Stress Limitations

**Top of beam:**
Compressive stress $4.9274 \text{ N/mm}^2$ is less than allowable stresses of $0.5 \times f_{ci}=20 \text{ N/mm}^2$ and $0.4 \times f_{cu}=20 \text{ N/mm}^2$.

**Bottom of beam:**
Compressive stress $13.004 \text{ N/mm}^2$ is less than allowable stresses of $0.5 \times f_{ci}=20 \text{ N/mm}^2$ and $0.4 \times f_{cu}=20 \text{ N/mm}^2$.

#### Long Term Losses

Relaxation after transfer( % ) $slr(1)=2$
Shrinkage per unit length $sul(1)=0.3E-3$

#### Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>59.16</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>434.74</td>
<td>7.3485</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>493.9</td>
<td>8.3485</td>
</tr>
<tr>
<td>Relaxation</td>
<td>118.32</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>283.56</td>
<td>4.7931</td>
</tr>
<tr>
<td>Creep</td>
<td>502.18</td>
<td>8.4886</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>904.06</td>
<td>15.282</td>
</tr>
<tr>
<td>All Losses</td>
<td>1398</td>
<td>23.63</td>
</tr>
</tbody>
</table>

**Losses - Long term**

- **Initial force** $5916 \text{ kN}$
- **Final force** $P_f=P_0-loftl=4518 \text{ kN}$
- **Unit-weight of in-situ concrete** $w_c=24 \text{ kN/m}^3$
Reinforced & prestressed concrete: BS5400 & Eurocode

Made by: IFB

Date: 02/12/19

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### Composite section

Height from beam soffit to bottom of in-situ concrete: \( h_{sis} = 1065 \text{ mm} \)

Per cent of shrinkage remaining in cement paste: \( p_s = 25 \% \)

Percentage of shrinkage remaining in concrete: \( p_c = 25 \% \)

Percentage of shrinkage remaining in steel: \( s_T = 67 \% \)

The net moment is: \( M_c = F*e/1E3 - M_b = -7.8846 \text{ kNm} \)

Stress at top of in-situ concrete:

\[
\text{ftisds} = -\left( \frac{F}{(A_i \cdot mr)} - \frac{F}{(A_i \cdot mr + A_c)} - M_c \cdot 1E3 \cdot (d - Y_c)/I_c \right) \cdot mr \cdot 1E3
\]

Stress at bottom of in-situ concrete:

\[
\text{fbisds} = -\left( \frac{F}{(A_i \cdot mr)} - \frac{F}{(A_i \cdot mr + A_c)} - M_c \cdot 1E3 \cdot h_{sis} - Y_c)/I_c \right) \cdot mr \cdot 1E3
\]

Stress at top of precast concrete beam:

\[
\text{ftpbsd} = \left( \frac{F}{(A_i \cdot mr + A_c)} + M_c \cdot 1E3 \cdot (d - Y_c)/I_c + M_b \cdot 1E3 \cdot Y_t/I_{xx} \right) \cdot 1E3
\]

Stress at bottom of precast concrete beam:

\[
\text{fbpdbd} = \left( \frac{F}{(A_i \cdot mr + A_c)} - M_c \cdot 1E3 \cdot Y_c/I_c - M_b \cdot 1E3 \cdot Y_b/I_{xx} \right) \cdot 1E3
\]

---

### Combination 1 loading

Applied bending moment: \( c_{lb(1)} = 1440 \text{ kNm} \)

---

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.26514</td>
</tr>
<tr>
<td>Combination 1</td>
<td>7.715</td>
</tr>
<tr>
<td>Total</td>
<td>7.4498</td>
</tr>
</tbody>
</table>

---

Combination 1 Loading - Stresses
**Combination 2 to 5 loading**

Applied bending moment \(c2b(1)=1907 \text{ kNm}\)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>7.3459</td>
<td>0</td>
<td>8.2164</td>
</tr>
<tr>
<td>Diff. shrinkage Combination 2-5</td>
<td>-0.26514</td>
<td>0.44251</td>
<td>-0.2557</td>
<td>-0.16716</td>
</tr>
<tr>
<td>Total</td>
<td>6.8252</td>
<td>13.04</td>
<td>4.5518</td>
<td>-0.64994</td>
</tr>
</tbody>
</table>

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 6.8252 N/mm² is less than the maximum allowable stress of 0.5*fcuc=20 N/mm².

Top of beam:
Compressive stress 13.04 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².

Bottom of in-situ concrete:
Compressive stress 4.5518 N/mm² is less than the maximum allowable stress of 0.5*fcuc=20 N/mm².

Bottom of beam:
Tensile stress 0.64994 N/mm² is less than the maximum allowable tensile stress of 3.182 N/mm².
Location: Ex2 - 25 metre span U10 beam

Precast prestressed concrete U beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam  span=25 m

Beam U 10 section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=1400 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=677357 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=634.3 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>Zt=187.93E6 mm³</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>Zb=226.9E6 mm³</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=24.5 kN/m³</td>
</tr>
</tbody>
</table>

Concrete strengths

<table>
<thead>
<tr>
<th>Strength</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength at transfer</td>
<td>fci=45 N/mm²</td>
</tr>
<tr>
<td>Characteristic strength</td>
<td>fcu=50 N/mm²</td>
</tr>
</tbody>
</table>

Concrete moduli

<table>
<thead>
<tr>
<th>Modulus of elasticity</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity at transfer for concrete strength 45 N/mm² is:</td>
<td>Eci=32.5 kN/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity for concrete strength 50 N/mm² is:</td>
<td>Ecb=34 kN/mm²</td>
</tr>
</tbody>
</table>

Composite section properties

<table>
<thead>
<tr>
<th>Char strength (in-situ concrete)</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity for concrete strength 35 N/mm² is:</td>
<td>Ecc=29.5 kN/mm²</td>
</tr>
</tbody>
</table>

Item No. 1

<table>
<thead>
<tr>
<th>Item</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Item weighting</td>
<td>iw1=0.86765</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(1)=0 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(1)=1380 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(2)=370 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(2)=1380 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(3)=370 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(3)=1400 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(4)=605 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(4)=1400 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(5)=605 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(5)=1380 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(6)=1246 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(6)=1380 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(7)=1246 mm</td>
</tr>
<tr>
<td>Y co-ordinate</td>
<td>Y(7)=1400 mm</td>
</tr>
<tr>
<td>X co-ordinate</td>
<td>X(8)=1481 mm</td>
</tr>
</tbody>
</table>
Item No. 2

Second moment of area  \( I_X X^2 = 143.92 \times 10^9 \) mm\(^4\)
Area of item  \( A_2 = 677357 \) mm\(^2\)
Centroid above soffit  \( Y_{C2} = 634.3 \) mm
Item weighting  \( i w_2 = 1 \)
Depth of composite section  \( d_i = 1680 \) mm
Initial force per strand  \( i f = 90 \) kN
Modulus of elasticity  \( E_s = 200 \) kN/mm\(^2\)

Cross-section 1

Distance from mid-span of beam  \( c_{sd(1)} = 4.5 \) m
Number of layers of strand  \( n_{ls(1)} = 4 \)
Height above soffit  \( d_s(1) = 60 \) mm
Number of strands  \( n_s(1) = 16 \)
Height above soffit  \( d_s(2) = 110 \) mm
Number of strands  \( n_s(2) = 12 \)
Height above soffit  \( d_s(3) = 800 \) mm
Number of strands  \( n_s(3) = 4 \)
Height above soffit  \( d_s(4) = 1300 \) mm
Number of strands  \( n_s(4) = 4 \)

Short Term Losses

Relaxation at transfer ( % )  \( str = 1.25 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.5</td>
<td>1.25</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>120.59</td>
<td>3.7219</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>161.09</td>
<td>4.9719</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force  \( 3240 \) kN
Force at transfer  \( Pt=P_0-\text{loft}=3078.9 \text{ kN} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.98512</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.0037</td>
</tr>
<tr>
<td>Total</td>
<td>5.0186</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 5.0186 N/mm\(^2\) is less than allowable stresses of 0.5\(\times\)fci=22.5 N/mm\(^2\) and 0.4\(\times\)fcu=20 N/mm\(^2\).

Bottom of beam:
Compressive stress 4.1535 N/mm\(^2\) is less than allowable stresses of 0.5\(\times\)fci=22.5 N/mm\(^2\) and 0.4\(\times\)fcu=20 N/mm\(^2\).

**Long Term Losses**

Relaxation after transfer( % ) \( slr(1)=1 \)
Shrinkage per unit length \( sul(1)=0.3\times10^{-3} \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.5</td>
<td>1.25</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>120.59</td>
<td>3.7219</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>161.09</td>
<td>4.9719</td>
</tr>
<tr>
<td>Relaxation</td>
<td>32.4</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>162</td>
<td>5</td>
</tr>
<tr>
<td>Creep</td>
<td>112.41</td>
<td>3.4695</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>306.81</td>
<td>9.4695</td>
</tr>
<tr>
<td>All Losses</td>
<td>467.9</td>
<td>14.441</td>
</tr>
</tbody>
</table>

**Losses - Long term**

Initial force 3240 kN
Final force \( Pf=P_0-\text{loft}=2772.1 \text{ kN} \)
Unit-weight of in-situ concrete \( wc=23.5 \text{ kN/m}^3 \)
### Loading

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.88695</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.0037</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.7135</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>8.8303</td>
</tr>
</tbody>
</table>

#### Beam concrete stresses - Long term

**Composite section**

- Height from beam soffit to bottom of in-situ concrete: \( h_{sis} = 1380 \text{ mm} \)
- Percentage of shrinkage remaining: \( p_s = 45 \% \)
- Percentage: \( p_{cr} = 45 \)\%
- Percentage: \( s_{T} = 50 \% \)

The net moment is: \( M_{c} = F \times e / 10^3 - M_{b} = 411.5 \text{ kNm} \)

Stress at top of in-situ concrete:
\[
ft_{isds} = -\left(\frac{F}{(A_i \times mr)} - \frac{F}{(A_i \times mr + Ac)} - M_{c} \times 10^3 \times (d_i - Y_c) / I_c\right) \times mr \times 10^3
\]
\[= 1.1307 \text{ N/mm}^2 \]

Stress at bottom of in-situ concrete:
\[
fb_{isds} = -\left(\frac{F}{(A_i \times mr)} - \frac{F}{(A_i \times mr + Ac)} - M_{c} \times 10^3 \times (h_{sis} - Y_c) / I_c\right) \times mr \times 10^3
\]
\[= 0.80381 \text{ N/mm}^2 \]

Stress at top of precast concrete beam:
\[
ft_{pbds} = \left(\frac{F}{(A_i \times mr + Ac)} + M_{c} \times 10^3 \times (d - Y_c) / I_c + M_{b} \times 10^3 \times Y_t / I_{xx}\right) \times 10^3
\]
\[= -2.4914 \text{ N/mm}^2 \]

Stress at bottom of precast concrete beam:
\[
fb_{pbds} = \left(\frac{F}{(A_i \times mr + Ac)} - M_{c} \times 10^3 \times Y_c / I_c - M_{b} \times 10^3 \times Y_b / I_{xx}\right) \times 10^3
\]
\[= 0.96305 \text{ N/mm}^2 \]

**Combination 1 loading**

Applied bending moment: \( c_{1b}(l) = 1000 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ.</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>1.1307</td>
</tr>
<tr>
<td>Combination 1</td>
<td>2.1424</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>3.2731</td>
</tr>
</tbody>
</table>

**Combination 1 Loading - Stresses**
WARNING:

Bottom of beam:

Tensile stress 1.7523 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

Combination 2 to 5 loading

Applied bending moment: \( c2b(1) = 1250 \) kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ Dif. shrinkage</td>
<td>0</td>
<td>8.8303</td>
<td>0</td>
<td>0.16775</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>1.1307</td>
<td>-2.4914</td>
<td>0.80381</td>
<td>0.96305</td>
</tr>
<tr>
<td></td>
<td>2.4345</td>
<td>1.7376</td>
<td>1.4414</td>
<td>-3.6039</td>
</tr>
<tr>
<td>Total</td>
<td>3.5653</td>
<td>8.0766</td>
<td>2.2452</td>
<td>-2.4731</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:

Compressive stress 3.5653 N/mm² is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm².

Top of beam:

Compressive stress 8.0766 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².

Bottom of in-situ concrete:

Compressive stress 2.2452 N/mm² is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm².

Bottom of beam:

Tensile stress 2.4731 N/mm² is less than the maximum allowable tensile stress of 3.182 N/mm².
Location: Ex3 - 15 metre span U3 beam

Precast prestressed concrete U beams with in-situ top slab

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=15 m

Beam U 3 section properties

- Depth of beam $d=900 \text{ mm}$
- Cross-sectional area $A_c=510457 \text{ mm}^2$
- Centroid above soffit $Y_b=398.4 \text{ mm}$
- Section modulus top $Z_t=84.39 \times 10^6 \text{ mm}^3$
- Section modulus bottom $Z_b=106.27 \times 10^6 \text{ mm}^3$
- Unit weight of beam $u_{wb}=25 \text{ kN/m}^3$

Concrete strengths

- Strength at transfer $f_{ci}=35 \text{ N/mm}^2$
- Characteristic strength $f_{cu}=40 \text{ N/mm}^2$

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 35 N/mm$^2$ is: $E_{ci}=29.5 \text{ kN/mm}^2$
- Modulus of elasticity for concrete strength 40 N/mm$^2$ is: $E_{cb}=31 \text{ kN/mm}^2$

Composite section properties

- Char strength (in-situ concrete) $f_{cu,c}=35 \text{ N/mm}^2$
- Modulus of elasticity for concrete strength 35 N/mm$^2$ is: $E_{cc}=29.5 \text{ kN/mm}^2$
Second moment of area:
Depth of composite section  \( d_i = 1085 \text{ mm} \)
Initial force per strand  \( i_f = 150 \text{ kN} \)
Modulus of elasticity  \( E_s = 200 \text{ kN/mm}^2 \)

Cross-section 1

Distance from mid-span of beam  \( c_{sd}(1) = 2.4 \text{ m} \)
Number of layers of strand  \( n_{ls}(1) = 4 \)
Height above soffit  \( d_s(1) = 50 \text{ mm} \)
Number of strands  \( n_s(1) = 10 \)

**WARNING:**
Strand location is non-standard.

Height above soffit  \( d_s(2) = 105 \text{ mm} \)
Number of strands  \( n_s(2) = 10 \)

**WARNING:**
Strand location is non-standard.

Height above soffit  \( d_s(3) = 490 \text{ mm} \)
Number of strands  \( n_s(3) = 4 \)

**WARNING:**
Strand location is non-standard.

Height above soffit  \( d_s(4) = 790 \text{ mm} \)
Number of strands  \( n_s(4) = 4 \)

**WARNING:**
Strand location is non-standard.

Bending moment from temp.loads  \( t_{lb}(1) = 600 \text{ kNm} \)
Bending moment from other loads  \( o_{lb}(1) = 400 \text{ kNm} \)

**Short Term Losses**

Relaxation at transfer ( % )  \( str = 0.75 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.5</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>304.47</td>
<td>7.2492</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>335.97</td>
<td>7.9992</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 4200 kN
Force at transfer $P_t = P_0 - \text{loft} = 3864$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>0.23658</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.8168</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>7.1085</td>
</tr>
<tr>
<td>Other Loads</td>
<td>4.739</td>
</tr>
<tr>
<td>Total</td>
<td>15.901</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
Compressive stress 15.901 N/mm² is less than allowable stresses of 0.5*fci=17.5 N/mm² and 0.4*fcu=16 N/mm².

Bottom of beam:
Compressive stress 0.95265 N/mm² is less than allowable stresses of 0.5*fci=17.5 N/mm² and 0.4*fcu=16 N/mm².

Long Term Losses

Relaxation after transfer( % ) slr(1)=0.5
Shrinkage per unit length sul(1)=0.3E-3
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.5</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>304.47</td>
<td>7.2492</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>335.97</td>
<td>7.9992</td>
</tr>
<tr>
<td>Relaxation</td>
<td>21</td>
<td>0.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>252</td>
<td>6</td>
</tr>
<tr>
<td>Creep</td>
<td>330.82</td>
<td>7.8767</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>603.82</td>
<td>14.377</td>
</tr>
<tr>
<td>All Losses</td>
<td>939.79</td>
<td>22.376</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 4200 kN
Final force $P_f=P_0-\text{loss}=3260.2$ kN
Unit-weight of in-situ concrete $w_c=24.5$ kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>0.19961</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.8168</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.513</td>
</tr>
<tr>
<td>Total</td>
<td>6.5295</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete $h_{sis}=865$ mm
Percentage of shrinkage remaining $p_s=40\ %$
Percentage $p_c=40$
Percentage $s_t=67\ %$
The net moment is $M_c=F*e/1E3-M_b=-195.23$ kNm

Stress at top of in-situ concrete:
$$f_{tisds}=-(F/(A_i*mr)-F/(A_i*mr+Ac)-M_c*1E3*(di-Y_c)/Ic)*mr*1E3 =0.14148 \text{ N/mm}^2$$

Stress at bottom of in-situ concrete:
$$f_{bisds}=-(F/(A_i*mr)-F/(A_i*mr+Ac)-M_c*1E3*(hsis-Y_c)/Ic)*mr*1E3 =0.51663 \text{ N/mm}^2$$

Stress at top of precast concrete beam:
ftpbds=(F/(Ai*mr+Ac)+Mc*E3*(d-Yc)/Ic+Mb*E3*Yt/Ixx)*1E3
=-0.9234 N/mm²
Stress at bottom of precast concrete beam:
fbpbds=(F/(Ai*mr+Ac)-Mc*E3*Yc/Ic-Mb*E3*Yb/Ixx)*1E3=0.36122 N/mm²

**Combination 1 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ.</td>
<td>0</td>
<td>6.5295</td>
<td>0</td>
<td>6.2736</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.14148</td>
<td>-0.9234</td>
<td>0.51663</td>
<td>0.36122</td>
</tr>
<tr>
<td>Combination 1</td>
<td>6.8801</td>
<td>4.0523</td>
<td>3.5173</td>
<td>-8.3183</td>
</tr>
<tr>
<td>Total</td>
<td>7.0216</td>
<td>9.6584</td>
<td>4.0339</td>
<td>-1.6835</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

**WARNING:**
Bottom of beam:
Tensile stress 1.6835 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

**Combination 2 to 5 loading**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ.</td>
<td>0</td>
<td>6.5295</td>
<td>0</td>
<td>6.2736</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.14148</td>
<td>-0.9234</td>
<td>0.51663</td>
<td>0.36122</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>9.8287</td>
<td>5.0247</td>
<td>-13.864</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>9.9702</td>
<td>11.395</td>
<td>5.5413</td>
<td>-7.2291</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 9.9702 N/mm² is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm².
Top of beam:
Compressive stress 11.395 N/mm² is less than the maximum allowable stress of 0.4*fcu=16 N/mm².

Bottom of in-situ concrete:
Compressive stress 5.5413 N/mm² is less than the maximum allowable stress of 0.5*fcuc=17.5 N/mm².

WARNING:
Bottom of beam:
Tensile stress 7.2291 N/mm² exceeds maximum allowable tensile stress of 2.846 N/mm².
Location: Ex1 - 20 metre span U beam

Precast prestressed concrete U beams with in-situ top slab


Beam span under consideration span=20 m

Beam U 7 section properties

- Depth of beam d=1100 mm
- Cross-sectional area Ac=577217 mm²
- Centroid above soffit Yb=491.7 mm
- Section modulus top Zt=121.93E6 mm⁴
- Section modulus bottom Zb=150.83E6 mm⁴
- Unit weight of beam uwb=24 kN/m³

Concrete compressive strengths

- Characteristic cylinder strength fcki=32 N/mm² (transfer)
- Characteristic cylinder strength fck=40 N/mm²

Concrete moduli (short-term)

- Concrete modulus of elasticity Ecmi=33 kN/mm² (transfer)
- Concrete modulus of elasticity Ecmb=35 kN/mm²

Composite section properties

- Char strength (in-situ concrete) fckc=32 N/mm²
- Short-term modulus of elasticity for in-situ concrete Ecmc=33 kN/mm²
Second moment of area:
Depth of composite section: \(d_i = 1245\) mm
Modulus of elasticity (strand): \(E_p = 195\) kN/mm²

Cross-section 1

Distance from mid-span of beam: \(csd(1) = 0\) m
Number of layers of strand: \(nls(1) = 4\)
Height above soffit: \(ds(1) = 60\) mm
Number of strands: \(ns(1) = 14\)
Height above soffit: \(ds(2) = 110\) mm
Number of strands: \(ns(2) = 12\)
Height above soffit: \(ds(3) = 700\) mm
Number of strands: \(ns(3) = 4\)
Height above soffit: \(ds(4) = 1000\) mm
Number of strands: \(ns(4) = 4\)

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer (\%): \(str = 1\)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force (kN)</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>59.16</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>372.83</td>
<td>6.3021</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>431.99</td>
<td>7.3021</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force: 5916 kN
Force at transfer: \(P_t = P_0 - \text{loft} = 5484\) kN
<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress Self weight of Beam</td>
<td>-0.76254</td>
</tr>
<tr>
<td></td>
<td>5.6813</td>
</tr>
<tr>
<td>Total</td>
<td>4.9188</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations** *(BS EN 1992-1-1, Clause 5.10.2.2)*

**Top of beam:**
Compressive stress 4.9188 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Bottom of beam:**
Compressive stress 13.204 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Long Term Losses** *(BS EN 1992-1-1, Clause 5.10.6(1))*

Relaxation after transfer( % )  slr(1)=2
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(*=,to) crf=2

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation Elastic Shortening</td>
<td>59.16</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>372.83</td>
<td>6.3021</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>431.99</td>
<td>7.3021</td>
</tr>
<tr>
<td>Relaxation Shrinkage Creep</td>
<td>118.32</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>276.47</td>
<td>4.6733</td>
</tr>
<tr>
<td></td>
<td>590.83</td>
<td>9.987</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>985.62</td>
<td>16.66</td>
</tr>
<tr>
<td>All Losses</td>
<td>1417.6</td>
<td>23.962</td>
</tr>
</tbody>
</table>

**Losses - Long term**

| Initial force | 5916 kN |
| Final force   | Pf=P0-loftl=4498.4 kN |
| Unit-weight of in-situ concrete | wc=24 kN/m³ |
### Final Stresses N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-0.62549</td>
<td>14.598</td>
<td>10.951</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.6813</td>
<td>-4.5923</td>
<td>-2.131</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.2928</td>
<td>-1.8533</td>
<td>-0.86003</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>7.3487</td>
<td>8.1526</td>
<td>7.96</td>
</tr>
</tbody>
</table>

### Beam concrete stresses - Long term

#### Composite section

<table>
<thead>
<tr>
<th>Height from beam soffit to bottom of in-situ concrete</th>
<th>hsis=1065 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perc'age of shrinkage remaining</td>
<td>ps=25 %</td>
</tr>
<tr>
<td>Percentage</td>
<td>pcr=25</td>
</tr>
<tr>
<td>Percentage</td>
<td>stT=67 %</td>
</tr>
</tbody>
</table>

The net moment is: \( M_c = \frac{F*e}{1E3} - M_b = -77.531 \text{ kNm} \)

Stress at top of in-situ concrete:
\[
ft_{isds} = -\left(\frac{F}{(A_i*mr)} - \frac{F}{(A_i*mr+Ac)} - Mc*1E3*(di-Yc)/Ic\right) * mr*1E3
= -0.15054 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
fb_{isds} = -\left(\frac{F}{(A_i*mr)} - \frac{F}{(A_i*mr+Ac)} - Mc*1E3*(hsis-Yc)/Ic\right) * mr*1E3
= -0.057735 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:
\[
ft_{pbds} = \left(\frac{F}{(A_i*mr+Ac)} + Mc*1E3*(d-Yc)/Ic + Mb*1E3*Yt/Iyy\right) * 1E3
= 0.18909 \text{ N/mm}^2
\]

Stress at bottom of precast concrete beam:
\[
fb_{pbds} = \left(\frac{F}{(A_i*mr+Ac)} - Mc*1E3*Yc/Ic - Mb*1E3*Yb/Iyy\right) * 1E3
= -0.072259 \text{ N/mm}^2
\]

#### Loading Combination 1

(no tensile stress permitted in precast beam)

Characteristic bending moment: \( clb(1)=1440 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam and in-situ.</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-0.15054</td>
</tr>
<tr>
<td>Total</td>
<td>7.5644</td>
</tr>
</tbody>
</table>
**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment \( c2b(1) = 1907 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>-0.15054</td>
</tr>
<tr>
<td>Total</td>
<td>6.9398</td>
</tr>
</tbody>
</table>

**Loading Combination 2 - Stresses**

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 6.9398 N/mm² is less than the maximum allowable stress of 0.6*fckc=19.2 N/mm².

Top of beam:
Compressive stress 12.789 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

Bottom of in-situ concrete:
Compressive stress 4.7498 N/mm² is less than the maximum allowable stress of 0.6*fckc=19.2 N/mm².

Bottom of beam:
Tensile stress 0.6188 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
Location: Ex2 - 25 metre span U10 beam

Precast prestressed concrete U beams with in-situ top slab


Beam span under consideration span=25 m

Beam U 10 section properties

Depth of beam d=1400 mm
Cross-sectional area Ac=677357 mm²
Centroid above soffit Yb=634.3 mm
Section modulus top Zt=187.93E6 mm⁴
Section modulus bottom Zb=226.9E6 mm⁴
Unit weight of beam uwb=24.5 kN/m³

Concrete compressive strengths

Characteristic cylinder strength fcki=36 N/mm² (transfer)
Characteristic cylinder strength fck=40 N/mm²

Concrete moduli (short-term)

Concrete modulus of elasticity Ecmi=33 kN/mm² (transfer)
Concrete modulus of elasticity Ecmb=35 kN/mm²

Composite section properties

Char strength (in-situ concrete) fckc=28 N/mm²
Short-term modulus of elasticity for in-situ concrete Ecmc=34 kN/mm²

Item No. 1

Item weighting iw1=0.97143
X co-ordinate X(1)=0 mm
Y co-ordinate Y(1)=1380 mm
X co-ordinate X(2)=370 mm
Y co-ordinate Y(2)=1380 mm
X co-ordinate X(3)=370 mm
Y co-ordinate Y(3)=1400 mm
X co-ordinate X(4)=605 mm
Y co-ordinate Y(4)=1400 mm
X co-ordinate X(5)=605 mm
Y co-ordinate Y(5)=1380 mm
X co-ordinate X(6)=1246 mm
Y co-ordinate Y(6)=1380 mm
X co-ordinate X(7)=1246 mm
Y co-ordinate Y(7)=1400 mm
X co-ordinate X(8)=1481 mm
Y co-ordinate Y(8)=1400 mm
X co-ordinate X(9)=1481 mm
Item No. 2

Second moment of area \( I_{XX2} = 143.92 \times 10^9 \text{ mm}^4 \)
Area of item \( A_2 = 677357 \text{ mm}^2 \)
Centroid above soffit \( Y_{C2} = 634.3 \text{ mm} \)
Item weighting \( i_{w2} = 1 \)
Depth of composite section \( d_i = 1680 \text{ mm} \)
Modulus of elasticity (strand) \( E_p = 195 \text{ kN/mm}^2 \)

Cross-section 1

Distance from mid-span of beam \( c_{sd(1)} = 4.5 \text{ m} \)
Number of layers of strand \( n_{ls(1)} = 4 \)
Height above soffit \( d_{s(1)} = 60 \text{ mm} \)
Number of strands \( n_s(1) = 16 \)
Height above soffit \( d_{s(2)} = 110 \text{ mm} \)
Number of strands \( n_s(2) = 12 \)
Height above soffit \( d_{s(3)} = 800 \text{ mm} \)
Number of strands \( n_s(3) = 4 \)
Height above soffit \( d_{s(4)} = 1300 \text{ mm} \)
Number of strands \( n_s(4) = 4 \)

Short Term Losses ( BS EN 1992-1-1 )

Relaxation at transfer ( % ) \( s_{tr} = 1.25 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.5</td>
<td>1.25</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>111.75</td>
<td>3.4491</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>152.25</td>
<td>4.6991</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force \( 3240 \text{ kN} \)
Force at transfer \( P_t = P_0 - \text{lloff} = 3087.8 \text{ kN} \)
Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam: Compressive stress 5.0158 N/mm² is less than allowable stresses of 0.7*fcki=25.2 N/mm² and 0.6*fck=24 N/mm².

Bottom of beam: Compressive stress 4.1797 N/mm² is less than allowable stresses of 0.7*fcki=25.2 N/mm² and 0.6*fck=24 N/mm².

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer (%) slr(1)=1
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(*,to) crf=2

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>40.5</td>
<td>1.25</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>111.75</td>
<td>3.4491</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>152.25</td>
<td>4.6991</td>
</tr>
<tr>
<td>Relaxation</td>
<td>32.4</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>157.95</td>
<td>4.875</td>
</tr>
<tr>
<td>Creep</td>
<td>131.08</td>
<td>4.0457</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>321.43</td>
<td>9.9207</td>
</tr>
<tr>
<td>All Losses</td>
<td>473.68</td>
<td>14.62</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 3240 kN
Final force Pf=P0-lofl=2766.3 kN
Unit-weight of in-situ concrete wc=23.5 kN/m³
Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete \( h_{sis} = 1380 \) mm
Per cent age of shrinkage remaining \( ps = 45 \% \)
Percentage \( p_{cr} = 45 \)
Percentage \( stT = 50 \% \)
The net moment is \( M_c = F * e / 1E3 - M_b = 385.6 \) kNm

Stress at top of in-situ concrete:
\[
ft_{isds} = -(F/(A_i * mr) - F/(A_i * mr + A_c) - M_c * 1E3 * (d_i - Y_c)/I_c) * mr * 1E3 = 1.499 \text{ N/mm}^2
\]

Stress at bottom of in-situ concrete:
\[
f_{bisds} = -(F/(A_i * mr) - F/(A_i * mr + A_c) - M_c * 1E3 * (h_{sis} - Y_c)/I_c) * mr * 1E3 = 1.1696 \text{ N/mm}^2
\]

Stress at top of precast concrete beam:
\[
ft_{pbds} = (F/(A_i * mr + A_c) + M_c * 1E3 * (d - Y_c)/I_c + M_b * 1E3 * Y_t/I_{yy}) * 1E3 = -3.4651 \text{ N/mm}^2
\]

Stress at bottom of precast concrete beam:
\[
f_{pbds} = (F/(A_i * mr + A_c) - M_c * 1E3 * Y_c/I_c - M_b * 1E3 * Y_b/I_{yy}) * 1E3 = 1.3372 \text{ N/mm}^2
\]

Loading Combination 1

(no tensile stress permitted in precast beam)

Characteristic bending moment \( clb(1) = 1000 \) kN\( m \)
WARNING:
Bottom of beam:
Tensile stress 1.3472 N/mm² exceeds maximum allowable
tensile stress of 0 N/mm².

Loading Combination 2
(tensile stress permitted (no visible cracking))
Characteristic bending moment c2b(1)=1250 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2</th>
<th>Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>8.8322</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2</td>
<td>1.499</td>
<td>-3.4651</td>
</tr>
<tr>
<td></td>
<td>2.5359</td>
<td>1.5848</td>
</tr>
<tr>
<td>Total</td>
<td>4.0349</td>
<td>6.9519</td>
</tr>
</tbody>
</table>

Loading Combination 2 - Stresses
NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 4.0349 N/mm² is less than the maximum allowable stress of 0.6*fck=16.8 N/mm².

Top of beam:
Compressive stress 6.9519 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

Bottom of in-situ concrete:
Compressive stress 2.638 N/mm² is less than the maximum allowable stress of 0.6*fck=16.8 N/mm².

Bottom of beam:
Tensile stress 2.056 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
Location: Ex3 - 15 metre span U3 beam

Precast prestressed concrete U beams with in-situ top slab


Beam span under consideration  span=15 m

Beam U 3 section properties

Depth of beam  d=900 mm
Cross-sectional area  Ac=510457 mm²
Centroid above soffit  Yb=398.4 mm
Section modulus top  Zt=84.39E6 mm³
Section modulus bottom  Zb=106.27E6 mm³
Unit weight of beam  uwb=25 kN/m³

Concrete compressive strengths

Characteristic cylinder strength  fcki=28 N/mm² (transfer)
Characteristic cylinder strength  fck=32 N/mm²

Concrete moduli (short-term)

Concrete modulus of elasticity  Ecmi=32 kN/mm² (transfer)
Concrete modulus of elasticity  Ecmb=33 kN/mm²

Composite section properties

Char strength (in-situ concrete)  fckc=28 N/mm²
Short-term modulus of elasticity for in-situ concrete  Ecmc=32 kN/mm²
Second moment of area:
Depth of composite section \( d_i = 1085 \text{ mm} \)
Modulus of elasticity (strand) \( E_p = 195 \text{ kN/mm}^2 \)

**Cross-section 1**

Distance from mid-span of beam \( c_{sd}(1) = 2.4 \text{ m} \)
Number of layers of strand \( n_{ls}(1) = 4 \)
Height above soffit \( d_s(1) = 50 \text{ mm} \)
Number of strands \( n_s(1) = 10 \)

**WARNING:**
**Strand location is non-standard.**

Height above soffit \( d_s(2) = 105 \text{ mm} \)
Number of strands \( n_s(2) = 10 \)

**WARNING:**
**Strand location is non-standard.**

Height above soffit \( d_s(3) = 490 \text{ mm} \)
Number of strands \( n_s(3) = 4 \)

**WARNING:**
**Strand location is non-standard.**

Height above soffit \( d_s(4) = 790 \text{ mm} \)
Number of strands \( n_s(4) = 4 \)

**WARNING:**
**Strand location is non-standard.**

Bending moment from temp.loads \( t_{lb}(1) = 600 \text{ kNm} \)
Bending moment from other loads \( o_{lb}(1) = 400 \text{ kNm} \)

**Short Term Losses ( BS EN 1992-1-1 )**

Relaxation at transfer ( % ) \( s\text{tr} = 0.75 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.5</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>256.8</td>
<td>6.1144</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>288.3</td>
<td>6.8644</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force: 4200 kN
Force at transfer: Pt=P0-loft=3911.7 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>0.23949</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.8168</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>7.1085</td>
</tr>
<tr>
<td>Other Loads</td>
<td>4.739</td>
</tr>
<tr>
<td>Total</td>
<td>15.904</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam:
Compressive stress 15.904 N/mm² is less than allowable stresses of 0.7*fcki=19.6 N/mm² and 0.6*fck=19.2 N/mm².

Bottom of beam:
Compressive stress 1.1179 N/mm² is less than allowable stresses of 0.7*fcki=19.6 N/mm² and 0.6*fck=19.2 N/mm².

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer (%): slr(1)=0.5
Shrinkage per unit length: sul(1)=0.3E-3
Creep coefficient φ(ψ, to): crf=2
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>31.5</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>256.8</td>
<td>6.1144</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>288.3</td>
<td>6.8644</td>
</tr>
<tr>
<td>Relaxation</td>
<td>21</td>
<td>0.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>245.7</td>
<td>5.85</td>
</tr>
<tr>
<td>Creep</td>
<td>362.42</td>
<td>8.6291</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>629.12</td>
<td>14.979</td>
</tr>
<tr>
<td>All Losses</td>
<td>917.43</td>
<td>21.843</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 4200 kN
Final force Pf=P0-loftl=3282.6 kN
Unit-weight of in-situ concrete wc=24.5 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>0.20098</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.8168</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>2.513</td>
</tr>
<tr>
<td>Total</td>
<td>6.5308</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=865 mm
Perc'age of shrinkage remaining ps=40 %
Percentage pcr=40
Percentage stT=67 %
The net moment is Mc=F*e/1E3-Mb=-272.88 kNm

Stress at top of in-situ concrete:
ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3
=0.1937 N/mm²

Stress at bottom of in-situ concrete:
fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
=0.71806 N/mm²

Stress at top of precast concrete beam:
ftpbds= \(\frac{F}{(A_i \cdot mr + A_c)} + M_c \cdot 1E^3 \cdot (d - Y_c)/I_c + M_b \cdot 1E^3 \cdot Y_t/I_{yy}) \cdot 1E^3\)

Stress at bottom of precast concrete beam:

\(fbpbds=\left(\frac{F}{(A_i \cdot mr + A_c)} - M_c \cdot 1E^3 \cdot Y_c/I_c - M_b \cdot 1E^3 \cdot Y_b/I_{yy}\right) \cdot 1E^3\)

\(-1.2783 \text{ N/mm}^2\)

\(Stress \text{ at bottom of precast concrete beam:}\)

\(fbpbds=\left(\frac{F}{(A_i \cdot mr + A_c)} - M_c \cdot 1E^3 \cdot Y_c/I_c - M_b \cdot 1E^3 \cdot Y_b/I_{yy}\right) \cdot 1E^3\)

\(-0.49992 \text{ N/mm}^2\)

### Loading Combination 1

(no tensile stress permitted in precast beam)

Characteristic bending moment \(c_{lb(1)}=1500 \text{ kNm}\)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.1937</td>
</tr>
<tr>
<td>Combination 1</td>
<td>6.8801</td>
</tr>
<tr>
<td>Total</td>
<td>7.0738</td>
</tr>
</tbody>
</table>

### Loading Combination 1 - Stresses

**WARNING:**

**Bottom of beam:**

Tensile stress 1.4673 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

### Loading Combination 2

(tensile stress permitted (no visible cracking))

Characteristic bending moment \(c_{2b(1)}=2500 \text{ kNm}\)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.1937</td>
</tr>
<tr>
<td>Combination 2</td>
<td>9.8287</td>
</tr>
<tr>
<td>Total</td>
<td>10.022</td>
</tr>
</tbody>
</table>

### Loading Combination 2 - Stresses

**NOTE:** The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:

Compressive stress 10.022 N/mm² is less than the maximum allowable
stress of 0.6*fckc=16.8 N/mm².

Top of beam:
Compressive stress 11.042 N/mm² is less than the maximum allowable stress of 0.6*fck=19.2 N/mm².

Bottom of in-situ concrete:
Compressive stress 5.7427 N/mm² is less than the maximum allowable stress of 0.6*fckc=16.8 N/mm².

WARNING:

Bottom of beam:
Tensile stress 7.0129 N/mm² exceeds maximum allowable tensile stress of 2.8534 N/mm².
Location: Ex1 - 20 metre span wide box beam

Precast prestressed concrete Wide Box Beams

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam \( \text{span} = 20 \, \text{m} \)

Beam WB 7 section properties

- Depth of beam \( d = 960 \, \text{mm} \)
- Cross-sectional area \( A_c = 547245 \, \text{mm}^2 \)
- Centroid above soffit \( y_b = 475 \, \text{mm} \)
- Section modulus top \( Z_t = 137.42 \times 10^6 \, \text{mm}^3 \)
- Section modulus bottom \( Z_b = 140.61 \times 10^6 \, \text{mm}^3 \)
- Unit weight of beam \( u_{wb} = 25 \, \text{kN/m}^3 \)

Concrete strengths

- Strength at transfer \( f_{ci} = 40 \, \text{N/mm}^2 \)
- Characteristic strength \( f_{cu} = 50 \, \text{N/mm}^2 \)

Concrete moduli

Modulus of elasticity at transfer for concrete strength 40 N/mm\(^2\) is: \( E_{ci} = 31 \, \text{kN/mm}^2 \)

Modulus of elasticity for concrete strength 50 N/mm\(^2\) is: \( E_{cb} = 34 \, \text{kN/mm}^2 \)

Composite section properties

- Depth of in-situ concrete \( d_{isc} = 480 \, \text{mm} \)
- Char strength of in-situ concrete \( f_{cu} = 30 \, \text{N/mm}^2 \)
- Modulus of elasticity for concrete strength 30 N/mm\(^2\) is: \( E_{cc} = 28 \, \text{kN/mm}^2 \)
- Breadth of section \( b = 1280 \, \text{mm} \)
- Initial force per strand \( i_f = 225 \, \text{kN} \)
- Modulus of elasticity \( E_s = 200 \, \text{kN/mm}^2 \)

Cross-section 1

- Distance from mid-span of beam \( c_{sd(1)} = 0 \, \text{m} \)
- Number of layers of strand \( n_{ls(1)} = 4 \)
- Height from soffit \( d_s(1) = 65 \, \text{mm} \)
- Number of strands \( n_s(1) = 16 \)
- Height from soffit \( d_s(2) = 115 \, \text{mm} \)
- Number of strands \( n_s(2) = 8 \)
- Height from soffit \( d_s(3) = 700 \, \text{mm} \)
- Number of strands \( n_s(3) = 2 \)
- Height from soffit \( d_s(4) = 900 \, \text{mm} \)
- Number of strands \( n_s(4) = 2 \)
Short Term Losses

Relaxation at transfer ( % )\n\n\nSummary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>63</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>574.95</td>
<td>9.1261</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>637.95</td>
<td>10.126</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 6300 kN
Force at transfer \( P_t = P_0 - \text{loft} = 5662.1 \text{ kN} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of Beam</td>
<td>Bottom of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.6064</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.9673</td>
</tr>
<tr>
<td>Total</td>
<td>3.3609</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
Compressive stress 3.3609 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Bottom of beam:
Compressive stress 17.188 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Long Term Losses

Relaxation after transfer( % )\n\nShrinkage per unit length \( \text{sul(1)} = 0.3E-3 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>63</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>574.95</td>
<td>9.1261</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>637.95</td>
<td>10.126</td>
</tr>
<tr>
<td>Relaxation</td>
<td>94.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>277.2</td>
<td>4.4</td>
</tr>
<tr>
<td>Creep</td>
<td>737.77</td>
<td>11.711</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>1109.5</td>
<td>17.611</td>
</tr>
<tr>
<td>All Losses</td>
<td>1747.4</td>
<td>27.737</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 6300 kN
Final force Pf=P0-loftl=4552.6 kN
Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.2916</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.9673</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>0.39909</td>
</tr>
<tr>
<td>Total</td>
<td>4.0748</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=d-disc=480 mm
Perc'age of shrinkage remaining ps=25 %
Percentage pcr=25
Percentage stT=75 %
The net moment is Mc=F*e/1E3-Mb=-272.51 kNm
Stress at top of in-situ concrete:
\[\text{ftisds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(d-Yc)/Ic)*mr*1E3\]
\[=-1.3033 \text{ N/mm}^2\]
Stress at bottom of in-situ concrete:
\[\text{fbisds}=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3\]
\[=0.24611 \text{ N/mm}^2\]
Stress at top of precast concrete beam:
Stress at bottom of precast concrete beam:

\[
\text{fbpbds} = \frac{F}{(A_i \cdot m_r + A_c) - M_c \cdot 1E3 \cdot Y_c/I_c - M_b \cdot 1E3 \cdot Y_b/I_{xx}} \cdot 1E3 = -0.017406 \text{ N/mm}^2
\]

Combination 1 loading

Applied bending moment \( clb(1) = 1500 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Prestress, beam and in-situ</th>
<th>Diff. shrinkage</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
<td>-1.3033</td>
<td>9.4603</td>
</tr>
<tr>
<td>Top of in-situ</td>
<td>4.0748</td>
<td>0.10361</td>
<td>0.24611</td>
</tr>
<tr>
<td>Top of Beam</td>
<td>0</td>
<td>-0.18131</td>
<td>-10.576</td>
</tr>
<tr>
<td>Bottom of in-situ</td>
<td>0</td>
<td>0.24611</td>
<td>-10.576</td>
</tr>
<tr>
<td>Bottom of Beam</td>
<td>12.476</td>
<td>0.24611</td>
<td>-10.576</td>
</tr>
<tr>
<td>Total</td>
<td>8.157</td>
<td>15.666</td>
<td>0.064798</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

Combination 2 to 5 loading

Applied bending moment \( c2b(1) = 2000 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Prestress, beam and in-situ</th>
<th>Diff. shrinkage</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
<td>-1.3033</td>
<td>11.13</td>
</tr>
<tr>
<td>Top of in-situ</td>
<td>4.0748</td>
<td>0.10361</td>
<td>0.24611</td>
</tr>
<tr>
<td>Top of Beam</td>
<td>0</td>
<td>-0.24175</td>
<td>-14.102</td>
</tr>
<tr>
<td>Bottom of in-situ</td>
<td>0</td>
<td>0.24611</td>
<td>-14.102</td>
</tr>
<tr>
<td>Bottom of Beam</td>
<td>12.476</td>
<td>0.24611</td>
<td>-14.102</td>
</tr>
<tr>
<td>Total</td>
<td>9.8264</td>
<td>17.693</td>
<td>0.00436</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Top of in-situ concrete:
Compressive stress 9.8264 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².

Top of beam:
Compressive stress 17.693 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².

Bottom of in-situ concrete:
Compressive stress 0.00436 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².

Bottom of beam:
Tensile stress 1.6434 N/mm² is less than the maximum allowable tensile stress of 3.182 N/mm².
Location: Ex2 - 30 metre span wide box beam WB16

Precast prestressed concrete Wide Box Beams

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam  span=30 m

Beam WB 16 section properties

- Depth of beam  d=1460 mm
- Cross-sectional area  Ac=785520 mm²
- Centroid above soffit  Yb=696 mm
- Section modulus top  Zt=277.49E6 mm³
- Section modulus bottom  Zb=304.68E6 mm³
- Unit weight of beam  uwb=24.5 kN/m³

Concrete strengths

- Strength at transfer  fci=35 N/mm²
- Characteristic strength  fcu=45 N/mm²

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 35 N/mm² is: Eci=29.5 kN/mm²
- Modulus of elasticity for concrete strength 45 N/mm² is: Ecb=32.5 kN/mm²

Composite section properties

- Depth of in-situ concrete  disc=0 mm
- Nominal diameter  nomd=12 mm
- Nominal tensile strength  nomt=1600 N/mm²
- Nominal steel area  noma=90 mm²
- Characteristic breaking load  cbl=160 kN
- Initial force per strand  if=100 kN
- Modulus of elasticity  Es=200 kN/mm²

Cross-section 1

- Distance from mid-span of beam  csd(1)=6.7 m
- Number of layers of strand  nls(1)=6
- Height from soffit  ds(1)=60 mm
- Number of strands  ns(1)=18

WARNING:
Strand location is non-standard.

Height from soffit  ds(2)=120 mm
Number of strands  ns(2)=10

WARNING:
Strand location is non-standard.
**Short Term Losses**

Relaxation at transfer ( % )  \( \text{str}=0.75 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>27</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>151.78</td>
<td>4.216</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>178.78</td>
<td>4.966</td>
</tr>
</tbody>
</table>

Losses at Transfer

- Initial force: 3600 kN
- Force at transfer: 3421.2 kN

**Stresses at Transfer N/mm²**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-0.25867</td>
<td>8.5587</td>
<td>6.6161</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.2441</td>
<td>-5.6884</td>
<td>-3.0594</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>1.8014</td>
<td>-1.6411</td>
<td>-0.88262</td>
</tr>
<tr>
<td>Other Loads</td>
<td>2.1617</td>
<td>-1.9693</td>
<td>-1.0591</td>
</tr>
<tr>
<td>Total</td>
<td>9.9485</td>
<td>-0.73999</td>
<td>1.6149</td>
</tr>
</tbody>
</table>

**Stress Limitations**

Top of beam:
- Compressive stress 9.9485 N/mm² is less than allowable stresses of 0.5*fci=17.5 N/mm² and 0.4*fcu=18 N/mm².

Bottom of beam:
- The flexural tensile stress of 0.73999 N/mm² at transfer is less than the permissible value of 1 N/mm². Clause 6.3.2.4(b)(1).
Long Term Losses

Relaxation after transfer (\%) \( \text{slr}(1) = 0.5 \)
Shrinkage per unit length \( \text{sul}(1) = 0.3 \times 10^{-3} \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>27</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>151.78</td>
<td>4.216</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>178.78</td>
<td>4.966</td>
</tr>
<tr>
<td>Relaxation</td>
<td>18</td>
<td>0.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>194.4</td>
<td>5.4</td>
</tr>
<tr>
<td>Creep</td>
<td>88.781</td>
<td>2.4661</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>301.18</td>
<td>8.3661</td>
</tr>
<tr>
<td>All Losses</td>
<td>479.96</td>
<td>13.332</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force \( 3600 \text{ kN} \)
Final force \( P_f = P_0 - \text{loftl} = 3120 \text{ kN} \)

Loading

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.23589</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.2441</td>
</tr>
<tr>
<td>Total</td>
<td>6.0082</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Combination 1 loading

Applied bending moment \( c_{lb}(1) = 1000 \text{ kNm} \)
### Combination 1 Loading - Stresses

<table>
<thead>
<tr>
<th>Loading</th>
<th>Comb. 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress, beam</td>
<td>6.0082</td>
</tr>
<tr>
<td>Combination 1</td>
<td>4.5035</td>
</tr>
<tr>
<td></td>
<td>Total</td>
</tr>
</tbody>
</table>

WARNING:
Bottom of beam
Tensile stress 1.1652 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

### Combination 2 to 5 Loading

Applied bending moment  \( c_{2b(1)} = 1750 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Comb. 2-5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress, beam</td>
<td>6.0082</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>6.3049</td>
</tr>
<tr>
<td></td>
<td>Total</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

### Combination 3 Loading

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of beam:
Compressive stress 12.313 N/mm² is less than the maximum allowable stress of \( 0.4 \times f_{cu} = 18 \text{ N/mm}² \).

WARNING:
Bottom of beam:
Tensile stress 3.6268 N/mm² exceeds maximum allowable tensile stress of 3.0187 N/mm².
Location: Ex1 - 20 metre span wide box beam

Precast prestressed concrete Wide Box Beams


Span of beam: span=20 m

Beam WB 7 section properties

Depth of beam: d=960 mm
Cross-sectional area: Ac=547245 mm²
Centroid above soffit: Yb=475 mm
Section modulus top: Zt=137.42E6 mm©
Section modulus bottom: Zb=140.61E6 mm©
Unit weight of beam: uwb=25 kN/m©

Concrete compressive strengths

Characteristic cylinder strength: fcki=32 N/mm² (transfer)
Characteristic cylinder strength: fck=40 N/mm²

Concrete moduli (short-term)

Concrete modulus of elasticity: Ecmi=33 kN/mm² (transfer)
Concrete modulus of elasticity: Ecmb=35 kN/mm²

Composite section properties

Depth of in-situ concrete: disc=480 mm
Char strength (in-situ concrete): fckc=25 N/mm²
Short-term modulus of elasticity for in-situ concrete: Ecmc=31 kN/mm²
Breadth of section: b=1280 mm
Modulus of elasticity (strand): Ep=195 kN/mm²

Cross-section 1

Distance from mid-span of beam: csd(1)=0 m
Number of layers of strand: nls(1)=4
Height from soffit: ds(1)=65 mm
Number of strands: ns(1)=16
Height from soffit: ds(2)=115 mm
Number of strands: ns(2)=8
Height from soffit: ds(3)=700 mm
Number of strands: ns(3)=2
Height from soffit: ds(4)=900 mm
Number of strands: ns(4)=2

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer (%): str=1
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>63</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>485.6</td>
<td>7.7079</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>548.6</td>
<td>8.7079</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 6300 kN
Force at transfer \( P_t = P_0 - loft = 5751.4 \) kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-1.6318</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.9673</td>
</tr>
<tr>
<td>Total</td>
<td>3.3356</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
Compressive stress 3.3356 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

Bottom of beam:
Compressive stress 17.536 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer( % ) \( slr(1) = 1.5 \)
Shrinkage per unit length \( sul(1) = 0.3E-3 \)
Creep coefficient \( \phi(\omega,\tau) \) \( crf = 1.8 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>63</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>485.6</td>
<td>7.7079</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>548.6</td>
<td>8.7079</td>
</tr>
<tr>
<td>Relaxation</td>
<td>94.5</td>
<td>1.5</td>
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<tr>
<td>Shrinkage</td>
<td>270.27</td>
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<tr>
<td>Creep</td>
<td>794.83</td>
<td>12.616</td>
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<tr>
<td>Long-term Total</td>
<td>1159.6</td>
<td>18.406</td>
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<tr>
<td>All Losses</td>
<td>1708.2</td>
<td>27.114</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 6300 kN
Final force Pf=P0-loftl=4591.8 kN
Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>4.9673</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>0.39909</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>4.0637</td>
</tr>
<tr>
<td>Total</td>
<td></td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=d-disc=480 mm
Percentage of shrinkage remaining ps=25 %
Percentage pcr=25
Percentage stT=75 %
The net moment is Mc=F*e/1E3-Mb=-314.12 kNm
Stress at top of in-situ concrete:
\[ ftisds=\left(\frac{F}{(Ai*mr)}-\frac{F}{(Ai*mr+Ac)}-Mc*1E3*\frac{(d-Yc)}{Ic}\right)*mr*1E3 =-1.3445 \text{ N/mm}^2 \]
Stress at bottom of in-situ concrete:
\[ fbisds=\left(\frac{F}{(Ai*mr)}-\frac{F}{(Ai*mr+Ac)}-Mc*1E3*\frac{(hsis-Yc)}{Ic}\right)*mr*1E3 =0.57094 \text{ N/mm}^2 \]
Stress at top of precast concrete beam:
ftpbds=(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)*1E3
=0.085793 N/mm²

Stress at bottom of precast concrete beam:
fbpbds=(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)*1E3=-0.022513 N/mm²

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment clb(1)=1500 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-1.3445</td>
</tr>
<tr>
<td>Combination 1</td>
<td>10.123</td>
</tr>
<tr>
<td>Total</td>
<td>8.7784</td>
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</tbody>
</table>

Loading Combination 1 - Stresses

**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment c2b(1)=2000 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-1.3445</td>
</tr>
<tr>
<td>Combination 2</td>
<td>11.909</td>
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<tr>
<td>Total</td>
<td>10.565</td>
</tr>
</tbody>
</table>

Loading Combination 2 - Stresses

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 10.565 N/mm² is less than the maximum allowable stress of 0.6*fckc=15 N/mm².

Top of beam:
Compressive stress 17.595 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².
Bottom of in-situ concrete:
Compressive stress 0.28443 N/mm² is less than the maximum allowable stress of 0.6*fck=15 N/mm².

Bottom of beam:
Tensile stress 1.4868 N/mm² is less than the maximum allowable tensile stress of 2.846 N/mm².
Precast prestressed concrete Wide Box Beams


Span of beam: span=30 m

Beam WB 16 section properties

- Depth of beam: d=1460 mm
- Cross-sectional area: Ac=785520 mm²
- Centroid above soffit: Yb=696 mm
- Section modulus top: Zt=277.49E6 mm³
- Section modulus bottom: Zb=304.68E6 mm³
- Unit weight of beam: uwb=24.5 kN/m³

Concrete compressive strengths

- Characteristic cylinder strength: fcki=28 N/mm² (transfer)
- Characteristic cylinder strength: fck=36 N/mm²

Concrete moduli (short-term)

- Concrete modulus of elasticity: Ecmi=32 kN/mm² (transfer)
- Concrete modulus of elasticity: Ecmb=34 kN/mm²

Composite section properties

- Depth of in-situ concrete: disc=0 mm
- Nominal diameter: nomd=12 mm
- Nominal tensile strength: nomt=1600 N/mm²
- Nominal steel area: noma=90 mm²
- Characteristic breaking load: fpk=160 kN
- Initial force per strand: ipf=100 kN
- Char breaking load (strand): fpk=160 kN
- Initial force per strand: ipf=100 kN
- Modulus of elasticity (strand): Ep=200 kN/mm²

Cross-section 1

- Distance from mid-span of beam: csd(1)=6.7 m
- Number of layers of strand: nls(1)=6
- Height from soffit: ds (1)=60 mm
- Number of strands: ns (1)=18

WARNING:
Strand location is non-standard.

Height from soffit: ds (2)=120 mm
Number of strands: ns (2)=10

WARNING:
Strand location is non-standard.
Height from soffit                ds(3)=1000 mm
Number of strands                 ns(3)=2
Height from soffit                ds(4)=1100 mm
Number of strands                 ns(4)=2
Height from soffit                ds(5)=1200 mm
Number of strands                 ns(5)=2
Height from soffit                ds(6)=1350 mm
Number of strands                 ns(6)=2
Bending moment from temp.loads    tlb(1)=500 kNm
Bending moment from other loads   olb(1)=600 kNm

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer ( % )     str=0.75

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>27</td>
<td>0.75</td>
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<tr>
<td>Elastic Shortening</td>
<td>134.65</td>
<td>3.7402</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>161.65</td>
<td>4.4902</td>
</tr>
</tbody>
</table>

**Summary**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.25996</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.2441</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>1.8014</td>
</tr>
<tr>
<td>Other Loads</td>
<td>2.1617</td>
</tr>
<tr>
<td>Total</td>
<td>9.9472</td>
</tr>
</tbody>
</table>

**Beam Concrete Stresses at Transfer**

**Stress Limitations ( BS EN 1992-1-1, Clause 5.10.2.2 )**

Top of beam:
Compressive stress 9.9472 N/mm² is less than allowable stresses of 0.7*fcki=19.6 N/mm² and 0.6*fck=21.6 N/mm².

Bottom of beam:
The flexural tensile stress of 0.69714 N/mm² at transfer is less than the permissible value of 1 N/mm² (as was permitted in BS5400-4).
Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer (%) \( slr(1)=0.5 \)
Shrinkage per unit length \( sul(1)=0.3 \times 10^{-3} \)
Creep coefficient \( crf=2 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>27</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>134.65</td>
<td>3.7402</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>161.65</td>
<td>4.4902</td>
</tr>
<tr>
<td>Relaxation</td>
<td>18</td>
<td>0.5</td>
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<tr>
<td>Shrinkage</td>
<td>194.4</td>
<td>5.4</td>
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<tr>
<td>Creep</td>
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<tr>
<td>Long-term Total</td>
<td>308.86</td>
<td>8.5795</td>
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<tr>
<td>All Losses</td>
<td>470.51</td>
<td>13.07</td>
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</tbody>
</table>

Losses - Long term

Initial force 3600 kN
Final force \( Pf=P0-loftl=3129.5 \) kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-0.23661</td>
<td>7.8289</td>
<td>6.0519</td>
<td></td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.2441</td>
<td>-5.6884</td>
<td>-3.0594</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>6.0075</td>
<td>2.1405</td>
<td>2.9925</td>
<td></td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Loading Combination 1

(no tensile stress permitted in precast beam)
Characteristic bending moment \( clb(1)=1000 \) kNm
### Loading Combination 1 Loading - Stresses

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam Combination 1</td>
<td>6.0075</td>
<td>2.1405</td>
</tr>
<tr>
<td></td>
<td>4.5035</td>
<td>-3.2821</td>
</tr>
<tr>
<td>Total</td>
<td>10.511</td>
<td>-1.1416</td>
</tr>
</tbody>
</table>

**WARNING:**

Bottom of beam

Tensile stress 1.1416 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

### Loading Combination 2

(tensile stress permitted (no visible cracking))

Characteristic bending moment c2b(1)=1750 kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam Combination 2</td>
<td>6.0075</td>
<td>2.1405</td>
</tr>
<tr>
<td></td>
<td>6.3049</td>
<td>-5.7437</td>
</tr>
<tr>
<td>Total</td>
<td>12.312</td>
<td>-3.6032</td>
</tr>
</tbody>
</table>

**NOTE:** The stresses from temperature differences should be added to the total stresses.

Top of beam:
Compressive stress 12.312 N/mm² is less than the maximum allowable stress of 0.6*fck=21.6 N/mm².

**WARNING:**

Bottom of beam:
Tensile stress 3.6032 N/mm² exceeds maximum allowable tensile stress of 2.7 N/mm².
Location: Ex1 - 20 metre span box beam

Precast prestressed concrete Box Beams

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=20 m

Beam B 7 section properties

Depth of beam d=960 mm
Cross-sectional area Ac=451208 mm²
Centroid above soffit Yb=476 mm
Section modulus top Zt=106.78E6 mm⁴
Section modulus bottom Zb=108.58E6 mm⁴
Unit weight of beam uwb=25 kN/m³

Concrete strengths

Strength at transfer fci=40 N/mm²
Characteristic strength fcu=50 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 40 N/mm² is:
Eci=31 kN/mm²
Modulus of elasticity for concrete strength 50 N/mm² is:
Ecb=34 kN/mm²

Composite section properties

Depth of in-situ concrete disc=480 mm
Char strength of in-situ concrete fcuc=30 N/mm²
Modulus of elasticity for concrete strength 30 N/mm² is:
Ecc=28 kN/mm²
Breadth of section b=1000 mm
Initial force per strand if=225 kN
Modulus of elasticity Es=200 kN/mm²

Cross-section 1

Distance from mid-span of beam csd(1)=0 m
Number of layers of strand nls(1)=4
Height from soffit ds(1)=65 mm
Number of strands ns(1)=10
Height from soffit ds(2)=115 mm
Number of strands ns(2)=6
Height from soffit ds(3)=700 mm
Number of strands ns(3)=2
Height from soffit ds(4)=900 mm
Number of strands ns(4)=2
Short Term Losses

Relaxation at transfer ( % )  str=1

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>45</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>323.99</td>
<td>7.1998</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>368.99</td>
<td>8.1998</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 4500 kN
Force at transfer Pt=P0-loft=4131 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.47717</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.2817</td>
</tr>
<tr>
<td>Total</td>
<td>4.8046</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations

Top of beam:
Compressive stress 4.8046 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Bottom of beam:
Compressive stress 13.434 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Long Term Losses

Relaxation after transfer( % )  slr(1)=1.5
Shrinkage per unit length  sul(1)=0.3E-3
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>45</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>323.99</td>
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<td>Transfer Total</td>
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<td>8.1998</td>
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<tr>
<td>Relaxation</td>
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<tr>
<td>Shrinkage</td>
<td>198</td>
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<tr>
<td>Creep</td>
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<td>Long-term Total</td>
<td>627.82</td>
<td>13.952</td>
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<tr>
<td>All Losses</td>
<td>996.82</td>
<td>22.151</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 4500 kN
Final force Pf=P0-loftl=3503.2 kN
Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<tr>
<td>Prestress</td>
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<td>Self weight of Beam</td>
<td>5.2817</td>
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<tr>
<td>In-situ concrete</td>
<td>0.61216</td>
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<tr>
<td>Total</td>
<td>5.4892</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from soffit of in-situ concrete hsis=d-disc=480 mm
Perc'age of shrinkage remaining ps=25 %
Percentage pcr=25
Percentage stT=75 %
The net moment is Mc=F*e/1E3-Mb=-96.572 kNm
Stress at top of in-situ concrete:
ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(d-Yc)/Ic)*mr*1E3
=-0.97688 N/mm²
Stress at bottom of in-situ concrete:
fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3
=-0.28093 N/mm²
Stress at top of precast concrete beam:
ftpbds=(F/(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3
=0.16168 N/mm²

Stress at bottom of precast concrete beam:

fbpdb=(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3=-0.014425 N/mm²

**Combination 1 loading**

Applied bending moment  \( clb(1)=1100 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
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<tr>
<td>Prestress,beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage Combination 1</td>
<td>-0.97688</td>
</tr>
<tr>
<td>Total</td>
<td>7.3601</td>
</tr>
</tbody>
</table>

**Combination 1 Loading - Stresses**

**Combination 2 to 5 loading**

Applied bending moment  \( c2b(1)=1400 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress,beam and in-situ</td>
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</tr>
<tr>
<td>Dif. shrinkage Combination 2-5</td>
<td>-0.97688</td>
</tr>
<tr>
<td>Total</td>
<td>8.7496</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 Loading - Stresses**

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 8.7496 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².

Top of beam:
Compressive stress 17.462 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².

Bottom of in-situ concrete:
Tensile stress 0.64372 N/mm² is less than the maximum allowable tensile stress of 3.6 N/mm² from Table 32.
Bottom of beam:

Tensile stress 2.705 N/mm² is less than the maximum allowable tensile stress of 3.182 N/mm².
Precast prestressed concrete Box Beams

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam \( \text{span}=30 \text{ m} \)

Beam B 16 section properties

- Depth of beam \( d=1460 \text{ mm} \)
- Cross-sectional area \( A_c=678083 \text{ mm}^2 \)
- Centroid above soffit \( Y_b=703 \text{ mm} \)
- Section modulus top \( Z_t=223.47\times10^6 \text{ mm}^3 \)
- Section modulus bottom \( Z_b=240.63\times10^6 \text{ mm}^3 \)
- Unit weight of beam \( u_w=24.5 \text{ kN/m}^3 \)

Concrete strengths

- Strength at transfer \( f_{ci}=35 \text{ N/mm}^2 \)
- Characteristic strength \( f_{cu}=45 \text{ N/mm}^2 \)

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 35 \text{ N/mm}^2 is: \( E_{ci}=29.5 \text{ kN/mm}^2 \)
- Modulus of elasticity for concrete strength 45 \text{ N/mm}^2 is: \( E_{cb}=32.5 \text{ kN/mm}^2 \)

Composite section properties

- Depth of in-situ concrete \( d_{isc}=0 \text{ mm} \)
- Nominal diameter \( d_{nom}=12 \text{ mm} \)
- Nominal tensile strength \( f_{nomt}=1600 \text{ N/mm}^2 \)
- Nominal steel area \( A_{nom}=90 \text{ mm}^2 \)
- Characteristic breaking load \( c_{bl}=160 \text{ kN} \)
- Initial force per strand \( f_{i,t}=100 \text{ kN} \)
- Modulus of elasticity \( E_s=200 \text{ kN/mm}^2 \)

Cross-section 1

- Distance from mid-span of beam \( c_{sd}(1)=5.2 \text{ m} \)
- Number of layers of strand \( n_{ls}(1)=6 \)
- Height from soffit \( d_s(1)=70 \text{ mm} \)
- Number of strands \( n_s(1)=13 \)

WARNING:
Strand location is non-standard.

- Height from soffit \( d_s(2)=125 \text{ mm} \)
- Number of strands \( n_s(2)=13 \)

WARNING:
Strand location is non-standard.
Height from soffit: $ds(3)=600 \text{ mm}$
Number of strands: $ns(3)=2$
Height from soffit: $ds(4)=1000 \text{ mm}$
Number of strands: $ns(4)=2$
Height from soffit: $ds(5)=1300 \text{ mm}$
Number of strands: $ns(5)=2$
Height from soffit: $ds(6)=1400 \text{ mm}$
Number of strands: $ns(6)=2$
Bending moment from temp.loads: $tlb(1)=700 \text{ kNm}$
Bending moment from other loads: $olb(1)=600 \text{ kNm}$

**Short Term Losses**

Relaxation at transfer ($\%$): $str=0.75$

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force $\text{kN}$</th>
<th>% Loss $%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>25.5</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>161.59</td>
<td>4.7528</td>
</tr>
<tr>
<td><strong>Transfer Total</strong></td>
<td><strong>187.09</strong></td>
<td><strong>5.5028</strong></td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force: $3400 \text{ kN}$
Force at transfer: $Pt=P_0-loft=3212.9 \text{ kN}$

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer $\text{N/mm}^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.6606</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.3585</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>3.1325</td>
</tr>
<tr>
<td>Other Loads</td>
<td>2.685</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>12.515</strong></td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress $12.515 \text{ N/mm}^2$ is less than allowable stresses of $0.5*\text{fcI}=17.5 \text{ N/mm}^2$ and $0.4*\text{fcu}=18 \text{ N/mm}^2$.

Bottom of beam:
WARNING:
The flexural tensile stress 2.4841 N/mm² at transfer in the concrete due to prestress and co-existent dead and temporary loads during erection should not exceed 1 N/mm².
Clause 6.3.2.4(b)(1).

Long Term Losses

Relaxation after transfer (%) \( slr(1) = 0.5 \)
Shrinkage per unit length \( sul(1) = 0.3E-3 \)

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>25.5</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>161.59</td>
<td>4.7528</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>187.09</td>
<td>5.5028</td>
</tr>
<tr>
<td>Relaxation</td>
<td>17</td>
<td>0.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>183.6</td>
<td>5.4</td>
</tr>
<tr>
<td>Creep</td>
<td>81.726</td>
<td>2.4037</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>282.33</td>
<td>8.3037</td>
</tr>
<tr>
<td>All Losses</td>
<td>469.42</td>
<td>13.806</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 3400 kN
Final force \( Pf = P_0 - loftl = 2930.6 \) kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.60255</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.3585</td>
</tr>
<tr>
<td>Total</td>
<td>6.7559</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Combination 1 loading

Applied bending moment \( clb(1) = 1200 \) kNm
Loading | Comb 1. Stresses N/mm²  
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
<td>Bottom of Beam</td>
</tr>
<tr>
<td>Prestress,beam</td>
<td>6.7559</td>
<td>2.0614</td>
</tr>
<tr>
<td>Combination 1</td>
<td>6.265</td>
<td>-4.9869</td>
</tr>
<tr>
<td>Total</td>
<td>13.021</td>
<td>-2.9255</td>
</tr>
</tbody>
</table>

**Combination 1 Loading - Stresses**

**WARNING:**
Bottom of beam:
Tensile stress 2.9255 N/mm² exceeds maximum allowable
tensile stress of 0 N/mm².

**Combination 2 to 5 loading**

| Applied bending moment | c2b(1)=2250 kNm |

| Loading                | Comb. 2 - 5 Stresses N/mm²  
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
<td>Bottom of Beam</td>
</tr>
<tr>
<td>Prestress,beam</td>
<td>6.7559</td>
<td>2.0614</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>10.069</td>
<td>-9.3505</td>
</tr>
<tr>
<td>Total</td>
<td>16.825</td>
<td>-7.289</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 Loading - Stresses**

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of beam:
Compressive stress 16.825 N/mm² is less than the maximum allowable stress of 0.4*fcu=18 N/mm².

**WARNING:**
Bottom of beam:
Tensile stress 7.289 N/mm² exceeds maximum allowable
tensile stress of 3.0187 N/mm².
Location: Ex1 - 20 metre span box beam

Precast prestressed concrete Box Beams


Span of beam span=20 m

Beam B 7 section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=960 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=451208 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=476 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>2t=106.78E6 mm³</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>2b=108.58E6 mm³</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=25 kN/m³</td>
</tr>
</tbody>
</table>

Concrete compressive strengths

<table>
<thead>
<tr>
<th>Characteristic cylinder strength</th>
<th>fcki=32 N/mm² (transfer)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic cylinder strength</td>
<td>fck=40 N/mm²</td>
</tr>
</tbody>
</table>

Concrete moduli (short-term)

<table>
<thead>
<tr>
<th>Concrete modulus of elasticity for in-situ concrete</th>
<th>Ecmc=31 kN/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete modulus of elasticity</td>
<td>Ecmb=35 kN/mm²</td>
</tr>
</tbody>
</table>

Composite section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of in-situ concrete</td>
<td>disc=480 mm</td>
</tr>
<tr>
<td>Char strength (in-situ concrete)</td>
<td>fckc=25 N/mm²</td>
</tr>
<tr>
<td>Short-term modulus of elasticity for in-situ concrete</td>
<td>Ecmc=31 kN/mm²</td>
</tr>
<tr>
<td>Breadth of section</td>
<td>b=1000 mm</td>
</tr>
<tr>
<td>Modulus of elasticity (strand)</td>
<td>Ep=195 kN/mm²</td>
</tr>
</tbody>
</table>

Cross-section 1

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance from mid-span of beam</td>
<td>csd(1)=0 m</td>
</tr>
<tr>
<td>Number of layers of strand</td>
<td>nls(1)=4</td>
</tr>
<tr>
<td>Height from soffit</td>
<td>ds(1)=65 mm</td>
</tr>
<tr>
<td>Number of strands</td>
<td>ns(1)=10</td>
</tr>
<tr>
<td>Height from soffit</td>
<td>ds(2)=115 mm</td>
</tr>
<tr>
<td>Number of strands</td>
<td>ns(2)=6</td>
</tr>
<tr>
<td>Height from soffit</td>
<td>ds(3)=700 mm</td>
</tr>
<tr>
<td>Number of strands</td>
<td>ns(3)=2</td>
</tr>
<tr>
<td>Height from soffit</td>
<td>ds(4)=900 mm</td>
</tr>
<tr>
<td>Number of strands</td>
<td>ns(4)=2</td>
</tr>
</tbody>
</table>

Short Term Losses (BS EN 1992-1-1)

| Relaxation at transfer ( % ) | str=1                 |
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>45</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>278.21</td>
<td>6.1826</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>323.21</td>
<td>7.1826</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force: 4500 kN
Force at transfer: Pt=P0-loft=4176.8 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.48246</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.2817</td>
</tr>
<tr>
<td>Total</td>
<td>4.7993</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
Compressive stress 4.7993 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

Bottom of beam:
Compressive stress 13.641 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer( % ) slr(1)=1.5
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient ϕ(∞,to) crf=1.8
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>45</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>278.21</td>
<td>6.1826</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>323.21</td>
<td>7.1826</td>
</tr>
<tr>
<td>Relaxation</td>
<td>67.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>193.05</td>
<td>4.29</td>
</tr>
<tr>
<td>Creep</td>
<td>382.24</td>
<td>8.4943</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>642.79</td>
<td>14.284</td>
</tr>
<tr>
<td>All Losses</td>
<td>966.01</td>
<td>21.467</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 4500 kN
Final force Pf=P0-loftl=3534 kN
Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.40821</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.2817</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>0.61216</td>
</tr>
<tr>
<td>Total</td>
<td>5.4857</td>
</tr>
</tbody>
</table>

The net moment is Mc=F*e/1E3-Mb=-111.83 kNm

Stress at top of in-situ concrete:
\[ ftisds = \frac{F}{(Ai*mr)} - \frac{F}{(Ai*mr+Ac)} - Mc*1E3*(d-Yc)/Ic]*mr*1E3 = -0.98164 N/mm²

Stress at bottom of in-situ concrete:
\[ fbisds = \frac{F}{(Ai*mr)} - \frac{F}{(Ai*mr+Ac)} - Mc*1E3*(hsis-Yc)/Ic]*mr*1E3 = -0.11838 N/mm²

Stress at top of precast concrete beam:
\[ ftpbds = \frac{F}{(Ai*mr+Ac)} + Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)*1E3 = 0.14638 N/mm²

Stress at bottom of precast concrete beam:
fbpbds=(F/(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)*1E3=-0.01752 N/mm²

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment  \( C_{1b}(1) = 1100 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.98164</td>
</tr>
<tr>
<td>Combination 1</td>
<td>8.9021</td>
</tr>
<tr>
<td>Total</td>
<td>7.9205</td>
</tr>
</tbody>
</table>

**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment  \( C_{2b}(1) = 1400 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.98164</td>
</tr>
<tr>
<td>Combination 2</td>
<td>10.386</td>
</tr>
<tr>
<td>Total</td>
<td>9.4042</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 9.4042 N/mm² is less than the maximum allowable stress of 0.6\( f_{ckc} \)=15 N/mm².

Top of beam:
Compressive stress 17.358 N/mm² is less than the maximum allowable stress of 0.6\( f_{ck} \)=24 N/mm².

Bottom of in-situ concrete:
Tensile stress 0.53978 N/mm² is less than the maximum allowable tensile stress of 2.8721 N/mm² (EN 1992-1-1 expression 3.23).
Bottom of beam:
Tensile stress 2.5549 N/mm² is less than the maximum allowable
tensile stress of 2.846 N/mm².
Location: Ex2 - 30 metre span box beam B16

Precast prestressed concrete Box Beams


Span of beam  span=30 m

Beam B 16 section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of beam</td>
<td>d=1460 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>Ac=678083 mm²</td>
</tr>
<tr>
<td>Centroid above soffit</td>
<td>Yb=703 mm</td>
</tr>
<tr>
<td>Section modulus top</td>
<td>Zt=223.47E6 mm²</td>
</tr>
<tr>
<td>Section modulus bottom</td>
<td>Zb=240.63E6 mm²</td>
</tr>
<tr>
<td>Unit weight of beam</td>
<td>uwb=24.5 kN/m³</td>
</tr>
</tbody>
</table>

Concrete compressive strengths

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic cylinder strength</td>
<td>fcki=28 N/mm² (transfer)</td>
</tr>
<tr>
<td>Characteristic cylinder strength</td>
<td>fck=36 N/mm²</td>
</tr>
</tbody>
</table>

Concrete moduli (short-term)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete modulus of elasticity</td>
<td>Ecmi=32 kN/mm² (transfer)</td>
</tr>
<tr>
<td>Concrete modulus of elasticity</td>
<td>Ecmb=34 kN/mm²</td>
</tr>
</tbody>
</table>

Composite section properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of in-situ concrete</td>
<td>disc=0 mm</td>
</tr>
<tr>
<td>Nominal diameter</td>
<td>nomd=12 mm</td>
</tr>
<tr>
<td>Nominal tensile strength</td>
<td>nomt=1600 N/mm²</td>
</tr>
<tr>
<td>Nominal steel area</td>
<td>noma=90 mm²</td>
</tr>
<tr>
<td>Characteristic breaking load</td>
<td>fpk=160 kN</td>
</tr>
<tr>
<td>Initial force per strand</td>
<td>ipf=100 kN</td>
</tr>
<tr>
<td>Char breaking load (strand)</td>
<td>fpk=160 kN</td>
</tr>
<tr>
<td>Initial force per strand</td>
<td>ipf=100 kN</td>
</tr>
<tr>
<td>Modulus of elasticity (strand)</td>
<td>Ep=195 kN/mm²</td>
</tr>
</tbody>
</table>

Cross-section 1

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance from mid-span of beam</td>
<td>csd(1)=5.2 m</td>
</tr>
<tr>
<td>Number of layers of strand</td>
<td>nls(1)=6</td>
</tr>
<tr>
<td>Height from soffit</td>
<td>ds(1)=70 mm</td>
</tr>
<tr>
<td>Number of strands</td>
<td>ns(1)=13</td>
</tr>
</tbody>
</table>

WARNING: Strand location is non-standard.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height from soffit</td>
<td>ds(2)=125 mm</td>
</tr>
<tr>
<td>Number of strands</td>
<td>ns(2)=13</td>
</tr>
</tbody>
</table>

WARNING: Strand location is non-standard.
Height from soffit ds(3)=600 mm
Number of strands ns(3)=2
Height from soffit ds(4)=1000 mm
Number of strands ns(4)=2
Height from soffit ds(5)=1300 mm
Number of strands ns(5)=2
Height from soffit ds(6)=1400 mm
Number of strands ns(6)=2
Bending moment from temp.loads tlb(1)=700 kNm
Bending moment from other loads olb(1)=600 kNm

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer ( % ) str=0.75

Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>25.5</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>139.25</td>
<td>4.0956</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>164.75</td>
<td>4.8456</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 3400 kN
Force at transfer Pt=P0-loft=3235.2 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.66519</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.3585</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>3.1325</td>
</tr>
<tr>
<td>Other Loads</td>
<td>2.685</td>
</tr>
<tr>
<td>Total</td>
<td>12.511</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations ( BS EN 1992-1-1, Clause 5.10.2.2 )

Top of beam:
Compressive stress 12.511 N/mm² is less than allowable stresses of 0.7*fck=19.6 N/mm² and 0.6*fck=21.6 N/mm².

Bottom of beam:
WARNING:

The flexural tensile stress 2.4163 N/mm² at transfer in the concrete due to prestress and co-existent dead and temporary loads during erection should not exceed 1 N/mm² (as was permitted in BS5400-4).

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>25.5</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>139.25</td>
<td>4.0956</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>164.75</td>
<td>4.8456</td>
</tr>
<tr>
<td>Relaxation</td>
<td>17</td>
<td>0.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>179.01</td>
<td>5.265</td>
</tr>
<tr>
<td>Creep</td>
<td>78.529</td>
<td>2.3097</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>274.54</td>
<td>8.0747</td>
</tr>
<tr>
<td>All Losses</td>
<td>439.29</td>
<td>12.92</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 3400 kN
Final force Pf=P₀-loft₁=2960.7 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>-0.60875</td>
<td>8.9864</td>
<td>6.8341</td>
<td></td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>7.3585</td>
<td>-6.8336</td>
<td>-3.6501</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>6.7497</td>
<td>2.1529</td>
<td>3.184</td>
<td></td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Loading Combination 1

(no tensile stress permitted in precast beam)
Characteristic bending moment clb(1)=1200 kNm
### Loading Combination 1 Loading - Stresses

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
<th>Top of Beam</th>
<th>Bottom of beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam</td>
<td>6.7497</td>
<td>6.265</td>
<td>2.1529</td>
</tr>
<tr>
<td>Combination 1</td>
<td></td>
<td></td>
<td>-4.9869</td>
</tr>
<tr>
<td>Total</td>
<td>13.015</td>
<td></td>
<td>-2.834</td>
</tr>
</tbody>
</table>

**WARNING:**

Bottom of beam:
Tensile stress 2.834 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

### Loading Combination 2 Loading - Stresses

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
<th>Top of Beam</th>
<th>Bottom of beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam</td>
<td>6.7497</td>
<td>10.069</td>
<td>2.1529</td>
</tr>
<tr>
<td>Combination 2</td>
<td></td>
<td></td>
<td>-9.3505</td>
</tr>
<tr>
<td>Total</td>
<td>16.818</td>
<td></td>
<td>-7.1976</td>
</tr>
</tbody>
</table>

**NOTE:** The stresses from temperature differences should be added to the total stresses.

Top of beam:
Compressive stress 16.818 N/mm² is less than the maximum allowable stress of 0.6*fck=21.6 N/mm².

**WARNING:**

Bottom of beam:
Tensile stress 7.1976 N/mm² exceeds maximum allowable tensile stress of 2.7 N/mm².
Location: Ex1 - 23 metre span I beam

Precast prestressed concrete beam with non-standard section

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam                      span=23 m

Beam section properties

Depth of beam                     d=1425 mm
Cross-sectional area              Ac=411675 mm²
Centroid above soffit             Yb=595 mm
Section modulus top               Zt=113.14E6 mm³
Section modulus bottom            Zb=157.87E6 mm³
Unit weight of beam               uwb=24 kN/m³

Concrete strengths

Strength at transfer              fci=40 N/mm²
Characteristic strength           fcu=50 N/mm²

Concrete moduli

Modulus of elasticity at transfer for concrete strength 40 N/mm² is:
Eci=31 kN/mm²
Modulus of elasticity for concrete strength 50 N/mm² is:
Modulus                           Ecb=34 kN/mm²

Composite section properties

Char strength (in-situ concrete)  fcuc=30 N/mm²
Modulus of elasticity for concrete strength 30 N/mm² is:
Ecc=28 kN/mm²

Item No. 1

Item weighting                    iw1=0.82353
X co-ordinate                     X(1)=445 mm
Y co-ordinate                     Y(1)=0 mm
X co-ordinate                     X(2)=1055 mm
Y co-ordinate                     Y(2)=0 mm
X co-ordinate                     X(3)=1055 mm
Y co-ordinate                     Y(3)=200 mm
X co-ordinate                     X(4)=825 mm
Y co-ordinate                     Y(4)=430 mm
X co-ordinate                     X(5)=825 mm
Y co-ordinate                     Y(5)=1120 mm
X co-ordinate                     X(6)=930 mm
Y co-ordinate                     Y(6)=1125 mm
X co-ordinate                     X(7)=930 mm
Y co-ordinate                     Y(7)=1425 mm
X co-ordinate                     X(8)=1500 mm
Y co-ordinate                     Y(8)=1425 mm
X co-ordinate                     X(9)=1500 mm
Y co-ordinate                     Y(9)=1605 mm
X co-ordinate                     X(10)=0 mm
Y co-ordinate                     Y(10)=1605 mm
X co-ordinate                     X(11)=0 mm
Y co-ordinate                     Y(11)=1425 mm
X co-ordinate                     X(12)=570 mm
Y co-ordinate                     Y(12)=1425 mm
X co-ordinate                     X(13)=570 mm
Y co-ordinate                     Y(13)=1225 mm
X co-ordinate                     X(14)=675 mm
Y co-ordinate                     Y(14)=1120 mm
X co-ordinate                     X(15)=675 mm
Y co-ordinate                     Y(15)=430 mm
X co-ordinate                     X(16)=445 mm
Y co-ordinate                     Y(16)=200 mm
X co-ordinate                     X(17)=445 mm
Y co-ordinate                     Y(17)=0 mm

**Item No. 2**

Second moment of area             IXX2=93.933E9 mm^4
Area of item                      A2=411675 mm^2
Centroid from X-X axis            YC2=595 mm
Item weighting                    iw2=0.17647
Depth of composite section        di=1605 mm
Initial force per strand          if=159 kN
Modulus of elasticity             Es=195 kN/mm^2

**Cross-section 1**

Distance from mid-span of beam     csd(1)=0 m
Number of layers of strand        nls(1)=5
Height above soffit               ds(1)=40 mm
Number of strands                 ns(1)=14
Height above soffit               ds(2)=80 mm
Number of strands                 ns(2)=14
Height above soffit               ds(3)=120 mm
Number of strands                 ns(3)=2
Height above soffit               ds(4)=160 mm
Number of strands                 ns(4)=2
Height above soffit               ds(5)=1385 mm
Number of strands                 ns(5)=4

**Short Term Losses**

Relaxation at transfer ( % )       str=1
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force (kN)</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>57.24</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>705.89</td>
<td>12.332</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>763.13</td>
<td>13.332</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 5724 kN
Force at transfer Pt=P0-loft=4960.9 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.5581</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.7729</td>
</tr>
<tr>
<td>Total</td>
<td>1.2148</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 1.2148 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

Bottom of beam:
Compressive stress 19.818 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

**Long Term Losses**

Relaxation after transfer ( % ) slr(1)=1
Shrinkage per unit length sul(1)=0.3E-3
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>57.24</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>705.89</td>
<td>12.332</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>763.13</td>
<td>13.332</td>
</tr>
<tr>
<td>Relaxation</td>
<td>57.24</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>292.73</td>
<td>5.1142</td>
</tr>
<tr>
<td>Creep</td>
<td>989.68</td>
<td>17.29</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>1339.7</td>
<td>23.404</td>
</tr>
<tr>
<td>All Losses</td>
<td>2102.8</td>
<td>36.736</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force: 5724 kN
Final force: Pf=P0-loftl=3621.2 kN
Unit-weight of in-situ concrete: wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.3272</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>5.7729</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.9439</td>
</tr>
<tr>
<td>Total</td>
<td>6.3896</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete: hsis=1425 mm
Percentage of shrinkage remaining: ps=25 %
Percentage: pcr=25
Percentage: stT=50 %
The net moment is: Mc=F*e*1E3-Mb=-367.6 kNm
Stress at top of in-situ concrete:
\[ ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3 \]
\[ =-0.41487 \text{ N/mm}^2 \]
Stress at bottom of in-situ concrete:
\[ fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3 \]
\[ =-0.16346 \text{ N/mm}^2 \]
Stress at top of precast concrete beam:
\[
ftpbds = \frac{F}{(Ai*mr+Ac)} + \frac{Mc*1E3*(d-Yc)}{Ic} + \frac{Mb*1E3*Yt/Ixx}{1E3} = 0.83341 \text{ N/mm}^2
\]
Stress at bottom of precast concrete beam:
\[
fbpbds = \frac{F}{(Ai*mr+Ac)} - \frac{Mc*1E3*Yc/Ic}{Ic} - \frac{Mb*1E3*Yb/Ixx}{Ixx} = -0.27209 \text{ N/mm}^2
\]

**Combination 1 loading**

Applied bending moment \( c_{lb}(1) = 2300 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses ( \text{N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.41487</td>
</tr>
<tr>
<td>Combination 1</td>
<td>6.2685</td>
</tr>
<tr>
<td>Total</td>
<td>5.8536</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses

**Combination 2 to 5 loading**

Applied bending moment \( c_{2b}(1) = 2600 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses ( \text{N/mm}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.41487</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>6.7909</td>
</tr>
<tr>
<td>Total</td>
<td>6.376</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 6.376 \( \text{N/mm}^2 \) is less than the maximum allowable stress of 0.5*fcu\(_{\text{uc}}\)=15 \( \text{N/mm}^2 \).
Top of beam:
Compressive stress 13.31 \( \text{N/mm}^2 \) is less than the maximum allowable stress of 0.4*fcu\(_{\text{cu}}\)=20 \( \text{N/mm}^2 \).
Bottom of in-situ concrete:
Compressive stress $4.8492 \text{ N/mm}^2$ is less than the maximum allowable stress of $0.5 \times f_{cu,c}=15 \text{ N/mm}^2$.

Bottom of beam:
Tensile stress $0.75768 \text{ N/mm}^2$ is less than the maximum allowable tensile stress of $3.182 \text{ N/mm}^2$. 
Location: Ex2 - 20 metre span beam

Precast prestressed concrete beam with non-standard section

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=20 m

Beam section properties

- Depth of beam \( d = 1040 \, \text{mm} \)
- Cross-sectional area \( A_c = 387050 \, \text{mm}^2 \)
- Centroid above soffit \( Y_b = 409 \, \text{mm} \)
- Section modulus top \( Z_t = 75.39 \times 10^6 \, \text{mm}^3 \)
- Section modulus bottom \( Z_b = 116.23 \times 10^6 \, \text{mm}^3 \)
- Unit weight of beam \( u_{wb} = 24 \, \text{kN/m}^3 \)

Concrete strengths

- Strength at transfer \( f_{ci} = 40 \, \text{N/mm}^2 \)
- Characteristic strength \( f_{cu} = 50 \, \text{N/mm}^2 \)

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 40 N/mm² is: \( E_{ci} = 31 \, \text{kN/mm}^2 \)
- Modulus of elasticity for concrete strength 50 N/mm² is: \( E_{cb} = 34 \, \text{kN/mm}^2 \)

Composite section properties

- Char strength (in-situ concrete) \( f_{cuc} = 40 \, \text{N/mm}^2 \)
- Modulus of elasticity for concrete strength 40 N/mm² is: \( E_{cc} = 31 \, \text{kN/mm}^2 \)
- Depth of composite section \( d_i = 1165 \, \text{mm} \)
- Area of in-situ concrete \( A_i = 229500 \, \text{mm}^2 \)
- Centroid of in-situ concrete \( Y_i = 1087.9 \, \text{mm} \)
- Centroid of composite section \( Y_c = 661.69 \, \text{mm} \)
- Second moment of area \( I_c = 114.4 \times 10^9 \, \text{mm}^4 \)
- Initial force per strand \( i_f = 174 \, \text{kN} \)
- Modulus of elasticity \( E_s = 195 \, \text{kN/mm}^2 \)

Cross-section 1

- Distance from mid-span of beam \( c_{sd(1)} = 0 \, \text{m} \)
- Number of layers of strand \( n_{ls(1)} = 4 \)
- Height above soffit \( d_s(1) = 60 \, \text{mm} \)
- Number of strands \( n_s(1) = 11 \)
- Height above soffit \( d_s(2) = 110 \, \text{mm} \)
- Number of strands \( n_s(2) = 14 \)
- Height above soffit \( d_s(3) = 880 \, \text{mm} \)
- Number of strands \( n_s(3) = 2 \)
- Height above soffit \( d_s(4) = 930 \, \text{mm} \)
Number of strands \( ns(4) = 2 \)

**Short Term Losses**

Relaxation at transfer ( % ) \( \text{str}=1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>442.89</td>
<td>8.7771</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>493.35</td>
<td>9.7771</td>
</tr>
</tbody>
</table>

**Losses at Transfer**

Initial force 5046 kN

Force at transfer \( Pt=P0-\text{loft}=4552.6 \) kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.82575</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.165</td>
</tr>
<tr>
<td>Total</td>
<td>5.3393</td>
</tr>
</tbody>
</table>

**Stress Limitations**

Top of beam:
Compressive stress 5.3393 N/mm\(^2\) is less than allowable stresses of 0.5*\(fci=20\) N/mm\(^2\) and 0.4*\(fcu=20\) N/mm\(^2\).

Bottom of beam:
Compressive stress 15.926 N/mm\(^2\) is less than allowable stresses of 0.5*\(fci=20\) N/mm\(^2\) and 0.4*\(fcu=20\) N/mm\(^2\).

**Long Term Losses**

Relaxation after transfer( % ) \( \text{slr}(1)=2 \)

Shrinkage per unit length \( \text{sul}(1)=0.3\text{E}-3 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>442.89</td>
<td>8.7771</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>493.35</td>
<td>9.7771</td>
</tr>
<tr>
<td>Relaxation</td>
<td>100.92</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>235.81</td>
<td>4.6733</td>
</tr>
<tr>
<td>Creep</td>
<td>574.73</td>
<td>11.39</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>911.46</td>
<td>18.063</td>
</tr>
<tr>
<td>All Losses</td>
<td>1404.8</td>
<td>27.84</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 5046 kN
Final force Pf=P0-loftl=3641.2 kN
Unit-weight of in-situ concrete wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.66043</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.165</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.6555</td>
</tr>
<tr>
<td>Total</td>
<td>9.1602</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=1005 mm
Perc'age of shrinkage remaining ps=25 %
Percentage pcr=25
Percentage stT=67 %
The net moment is Mc=F*e/1E3-Mb=-85.455 kNm
Stress at top of in-situ concrete:
\[ ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3 =-0.030723 \text{ N/mm}^2 \]
Stress at bottom of in-situ concrete:
\[ fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3 =0.088794 \text{ N/mm}^2 \]
Stress at top of precast concrete beam:

\[ \text{ftpbd} = \frac{F}{(A_i \times m_r+ A_c)} + M_c \times 1E3 \times (d-Y_c)/I_c + M_b \times 1E3 \times Y_t/I_{xx} \times 1E3 \]

\[ = -0.067041 \text{ N/mm}^2 \]

Stress at bottom of precast concrete beam:

\[ \text{fbpdb} = \frac{F}{(A_i \times m_r+ A_c)} - M_c \times 1E3 \times Y_c/I_c - M_b \times 1E3 \times Y_b/I_{xx} \times 1E3 = 0.017159 \text{ N/mm}^2 \]

**Combination 1 loading**

Applied bending moment \( c_{lb}(1)=1446 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.030723</td>
</tr>
<tr>
<td>Combination 1</td>
<td>9.1291</td>
</tr>
<tr>
<td>Total</td>
<td>9.0984</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 loading**

Applied bending moment \( c_{2b}(1)=1907 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.030723</td>
</tr>
<tr>
<td>Combination 2-5</td>
<td>8.39</td>
</tr>
<tr>
<td>Total</td>
<td>8.3592</td>
</tr>
</tbody>
</table>

**Combination 3 loading.** The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 8.3592 N/mm² is less than the maximum allowable stress of 0.5*fucc=20 N/mm².

Top of beam:
Compressive stress 15.399 N/mm² is less than the maximum allowable stress of 0.4*fucc=20 N/mm².
Bottom of in-situ concrete:
Compressive stress 5.8116 N/mm² is less than the maximum allowable stress of 0.5*f_{cuc}=20 N/mm².
Bottom of beam:
Tensile stress 1.445 N/mm² is less than the maximum allowable tensile stress of 3.182 N/mm².
Location: Ex3 - 13.5 metre span solid edge beam

Precast prestressed concrete beam with non-standard section

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam span=13.5 m

Beam section properties

- Depth of beam d=615 mm
- Cross-sectional area Ac=130675 mm²
- Centroid above soffit Yb=244 mm
- Section modulus top Zt=14.3E6 mm⁴
- Section modulus bottom Zb=21.81E6 mm⁴
- Unit weight of beam uwb=25 kN/m³

Concrete strengths

- Strength at transfer fci=40 N/mm²
- Characteristic strength fcu=50 N/mm²

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 40 N/mm² is: Eci=31 kN/mm²
- Modulus of elasticity for concrete strength 50 N/mm² is: Ecb=34 kN/mm²

Composite section properties

- Char strength (in-situ concrete) fcuc=30 N/mm²
- Modulus of elasticity for concrete strength 30 N/mm² is: Ecc=28 kN/mm²

Item No. 1

- Item weighting iw1=0.82353
- X co-ordinate X{(1)}=0 mm
- Y co-ordinate Y{(1)}=0 mm
- X co-ordinate X{(2)}=508 mm
- Y co-ordinate Y{(2)}=0 mm
- X co-ordinate X{(3)}=508 mm
- Y co-ordinate Y{(3)}=250 mm
- X co-ordinate X{(4)}=650 mm
- Y co-ordinate Y{(4)}=350 mm
- X co-ordinate X{(5)}=650 mm
- Y co-ordinate Y{(5)}=790 mm
- X co-ordinate X{(6)}=150 mm
- Y co-ordinate Y{(6)}=790 mm
- X co-ordinate X{(7)}=150 mm
- Y co-ordinate Y{(7)}=690 mm
- X co-ordinate X{(8)}=0 mm
**Item No. 2**

- **Second moment of area**: $I_{XX2} = 5.3216E9 \text{ mm}^4$
- **Area of item**: $A_2 = 130675 \text{ mm}^2$
- **Centroid from X-X axis**: $Y_{C2} = 244 \text{ mm}$
- **Item weighting**: $iw_2 = 0.17647$
- **Depth of composite section**: $d_i = 790 \text{ mm}$
- **Nominal diameter**: $nomd = 12 \text{ mm}$
- **Nominal tensile strength**: $nomt = 1700 \text{ N/mm}^2$
- **Nominal steel area**: $noma = 100 \text{ mm}^2$
- **Characteristic breaking load**: $cbl = 150 \text{ kN}$
- **Initial force per strand**: $if = 100 \text{ kN}$
- **Modulus of elasticity**: $E_s = 195 \text{ kN/mm}^2$

**Cross-section 1**

- **Distance from mid-span of beam**: $csd(1) = 2 \text{ m}$
- **Number of layers of strand**: $nls(1) = 4$
- **Height above soffit**: $ds(1) = 49 \text{ mm}$
- **Number of strands**: $ns(1) = 10$
- **Height above soffit**: $ds(2) = 85 \text{ mm}$
- **Number of strands**: $ns(2) = 8$
- **Height above soffit**: $ds(3) = 520 \text{ mm}$
- **Number of strands**: $ns(3) = 2$
- **Height above soffit**: $ds(4) = 580 \text{ mm}$
- **Number of strands**: $ns(4) = 2$
- **Bending moment from temp.loads**: $tlb(1) = 80 \text{ kNm}$
- **Bending moment from other loads**: $olb(1) = 50 \text{ kNm}$

**Short Term Losses**

- **Relaxation at transfer (%)**: $str = 1$

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force (kN)</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>277.37</td>
<td>12.608</td>
</tr>
<tr>
<td><strong>Transfer Total</strong></td>
<td>299.37</td>
<td>13.608</td>
</tr>
</tbody>
</table>

**Losses at Transfer**

- **Initial force**: 2200 kN
- **Force at transfer**: $P_t = P_0 - loft = 1900.6 \text{ kN}$
<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>2.511</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.733</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>5.5772</td>
</tr>
<tr>
<td>Other Loads</td>
<td>3.4858</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>16.307</strong></td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations**

Top of beam:
Compressive stress 16.307 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².
Bottom of beam:
Compressive stress 13.386 N/mm² is less than allowable stresses of 0.5*fci=20 N/mm² and 0.4*fcu=20 N/mm².

**Long Term Losses**

Relaxation after transfer( % )   slr(1)=1.5
Shrinkage per unit length        sul(1)=0.3E-3

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>277.37</td>
<td>12.608</td>
</tr>
<tr>
<td><strong>Transfer Total</strong></td>
<td><strong>299.37</strong></td>
<td><strong>13.608</strong></td>
</tr>
<tr>
<td>Relaxation</td>
<td>33</td>
<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>128.7</td>
<td>5.85</td>
</tr>
<tr>
<td>Creep</td>
<td>363.2</td>
<td>16.509</td>
</tr>
<tr>
<td><strong>Long-term Total</strong></td>
<td><strong>524.9</strong></td>
<td><strong>23.859</strong></td>
</tr>
<tr>
<td><strong>All Losses</strong></td>
<td><strong>824.27</strong></td>
<td><strong>37.467</strong></td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2200 kN
Final force Pf=P0-loftl=1375.7 kN
Unit-weight of in-situ concrete wc=25 kN/m³
Loading | Final Stresses N/mm²
---|---
| Top of Beam | Bottom of Beam | Centroid of Strands |
---|---|---|---|
Prestress | 1.8175 | 16.256 | 12.66 |
Self weight of Beam | 4.733 | -3.1128 | -1.1586 |
In-situ concrete | 11.779 | -7.7471 | -2.8835 |
---|---|---|---|
Total | 18.33 | 5.3966 | 8.618 |

Beam concrete stresses - Long term

**Composite section**

Height from beam soffit to bottom of in-situ concrete: $h_{sis}=100$ mm

Percentage of shrinkage remaining: $p_s=40\%$

Percentage: $p_{cr}=40$

Percentage: $sT=50\%$

The net moment is $M_c=F*e/1E3-M_b=-75.167$ kNm

Stress at top of in-situ concrete:

$$f_{tisds}=-(F/(A_i*mr)-F/(A_i*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3$$

$$=0.60297 \text{ N/mm}^2$$

Stress at bottom of in-situ concrete:

$$f_{bisds}=-(F/(A_i*mr)-F/(A_i*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3$$

$$=2.7803 \text{ N/mm}^2$$

Stress at top of precast concrete beam:

$$f_{tpbds}=-(F/(A_i*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Ixx)*1E3=-9.696 \text{ N/mm}^2$$

Stress at bottom of precast concrete beam:

$$f_{bpbds}=(F/(A_i*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Ixx)*1E3=-0.25506 \text{ N/mm}^2$$

**Combination 1 loading**

Applied bending moment $c_{lb(1)}=550$ kNm

<p>| Loading | Combination 1 Stresses N/mm² |
|---|---|---|---|---|</p>
<table>
<thead>
<tr>
<th></th>
<th>Top of in-situ</th>
<th>Top of Beam</th>
<th>Bottom of in-situ</th>
<th>Bottom of Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
<td>18.33</td>
<td>0</td>
<td>5.3966</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>0.60297</td>
<td>-9.696</td>
<td>2.7803</td>
<td>-0.25506</td>
</tr>
<tr>
<td>Combination 1</td>
<td>9.8989</td>
<td>6.6675</td>
<td>-6.858</td>
<td>-11.131</td>
</tr>
<tr>
<td>Total</td>
<td>10.502</td>
<td>15.301</td>
<td>-4.0777</td>
<td>-5.9899</td>
</tr>
</tbody>
</table>

Combination 1 Loading - Stresses
WARNING:
Bottom of beam:
Tensile stress 5.9899 N/mm² exceeds maximum allowable
tensile stress of 0 N/mm².

Combination 2 to 5 loading

Applied bending moment  c2b(1)=625 kNm

WARNING:
Bottom of beam:
Tensile stress 7.5078 N/mm² exceeds modified allowable
tensile stress of 1.382 N/mm².

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam</td>
<td>0</td>
</tr>
<tr>
<td>and in-situ</td>
<td></td>
</tr>
<tr>
<td>Dif. shrinkage Combination 2-5</td>
<td>0.60297</td>
</tr>
<tr>
<td></td>
<td>10.311</td>
</tr>
<tr>
<td>Total</td>
<td>10.914</td>
</tr>
</tbody>
</table>

Combination 2 to 5 Loading - Stresses

Combination 3 loading. The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 10.914 N/mm² is less than the maximum allowable stress of 0.5*fcuc=15 N/mm².
Top of beam:
Compressive stress 15.579 N/mm² is less than the maximum allowable stress of 0.4*fcu=20 N/mm².
Bottom of in-situ concrete:
Tensile stress 5.0129 N/mm² is less than the modified maximum allowable tensile stress of 5.4 N/mm² from Table 32.

WARNING:
Bottom of beam:
Tensile stress 7.5078 N/mm² exceeds modified max allowable
tensile stress of 1.382 N/mm².
Location: Ex4 - 17.0 metre span solid section

Precast prestressed concrete beam with non-standard section

Calculations for stresses at the Serviceability Limit State (SLS) to BS5400-4:1990 implemented by DOT Departmental Standard BD 44/15.

Span of beam: span=17 m

Beam section properties

- Depth of beam: d=775 mm
- Cross-sectional area: Ac=163360 mm²
- Centroid above soffit: Yb=334 mm
- Section modulus top: Zt=24.31E6 mm⁴
- Section modulus bottom: Zb=32.1E6 mm⁴
- Unit weight of beam: uwb=25 kN/m³

Concrete strengths

- Strength at transfer: fci=35 N/mm²
- Characteristic strength: fcu=40 N/mm²

Concrete moduli

- Modulus of elasticity at transfer for concrete strength 35 N/mm² is: Eci=29.5 kN/mm²
- Modulus of elasticity for concrete strength 40 N/mm² is: Ecb=31 kN/mm²

Composite section properties

- Char strength (in-situ concrete): fcuc=35 N/mm²
- Modulus of elasticity for concrete strength 35 N/mm² is: Ecc=29.5 kN/mm²
- Depth of composite section: di=850 mm
- Area of in-situ concrete: Ai=304140 mm²
- Centroid of composite section: Yc=425 mm
- Second moment of area: Ic=28.147E9 mm⁶
- Initial force per strand: if=200 kN
- Modulus of elasticity: Es=195 kN/mm²

Cross-section 1

- Distance from mid-span of beam: csd(1)=6 m
- Number of layers of strand: nls(1)=6
- Height above soffit: ds(1)=50 mm
- Number of strands: ns(1)=4
- Height above soffit: ds(2)=90 mm
- Number of strands: ns(2)=3
- Height above soffit: ds(3)=260 mm
- Number of strands: ns(3)=1
- Height above soffit: ds(4)=500 mm
- Number of strands: ns(4)=1
Height above soffit          \( ds(5) = 620 \text{ mm} \)
Number of strands          \( ns(5) = 1 \)
Height above soffit          \( ds(6) = 740 \text{ mm} \)
Number of strands          \( ns(6) = 1 \)

**Short Term Losses**

Relaxation at transfer ( % )  \( str = 1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>248.17</td>
<td>11.281</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>270.17</td>
<td>12.281</td>
</tr>
</tbody>
</table>

**Losses at Transfer**

Initial force  2200 kN
Force at transfer  \( Pt = P_0 - \text{loft} = 1929.8 \text{ kN} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>3.9909</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.0447</td>
</tr>
<tr>
<td>Total</td>
<td>7.0356</td>
</tr>
</tbody>
</table>

**Beam Concrete Stresses at Transfer**

**Stress Limitations**

Top of beam:  
Compressive stress 7.0356 N/mm² is less than allowable stresses of 0.5*fci=17.5 N/mm² and 0.4*fcu=16 N/mm².

Bottom of beam:  
Compressive stress 15.432 N/mm² is less than allowable stresses of 0.5*fci=17.5 N/mm² and 0.4*fcu=16 N/mm².

**Long Term Losses**

Relaxation after transfer( % )  \( slr(1) = 2 \)
Shrinkage per unit length       \( sul(1) = 0.3E-3 \)
**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22 kN</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>248.17 kN</td>
<td>11.281</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>270.17 kN</td>
<td>12.281</td>
</tr>
<tr>
<td>Relaxation</td>
<td>44 kN</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>143.5 kN</td>
<td>6.5228</td>
</tr>
<tr>
<td>Creep</td>
<td>392.54 kN</td>
<td>17.843</td>
</tr>
<tr>
<td>Long-term Total</td>
<td>580.04 kN</td>
<td>26.365</td>
</tr>
<tr>
<td>All Losses</td>
<td>850.21 kN</td>
<td>38.646</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2200 kN
Final force Pf=P0-loftl=1349.8 kN
Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>2.7914</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.0447</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>5.6686</td>
</tr>
<tr>
<td>Total</td>
<td>11.505</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

**Composite section**

Height from beam soffit to bottom of in-situ concrete hsis=100 mm

** Combination 1 loading**

Applied bending moment clb(1)=200 kNm
**Combination 1 Loading - Stresses**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Combination 1</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>3.3973</td>
</tr>
</tbody>
</table>

**Combination 2 to 5 Loading**

Applied bending moment $c2b(1)=250$ kNm

<table>
<thead>
<tr>
<th>Loading</th>
<th>Combination 2 to 5 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ Combination 2-5</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>3.7748</td>
</tr>
</tbody>
</table>

**Combination 3 loading.** The stresses from the Proforma for temperature differences should be added to the total stresses.

Top of in-situ concrete:
- Compressive stress 3.7748 N/mm² is less than the maximum allowable stress of $0.5\times f_{cu}=17.5$ N/mm².
- Top of beam:
- Compressive stress 14.613 N/mm² is less than the maximum allowable stress of $0.4\times f_{cu}=16$ N/mm².
Bottom of in-situ concrete:
- Tensile stress 2.8866 N/mm² is less than the maximum allowable tensile stress of 4 N/mm² from Table 32.
- Bottom of beam:
- Compressive stress 2.0324 N/mm² is less than the maximum allowable stress of $0.4\times f_{cu}=16$ N/mm².
Precast prestressed concrete beam with non-standard section


Beam span under consideration span=24 m

Beam section properties

Depth of beam \( d = 1425 \) mm
Cross-sectional area \( A_c = 411675 \) mm\(^2\)
Centroid above soffit \( y_b = 595 \) mm
Section modulus top \( Z_t = 113.14E6 \) mm\(^3\)
Section modulus bottom \( Z_b = 157.87E6 \) mm\(^3\)
Unit weight of beam \( u_wb = 24 \) kN/m\(^3\)

Concrete compressive strengths

Characteristic cylinder strength \( f_{ck} = 32 \) N/mm\(^2\) (transfer)
Characteristic cylinder strength \( f_{ck} = 40 \) N/mm\(^2\)

Concrete moduli (short-term)

Concrete modulus of elasticity \( E_{cmi} = 33 \) kN/mm\(^2\) (transfer)
Concrete modulus of elasticity \( E_{cmb} = 35 \) kN/mm\(^2\)

Composite section properties

Char strength (in-situ concrete) \( f_{ckc} = 25 \) N/mm\(^2\)
Short-term modulus of elasticity for in-situ concrete \( E_{cmc} = 31 \) kN/mm\(^2\)

Item No. 1

Item weighting \( iw_1 = 0.88571 \)
X co-ordinate \( X(1) = 445 \) mm
Y co-ordinate \( Y(1) = 0 \) mm
X co-ordinate \( X(2) = 1055 \) mm
Y co-ordinate \( Y(2) = 0 \) mm
X co-ordinate \( X(3) = 1055 \) mm
Y co-ordinate \( Y(3) = 200 \) mm
X co-ordinate \( X(4) = 825 \) mm
Y co-ordinate \( Y(4) = 430 \) mm
X co-ordinate \( X(5) = 825 \) mm
Y co-ordinate \( Y(5) = 1120 \) mm
X co-ordinate \( X(6) = 930 \) mm
Y co-ordinate \( Y(6) = 1225 \) mm
X co-ordinate \( X(7) = 930 \) mm
Y co-ordinate \( Y(7) = 1425 \) mm
X co-ordinate \( X(8) = 1500 \) mm
Y co-ordinate \( Y(8) = 1425 \) mm
X co-ordinate \( X(9) = 1500 \) mm
Item No. 2

Second moment of area $I_{XX2}=93.933\times 10^9$ mm$^4$
Area of item $A_2=411675$ mm$^2$
Centroid from X-X axis $Y_{C2}=595$ mm
Item weighting $i_w_2=0.11429$
Depth of composite section $d_i=1605$ mm
Modulus of elasticity (strand) $E_p=195$ kN/mm$^2$

Cross-section 1

Distance from mid-span of beam $c_{sd(1)}=0$ m
Number of layers of strand $n_{ls(1)}=5$
Height above soffit $d_{s(1)}=40$ mm
Number of strands $n_s(1)=14$
Height above soffit $d_{s(2)}=80$ mm
Number of strands $n_s(2)=14$
Height above soffit $d_{s(3)}=120$ mm
Number of strands $n_s(3)=2$
Height above soffit $d_{s(4)}=160$ mm
Number of strands $n_s(4)=2$
Height above soffit $d_{s(5)}=1385$ mm
Number of strands $n_s(5)=4$

Short Term Losses (BS EN 1992-1-1)

Relaxation at transfer (%) $\text{str}=1$
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>57.24</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>593.64</td>
<td>10.371</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>650.88</td>
<td>11.371</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 5724 kN
Force at transfer Pt=P0-loft=5073.1 kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-4.6612</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.2858</td>
</tr>
<tr>
<td>Total</td>
<td>1.6246</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations ( BS EN 1992-1-1, Clause 5.10.2.2 )**

Top of beam:
Compressive stress 1.6246 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².
Bottom of beam:
Compressive stress 19.993 N/mm² is less than allowable stresses of 0.7*fcki=22.4 N/mm² and 0.6*fck=24 N/mm².

**Long Term Losses ( BS EN 1992-1-1, Clause 5.10.6(1) )**

Relaxation after transfer( % ) slr(1)=1
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(∞,to) crf=1.8
## Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>57.24</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>593.64</td>
<td>10.371</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>650.88</td>
<td>11.371</td>
</tr>
<tr>
<td>Relaxation</td>
<td>57.24</td>
<td>1</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>292.73</td>
<td>5.1142</td>
</tr>
<tr>
<td>Creep</td>
<td>1122.3</td>
<td>19.607</td>
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<tr>
<td>Long-term Total</td>
<td>1472.3</td>
<td>25.721</td>
</tr>
<tr>
<td>All Losses</td>
<td>2123.2</td>
<td>37.092</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 5724 kN
Final force Pf=P0-loftl=3600.8 kN
Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-3.3085</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.2858</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>4.2944</td>
</tr>
<tr>
<td>Total</td>
<td>7.2717</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

**Composite section**

Height from beam soffit to bottom of in-situ concrete: hsis=1425 mm
Per centage of shrinkage remaining: ps=25 %
Percentage: pcr=25%
Percentage: stT=50 %
The net moment is Mc=F*e/1E3-Mb=-462.11 kNm
Stress at top of in-situ concrete:
\[ ftisds=\frac{-F}{(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3}{1E3} \]
\[-0.28831 \text{ N/mm}^2 \]
Stress at bottom of in-situ concrete:
\[ fbisds=\frac{-F}{(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3}{1E3} \]
\[=0.042621 \text{ N/mm}^2 \]
Stress at top of precast concrete beam:
\[ ftpbds = \left( \frac{F}{(A_i + m_r + A_c)} \right) + Mc1E3(d-Yc)/Ic + Mb1E3*Yt/Iyy) * 1E3 = 0.3622 \text{ N/mm}^2 \]

Stress at bottom of precast concrete beam:
\[ fbpbds = \left( \frac{F}{(A_i + m_r + A_c)} \right) - Mc1E3(Yc/Ic - Mb1E3*Yb/Iyy) * 1E3 = -0.12167 \text{ N/mm}^2 \]

**Loading Combination 1**

(no tensile stress permitted in precast beam)

Characteristic bending moment \( c_{lb}(1) = 2300 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.28831</td>
</tr>
<tr>
<td>Combination 1</td>
<td>6.4166</td>
</tr>
<tr>
<td>Total</td>
<td>6.1282</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses

**Loading Combination 2**

(tensile stress permitted (no visible cracking))

Characteristic bending moment \( c_{lb}(1) = 2600 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
<td>-0.28831</td>
</tr>
<tr>
<td>Combination 2</td>
<td>6.9513</td>
</tr>
<tr>
<td>Total</td>
<td>6.663</td>
</tr>
</tbody>
</table>

Loading Combination 2 - Stresses

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 6.663 N/mm\(^2\) is less than the maximum allowable stress of 0.6*fckc=15 N/mm\(^2\).

Top of beam:
Compressive stress 13.38 N/mm\(^2\) is less than the maximum allowable stress of 0.6*fck=24 N/mm\(^2\).
Bottom of in-situ concrete:
Compressive stress 5.132 N/mm² is less than the maximum allowable stress of 0.6*fckc=15 N/mm².

Bottom of beam:
Tensile stress 1.2136 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
Location: Ex2 - 20 metre span beam

Precast prestressed concrete beam with non-standard section


Beam span under consideration  span=20 m

Beam section properties

Depth of beam  d=1040 mm
Cross-sectional area  Ac=387050 mm²
Centroid above soffit  Yb=409 mm
Section modulus top  Zt=75.39E6 mm⁴
Section modulus bottom  Zb=116.23E6 mm⁴
Unit weight of beam  uwb=24 kN/m³

Concrete compressive strengths

Characteristic cylinder strength  fcki=32 N/mm² (transfer)
Characteristic cylinder strength  fck=40 N/mm²

Concrete moduli (short-term)

Concrete modulus of elasticity  Ecmi=33 kN/mm² (transfer)
Concrete modulus of elasticity  Ecmb=35 kN/mm²

Composite section properties

Char strength (in-situ concrete)  fckc=32 N/mm²
Short-term modulus of elasticity for in-situ concrete  Ecmc=33 kN/mm²
Depth of composite section  di=1165 mm
Area of in-situ concrete  Ai=229500 mm²
Centroid of in-situ concrete  Yi=1087.9 mm
Centroid of composite section  Yc=661.69 mm
Second moment of area  Ic=114.4E9 mm⁶
Modulus of elasticity (strand)  Ep=195 kN/mm²

Cross-section 1

Distance from mid-span of beam  csd(1)=0 m
Number of layers of strand  nls(1)=4
Height above soffit  ds(1)=60 mm
Number of strands  ns(1)=11
Height above soffit  ds(2)=110 mm
Number of strands  ns(2)=14
Height above soffit  ds(3)=880 mm
Number of strands  ns(3)=2
Height above soffit  ds(4)=930 mm
Number of strands  ns(4)=2
Short Term Losses ( BS EN 1992-1-1 )

Relaxation at transfer ( % ) \( \text{str}=1 \)

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>384.06</td>
<td>7.6112</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>434.52</td>
<td>8.6112</td>
</tr>
</tbody>
</table>

Losses at Transfer

Initial force 5046 kN
Force at transfer \( P_t=P_0-\text{loft}=4611.5 \) kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm(^2)</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td></td>
<td>-0.83642</td>
<td>20.179</td>
<td>16.124</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td></td>
<td>6.165</td>
<td>-3.996</td>
<td>-2.0352</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>5.3286</td>
<td>16.183</td>
<td>14.089</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

**Stress Limitations ( BS EN 1992-1-1, Clause 5.10.2.2 )**

Top of beam:
Compressive stress 5.3286 N/mm\(^2\) is less than allowable stresses of 0.7*fcki=22.4 N/mm\(^2\) and 0.6*fck=24 N/mm\(^2\).

Bottom of beam:
Compressive stress 16.183 N/mm\(^2\) is less than allowable stresses of 0.7*fcki=22.4 N/mm\(^2\) and 0.6*fck=24 N/mm\(^2\).

**Long Term Losses ( BS EN 1992-1-1, Clause 5.10.6(1) )**

Relaxation after transfer ( % ) \( \text{slr}(1)=2 \)
Shrinkage per unit length \( \text{sul}(1)=0.3\times10^{-3} \)
Creep coefficient \( \varphi(\text{m},\text{to}) \)
\( \text{crf}=1.8 \)
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>50.46</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>384.06</td>
<td>7.6112</td>
</tr>
<tr>
<td><strong>Transfer Total</strong></td>
<td><strong>434.52</strong></td>
<td><strong>8.6112</strong></td>
</tr>
<tr>
<td>Relaxation</td>
<td>100.92</td>
<td>2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>235.81</td>
<td>4.6733</td>
</tr>
<tr>
<td>Creep</td>
<td>619.18</td>
<td>12.271</td>
</tr>
<tr>
<td><strong>Long-term Total</strong></td>
<td><strong>955.92</strong></td>
<td><strong>18.944</strong></td>
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<tr>
<td>All Losses</td>
<td>1390.4</td>
<td>27.555</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 5046 kN
Final force Pf=P0-loftl=3655.6 kN
Unit-weight of in-situ concrete wc=24 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>-0.66303</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>6.165</td>
</tr>
<tr>
<td>In-situ concrete</td>
<td>3.6555</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>9.1576</strong></td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=1005 mm
Percentage of shrinkage remaining ps=25 %
Percentage pcr=25
Percentage stT=67 %

The net moment is Mc=F*e/1E3-Mb=-119.61 kNm
Stress at top of in-situ concrete:
\[ ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3 \]
=0.0028135 N/mm²
Stress at bottom of in-situ concrete:
\[ fbisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(hsis-Yc)/Ic)*mr*1E3 \]
=0.17009 N/mm²
Stress at top of precast concrete beam:
\[
ftpbds = \frac{F}{(Ai*mr+Ac)+Mc*1E3*(d-Yc)/Ic+Mb*1E3*Yt/Iyy)}*1E3
\]
\[-0.21575 \text{ N/mm}^2\]

Stress at bottom of precast concrete beam:
\[
fbpbds = \frac{F}{(Ai*mr+Ac)-Mc*1E3*Yc/Ic-Mb*1E3*Yb/Iyy)}*1E3 = 0.058269 \text{ N/mm}^2
\]

Loading Combination 1

(no tensile stress permitted in precast beam)
Characteristic bending moment \( c1b(1)=1446 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading Combination 1 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of in-situ</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
</tr>
<tr>
<td>Combination 1</td>
</tr>
<tr>
<td>Total</td>
</tr>
</tbody>
</table>

Loading Combination 1 - Stresses

Loading Combination 2

(tensile stress permitted (no visible cracking))
Characteristic bending moment \( c2b(1)=1907 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading Combination 2 Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of in-situ</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>Prestress, beam and in-situ</td>
</tr>
<tr>
<td>Dif. shrinkage</td>
</tr>
<tr>
<td>Combination 2</td>
</tr>
<tr>
<td>Total</td>
</tr>
</tbody>
</table>

Loading Combination 2 - Stresses

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 8.3928 N/mm² is less than the maximum allowable stress of 0.6*fckc=19.2 N/mm².

Top of beam:
Compressive stress 15.248 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².
Bottom of in-situ concrete:
Compressive stress 5.8929 N/mm$^2$ is less than the maximum allowable stress of $0.6f_{ckc}=19.2$ N/mm$^2$.

Bottom of beam:
Tensile stress 1.3411 N/mm$^2$ is less than the maximum allowable tensile stress of 3.0263 N/mm$^2$. 
Location: Ex3 - 13.5 metre span solid edge beam

Precast prestressed concrete beam with non-standard section


Beam span under consideration span=13.5 m

Beam section properties

- Depth of beam d=615 mm
- Cross-sectional area Ac=130675 mm²
- Centroid above soffit Yb=244 mm
- Section modulus top Zt=14.3E6 mm³
- Section modulus bottom Zb=21.81E6 mm³
- Unit weight of beam uwb=25 kN/m³

Concrete compressive strengths

- Characteristic cylinder strength fcki=32 N/mm² (transfer)
- Characteristic cylinder strength fck=40 N/mm²

Concrete moduli (short-term)

- Concrete modulus of elasticity Ecmi=33 kN/mm² (transfer)
- Concrete modulus of elasticity Ecmb=35 kN/mm²

Composite section properties

- Char strength (in-situ concrete) fckc=28 N/mm²
- Short-term modulus of elasticity for in-situ concrete Ecmc=32 kN/mm²

Item No. 1

- Item weighting iw1=0.91429
- X co-ordinate X(1)=0 mm
- Y co-ordinate Y(1)=0 mm
- X co-ordinate X(2)=508 mm
- Y co-ordinate Y(2)=0 mm
- X co-ordinate X(3)=508 mm
- Y co-ordinate Y(3)=250 mm
- X co-ordinate X(4)=650 mm
- Y co-ordinate Y(4)=350 mm
- X co-ordinate X(5)=650 mm
- Y co-ordinate Y(5)=790 mm
- X co-ordinate X(6)=150 mm
- Y co-ordinate Y(6)=790 mm
- X co-ordinate X(7)=150 mm
- Y co-ordinate Y(7)=690 mm
- X co-ordinate X(8)=0 mm
- Y co-ordinate Y(8)=690 mm
- X co-ordinate X(9)=0 mm
Y co-ordinate

**Item No. 2**

- Second moment of area: $I_{XX2} = 5.3216E9 \text{ mm}^4$
- Area of item: $A_2 = 130675 \text{ mm}^2$
- Centroid from X-X axis: $Y_{C2} = 244 \text{ mm}$
- Item weighting: $i_w = 0.085714$
- Depth of composite section: $d_i = 790 \text{ mm}$
- Nominal diameter: $n_{omd} = 12 \text{ mm}$
- Nominal tensile strength: $n_{omt} = 1700 \text{ N/mm}^2$
- Nominal steel area: $n_{oma} = 100 \text{ mm}^2$
- Char breaking load (strand): $f_{pk} = 150 \text{ kN}$
- Initial force per strand: $i_{pf} = 150 \text{ kN}$
- Modulus of elasticity (strand): $E_p = 195 \text{ kN/mm}^2$

**Cross-section 1**

- Distance from mid-span of beam: $c_{sd(1)} = 2 \text{ m}$
- Number of layers of strand: $n_{ls(1)} = 4$
- Height above soffit: $d_{s(1)} = 49 \text{ mm}$
- Number of strands: $n_{s(1)} = 10$
- Height above soffit: $d_{s(2)} = 85 \text{ mm}$
- Number of strands: $n_{s(2)} = 8$
- Height above soffit: $d_{s(3)} = 520 \text{ mm}$
- Number of strands: $n_{s(3)} = 2$
- Height above soffit: $d_{s(4)} = 580 \text{ mm}$
- Number of strands: $n_{s(4)} = 2$
- Bending moment from temp.loads: $t_{lb(1)} = 80 \text{ kNm}$
- Bending moment from other loads: $o_{lb(1)} = 50 \text{ kNm}$

**Short Term Losses (BS EN 1992-1-1)**

- Relaxation at transfer (%): $s_{tr} = 1$

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force (kN)</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>33</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>349.08</td>
<td>10.578</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>382.08</td>
<td>11.578</td>
</tr>
</tbody>
</table>

Losses at Transfer

- Initial force: $3300 \text{ kN}$
- Force at transfer: $P_t = P_0 - \Delta f = 2917.9 \text{ kN}$
Stresses at Transfer N/mm²

<table>
<thead>
<tr>
<th>Loading</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>3.855</td>
<td>34.48</td>
<td>26.852</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>4.733</td>
<td>-3.1128</td>
<td>-1.1586</td>
</tr>
<tr>
<td>Temporary Loads</td>
<td>5.5772</td>
<td>-3.668</td>
<td>-1.3653</td>
</tr>
<tr>
<td>Other Loads</td>
<td>3.4858</td>
<td>-2.2925</td>
<td>-0.85329</td>
</tr>
<tr>
<td>Total</td>
<td>17.651</td>
<td>25.407</td>
<td>23.475</td>
</tr>
</tbody>
</table>

Beam Concrete Stresses at Transfer

Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)

Top of beam:
Compressive stress 17.651 N/mm² is less than allowable stresses
of 0.7*fck=22.4 N/mm² and 0.6*fck=24 N/mm².

Bottom of beam:

WARNING:
Compressive stress 25.407 N/mm² exceeds maximum allowable
stress of 0.7*fck=22.4 N/mm².

Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))

Relaxation after transfer( % ) slr(1)=1.5
Shrinkage per unit length sul(1)=0.3E-3
Creep coefficient φ(ω,to) crf=1.9

WARNING:
Maximum stress at transfer exceeds 20 N/mm²
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>33</td>
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<tr>
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<td>10.578</td>
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<tr>
<td>Transfer Total</td>
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<td>11.578</td>
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<td>Relaxation</td>
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<td>1.5</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>128.7</td>
<td>3.9</td>
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<tr>
<td>Creep</td>
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<td>21.913</td>
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<td>27.313</td>
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<tr>
<td>All Losses</td>
<td>1283.4</td>
<td>38.891</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 3300 kN
Final force \( Pf=P_0-loftl=2016.6 \) kN
Unit-weight of in-situ concrete \( wc=25 \) kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
</tr>
<tr>
<td>Prestress</td>
<td>2.6642</td>
</tr>
<tr>
<td>Self weight of Beam</td>
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<tr>
<td>In-situ concrete</td>
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<tr>
<td>Total</td>
<td>19.177</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete \( hsis=100 \) mm
Perc'age of shrinkage remaining \( ps=40 \) %

WARNING:
Maximum stress at transfer exceeds 20 N/mm²

Percentage \( pcr=40 \)
Percentage \( stT=50 \) %
The net moment is \( Mc=F*e/1E3-Mb=-258.58 \) kNm
Stress at top of in-situ concrete:
\[
ftisds=-(F/(Ai*mr)-F/(Ai*mr+Ac)-Mc*1E3*(di-Yc)/Ic)*mr*1E3=-0.76411 \text{ N/mm}^2
\]
Stress at bottom of in-situ concrete:
\[ fb_{isds} = -\frac{F}{(A_i \cdot mr)} - \frac{F}{(A_i \cdot mr + A_c)} - \frac{Mc}{I_c} \cdot (h_{sis} - Y_c) \cdot 10^3 \cdot mr \cdot 10^3 \]
\[ = 7.0621 \text{ N/mm}^2 \]

Stress at top of precast concrete beam:
\[ ft_{pbds} = -\frac{F}{(A_i \cdot mr + A_c)} + \frac{Mc}{I_c} \cdot (d - Y_t) \cdot 10^3 + \frac{Mb}{I_{yy}} \cdot Y_t \cdot 10^3 \cdot 10^3 \]
\[ = -12.423 \text{ N/mm}^2 \]

Stress at bottom of precast concrete beam:
\[ fb_{pbds} = \frac{F}{(A_i \cdot mr + A_c)} - \frac{Mc}{I_c} \cdot Y_c \cdot 10^3 - \frac{Mb}{I_{yy}} \cdot Y_b \cdot 10^3 \cdot 10^3 \]
\[ = -3.573 \text{ N/mm}^2 \]

### Loading Combination 1

(no tensile stress permitted in precast beam)

Characteristic bending moment \( c_{1b}(1) = 550 \text{ kNm} \)

<table>
<thead>
<tr>
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<th>Loading Combination 1 Stresses N/mm²</th>
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<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
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<tr>
<td>Prestress, beam and in-situ</td>
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</tr>
<tr>
<td>Dif. shrinkage</td>
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<tr>
<td>Combination 1</td>
<td>10.207</td>
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<tr>
<td>Total</td>
<td>9.4433</td>
</tr>
</tbody>
</table>

**Loading Combination 1 - Stresses**

**WARNING:**

Bottom of beam:

Tensile stress 1.2154 N/mm² exceeds maximum allowable tensile stress of 0 N/mm².

### Loading Combination 2

(tensile stress permitted (no visible cracking))

Characteristic bending moment \( c_{2b}(1) = 625 \text{ kNm} \)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
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<tbody>
<tr>
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<td>Top of in-situ</td>
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<tr>
<td>Prestress, beam and in-situ</td>
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<tr>
<td>Dif. shrinkage</td>
<td>-0.76411</td>
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<td>10.633</td>
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<td>Total</td>
<td>9.8686</td>
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</tbody>
</table>

**Loading Combination 2 - Stresses**
NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 9.8686 N/mm² is less than the maximum allowable stress of 0.6*fckc=16.8 N/mm².

Top of beam:
Compressive stress 13.135 N/mm² is less than the maximum allowable stress of 0.6*fck=24 N/mm².

Bottom of in-situ concrete:
Tensile stress 1.2218 N/mm² is less than the maximum allowable tensile stress of 2.7656 N/mm² (EN 1992-1-1 expression 3.23)

Bottom of beam:
Tensile stress 2.6625 N/mm² is less than the maximum allowable tensile stress of 3.0263 N/mm².
Location: Ex4 - 17.0 metre span solid section

Precast prestressed concrete beam with non-standard section


Beam span under consideration span=17 m

Beam section properties

- Depth of beam $d=775$ mm
- Cross-sectional area $A_c=163360$ mm$^2$
- Centroid above soffit $Y_b=334$ mm
- Section modulus top $Z_t=24.31E6$ mm$^3$
- Section modulus bottom $Z_b=32.1E6$ mm$^3$
- Unit weight of beam $u_{wb}=25$ kN/m$^3$

Concrete compressive strengths

- Characteristic cylinder strength $f_{ck}=32$ N/mm$^2$ (transfer)
- Characteristic cylinder strength $f_{ck}=32$ N/mm$^2$

Concrete moduli (short-term)

- Concrete modulus of elasticity $E_{cmi}=32$ kN/mm$^2$ (transfer)
- Concrete modulus of elasticity $E_{cmb}=33$ kN/mm$^2$

Composite section properties

- Char strength (in-situ concrete) $f_{ckc}=30$ N/mm$^2$
- Short-term modulus of elasticity for in-situ concrete $E_{cmc}=32$ kN/mm$^2$
- Depth of composite section $d_i=850$ mm
- Area of in-situ concrete $A_i=304140$ mm$^2$
- Centroid of composite section $Y_c=425$ mm
- Second moment of area $I_c=28.147E9$ mm$^4$
- Modulus of elasticity (strand) $E_p=195$ kN/mm$^2$

Cross-section 1

- Distance from mid-span of beam $c_{sd(1)}=6$ m
- Number of layers of strand $n_{ls(1)}=6$
- Height above soffit $d_{s(1)}=50$ mm
- Number of strands $n_{s(1)}=4$
- Height above soffit $d_{s(2)}=90$ mm
- Number of strands $n_{s(2)}=3$
- Height above soffit $d_{s(3)}=260$ mm
- Number of strands $n_{s(3)}=1$
- Height above soffit $d_{s(4)}=500$ mm
- Number of strands $n_{s(4)}=1$
- Height above soffit $d_{s(5)}=620$ mm
- Number of strands $n_{s(5)}=1$
- Height above soffit $d_{s(6)}=740$ mm
Number of strands $ns(6)=1$

**Short Term Losses (BS EN 1992-1-1)**

Relaxation at transfer (%) $str=1$

**Summary**

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force kN</th>
<th>% Loss</th>
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</thead>
<tbody>
<tr>
<td>Relaxation</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Elastic Shortening</td>
<td>207.04</td>
<td>9.4107</td>
</tr>
<tr>
<td>Transfer Total</td>
<td>229.04</td>
<td>10.411</td>
</tr>
</tbody>
</table>

**Losses at Transfer**

Initial force $2200$ kN
Force at transfer $Pt=P_0-loft=1971$ kN

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stresses at Transfer N/mm²</th>
<th>Top of Beam</th>
<th>Bottom of Beam</th>
<th>Centroid of Strands</th>
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</thead>
<tbody>
<tr>
<td>Prestress</td>
<td></td>
<td>4.076</td>
<td>18.116</td>
<td>13.85</td>
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<tr>
<td>Self weight of Beam</td>
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<td>3.0447</td>
<td>-2.306</td>
<td>-0.68038</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>7.1207</td>
<td>15.81</td>
<td>13.17</td>
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</tbody>
</table>

**Beam Concrete Stresses at Transfer**

**Stress Limitations (BS EN 1992-1-1, Clause 5.10.2.2)**

Top of beam:
Compressive stress $7.1207$ N/mm² is less than allowable stresses of $0.7*f_{ck,i}=19.6$ N/mm² and $0.6*f_{ck}=19.2$ N/mm².

Bottom of beam:
Compressive stress $15.81$ N/mm² is less than allowable stresses of $0.7*f_{ck,i}=19.6$ N/mm² and $0.6*f_{ck}=19.2$ N/mm².

**Long Term Losses (BS EN 1992-1-1, Clause 5.10.6(1))**

Relaxation after transfer (%) $slr(1)=2$
Shrinkage per unit length $sul(1)=0.3E-3$
Creep coefficient $\phi(\psi, to) = 2.2$
Summary

<table>
<thead>
<tr>
<th>Loss</th>
<th>Loss of Force</th>
<th>% Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kN</td>
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<tr>
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<td>1</td>
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<td>Elastic Shortening</td>
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<td>9.4107</td>
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<td>Transfer Total</td>
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<tr>
<td>Relaxation</td>
<td>44</td>
<td>2</td>
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<td>Shrinkage</td>
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<td>6.5228</td>
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<td>Creep</td>
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<td>Long-term Total</td>
<td>686.28</td>
<td>31.195</td>
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<tr>
<td>All Losses</td>
<td>915.32</td>
<td>41.605</td>
</tr>
</tbody>
</table>

Losses - Long term

Initial force 2200 kN
Final force Pf=P0-loftl=1284.7 kN
Unit-weight of in-situ concrete wc=25 kN/m³

<table>
<thead>
<tr>
<th>Loading</th>
<th>Final Stresses N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top of Beam</td>
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<tr>
<td>Prestress</td>
<td>2.6567</td>
</tr>
<tr>
<td>Self weight of Beam</td>
<td>3.0447</td>
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<tr>
<td>In-situ concrete</td>
<td>5.6686</td>
</tr>
<tr>
<td>Total</td>
<td>11.37</td>
</tr>
</tbody>
</table>

Beam concrete stresses - Long term

Composite section

Height from beam soffit to bottom of in-situ concrete hsis=100 mm

Loading Combination 1

(no tensile stress permitted in precast beam)
Characteristic bending moment clb(1)=200 kNm
Loading Combination 1 - Stresses

<table>
<thead>
<tr>
<th>Loading</th>
<th>Loading Combination 1 Stresses N/mm²</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Top of in-situ</td>
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<tr>
<td>Prestress, beam and in-situ</td>
<td>0</td>
</tr>
<tr>
<td>Combination 1</td>
<td>3.3973</td>
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<tr>
<td>Total</td>
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</tbody>
</table>

Loading Combination 2 - Stresses

<table>
<thead>
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<th>Loading</th>
<th>Loading Combination 2 Stresses N/mm²</th>
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</thead>
<tbody>
<tr>
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<tr>
<td>Prestress, beam and in-situ</td>
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<tr>
<td>Combination 2</td>
<td>3.7748</td>
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<tr>
<td>Total</td>
<td>3.7748</td>
</tr>
</tbody>
</table>

NOTE: The stresses from temperature differences should be added to the total stresses.

Top of in-situ concrete:
Compressive stress 3.7748 N/mm² is less than the maximum allowable stress of 0.6*fckc=18 N/mm².

Top of beam:
Compressive stress 14.479 N/mm² is less than the maximum allowable stress of 0.6*fck=19.2 N/mm².

Bottom of in-situ concrete:
Tensile stress 2.8866 N/mm² is less than the maximum allowable tensile stress of 2.8958 N/mm² (EN 1992-1-1 expression 3.23)

Bottom of beam:
Compressive stress 1.434 N/mm² is less than the maximum allowable stress of 0.6*fck=19.2 N/mm².
Mix title: Example 1, Design of Normal Concrete Mixes
Location: Ex1 - Column on grid line C/12

Rectangular column interaction diagrams for uniaxial bending

The calculation produces a column interaction diagram and tabular results for axial load and uniaxial bending on a rectangular section. The analysis is in accordance with BS5400-4:1990 as implemented by DOT Standard BD 44/15. The method used is that described in Clause 5.5.3.2 with the stresses in the concrete in compression derived from the stress-strain curve in Figure 1. The axial force is assumed to act at the centroid of the gross concrete section.

Rectangular column section

Consider bending about X-X the major axis.

<table>
<thead>
<tr>
<th>Rectangular column section</th>
</tr>
</thead>
<tbody>
<tr>
<td>b=350 mm</td>
</tr>
<tr>
<td>h=450 mm</td>
</tr>
<tr>
<td>fcu=40 N/mm²</td>
</tr>
<tr>
<td>n=2</td>
</tr>
<tr>
<td>gamc=1.50</td>
</tr>
<tr>
<td>gams=1.15</td>
</tr>
<tr>
<td>fym=460 N/mm²</td>
</tr>
<tr>
<td>Em=200000 N/mm²</td>
</tr>
<tr>
<td>A(1)=1608.5 mm²</td>
</tr>
<tr>
<td>d(1)=60 mm</td>
</tr>
<tr>
<td>A(2)=981.75 mm²</td>
</tr>
<tr>
<td>d(2)=390 mm</td>
</tr>
</tbody>
</table>
## Tabular Results

<table>
<thead>
<tr>
<th>Depth to Neutral Axis (mm)</th>
<th>Moment Capacity (kNm)</th>
<th>Axial Load Capacity (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>∞</td>
<td>32.624</td>
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</tr>
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<td>Axial Load (kN)</td>
<td>Depth to Neutral Axis (mm)</td>
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</tr>
<tr>
<td>----------------</td>
<td>---------------------------</td>
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<tr>
<td>3631.1</td>
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Moment (kNm)
Location: Ex1 - Column on grid line C/12

Rectangular column interaction diagrams for uniaxial bending

The calculation produces a column interaction diagram and tabular results for axial load and uniaxial bending on a rectangular section. The analysis is in accordance with BS EN 1992-2:2005 and the NA to BS EN 1992-2:2005 and utilises the stresses in the concrete in compression derived from the stress-strain curve in Figure 3.3. The axial force is assumed to act at the centroid of the gross concrete section.

![Rectangular column section diagram](image)

Consider bending about Y-Y the major axis.

Breadth of concrete section \( b = 350 \text{ mm} \)
Depth of concrete section \( h = 450 \text{ mm} \)
Number of layers of reinforcement \( n = 2 \)

The calculations utilise the following material factors which are in accordance with BS EN 1992-1-1 (Table 2.1N):

- Concrete material factor \( \gamma_c \) \( \gamma_c = 1.50 \)
- Steel material factor \( \gamma_s \) \( \gamma_s = 1.15 \)
- Char. strength of reinforcement \( f_y = 500 \text{ N/mm}^2 \)
- Young's modulus for reinforcement \( E_m = 200000 \text{ N/mm}^2 \)
- Area of reinforcement \( A(1) = 1608.5 \text{ mm}^2 \)
- Depth from compression face \( d(1) = 60 \text{ mm} \)
- Area of reinforcement \( A(2) = 981.75 \text{ mm}^2 \)
- Depth from compression face \( d(2) = 390 \text{ mm} \)
Tabular Results

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Moment (kNm)
Location: Ex1 - Column A

Rectangular column interaction diagrams for uniaxial bending

Breadth of concrete section \( b = 350 \text{ mm} \)
Depth of concrete section \( h = 450 \text{ mm} \)
Strength of concrete \( f_{cu} = 40 \text{ N/mm}^2 \)
Number of layers of reinforcement \( n = 2 \)
Strength of reinforcement \( f_{ym} = 460 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E_m = 200000 \text{ N/mm}^2 \)
Area of reinforcement \( A(1) = 1608.5 \text{ mm}^2 \)
Depth from compression face \( d(1) = 60 \text{ mm} \)
Area of reinforcement \( A(2) = 981.75 \text{ mm}^2 \)
Depth from compression face \( d(2) = 390 \text{ mm} \)
Factor for reinforcement \( g_{ms} = 1.15 \)
Factor for concrete \( g_{mc} = 1.5 \)
### Tabular Results

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Moment (kN-m)

 SCALE 5.48     Office 1007     Proforma 152
Circular column interaction diagrams for uniaxial bending

The calculation produces a column interaction diagram and tabular results for axial load and uniaxial bending on a circular section. The analysis is in accordance with BS5400-4:1990 as implemented by DOT Standard BD 44/15. The method used is that described in Clause 5.5.3.2 with the stresses in the concrete in compression derived from the stress-strain curve in Figure 1. The axial force is assumed to act at the centroid of the gross concrete section.

Diameter of concrete section \( d = 900 \text{ mm} \)
Char. strength of concrete \( f_{cu} = 40 \text{ N/mm}^2 \)
Total area of reinforcement \( A_r = 7977 \text{ mm}^2 \)
Char. strength of reinforcement \( f_{ym} = 460 \text{ N/mm}^2 \)
Young's modulus for r'ment \( E_m = 200000 \text{ N/mm}^2 \)
Concrete material factor \( \gamma_m \) \( \gamma_m = 1.50 \)
Steel material factor \( \gamma_m \) \( \gamma_m = 1.15 \)
Number of bars \( n_o = 16 \)
Radial distance to centre of bars \( d_{tc} = 62 \text{ mm} \)
## Tabular Results

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Moment (kNm)
Location: Ex1 - Column on grid line A/2

Circular column interaction diagrams for uniaxial bending

The calculation produces a column interaction diagram and tabular results for axial load and uniaxial bending on a circular section. The analysis is in accordance with BS EN 1992-2:2005 and the NA to BS EN 1992-2:2005 and utilises the stresses in the concrete in compression derived from the stress-strain curve in Figure 3.3.

Circular column section

Consider bending about the Y-Y axis.

Diameter of concrete section $d=900$ mm
Total area of reinforcement $A_r=7977$ mm$^2$
Char. strength of reinforcement $f_{yk}=500$ N/mm$^2$
Young's modulus for r'ment $E_m=200000$ N/mm$^2$
The calculations utilise the following material factors which are in accordance with BS EN 1992-1-1 (Table 2.1N):
Concrete material factor $\gamma_c$ $\gamma_{mc}=1.50$
Steel material factor $\gamma_s$ $\gamma_{ms}=1.15$
Number of bars $n_{ob}=16$
Radial distance to centre of bars $d_{tc}=62$ mm
# Tabular Results

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Moment (kNm)
**Location: Ex1 - Column A**

**Circular column interaction diagrams for uniaxial bending**

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## Tabular Results

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Moment (kNm)
Interaction diagrams for uniaxial bending of a general column

The calculation produces a column interaction diagram and tabular results for axial load and uniaxial bending on a general column section. The analysis is in accordance with BS5400-4:1990 as implemented by DOT Standard BD 44/15. The method used is that described in Clause 5.5.3.2 with the stresses in the concrete in compression derived from the stress-strain curve in Figure 1. The axial force is assumed to act at the centroid of the gross concrete section.

Char. strength of concrete     $f_{cum}=40 \text{ N/mm}^2$
Number of cross-sections      $n_{cs}=2$
Cross-section 1
Cross-section 2
Number of layers of reinforcement $n=2$
Char. strength of reinforcement $f_{ym}=460 \text{ N/mm}^2$
Young's modulus of reinforcement $E_m=200000 \text{ N/mm}^2$
Concrete material factor $g_m=1.50$
Steel material factor $g_{ms}=1.15$
Reinforcement layer 1 :
Area of reinforcement         $A(1)=1608.5 \text{ mm}^2$
Depth from compression face   $d(1)=60 \text{ mm}$
Reinforcement layer 2 :
Area of reinforcement         $A(2)=981.75 \text{ mm}^2$
Depth from compression face   $d(2)=390 \text{ mm}$
### Tabular Results

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Moment (kNm)
Location: Ex1 - Column A

Interaction diagrams for uniaxial bending of a general column section

The calculation produces a column interaction diagram and tabular results for axial load and uniaxial bending on a general column section. The analysis is in accordance with BS EN 1992-2:2005 and the NA to BS EN 1992-2:2005 and utilises the stresses in the concrete in compression derived from the stress-strain curve in Figure 3.3. The axial force is assumed to act at the centroid of the gross concrete section.

Number of cross-sections          ncs=2
Cross-section 1                   
Cross-section 2                   
Number of layers of r'ment        n=2
Char. strength of reinforcement   fyk=500 N/mm²
Young's modulus of r'ment         Em=200000 N/mm²
The calculations utilise the following material factors which are in accordance with BS EN 1992-1-1 (Table 2.1N):
Concrete material factor γm       γmc=1.50
Steel material factor γm          γms=1.15
Reinforcement layer 1 :           
Area of reinforcement             A(1)=1608.5 mm²
Depth from compression face       d(1)=60 mm
Reinforcement layer 2 :           
Area of reinforcement             A(2)=981.75 mm²
Depth from compression face       d(2)=390 mm
## Tabular Results

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Location: Ex1 - Column A

Interaction diagrams for uniaxial bending of a general column

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Moment (kNm)
Location: Ex1 - Column A

Rectangular column interaction diagrams for biaxial bending

The calculation produces a column interaction diagram and tabular results for axial load and biaxial bending on a rectangular section. The analysis is in accordance with BS5400-4:1990 as implemented by DOT Standard BD 44/15. The method used is that described in Clause 5.5.3.2 with the stresses in the concrete in compression derived from the stress-strain curve in Figure 1. The axial force is assumed to act at the centroid of the gross concrete section.

![Rectangular column section with biaxial bending.](image)

- Breadth of concrete section: \( b = 350 \text{ mm} \)
- Depth of concrete section: \( h = 450 \text{ mm} \)
- Char. strength of concrete: \( f_{cu} = 40 \text{ N/mm}^2 \)
- Number of layers of reinforcement: \( n = 2 \)
- Char. strength of reinforcement: \( f_{ym} = 460 \text{ N/mm}^2 \)
- Young's modulus for reinforcement: \( E_m = 200000 \text{ N/mm}^2 \)
- Concrete material factor: \( \gamma_m = 1.50 \)
- Steel material factor: \( \gamma_{ms} = 1.15 \)
- Reinforcement layer 1:
  - Number of bars: \( n_{ob}(1) = 7 \)
  - x co-ordinate: \( x_s(1) = 50 \text{ mm} \)
  - y co-ordinate: \( y_s(1) = 50 \text{ mm} \)
  - Location of last bar: \( x_f(1) = 300 \text{ mm} \)
  - Diameter of bars: \( d_{ob}(1) = 16 \text{ mm} \)
- Reinforcement layer 2:
  - Number of bars: \( n_{ob}(2) = 7 \)
  - x co-ordinate: \( x_s(2) = 50 \text{ mm} \)
  - y co-ordinate: \( y_s(2) = 400 \text{ mm} \)
  - Location of last bar: \( x_f(2) = 300 \text{ mm} \)
  - Diameter of bars: \( d_{ob}(2) = 16 \text{ mm} \)
Angle $\theta=45^\circ$

Faces 1 and 2 under compression from actions of biaxial bending.
### Tabular Results

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<tr>
<th>Depth to Neutral Axis (mm)</th>
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<th>Axial Load Capacity (kN)</th>
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Moment (kNm)
Location: Ex1 - Column A

Rectangular column interaction diagrams for biaxial bending

The calculation produces a column interaction diagram and tabular results for axial load and biaxial bending on a rectangular section. The analysis is in accordance with BS EN 1992-2:2005 and the NA to BS EN 1992-2:2005 and utilises the stresses in the concrete in compression derived from the stress-strain curve in Figure 3.3.

Rectangular column section with biaxial bending.

Breadth of concrete section \( b=350 \) mm
Depth of concrete section \( h=450 \) mm
Number of layers of r'ment \( n=2 \)
Char. strength of reinforcement \( f_{yk}=500 \) N/mm²
Young's modulus for r'ment \( E_m=200000 \) N/mm²

The calculations utilise the following material factors which are in accordance with BS EN 1992-1-1 (Table 2.1N):
Concrete material factor \( \gamma_c \) \( \gamma_c=1.50 \)
Steel material factor \( \gamma_s \) \( \gamma_s=1.15 \)
Reinforcement layer 1 :
Number of bars \( n_{ob}(1)=7 \)
x co-ordinate \( x_s(1)=50 \) mm
y co-ordinate \( y_s(1)=50 \) mm
Location of last bar \( x f(1)=300 \) mm
Diameter of bars \( d_{ob}(1)=16 \) mm
Reinforcement layer 2 :
Number of bars \( n_{ob}(2)=7 \)
x co-ordinate \( x_s(2)=50 \) mm
y co-ordinate \( y_s(2)=400 \) mm
Location of last bar \( x f(2)=300 \) mm
Diameter of bars \( d_{ob}(2)=16 \) mm
Depth to Neutral Axis measured from corner A along a line normal to axis of bending.

Angle $\theta=45^\circ$

Faces 1 and 2 under compression from actions of biaxial bending.
### Tabular Results

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## Graphical Results

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Moment (kNm)
Location: Ex1 - Column A

Rectangular column interaction diagrams for biaxial bending

Breadth of concrete section \( b = 350 \text{ mm} \)
Depth of concrete section \( h = 450 \text{ mm} \)
Strength of concrete \( f_{cu} = 40 \text{ N/mm}^2 \)
Number of layers of reinforcement \( n = 2 \)
Strength of reinforcement \( f_{ym} = 460 \text{ N/mm}^2 \)
Young's modulus for reinforcement \( E_m = 200000 \text{ N/mm}^2 \)
Layer 1:
Number of bars \( n_1 = 7 \)
x co-ordinate \( x_s(1) = 50 \text{ mm} \)
y co-ordinate \( y_s(1) = 50 \text{ mm} \)
x co-ordinate \( x_f(1) = 300 \text{ mm} \)
Diameter of bars \( d_{ob}(1) = 16 \text{ mm} \)
Layer 2:
Number of bars \( n_2 = 7 \)
x co-ordinate \( x_s(2) = 50 \text{ mm} \)
y co-ordinate \( y_s(2) = 400 \text{ mm} \)
x co-ordinate \( x_f(2) = 300 \text{ mm} \)
Diameter of bars \( d_{ob}(2) = 16 \text{ mm} \)
Factor for reinforcement \( g_{ms} = 1.15 \)
Factor for concrete \( g_{mc} = 1.5 \)

Face 1

\[ \theta = 45^\circ \]

Depth to Neutral Axis measured from corner A along a line normal to axis of bending.

Faces 1 and 2 under compression from actions of biaxial bending.
### Tabular Results

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<td>1744.1 * 216.3</td>
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<tr>
<td>1414.8 * 226.04</td>
<td>282.84</td>
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<tr>
<td>1083.8 * 228.16</td>
<td>254.56</td>
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<tr>
<td>758.87 * 222.63</td>
<td>226.27</td>
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<tr>
<td>449.59 * 210.53</td>
<td>197.99</td>
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<tr>
<td>165.19 * 191.03</td>
<td>180.04</td>
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<tr>
<td>0              * 177.48</td>
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</tbody>
</table>
Location: Combined compression and bending on substantial section.

Analysis of wall/slab section to requirements of BS8007
and/or BS8110

Calculations are undertaken for a section 1 metre in width. Section is assumed to be subjected primarily to bending (i.e. formulae 2 and 3 in Clause B.4 of BS8007 are used to determine stiffening effect of concrete, etc.)

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

Ultimate moment per metre width \( M = 950 \text{ kNm/m} \)

Service moment per metre width \( M_z' = 665 \text{ kNm/m} \)

Maximum permissible crack width \( C_w = 0.2 \text{ mm} \)

Shear force due to ultimate loads \( V = 450 \text{ kN/m} \)

Ultimate axial load (-ve tension) \( N_{ult}' = 400 \text{ kN/m} \)

Service axial load (-ve tension) \( N' = 280 \text{ kN/m} \)

Characteristic concrete strength \( f_{cu} = 35 \text{ N/mm}^2 \)

Elastic modulus of concrete \( E_c = 12000 \text{ N/mm}^2 \)

Characteristic steel strength \( f_y = 500 \text{ N/mm}^2 \)

Overall depth of concrete section \( h = 1000 \text{ mm} \)

Size of tension bars \( d_{iat} = 25 \text{ mm} \)

Spacing of tension bars \( s_{pac} = 150 \text{ mm} \)

Maximum dia.of coarse aggregate \( h_{agg} = 20 \text{ mm} \)

Size of compression bars \( d_{iac} = 20 \text{ mm} \)

Spacing of compression bars \( s_{pac} = 150 \text{ mm} \)

Designated exposure class is XC4

Specified fixing tolerance \( t_{ol} = 10 \text{ mm} \)

Nominal concrete cover \( c_{over} = 40 \text{ mm} \)

Chosen effective depth of section \( d = 947.5 \text{ mm} \)

Chosen depth to compression bars \( d' = 50 \text{ mm} \)

Temp.zone depth for tension steel \( t_rz = 250 \text{ mm} \)

Temp.zone depth for comp.steel \( c_rz = 250 \text{ mm} \)

Temp.fall: hydration peak and ambient \( T_1 = 20 \degree C \)

Seasonal variation in temperature \( T_2 = 5 \degree C \)

Chosen ratio of \( f_{ct}/f_b \) \( f_{ratio} = 0.67 \)

External restraint factor \( R = 0.5 \)

Thermal expansion coeff.for concrete \( \alpha = 10E^{-6} \)

Direct tensile strength \( f_{ct} = 1.6 \text{ N/mm}^2 \)
### REINFORCEMENT

**Tension steel:**
- **Size:** 25 mm
- **Spacing:** 150 mm
- **Area required:** 2427.4 mm²/m
- **Area provided:** 3272.5 mm²/m
- **Percentages:** (pcrit) 1.309 %
  
  provided : (gross) 0.32725 %
- **Weight provided:** 25.689 kg/m²

**Compression steel:**
- **Size:** 20 mm
- **Spacing:** 150 mm
- **Area required:** 0 mm²/m
- **Area provided:** 2094.4 mm²/m
- **Percentages:** (pcrit) 0.83776 %
  
  provided : (gross) 0.20944 %
- **Weight provided:** 16.441 kg/m²

### SUMMARY

- **Cover to all steel:** 40 mm
- **Total weight of steel:** 42.13 kg/m²

**satisfies:**
1. Ultimate limit-state
2. Service limit-state cracking in flexure
3. Temperature-cracking requirements

**Calculated crack widths**
- **Flexure:** In tension face: between bars 0.19345 mm
- **Temperature change:** In tension face 0.079975 mm
  
  In compression face 0.099969 mm
Location: Combined tension and bending

Analysis of wall/slab section to requirements of BS8007

and/or BS8110

Calculations are undertaken for a section 1 metre in width. Section is assumed to be subjected primarily to bending (i.e. formulae 2 and 3 in Clause B.4 of BS8007 are used to determine stiffening effect of concrete, etc.)

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Ultimate moment per metre width  M=950 kNm/m
Service moment per metre width  Mz'=665 kNm/m
Maximum permissible crack width  Cw=0.2 mm
Shear force due to ultimate loads  V=325 kN/m
Ultimate axial load (-ve tension) Nult'=-1100 kN/m
Service axial load (-ve tension)  N'=-770 kN/m
Characteristic concrete strength  fcu=35 N/mm²
Elastic modulus of concrete  Ec=12000 N/mm²
Characteristic steel strength  fy=500 N/mm²
Overall depth of concrete section  h=1000 mm
Size of tension bars  diat=25 mm
Spacing of tension bars  spac=85 mm
Maximum dia.of coarse aggregate  hagg=20 mm
Size of compression bars  diac=20 mm
Spacing of compression bars  spacc=150 mm
Designated exposure class is XC4
Specified fixing tolerance  tol=10 mm
Nominal concrete cover  cover=40 mm
Chosen effective depth of section  d=947.5 mm
Chosen depth to compression bars  d'=50 mm
Temp.zone depth for tension steel  trz=250 mm
Temp.zone depth for comp.steel  crz=250 mm
Temp.fall: hydration peak and ambient  T1=20 °C
Seasonal variation in temperature  T2=5 °C
Chosen ratio of fct/fb  fratio=0.67
External restraint factor  R=0.5
Thermal expansion coeff.for concrete  alpha=10E-6
Direct tensile strength  fct=1.6 N/mm²

WARNING:
As the actual shearing stress of 0.34301 N/mm² exceeds the critical value of 0.2574 N/mm², the section is thus inadequate in shear.
Either a higher concrete grade must be employed or a thicker section must be adopted.
Location: Combined tension & bending (as Ex.2 but greater tension).

Analysis of wall/slab section to requirements of BS8007
and/or BS8110

Calculations are undertaken for a section 1 metre in width. Section is assumed to be subjected primarily to bending (i.e. formulae 2 and 3 in Clause B.4 of BS8007 are used to determine stiffening effect of concrete, etc.)

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Ultimate moment per metre width $M=950\ \text{kNm/m}$
Service moment per metre width $M_z'=665\ \text{kNm/m}$
Maximum permissible crack width $C_w=0.2\ \text{mm}$
Shear force due to ultimate loads $V=325\ \text{kN/m}$
Ultimate axial load (-ve tension) $N_{ult}'=-1200\ \text{kN/m}$
Service axial load (-ve tension) $N'=-840\ \text{kN/m}$
Characteristic concrete strength $f_{cu}=35\ \text{N/mm}^2$
Elastic modulus of concrete $E_c=12000\ \text{N/mm}^2$
Characteristic steel strength $f_y=500\ \text{N/mm}^2$
Overall depth of concrete section $h=1000\ \text{mm}$
Size of tension bars $d_{iat}=25\ \text{mm}$
Spacing of tension bars $s_{pac}=85\ \text{mm}$
Maximum dia. of coarse aggregate $h_{agg}=20\ \text{mm}$
Size of compression bars $d_{iac}=20\ \text{mm}$
Spacing of compression bars $s_{pac}=150\ \text{mm}$
Designated exposure class is XC4
Specified fixing tolerance $t_{ol}=10\ \text{mm}$
Nominal concrete cover $c_{over}=40\ \text{mm}$
Chosen effective depth of section $d=947.5\ \text{mm}$
Chosen depth to compression bars $d_{pc}'=50\ \text{mm}$
Temp. zone depth for tension steel $trz=250\ \text{mm}$
Temp. zone depth for comp. steel $crz=250\ \text{mm}$
Temp. fall: hydration peak and ambient $T_1=20\ ^\circ\text{C}$
Seasonal variation in temperature $T_2=5\ ^\circ\text{C}$
Chosen ratio of $f_{ct}/f_{bc}$ $f_{ratio}=0.67$
External restraint factor $R=0.5$
Thermal expansion coeff. for concrete $\alpha_{10E6}=10\ \text{E-6}$
Direct tensile strength $f_{ct}=1.6\ \text{N/mm}^2$

WARNING:
As the actual shearing stress of $0.34301\ \text{N/mm}^2$ exceeds the critical value of $0.23688\ \text{N/mm}^2$, the section is thus inadequate in shear.
Either a higher concrete grade must be employed or a thicker section must be adopted.
Location: Shallow slab. Bending only. Mild steel reinforcement.

Analysis of wall/slab section to requirements of BS8007

and/or BS8110

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Ultimate moment per metre width \( M=30 \text{ kNm/m} \)

Service moment per metre width \( Mz'=21 \text{ kNm/m} \)

Maximum permissible crack width \( Cw=0.1 \text{ mm} \)

Shear force due to ultimate loads \( V=50 \text{ kN/m} \)

Ultimate axial load (−ve tension) \( Nult'=0 \text{ kN/m} \)

Service axial load (−ve tension) \( N'=0 \text{ kN/m} \)

Characteristic concrete strength \( fcu=35 \text{ N/mm}^2 \)

Elastic modulus of concrete \( Ec=12000 \text{ N/mm}^2 \)

Characteristic steel strength \( fy=250 \text{ N/mm}^2 \)

Overall depth of concrete section \( h=200 \text{ mm} \)

Size of tension bars \( diat=20 \text{ mm} \)

Spacing of tension bars \( spac=100 \text{ mm} \)

Maximum dia. of coarse aggregate \( hagg=20 \text{ mm} \)

Size of compression bars \( diac=12 \text{ mm} \)

Spacing of compression bars \( spacc=150 \text{ mm} \)

Designated exposure class is XC4

Specified fixing tolerance \( tol=10 \text{ mm} \)

Nominal concrete cover \( cover=40 \text{ mm} \)

Chosen effective depth of section \( d=150 \text{ mm} \)

Chosen depth to compression bars \( d'=46 \text{ mm} \)

Temp. zone depth for tension steel \( trz=100 \text{ mm} \)

Temp. zone depth for comp. steel \( crz=100 \text{ mm} \)

Temp. fall: hydration peak and ambient \( T_1=20 ^\circ C \)

Seasonal variation in temperature \( T_2=5 ^\circ C \)

Chosen ratio of \( fct/fb \) \( fratio=1 \)

External restraint factor \( R=0.5 \)

Thermal expansion coeff. for concrete \( \alpha=10E-6 \)

Direct tensile strength \( fct=1.6 \text{ N/mm}^2 \)
REINFORCEMENT

Tension steel:  
Size: 20 mm

SUMMARY

Spacing: 100 mm
Area required: 968.42 mm²/m
Area provided: 3141.6 mm²/m
Percentages: (pcrit) 3.1416 %
  provided: (gross) 1.5708 %
Weight provided: 24.662 kg/m²

Compression steel:  
Size: 12 mm
Spacing: 150 mm
Area required: 0 mm²/m
Area provided: 753.98 mm²/m
Percentages: (pcrit) 0.75398 %
  provided: (gross) 0.37699 %
Weight provided: 5.9188 kg/m²

Cover to all steel: 40 mm
Total weight of steel: 30.58 kg/m²

satisfies:
1. Ultimate limit-state
2. Service limit-state cracking in flexure
3. Temperature-cracking requirements

Calculated

Flexure: In tension face: between bars 0.042077 mm
Turmpature change: In tension face 0.039789 mm
In compression face 0.099472 mm
Location: Ex1 - Compression & bending on substantial section

Analysis of wall/slab section to EC2 Part 3 and/or EC2 Part 1-1

Calculations are undertaken for a section 1.0 m in width.

Section under consideration is assumed to be subjected primarily to bending.

Design partial safety factor \( g_{am}=1.15 \) (reinforcement)

Design BM before redistribution \( M_{Ed}'=950 \) kNm/m

Design BM after redistribution \( M_{Ed}=950 \) kNm/m

Maximum permissible crack width \( w_k=0.2 \) mm

Design shear force \( V_{Ed}=300 \) kNm/m

Design axial load (-ve tension) \( N_{Ed}'=400 \) kN/m

Service axial load (-ve tension) \( N'=280 \) kN/m

Characteristic steel strength \( f_yk=500 \) N/mm²

Diameter of tension bars \( d_{iat}=32 \) mm

Spacing of tension bars \( d_{spac}=125 \) mm

Maximum dia. of coarse aggregate \( h_{agg}=20 \) mm

Diameter of compression bars \( d_{iac}=25 \) mm

Spacing of compression bars \( d_{spacc}=200 \) mm

Chosen effective depth of section \( d=944 \) mm

Chosen depth to compression bars \( d'=52.5 \) mm

Temp.fall: hydrat. peak & ambient \( T_1=20 \) °C

Long-term drop in temperature \( T_2=5 \) °C

### DESIGN

- Overall depth: 1000 mm
- Effective depth: 944 mm

### SUMMARY

- Parameter \( K \): 0.0381
- Parameter \( K' \): 0.1684

### FLEXURE

- Lever arm ratio \( z/d \): 0.95
- Steel area (tension): 2436 mm²/m
- Steel percentage req.: 0.2581%
- Minimum area of steel: 1358 mm²/m
- Maximum area of steel: 40000 mm²/m
- Distribution steel: 1358 mm²/m

### Shear resistance

- Term for shear resistance: \( v_{Rdc} = (\rho_1 \cdot f_{ck})^{1/3} = 2.672 \)
- Shear resistance: \( V_{Rdc} = C_{Rdc} \cdot k_s \cdot v_{Rdc} \cdot b \cdot w / d / 1000 = 442.1 \) kN
- Minimum stress: \( v_{min} = 0.035 \cdot k_s \cdot 1.5 \cdot f_{ck} \cdot 0.5 = 0.3268 \) N/mm²
- Minimum shear resistance: \( V_{Rdm} = v_{min} \cdot b \cdot w / d / 1000 = 308.5 \) kN
SHEAR SUMMARY

Design shear force 300 kN
Design shear resistance 442.1 kN

NOTE: The axial load has been ignored in the above shear calculation.

Flexural cracking caused by loading

Coefficient (ribbed bars) k1=1.14
Maximum crack spacing (Exp. 7.11) srmax=3.4*c+0.425*k1*k2*diat/ppeff =304.68 mm
Crack width (Expression 7.8) wk1=srmax*ecr=0.13469 mm
The crack width 0.13469 mm is less than the permissible value of 0.2 mm hence satisfactory.

Shrinkage & thermal cracking caused by edge restraint to movement

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is -d). Coefficient of thermal expansion aTc=10 με/øC Coefficient k (to CIRIA C660) k=0.65 The full thickness of the section will be used below for evaluating the area of concrete within the tensile zone.

Area of conc.within tensile zone Act=b*h=1E6 mm²
The minimum steel area below is applicable in both directions.
Minimum steel area required Asmin=kc*k*Act*fcteff/fyk=3614 mm²/m
Min steel area required each face Asmin'=1807 mm²/m
The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement (outside layer):
Shrinkage strain (short-term) ecas=13.6 με
Shrinkage strain (long-term) ecal=45 με
The short-term crack width 0.033135 mm is less than the permissible value of 0.2 mm hence satisfactory.
The long-term crack width 0.069726 mm is less than the permissible value of 0.2 mm hence satisfactory.

(B) Compression reinforcement (outside layer):
Effective concrete area 1 a1c=b*2.5*(c'+diac/2)=131250 mm²
Effective concrete area 2 a2c=b*h/2=500000 mm²
The short-term crack width 0.054861 mm is less than the permissible value of 0.2 mm hence satisfactory.
The long-term crack width 0.11544 mm is less than the permissible value of 0.2 mm hence satisfactory.
<table>
<thead>
<tr>
<th></th>
<th>Size</th>
<th>32 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tension: steel</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Spacing</strong></td>
<td>125 mm</td>
<td></td>
</tr>
<tr>
<td><strong>Area required</strong></td>
<td>2436.4 mm²/m</td>
<td></td>
</tr>
<tr>
<td><strong>REINFORCEMENT</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Area provided</strong></td>
<td>6434 mm²/m</td>
<td></td>
</tr>
<tr>
<td><strong>Percentage provided</strong></td>
<td>0.6434 %</td>
<td></td>
</tr>
<tr>
<td><strong>Weight provided</strong></td>
<td>50.507 kg/m²</td>
<td></td>
</tr>
<tr>
<td><strong>Compression: steel</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Spacing</strong></td>
<td>200 mm</td>
<td></td>
</tr>
<tr>
<td><strong>Area required</strong></td>
<td>0 mm²/m</td>
<td></td>
</tr>
<tr>
<td><strong>Area provided</strong></td>
<td>2454.4 mm²/m</td>
<td></td>
</tr>
<tr>
<td><strong>Percentage provided</strong></td>
<td>0.24544 %</td>
<td></td>
</tr>
<tr>
<td><strong>Weight provided</strong></td>
<td>19.267 kg/m²</td>
<td></td>
</tr>
<tr>
<td><strong>Distrib.steel:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Area of steel</strong></td>
<td>2010.6 mm²/m</td>
<td></td>
</tr>
<tr>
<td><strong>tension face</strong></td>
<td>Diameter</td>
<td>16 mm</td>
</tr>
<tr>
<td><strong>Spacing</strong></td>
<td>100 mm</td>
<td></td>
</tr>
<tr>
<td><strong>compr.face</strong></td>
<td>Diameter</td>
<td>16 mm</td>
</tr>
<tr>
<td><strong>Spacing</strong></td>
<td>100 mm</td>
<td></td>
</tr>
<tr>
<td><strong>Nominal cover to all tension face steel</strong></td>
<td>40 mm</td>
<td></td>
</tr>
<tr>
<td><strong>Nominal cover to all compr.face steel</strong></td>
<td>40 mm</td>
<td></td>
</tr>
<tr>
<td><strong>Total weight of reinforcement</strong></td>
<td>85.557 kg/m²</td>
<td></td>
</tr>
</tbody>
</table>

**satisfies:**
1. Ultimate limit-state
2. Service limit-state cracking in flexure
3. Temperature-cracking requirements
4. Fire-resistance requirements

**Calculated crack widths**
- **Flexure:** In tension face: between bars 0.13469 mm
- **Temperature change:**
  - Short-term (tension face) 0.033135 mm
  - Long-term (tension face) 0.069726 mm
  - Short-term (compression face) 0.054861 mm
  - Long-term (compression face) 0.11544 mm

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**Location: Ex2 - Tension and bending**

Analysis of wall/slab section to EC2 Part 3 and/or EC2 Part 1-1

Calculations are undertaken for a section 1.0 m in width.

Section under consideration is assumed to be subjected primarily to bending.

Design partial safety factor \( \gamma_{ms} = 1.15 \) (reinforcement)
Design BM before redistribution \( M_{Ed}' = 950 \text{ kNm/m} \)
Design BM after redistribution \( M_{Ed} = 950 \text{ kNm/m} \)
Maximum permissible crack width \( w_k = 0.2 \text{ mm} \)
Design shear force \( V_{Ed} = 325 \text{ kN/m} \)
Design axial load (-ve tension) \( N_{Ed}' = -1100 \text{ kN/m} \)
Service axial load (-ve tension) \( N' = -770 \text{ kN/m} \)
Characteristic steel strength \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of tension bars \( d_i = 25 \text{ mm} \)
Spacing of tension bars \( s_p = 85 \text{ mm} \)
Maximum dia. of coarse aggregate \( h_{agg} = 20 \text{ mm} \)
Diameter of compression bars \( d_{ac} = 20 \text{ mm} \)
Spacing of compression bars \( s_{acc} = 150 \text{ mm} \)
Chosen effective depth of section \( d = 947.5 \text{ mm} \)
Chosen depth to compression bars \( d' = 50 \text{ mm} \)
Temp. fall: hydrat. peak & ambient \( T_1 = 20 \degree \text{ C} \)
Long-term drop in temperature \( T_2 = 5 \degree \text{ C} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Overall depth</th>
<th>1000 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Effective depth</td>
<td>947.5 mm</td>
</tr>
<tr>
<td>FLEXURE</td>
<td>Parameter K</td>
<td>0.0378</td>
</tr>
<tr>
<td></td>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td></td>
<td>Lever arm ratio z/d</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>Steel area (tension)</td>
<td>2427 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Steel percentage req.</td>
<td>0.2562 %</td>
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<tr>
<td></td>
<td>Minimum area of steel</td>
<td>1363 mm²/m</td>
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<td></td>
<td>Maximum area of steel</td>
<td>40000 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Distribution steel</td>
<td>1363 mm²/m</td>
</tr>
</tbody>
</table>

**Shear resistance**

Term for shear resistance \( \nu_{Rdc} = (\rho_1 f_{ck})^{1/3} = 2.575 \)
Shear resistance \( \nu_{Rdc} CR_{dc} ks V_{Rdc} bw d/1000 = 427.2 \text{ kN} \)
Minimum stress \( \nu_{min} = 0.035 ks^{1.5} f_{ck} 0.5 = 0.3265 \text{ N/mm}^2 \)
Minimum shear resistance \( \nu_{Rdm} = \nu_{min} bw d/1000 = 309.4 \text{ kN} \)
SHEAR SUMMARY

Design shear force 325 kN
Design shear resistance 427.2 kN

NOTE: The axial load has been ignored in the above shear calculation.

Flexural cracking caused by loading

Coefficient (ribbed bars) $k_1=1.14$
Maximum crack spacing (Expression 7.11) $s_{rmax}=3.4c+0.425k_1k_2\frac{d_{iat}}{p_{peff}}=273.64$ mm
Crack width (Expression 7.8) $w_{k_1}=s_{rmax}ec_{r}=0.1367$ mm
The crack width 0.1367 mm is less than the permissible value of 0.2 mm hence satisfactory.

Shrinkage & thermal cracking caused by edge restraint to movement

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is $-\infty$).
Coefficient of thermal expansion $a_{Tc}=10 \mu e/\circ C$
Coefficient $k$ (to CIRIA C660) $k=0.65$
The full thickness of the section will be used below for evaluating the area of concrete within the tensile zone.
Area of conc. within tensile zone $A_{ct}=bh=1E6$ mm$^2$
The minimum steel area below is applicable in both directions.
Minimum steel area required $A_{smin}=kc_kA_{ct}f_{cet eff}/f_{yk}=3614$ mm$^2/m$
Min steel area required each face $A_{smin}'=1807$ mm$^2/m$
The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement (outside layer):
Shrinkage strain (short-term) $e_{cas}=13.6 \mu e$
Shrinkage strain (long-term) $e_{cal}=45 \mu e$
The short-term crack width 0.02879 mm is less than the permissible value of 0.2 mm hence satisfactory.
The long-term crack width 0.060582 mm is less than the permissible value of 0.2 mm hence satisfactory.

(B) Compression reinforcement (outside layer):
Effective concrete area 1 $a_{1c}=b(2.5(c'+d_{iac}/2))=125000$ mm$^2$
Effective concrete area 2 $a_{2c}=bh/2=500000$ mm$^2$
The short-term crack width 0.050003 mm is less than the permissible value of 0.2 mm hence satisfactory.
The long-term crack width 0.10522 mm is less than the permissible value of 0.2 mm hence satisfactory.
<table>
<thead>
<tr>
<th></th>
<th>Size</th>
<th>Spacing</th>
<th>Area required</th>
<th>Percentage provided</th>
<th>Weight provided</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tension:</strong></td>
<td>25 mm</td>
<td>85 mm</td>
<td>4957.4 mm²/m</td>
<td>0.5775 %</td>
<td>45.334 kg/m²</td>
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<tr>
<td><strong>steel</strong></td>
<td></td>
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<td></td>
</tr>
<tr>
<td><strong>REINFORCEMENT</strong></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Compression:</strong></td>
<td>20 mm</td>
<td>150 mm</td>
<td>2094.4 mm²/m</td>
<td>0.20944 %</td>
<td>16.441 kg/m²</td>
</tr>
<tr>
<td><strong>steel</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Nominal cover to all tension face steel: 40 mm  
Nominal cover to all compr. face steel: 40 mm  
Total weight of reinforcement: 61.775 kg/m²  
satisfies:  
1. Ultimate limit-state  
2. Service limit-state cracking in flexure  
3. Temperature-cracking requirements  
4. Fire-resistance requirements  

Calculated crack widths:  
- Flexure: In tension face: between bars: 0.13671 mm  
- Temperature change: Short-term (tension face): 0.02879 mm  
- Temperature change: Short-term (compression face): 0.050003 mm  
- Long-term (tension face): 0.060582 mm  
- Long-term (compression face): 0.10522 mm
Location: Ex3 - Tension & bending (as Ex.2 but greater tension)

Analysis of wall/slab section to EC2 Part 3 and/or EC2 Part 1-1

Calculations are undertaken for a section 1.0 m in width.

Section under consideration is assumed to be subjected primarily to bending.

Design partial safety factor \( g_{am5} = 1.15 \) (reinforcement)

Design BM before redistribution \( M_{Ed'} = 950 \text{ kNm/m} \)

Design BM after redistribution \( M_{Ed} = 950 \text{ kNm/m} \)

Maximum permissible crack width \( w_k = 0.2 \text{ mm} \)

Design shear force \( V_{Ed} = 325 \text{ kN/m} \)

Service axial load (-ve tension) \( N' = -840 \text{ kN/m} \)

Characteristic steel strength \( f_{yk} = 500 \text{ N/mm}^2 \)

Diameter of tension bars \( d_{iat} = 25 \text{ mm} \)

Spacing of tension bars \( space = 85 \text{ mm} \)

Maximum dia. of coarse aggregate \( h_{agg} = 20 \text{ mm} \)

Diameter of compression bars \( d_{iac} = 20 \text{ mm} \)

Spacing of compression bars \( spacec = 150 \text{ mm} \)

Chosen effective depth of section \( d = 947.5 \text{ mm} \)

Chosen depth to compression bars \( d' = 50 \text{ mm} \)

Temp.fall: hydrat. peak & ambient \( T_1 = 20 \, ^\circ\text{C} \)

Long-term drop in temperature \( T_2 = 5 \, ^\circ\text{C} \)

**DESIGN**

- Overall depth: 1000 mm
- Effective depth: 947.5 mm

**SUMMARY**

- Parameter K: 0.0378
- Parameter K': 0.1684
- Lever arm ratio \( z/d \): 0.95
- Steel area (tension): 2427 mm²/m
- Steel percentage req.: 0.2562 %
- Minimum area of steel: 1363 mm²/m
- Maximum area of steel: 40000 mm²/m
- Distribution steel: 1363 mm²/m

**FLEXURE**

- Term for shear resistance: \( \nu_{Rdc} = (\rho_1 \cdot f_{ck})^{(1/3)} = 2.575 \)
- Shear resistance: \( \nu_{Rdc} = 0.035 \cdot k_s \cdot \nu_{Rdc} \cdot b \cdot d / 1000 = 427.2 \text{ kN} \)
- Minimum stress: \( \nu_{m} = 0.3265 \text{ N/mm}^2 \)
- Minimum shear resistance: \( \nu_{Rdm} = b \cdot d / 1000 = 309.4 \text{ kN} \)
Shear Summary

Design shear force: 325 kN
Design shear resistance: 427.2 kN

NOTE: The axial load has been ignored in the above shear calculation.

Flexural cracking caused by loading

Coefficient (ribbed bars) $k_1 = 1.14$
Maximum crack spacing (Expression 7.11) $s_{rmax} = 3.4c + 0.425k_1k_2\frac{diat}{p_{eff}} = 273.64$ mm
Crack width (Expression 7.8) $w_{k1} = s_{rmax}ecr = 0.13671$ mm
The crack width 0.13671 mm is less than the permissible value of 0.2 mm hence satisfactory.

Shrinkage & thermal cracking caused by edge restraint to movement

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is $-\delta$).
Coefficient of thermal expansion $aTc = 10 \mu e/\circ C$
Coefficient $k$ (to CIRIA C660) $k = 0.65$
The full thickness of the section will be used below for evaluating the area of concrete within the tensile zone.
Area of conc. within tensile zone $A_{ct} = bh = 1E6$ mm$^2$
The minimum steel area below is applicable in both directions.
Minimum steel area required $A_{smin} = k_c k Actf_{ceff}/fyk = 3614$ mm$^2$/m
Min steel area required each face $A_{smin}' = 1807$ mm$^2$/m
The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement (outside layer):
Shrinkage strain (short-term) $\varepsilon_{cas} = 13.6 \mu e$
Shrinkage strain (long-term) $\varepsilon_{cal} = 45 \mu e$
The short-term crack width 0.02879 mm is less than the permissible value of 0.2 mm hence satisfactory.
The long-term crack width 0.060582 mm is less than the permissible value of 0.2 mm hence satisfactory.

(B) Compression reinforcement (outside layer):
Effective concrete area 1 $a_{1c} = b(2.5)(c' + diac/2) = 125000$ mm$^2$
Effective concrete area 2 $a_{2c} = bh/2 = 500000$ mm$^2$
The short-term crack width 0.050003 mm is less than the permissible value of 0.2 mm hence satisfactory.
The long-term crack width 0.10522 mm is less than the permissible value of 0.2 mm hence satisfactory.
<table>
<thead>
<tr>
<th></th>
<th>Size</th>
<th>Spacing</th>
<th>Area required</th>
<th>Area provided</th>
<th>Percentage provided</th>
<th>Weight provided</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tension: steel</strong></td>
<td>25 mm</td>
<td>85 mm</td>
<td>5187.4 mm²/m</td>
<td>5775 mm²/m</td>
<td>0.5775 %</td>
<td>45.334 kg/m²</td>
</tr>
<tr>
<td><strong>Compression: steel</strong></td>
<td>20 mm</td>
<td>150 mm</td>
<td>0 mm²/m</td>
<td>2094.4 mm²/m</td>
<td>0.20944 %</td>
<td>16.441 kg/m²</td>
</tr>
</tbody>
</table>

**REINFORCEMENT SUMMARY**

<p>| | | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Nominal cover to all tension face steel</strong></td>
<td>40 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Nominal cover to all compr.face steel</strong></td>
<td>40 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total weight of reinforcement</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>61.775 kg/m²</td>
</tr>
</tbody>
</table>

**satisfies:**

1. Ultimate limit-state
2. Service limit-state cracking in flexure
3. Temperature-cracking requirements
4. Fire-resistance requirements

**Calculated crack widths**

- Flexure: In tension face: between bars 0.13671 mm
- Temperature change: Short-term (tension face) 0.02879 mm
- Temperature change: Short-term (compression face) 0.050003 mm
- Long-term (tension face) 0.060582 mm
- Long-term (compression face) 0.10522 mm
Location: Ex4 - Shallow slab (bending only)

Analysis of wall/slab section to EC2 Part 3 and/or EC2 Part 1-1

Calculations are undertaken for a section 1.0 m in width.

Section under consideration is assumed to be subjected primarily to bending.

Design partial safety factor \( \gamma_{ms} = 1.15 \) (reinforcement)
Design BM before redistribution \( M_{Ed}' = 30 \) kNm/m
Design BM after redistribution \( M_{Ed} = 30 \) kNm/m
Maximum permissible crack width \( \omega_k = 0.2 \) mm
Design shear force \( V_{Ed} = 50 \) kN/m
Design axial load (-ve tension) \( N_{Ed}' = 0 \) kN/m
Service axial load (-ve tension) \( N' = 0 \) kN/m
Characteristic steel strength \( f_{yk} = 500 \) N/mm²
Diameter of tension bars \( d_{iat} = 20 \) mm
Spacing of tension bars \( \text{spac} = 150 \) mm
Maximum dia.of coarse aggregate \( h_{agg} = 20 \) mm
Diameter of compression bars \( d_{iac} = 12 \) mm
Spacing of compression bars \( \text{spacc} = 125 \) mm
Chosen effective depth of section \( d = 150 \) mm
Chosen depth to compression bars \( d' = 46 \) mm
Temp. fall: hydrat. peak & ambient \( T_1 = 20 ^\circ C \)
Long-term drop in temperature \( T_2 = 5 ^\circ C \)

**DESIGN**

<table>
<thead>
<tr>
<th>Overall depth</th>
<th>200 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective depth</td>
<td>150 mm</td>
</tr>
</tbody>
</table>

**SUMMARY**

<table>
<thead>
<tr>
<th>Parameter K</th>
<th>0.0476</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Lever arm ratio ( z/d )</th>
<th>0.95</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Steel area (tension)</th>
<th>484.2 mm²/m</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Steel percentage req.</th>
<th>0.3228 %</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Minimum area of steel</th>
<th>215.8 mm²/m</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Maximum area of steel</th>
<th>8000 mm²/m</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Distribution steel</th>
<th>215.8 mm²/m</th>
</tr>
</thead>
</table>

**Shear resistance**

<table>
<thead>
<tr>
<th>Term for shear resistance</th>
<th>( VR_{dc} = (\rho_1 f_{ck})^{(1/3)} = 3.394 )</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Shear resistance</th>
<th>( VR_{dc} = CR_{dc} * ks * bw * d/1000 = 122.2 ) kN</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Minimum stress</th>
<th>( v_{min} = 0.035 * ks * 1.5 * f_{ck} * 0.5 = 0.5238 ) N/mm²</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Minimum shear resistance</th>
<th>( VR_{dm} = v_{min} * bw * d/1000 = 78.57 ) kN</th>
</tr>
</thead>
</table>
### SHEAR SUMMARY

<table>
<thead>
<tr>
<th>Flexural cracking caused by loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear force</td>
</tr>
<tr>
<td>Design shear resistance</td>
</tr>
</tbody>
</table>

Coefficient (ribbed bars) \( k_1 = 1.14 \)

Maximum crack spacing (Exp. 7.11) \( s_{\text{rmax}} = 3.4c + 0.425k_1k_2\text{diat/peff} = 251.05 \text{ mm} \)

Crack width (Expression 7.8) \( w_{k1} = s_{\text{rmax}} \text{ecr} = 0.056751 \text{ mm} \)

The crack width 0.056751 mm is less than the permissible value of 0.2 mm hence satisfactory.

### Shrinkage & thermal cracking caused by edge restraint to movement

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is \( -\infty \)).

Coefficient of thermal expansion \( a_{TC} = 10 \text{ æî/øC} \)

The full thickness of the section will be used below for evaluating the area of concrete within the tensile zone.

Area of conc.within tensile zone \( A_{ct} = bh = 200000 \text{ mm\^2} \)

The minimum steel area below is applicable in both directions.

Minimum steel area required \( A_{smin} = k_c k A_{ct} f_{ct\text{eff}}/f_{yk} = 1112 \text{ mm\^2/m} \)

Min steel area required each face \( A_{smin'} = 556 \text{ mm\^2/m} \)

The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement (outside layer): Shrinkage strain (short-term) \( \varepsilon_{\text{cas}} = 13.6 \text{ æî} \)
Shrinkage strain (long-term) \( \varepsilon_{\text{cal}} = 45 \text{ æî} \)

The short-term crack width 0.041906 mm is less than the permissible value of 0.2 mm hence satisfactory.

The long-term crack width 0.09836 mm is less than the permissible value of 0.2 mm hence satisfactory.

(B) Compression reinforcement (outside layer):

Effective concrete area 1 \( a_{1c} = bh(2.5c') = 115000 \text{ mm\^2} \)

Effective concrete area 2 \( a_{2c} = bh/2 = 100000 \text{ mm\^2} \)

The short-term crack width 0.054501 mm is less than the permissible value of 0.2 mm hence satisfactory.

The long-term crack width 0.12792 mm is less than the permissible value of 0.2 mm hence satisfactory.
Tension:

- **Steel**
  - Size: 20 mm
  - Spacing: 150 mm
  - Area required: 484.21 mm²/m

**REINFORCEMENT**

**SUMMARY**

- **Compression**
  - Size: 12 mm
  - Spacing: 125 mm
  - Area provided: 904.78 mm²/m
  - Percentage provided: 0.45239 %
  - Weight provided: 7.1025 kg/m²

Nominal cover to all tension face steel: 40 mm
Nominal cover to all compr. face steel: 40 mm
Total weight of reinforcement: 23.544 kg/m²

satisfies:

1. Ultimate limit-state
2. Service limit-state cracking in flexure
3. Temperature-cracking requirements
4. Fire-resistance requirements

**Calculated Crack widths**

- Flexure: In tension face: between bars: 0.056751 mm
- Temperature change: Short-term (tension face): 0.041906 mm
- Long-term (tension face): 0.09836 mm
- Temperature change: Short-term (compression face): 0.054501 mm
- Long-term (compression face): 0.12792 mm
Location: Ex5 Edge restraint (TCC Concrete Basements Section 10)

Analysis of wall/slab section to EC2 Part 3 and/or EC2 Part 1-1

Calculations are undertaken for a section 1.0 m in width. Section under consideration is assumed to be subjected primarily to bending.

Design partial safety factor gams=1.15 (reinforcement)
Design BM before redistribution MEEd'=34.1 kNm/m
Design BM after redistribution MEEd=34.1 kNm/m
Maximum permissible crack width wk=0.2 mm
Design shear force VEd=82 kN/m
Design axial load (-ve tension) NEEd'=0 kN/m
Service axial load (-ve tension) N'=0 kN/m
Characteristic steel strength fyk=500 N/mm²
Diameter of tension bars diat=16 mm
Spacing of tension bars spac=125 mm
Maximum dia. of coarse aggregate hagg=20 mm
Diameter of compression bars diac=12 mm
Spacing of compression bars spacc=100 mm
Effective depth of section d=176 mm
Chosen depth to compression bars d'=48 mm
Temp. fall: hydrat. peak & ambient T1=18 °C
Long-term drop in temperature T2=10 °C

DESIGN

Overall depth 250 mm
Effective depth 176 mm
Parameter K 0.0367
Parameter K' 0.1684
Lever arm ratio z/d 0.95
Steel area (tension) 469.1 mm²/m
Steel percentage req. 0.2665 %
Minimum area of steel 265.1 mm²/m
Maximum area of steel 10000 mm²/m
Distribution steel 265.1 mm²/m

SUMMARY

FLEXURE

Shear resistance

Term for shear resistance VRdc=(rho1*fck)^(1/3)=3.015
Shear resistance VRdc=CRdc*ks*VRdc*bw*d/1000=127.4 kN
Minimum stress vmin=0.035*ks^1.5*fck^0.5=0.5422 N/mm²
Minimum shear resistance VRdm=vmin*bw*d/1000=95.43 kN
**Shear Summary**

**Design shear force:** 82 kN  
**Design shear resistance:** 127.4 kN

**Flexural cracking caused by loading**

Coefficient (ribbed bars) \( k_1 = 0.8 \)

Maximum crack spacing (Exp. 7.11) \( s_{r_{\text{max}}} = 3.4 \cdot c + 0.425 \cdot k_1 \cdot k_2 \cdot \text{diat}/p_{\text{peff}} = 337.34 \text{ mm} \)

Crack width (Expression 7.8) \( w_{k_1} = s_{r_{\text{max}}} \cdot e_{\text{cr}} = 0.094186 \text{ mm} \)

The crack width 0.094186 mm is less than the permissible value of 0.2 mm hence satisfactory.

**Shrinkage & thermal cracking caused by edge restraint to movement**

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is \( -\infty \)).

Coefficient of thermal expansion \( a_{Tc} = 12 \cdot \mu \text{r}/\degree \text{C} \)

The full thickness of the section will be used below for evaluating the area of concrete within the tensile zone.

Area of conc. within tensile zone \( A_{t} = b \cdot h = 250000 \text{ mm}^2 \)

The minimum steel area below is applicable in both directions.

Minimum steel area required \( A_{s_{\min}} = k_c \cdot k \cdot A_{t} \cdot f_{c_{\text{teff}}} / f_{yk} = 1450 \text{ mm}^2 / \text{m} \)

Min steel area required each face \( A_{s_{\min}'} = 725 \text{ mm}^2 / \text{m} \)

The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension face reinforcement:

Shrinkage strain (short-term) \( e_{c_{as}} = 15 \cdot \mu \text{r} \)

Shrinkage strain (long-term) \( e_{c_{al}} = 50 \cdot \mu \text{r} \)

Creep coefficient \( K = 1 \)

Calculation assumes internal condition (i.e. ambient RH=45%).

Design crack width (long-term) \( w_{k_{2l}} = s_{r_{\text{max}}} \cdot e_{c_{rl}} / 10^6 = 0.19868 \text{ mm} \)

The short-term crack width 0.050155 mm is less than the permissible value of 0.2 mm hence satisfactory.

The long-term crack width 0.19868 mm is less than the permissible value of 0.2 mm hence satisfactory.

(B) Compression face reinforcement:

Effective concrete area 1 \( a_{1c} = b \cdot 2.5 \cdot (c' + \text{diac}/2) = 120000 \text{ mm}^2 \)

Effective concrete area 2 \( a_{2c} = b \cdot h / 2 = 125000 \text{ mm}^2 \)

The short-term crack width 0.044617 mm is less than the permissible value of 0.2 mm hence satisfactory.

The long-term crack width 0.17674 mm is less than the permissible value of 0.2 mm hence satisfactory.
<table>
<thead>
<tr>
<th></th>
<th>Size</th>
<th>Spacing</th>
<th>Area required</th>
<th>Area provided</th>
<th>Percentage provided</th>
<th>Weight provided</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tension:</strong> steel</td>
<td>16 mm</td>
<td>125 mm</td>
<td>469.08 mm²/m</td>
<td>1608.5 mm²/m</td>
<td>0.6434 %</td>
<td>12.627 kg/m²</td>
</tr>
<tr>
<td><strong>Compression:</strong> steel</td>
<td>12 mm</td>
<td>100 mm</td>
<td>0 mm²/m</td>
<td>1131 mm²/m</td>
<td>0.45239 %</td>
<td>8.8781 kg/m²</td>
</tr>
</tbody>
</table>

**SUMMARY**

- **Percentage provided**: 0.6434 %
- **Weight provided**: 12.627 kg/m²
- **Percentage provided**: 0.45239 %
- **Weight provided**: 8.8781 kg/m²

**Nominal cover to all tension face steel**: 50 mm
**Nominal cover to all compr.face steel**: 30 mm

**Total weight of reinforcement**: 21.505 kg/m²

**satisfies**:  
1. Ultimate limit-state  
2. Service limit-state cracking in flexure  
3. Temperature-cracking requirements  
4. Fire-resistance requirements

**Calculated crack widths**

- **Flexure**: In tension face: between bars 0.094186 mm  
- **Temperature change**: Short-term (tension face) 0.050155 mm  
- **Temperature change**: Long-term (tension face) 0.19868 mm  
- **Temperature change**: Short-term (compression face) 0.044617 mm  
- **Temperature change**: Long-term (compression face) 0.17674 mm
Location: Ex6 Slab with end restraint, TCC Concrete Basements Sec.10

Analysis of wall/slab section to EC2 Part 3 and/or EC2 Part 1-1

Calculations are undertaken for a section 1.0 m in width.

Section under consideration is assumed to be subjected primarily to bending.

Design partial safety factor \( \gamma_{ms} = 1.15 \) (reinforcement)

Design BM before redistribution \( M_{Ed}' = 78 \text{ kNm/m} \)

Design BM after redistribution \( M_{Ed} = 78 \text{ kNm/m} \)

Maximum permissible crack width \( w_k = 0.22 \text{ mm} \)

Design shear force \( V_{Ed} = 82 \text{ kN/m} \)

Design axial load (-ve tension) \( N_{Ed}' = 0 \text{ kN/m} \)

Service axial load (-ve tension) \( N' = 0 \text{ kN/m} \)

Characteristic steel strength \( f_{yk} = 500 \text{ N/mm}^2 \)

Diameter of tension bars \( d_{iat} = 20 \text{ mm} \)

Spacing of tension bars \( s_{pac} = 100 \text{ mm} \)

Maximum dia. of coarse aggregate \( h_{agg} = 20 \text{ mm} \)

Diameter of compression bars \( d_{iac} = 20 \text{ mm} \)

Spacing of compression bars \( s_{pac} = 100 \text{ mm} \)

Chosen effective depth of section \( d = 240 \text{ mm} \)

Chosen depth to compression bars \( d' = 40 \text{ mm} \)

Temp. fall: hydrat. peak & ambient \( T_1 = 18 \text{ °C} \)

Long-term drop in temperature \( T_2 = 10 \text{ °C} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
<th>FLEXURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall depth</td>
<td>300 mm</td>
<td></td>
</tr>
<tr>
<td>Effective depth</td>
<td>240 mm</td>
<td></td>
</tr>
<tr>
<td>Parameter K</td>
<td>0.0451</td>
<td></td>
</tr>
<tr>
<td>Parameter K'</td>
<td>0.1684</td>
<td></td>
</tr>
<tr>
<td>Lever arm ratio ( z/d )</td>
<td>0.95</td>
<td></td>
</tr>
<tr>
<td>Steel area (tension)</td>
<td>786.8 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Steel percentage req.</td>
<td>0.3279 %</td>
<td></td>
</tr>
<tr>
<td>Minimum area of steel</td>
<td>361.5 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Maximum area of steel</td>
<td>12000 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Distribution steel</td>
<td>361.5 mm²/m</td>
<td></td>
</tr>
</tbody>
</table>

Shear resistance

\( V_{Rdc} = (\rho_1 \cdot f_{ck})^{1/3} = 3.399 \)

Shear resistance \( V_{Rdc} = C_{Rdc} \cdot k_s \cdot V_{Rdc} \cdot bw \cdot d/1000 = 187.3 \text{ kN} \)

Minimum stress \( v_{min} = 0.035 \cdot k_s \cdot 1.5 \cdot f_{ck} \cdot 0.5 = 0.5072 \text{ N/mm}^2 \)

Minimum shear resistance \( V_{Rdm} = v_{min} \cdot bw \cdot d/1000 = 121.7 \text{ kN} \)
**Shear Summary**

- **Design shear force**: 82 kN
- **Design shear resistance**: 187.3 kN

**Flexural Cracking caused by Loading**

- Coefficient (ribbed bars): \( k_1 = 0.8 \)
- Maximum crack spacing (Exp. 7.11): \( sr_{max} = 3.4c + 0.425k_1k_2\frac{d_{iat}}{p_{eff}} = 249.93 \text{ mm} \)
- Crack width (Expression 7.8): \( wk_1 = sr_{max}ecr = 0.061381 \text{ mm} \)

The crack width 0.061381 mm is less than the permissible value of 0.22 mm hence satisfactory.

**Shrinkage & Thermal Cracking caused by End Restraint to Movement**

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is \(-\infty\)).

- Coefficient of thermal expansion: \( \alpha_T = 12 \mu \text{m/°C} \)
- The full thickness of the section will be used below for evaluating the area of concrete within the tensile zone.
- Area of conc. within tensile zone: \( Act = b\times h = 300000 \text{ mm}^2 \)
- The minimum steel area below is applicable in both directions.
- Minimum steel area required: \( As_{min} = kc\times k\times Act\times f_{ct\text{eff}}/fyk = 1740 \text{ mm}^2/m \)
- Min steel area required each face: \( As_{min}' = 870 \text{ mm}^2/m \)

The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement (outside layer):

- The short-term crack width 0.11512 mm is less than the permissible value of 0.22 mm hence satisfactory.
- The long-term crack width 0.19297 mm is less than the permissible value of 0.22 mm hence satisfactory.

(B) Compression reinforcement (outside layer):

- Effective concrete area 1: \( a_{1c} = b\times 2.5\times (c' + d_{iat}/2) = 100000 \text{ mm}^2 \)
- Effective concrete area 2: \( a_{2c} = b\times h/2 = 150000 \text{ mm}^2 \)

The short-term crack width 0.074108 mm is less than the permissible value of 0.22 mm hence satisfactory.
- The long-term crack width 0.12423 mm is less than the permissible value of 0.22 mm hence satisfactory.

**Reinforcement Summary**

<table>
<thead>
<tr>
<th>Tension: steel</th>
<th>Size</th>
<th>20 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing</td>
<td>100 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>786.84 \text{ mm}^2/m</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>3141.6 \text{ mm}^2/m</td>
<td></td>
</tr>
<tr>
<td>Percentage provided</td>
<td>1.0472 %</td>
<td></td>
</tr>
<tr>
<td>Weight provided</td>
<td>24.662 \text{ kg/m}^2</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Compression: steel</th>
<th>Size</th>
<th>20 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing</td>
<td>100 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>0 \text{ mm}^2/m</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>3141.6 \text{ mm}^2/m</td>
<td></td>
</tr>
<tr>
<td>Percentage provided</td>
<td>1.0472 %</td>
<td></td>
</tr>
<tr>
<td>Weight provided</td>
<td>24.662 \text{ kg/m}^2</td>
<td></td>
</tr>
</tbody>
</table>

| Distrib. steel: tension face | Area of steel | 3141.6 \text{ mm}^2/m |
|------------------------------|----------------|
| Diameter                     | 20 mm          |
| Spacing                      | 100 mm         |
Distrib.steel: Area of steel 3141.6 mm²/m
compr.face Diameter 20 mm
Spacing 100 mm
Nominal cover to all tension face steel 50 mm
Nominal cover to all compr.face steel 30 mm
Total weight of reinforcement 73.985 kg/m²

satisfies:
1. Ultimate limit-state
2. Service limit-state cracking in flexure
3. Temperature-cracking requirements
4. Fire-resistance requirements

Calculated crack widths
Flexure: In tension face: between bars 0.061381 mm
Temperature change: Short-term 0.11512 mm
(tension face) Long-term 0.19297 mm
Temperature change: Short-term 0.074108 mm
(compression face) Long-term 0.12423 mm
Location: Ex7 Based on Doug Kay’s presentation at IStructE 10.01.12

Analysis of wall/slab section to EC2 Part 3 and/or EC2 Part 1-1

Calculations are undertaken for a section 1.0 m in width.

Section under consideration is assumed to be subjected primarily to bending.

Design partial safety factor \( \gamma_{ams} = 1.15 \) (reinforcement)
Design BM before redistribution \( M_{Ed'} = 250 \) kNm/m
Design BM after redistribution \( M_{Ed} = 250 \) kNm/m
Maximum permissible crack width \( w_k = 0.169 \) mm
Design shear force \( V_{Ed} = 150 \) kN/m
Design axial load (-ve tension) \( N_{Ed'} = 0 \) kN/m
Service axial load (-ve tension) \( N' = 0 \) kN/m
Characteristic steel strength \( f_{yk} = 500 \) N/mm²
Diameter of tension bars \( d_{iat} = 25 \) mm
Spacing of tension bars \( d_{pac} = 140 \) mm
Maximum dia. of coarse aggregate \( h_{agg} = 20 \) mm
 Diameter of compression bars \( d_{iac} = 25 \) mm
Spacing of compression bars \( d_{pacc} = 140 \) mm
Effective depth of section \( d = 397.5 \) mm
Chosen depth to compression bars \( d' = 52.5 \) mm
Temp. fall: hydrat. peak & ambient \( T_1 = 20 \) °C
Long-term drop in temperature \( T_2 = 10 \) °C

DESIGN

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Overall depth</td>
<td>450 mm</td>
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<tr>
<td>Effective depth</td>
<td>397.5 mm</td>
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<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter K</td>
<td>0.0527</td>
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<tr>
<td>Parameter K'</td>
<td>0.1684</td>
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<tr>
<td>Lever arm ratio z/d</td>
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</tr>
<tr>
<td>Steel area (tension)</td>
<td>1523 mm²/m</td>
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<tr>
<td>Steel percentage req.</td>
<td>0.3831 %</td>
</tr>
<tr>
<td>Minimum area of steel</td>
<td>598.7 mm²/m</td>
</tr>
<tr>
<td>Maximum area of steel</td>
<td>18000 mm²/m</td>
</tr>
<tr>
<td>Distribution steel</td>
<td>598.7 mm²/m</td>
</tr>
</tbody>
</table>

Shear resistance

Term for shear resistance \( \nu_{Rdc} = (\rho_{1} \cdot f_{ck})^{1/3} = 2.98 \)
Shear resistance \( \nu_{Rdc} = C_{Rdc} \cdot k_{s} \cdot \nu_{Rdc} \cdot b \cdot w_{d} / 1000 = 243 \) kN
Minimum stress \( \nu_{min} = 0.035 \cdot k_{s} \cdot 1.5 \cdot f_{ck} \cdot 0.5 = 0.4284 \) N/mm²
Minimum shear resistance \( \nu_{Rdm} = \nu_{min} \cdot b \cdot w_{d} / 1000 = 170.3 \) kN
SHEAR SUMMARY

**Design shear force**: 150 kN
**Design shear resistance**: 243 kN

**Flexural cracking caused by loading**

Coefficient (ribbed bars) \( k_1 = 0.8 \)
Maximum crack spacing (Exp. 7.11) \( s_{r\text{max}} = 3.4 \times c + 0.425 \times k_1 \times k_2 \times \text{diat/ppeff} = 273.19 \text{ mm} \)
Crack width (Expression 7.8) \( w_k = s_{r\text{max}} \times e_{cr} = 0.16391 \text{ mm} \)
The crack width 0.16391 mm is less than the permissible value of 0.169 mm hence satisfactory.

**Shrinkage & thermal cracking caused by edge restraint to movement**

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is \( -\infty \)).
Coefficient of thermal expansion \( a_Tc = 12 \mu e/\circ C \)
The full thickness of the section will be used below for evaluating the area of concrete within the tensile zone.
Area of conc.within tensile zone \( A_t = b \times h = 450000 \text{ mm}^2 \)
The minimum steel area below is applicable in both directions.
Minimum steel area required \( A_{s\text{min}} = k_c \times k \times A_t \times f_{c\text{teff}}/f_{yk} = 2336 \text{ mm}^2/\text{m} \)
Min steel area required each face \( A_{s\text{min}'} = 1168 \text{ mm}^2/\text{m} \)
The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension face reinforcement:
Shrinkage strain (short-term) \( e_{c\text{as}} = 15 \mu e \)
Shrinkage strain (long-term) \( e_{c\text{al}} = 50 \mu e \)
Creep coefficient \( K = 1 \)
Calculation assumes internal condition (i.e. ambient RH=45%).
Design crack width (long-term) \( w_{k2l} = s_{r\text{max}} \times e_{cr1}/10^6 = 0.14488 \text{ mm} \)
The short-term crack width 0.040649 mm is less than the permissible value of 0.169 mm hence satisfactory.
The long-term crack width 0.14488 mm is less than the permissible value of 0.169 mm hence satisfactory.

(B) Compression face reinforcement:
Effective concrete area 1 \( a_{1c} = b \times 2.5 \times (c' + \text{diac}/2) = 131250 \text{ mm}^2 \)
Effective concrete area 2 \( a_{2c} = b \times h/2 = 225000 \text{ mm}^2 \)
The short-term crack width 0.040649 mm is less than the permissible value of 0.169 mm hence satisfactory.
The long-term crack width 0.14488 mm is less than the permissible value of 0.169 mm hence satisfactory.
### Tension:

<table>
<thead>
<tr>
<th>Size</th>
<th>Spacing</th>
<th>Area required</th>
<th>REINFORCEMENT</th>
<th>Area provided</th>
<th>Percentage provided</th>
<th>Weight provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>25 mm</td>
<td>140 mm</td>
<td>1522.7 mm²/m</td>
<td></td>
<td>3506.2 mm²/m</td>
<td>0.77916 %</td>
<td>27.524 kg/m²</td>
</tr>
</tbody>
</table>

### Compression:

<table>
<thead>
<tr>
<th>Size</th>
<th>Spacing</th>
<th>Area required</th>
<th>Percentage provided</th>
<th>Weight provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>25 mm</td>
<td>140 mm</td>
<td>0 mm²/m</td>
<td>0.77916 %</td>
<td>27.524 kg/m²</td>
</tr>
</tbody>
</table>

#### Nominal cover to all tension face steel 40 mm
#### Nominal cover to all compr.face steel 40 mm
#### Total weight of reinforcement 55.048 kg/m²

**satisfies:**
1. Ultimate limit-state
2. Service limit-state cracking in flexure
3. Temperature-cracking requirements

**Calculated crack widths**
- **Flexure:** In tension face: between bars 0.16391 mm
- **Temperature change:** Short-term (tension face) 0.040649 mm, Long-term 0.14488 mm
- **Temperature change:** Short-term (compression face) 0.040649 mm, Long-term 0.14488 mm
Location: Ex8 Crack width=0.3

Analysis of wall/slab section to EC2 Part 3 and/or EC2 Part 1-1

Calculations are undertaken for a section 1.0 m in width.

Section under consideration is assumed to be subjected primarily to bending.

Design partial safety factor \( \gamma_{ams} = 1.15 \) (reinforcement)
Design BM before redistribution \( M_{Ed}' = 10 \text{ kNm/m} \)
Design BM after redistribution \( M_{Ed} = 10 \text{ kNm/m} \)
Maximum permissible crack width \( w_k = 0.3 \text{ mm} \)
Design shear force \( V_{Ed} = 1 \text{ kN/m} \)
Design axial load (-ve tension) \( N_{Ed}' = 0 \text{ kN/m} \)
Service axial load (-ve tension) \( N' = 0 \text{ kN/m} \)
Characteristic steel strength \( f_yk = 500 \text{ N/mm}^2 \)
Diameter of tension bars \( d_{iat} = 12 \text{ mm} \)
Spacing of tension bars \( s_{pac} = 150 \text{ mm} \)
Maximum dia.of coarse aggregate \( h_{agg} = 20 \text{ mm} \)
 Diameter of compression bars \( d_{iac} = 12 \text{ mm} \)
Spacing of compression bars \( s_{pac} = 150 \text{ mm} \)
Effective depth of section \( d = 114 \text{ mm} \)
Chosen depth to compression bars \( d' = 61 \text{ mm} \)

**DESIGN**

| Parameter                        | Value
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<thead>
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<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>Overall depth</td>
<td>175 mm</td>
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<tr>
<td>Effective depth</td>
<td>114 mm</td>
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**SUMMARY**

| Parameter                        | Value
<table>
<thead>
<tr>
<th></th>
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</thead>
<tbody>
<tr>
<td>Parameter K</td>
<td>0.024</td>
</tr>
<tr>
<td>Parameter K'</td>
<td>0.1684</td>
</tr>
<tr>
<td>Lever arm ratio z/d</td>
<td>0.95</td>
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<tr>
<td>Steel area (tension)</td>
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</tr>
<tr>
<td>Steel percentage req.</td>
<td>0.1863 %</td>
</tr>
<tr>
<td>Minimum area of steel</td>
<td>179.3 mm²/m</td>
</tr>
<tr>
<td>Maximum area of steel</td>
<td>7000 mm²/m</td>
</tr>
<tr>
<td>Distribution steel</td>
<td>179.3 mm²/m</td>
</tr>
</tbody>
</table>

**FLEXURE**

**Shear resistance**

Term for shear resistance \( v_{Rdc} = (\rho_1 f_{ck})^{1/3} = 2.766 \)
Shear resistance \( V_{Rdc} = CR_{dc} * k_{s} * V_{Rdc} * b_{w} * d / 1000 = 75.68 \text{ kN} \)
Minimum stress \( v_{min} = 0.035 * k_{s} * 1.5 * f_{ck} * 0.5 = 0.56 \text{ N/mm}^2 \)
Minimum shear resistance \( V_{Rdm} = v_{min} * b_{w} * d / 1000 = 63.84 \text{ kN} \)
**SHEAR SUMMARY**

Design shear force  
1 kN

Design shear resistance  
75.68 kN

**Flexural cracking caused by loading**

Coefficient (ribbed bars)  
k1=0.8

Maximum crack spacing (Exp. 7.11)  
srmax=3.4*c+0.425*k1*k2*diat/ppeff  
=319.68 mm

Crack width (Expression 7.8)  
wk1=srmax*ecr=0.10123 mm

The crack width 0.10123 mm is less than the permissible value of 0.3 mm hence satisfactory.

<table>
<thead>
<tr>
<th>Tension:</th>
<th>Size</th>
<th>Spacing</th>
<th>Area required</th>
<th>Area provided</th>
<th>Percentage provided</th>
<th>Weight provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>steel</td>
<td>12 mm</td>
<td>150 mm</td>
<td>212.37 mm²/m</td>
<td>753.98 mm²/m</td>
<td>0.43085 %</td>
<td>5.9188 kg/m²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Compression:</th>
<th>Size</th>
<th>Spacing</th>
<th>Area required</th>
<th>Area provided</th>
<th>Percentage provided</th>
<th>Weight provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>steel</td>
<td>12 mm</td>
<td>150 mm</td>
<td>0 mm²/m</td>
<td>753.98 mm²/m</td>
<td>0.43085 %</td>
<td>5.9188 kg/m²</td>
</tr>
</tbody>
</table>

Nominal cover to all tension face steel 45 mm
Nominal cover to all compr.face steel 45 mm
Total weight of reinforcement 11.838 kg/m²

satisfies:
1. Ultimate limit-state
2. Service limit-state cracking in flexure

Calculated Flexure: In tension face: between bars 0.10123 mm crack widths
Location: Ex1 Combined compression & bending on substantial section

Analysis of rectangular beam section to BS8007 and/or BS8110

Section is assumed to be subjected primarily to bending (i.e. formulae 2 and 3 in Clause B.4 of BS8007 are used to determine stiffening effect of concrete, etc.)

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Ultimate moment $M=980$ kNm
Service moment $M'=686$ kNm
Maximum permissible crack width $C_w=0.2$ mm
Shear force due to ultimate loads $V=450$ kN
Ultimate axial load (-ve tension) $N_{ult}'=400$ kN
Service axial load (-ve tension) $N'=280$ kN
Characteristic concrete strength $f_{cu}=35$ N/mm²
Elastic modulus of concrete $E_c=12000$ N/mm²
Characteristic steel strength $f_y=500$ N/mm²
Overall depth of concrete section $h=1100$ mm
Breadth of concrete section $b_t=800$ mm
Spacing of bars in sides of beam $spacs=140$
Size of tension bars $dia_t=25$ mm
Number of tension bars $ntb=7$
Maximum dia. of coarse aggregate $hagg=20$ mm
Diameter of shear links $dia_l=10$ mm
Yield strength of links $f_{yl}=250$ N/mm²
Number of link legs $nlegs=2$
Designated exposure class is XC4
Specified fixing tolerance $tol=10$ mm
Nominal concrete cover $cover=40$ mm
Chosen effective depth of section $d=1037.5$ mm
Size of compression bars $dia_c=25$ mm
Number of compression bars $ncb=3$
Chosen depth to compression bars $d'=62.5$ mm
Temp. zone depth for tension steel $trz=250$ mm
Temp. zone depth for comp. steel $crz=250$ mm
Temp. fall: hydrat. peak & ambient $T_1=20$ °C
Seasonal variation in temperature $T_2=15$ °C
Chosen ratio of $f_{ct}/f_b$ $fratio=0.67$
External restraint factor $R=0.5$
Thermal expansion coeff. for concrete $alpha=10E-6$
Direct tensile strength $f_{ct}=1.6$ N/mm²
Chosen spacing of links $svinp=100$ mm
### REINFORCEMENT

**Tension steel:**
- **Size:** 25 mm
- **Number:** 7
- **Spacing:** 112.5 mm
- **Area required:** 2286.9 mm²
- **Area provided:** 3436.1 mm²
- **Percentages:**
  - (pcrit) 1.7181 %
  - (gross) 0.39047 %
- **Weight provided:** 26.974 kg/m

**Compression steel:**
- **Size:** 25 mm
- **Number:** 3
- **Spacing:** 337.5 mm
- **Area required:** 60 mm²
- **Area provided:** 1472.6 mm²
- **Percentages:**
  - (pcrit) 0.73631 %
  - (gross) 0.16734 %
- **Weight provided:** 11.56 kg/m

**In sides of beam**
- **Size:** 12 mm
- **Number (total):** 10
- **Spacing:** 140 mm
- **Weight provided:** 8.8781 kg/m

**Shear links:**
- **Size:** 10 mm
- **Spacing:** 100 mm
- **Number of legs:** 2
- **Weight (approx.):** 21.578 kg/m

**Cover to all steel:**
- **Cover:** 40 mm
- **Total weight of steel (approx.):** 68.989 kg/m

### SUMMARY

- **Weight provided:**
  - Tension steel: 26.974 kg/m
  - Compression steel: 11.56 kg/m
  - In sides of beam: 8.8781 kg/m
  - Shear links: 21.578 kg/m
  - Total steel: 68.989 kg/m

### Calculated

- **Flexure:**
  - In tension face: between bars 0.17774 mm
  - at corners 0.18642 mm
  - In side of beam 0.16251 mm

- **Temperature change:**
  - In tension face 0.085307 mm
  - In compression face 0.19905 mm

### Satisfies:

1. Ultimate limit-state
2. Service limit-state of cracking in flexure
3. Temperature-cracking requirements
Location: Ex2 - Combined tension and bending

Analysis of rectangular beam section to BS8007 and/or BS8110

Section is assumed to be subjected primarily to bending (i.e. formulae 2 and 3 in Clause B.4 of BS8007 are used to determine stiffening effect of concrete, etc.)

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Ultimate moment $M=1000$ kNm
Service moment $M'=700$ kNm
Maximum permissible crack width $C_w=0.2$ mm
Shear force due to ultimate loads $V=800$ kN
Ultimate axial load (-ve tension) $N_{ult}'=-600$ kN
Service axial load (-ve tension) $N'=-420$ kN
Characteristic concrete strength $f_{cu}=40$ N/mm$^2$
Elastic modulus of concrete $E_c=12000$ N/mm$^2$
Characteristic steel strength $f_y=500$ N/mm$^2$
Overall depth of concrete section $h=1100$ mm
Breadth of concrete section $b_t=800$ mm
Spacing of bars in sides of beam $spacs=140$
Size of tension bars $d_{iat}=25$ mm
Number of tension bars $n_{tb}=11$
Maximum dia. of coarse aggregate $h_{agg}=20$ mm
Diameter of shear links $d_{ial}=8$ mm
Yield strength of links $f_{yv}=500$ N/mm$^2$
Number of link legs $n_{legs}=4$
Designated exposure class is XC4
Specified fixing tolerance $tol=10$ mm
Nominal concrete cover $cover=40$ mm
Chosen effective depth of section $d=1039.5$ mm
Size of compression bars $d_{iac}=25$ mm
Number of compression bars $n_{cb}=4$
Chosen depth to compression bars $d'=60.5$ mm
Temp. zone depth for tension steel $t_{rz}=250$ mm
Temp. zone depth for comp. steel $c_{rz}=250$ mm
Temp. fall: hydrat. peak & ambient $T_1=20$ °C
Seasonal variation in temperature $T_2=10$ °C
Chosen ratio of $f_{ct}/f_b$ $fratio=0.67$
External restraint factor $R=0.5$
Thermal expansion coeff. for concrete $alpha=10E-6$
Direct tensile strength $f_{ct}=1.6$ N/mm$^2$
Chosen spacing of links $svinp=160$ mm
REINFORCEMENT

Tension steel:
Size 25 mm
Number 11
Spacing 67.9 mm
Area required 3709.1 mm²
Area provided 5399.6 mm²
Percentages: (pcrit) 2.6998 %
provided: (gross) 0.61359 %
Weight provided 42.387 kg/m

SUMMARY

Compression steel:
Size 25 mm
Number 4
Spacing 226.33 mm
Area required 60 mm²
Area provided 1963.5 mm²
Percentages: (pcrit) 0.98175 %
provided: (gross) 0.22312 %
Weight provided 15.413 kg/m

In sides of beam
Size 12 mm
Number (total) 10
Spacing 140 mm
Weight provided 8.8781 kg/m

Shear links:
Size 8 mm
Spacing 160 mm
Number of legs 4
Weight (approx.) 16.019 kg/m

Cover to all steel
40 mm
Total weight of steel (approx.) 82.698 kg/m

satisfies:
1. Ultimate limit-state
2. Service limit-state of cracking in flexure
3. Temperature-cracking requirements

Calculated

Flexure: In tension face: between bars 0.158 mm
at corners 0.19527 mm
Flexure: In side of beam 0.17623 mm
Temperature change: In tension face 0.046531 mm
In compression face 0.12796 mm
Location: Ex3 - Combined tension and bending

Analysis of rectangular beam section to BS8007 and/or BS8110

Section is assumed to be subjected primarily to bending (i.e. formulae 2 and 3 in Clause B.4 of BS8007 are used to determine stiffening effect of concrete, etc.)

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Ultimate moment
M=1000 kNm

Service moment
Mz'=-700 kNm

Maximum permissible crack width
Cw=0.2 mm

Shear force due to ultimate loads
V=800 kN

Ultimate axial load (-ve tension)
Nult'=-600 kN

Service axial load (-ve tension)
N'=-450 kN

Characteristic concrete strength
fcu=40 N/mm²

Elastic modulus of concrete
Ec=12000 N/mm²

Characteristic steel strength
fy=500 N/mm²

Overall depth of concrete section
h=1100 mm

Breadth of concrete section
bt=1000 mm

Spacing of bars in sides of beam
spacs=140

Size of tension bars
diat=25 mm

Number of tension bars
ntb=11

Maximum dia.of coarse aggregate
hagg=20 mm

Diameter of shear links
dial=8 mm

Yield strength of links
fyv=500 N/mm²

Number of link legs
nlegs=4

Designated exposure class is XC4

Specified fixing tolerance
tol=10 mm

Nominal concrete cover
cover=40 mm

Chosen effective depth of section
d=1039.5 mm

Size of compression bars
diac=25 mm

Number of compression bars
ncb=4

Chosen depth to compression bars
d'=60.5 mm

Temp.zone depth for tension steel
trz=250 mm

Temp.zone depth for comp.steel
crz=250 mm

Temp.fall: hydrat.peak & ambient
T1=20 °C

Seasonal variation in temperature
T2=10 °C

Chosen ratio of fct/fb
fratio=0.67

External restraint factor
R=0.5

Thermal expansion coeff.for concrete
alpha=10E-6

Direct tensile strength
fct=1.6 N/mm²

Chosen spacing of links
svinp=180 mm
### REINFORCEMENT

#### Tension steel:
- **Size:** 25 mm

#### SUMMARY

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<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
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<tr>
<td><strong>Spacing</strong></td>
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</tr>
<tr>
<td><strong>Area required</strong></td>
<td>3709.1 mm²</td>
</tr>
<tr>
<td><strong>Area provided</strong></td>
<td>5399.6 mm²</td>
</tr>
<tr>
<td><strong>Percentages:</strong></td>
<td></td>
</tr>
<tr>
<td>(pcrit)</td>
<td>2.1598 %</td>
</tr>
<tr>
<td>(gross)</td>
<td>0.49087 %</td>
</tr>
<tr>
<td><strong>Weight provided</strong></td>
<td>42.387 kg/m</td>
</tr>
</tbody>
</table>

#### Compression steel:
- **Size:** 25 mm

- **Number:** 4
- **Spacing:** 293 mm
- **Area required:** 75 mm²
- **Area provided:** 1963.5 mm²
- **Percentages:**
  - (pcrit): 0.7854 %
  - (gross): 0.1785 %
- **Weight provided:** 15.413 kg/m

#### In sides of beam
- **Size:** 12 mm
- **Number (total):** 10
- **Spacing:** 140 mm
- **Weight provided:** 8.8781 kg/m

#### Shear links:
- **Size:** 8 mm
- **Spacing:** 180 mm
- **Number of legs:** 4
- **Weight (approx.):** 15.554 kg/m

#### Cover to all steel
- **Cover:** 40 mm

#### Total weight of steel (approx.)
- **Weight:** 82.233 kg/m

### Satisfies:
1. Ultimate limit-state
2. Service limit-state of cracking in flexure
3. Temperature-cracking requirements

### Calculated Flexure:
- **In tension face:**
  - between bars: 0.15858 mm
  - at corners: 0.18146 mm
- **In side of beam:** 0.16476 mm
- **Temperature change:**
  - In tension face: 0.058164 mm
  - In compression face: 0.15995 mm
Location: Ex4 - Shallow beam with mild steel (bending only)

Analysis of rectangular beam section to BS8007 and/or BS8110

Section is assumed to be subjected primarily to bending (i.e. formulae 2 and 3 in Clause B.4 of BS8007 are used to determine stiffening effect of concrete, etc.)

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

- Ultimate moment $M=45$ kNm
- Service moment $M_z'=31.5$ kNm
- Maximum permissible crack width $C_w=0.2$ mm
- Shear force due to ultimate loads $V=50$ kN
- Ultimate axial load (-ve tension) $N_{ult}'=0$ kN
- Service axial load (-ve tension) $N'=0$ kN
- Characteristic concrete strength $f_{cu}=35$ N/mm$^2$
- Elastic modulus of concrete $E_c=12000$ N/mm$^2$
- Characteristic steel strength $f_y=250$ N/mm$^2$
- Overall depth of concrete section $h=250$ mm
- Breadth of concrete section $b_t=300$ mm
- Size of tension bars $d_i=25$ mm
- Number of tension bars $n_{tb}=3$
- Maximum dia. of coarse aggregate $hagg=20$ mm
- Diameter of shear links $d_i=8$ mm
- Yield strength of links $f_{yv}=250$ N/mm$^2$
- Number of link legs $n_{legs}=2$
- Designated exposure class is XC4
- Specified fixing tolerance $tol=10$ mm
- Nominal concrete cover $cover=40$ mm
- Chosen effective depth of section $d=189.5$ mm
- Size of compression bars $d_i=20$ mm
- Number of compression bars $n_{cb}=2$
- Chosen depth to compression bars $d'=58$ mm
- Temp. zone depth for tension steel $trz=125$ mm
- Temp. zone depth for comp. steel $crz=125$ mm
- Temp. fall: hydrat. peak & ambient $T_1=20$ °C
- Seasonal variation in temperature $T_2=5$ °C
- Chosen ratio of $f_{ct}/f_b$ $fratio=1$
- External restraint factor $R=0.5$
- Thermal expansion coeff. for concrete $\alpha=10E-6$
- Direct tensile strength $f_{ct}=1.6$ N/mm$^2$
- Chosen spacing of links $svinp=180$ mm
REINFORCEMENT

Tension steel:  
Size: 25 mm

SUMMARY

Number: 3
Spacing: 89.5 mm
Area required: 1296.4 mm²
Area provided: 1472.6 mm²
Percentages: (pcrit) 3.927 %
provided: (gross) 1.9635 %
Weight provided: 11.56 kg/m

Compression steel:  
Size: 20 mm

Number: 2
Spacing: 184 mm
Area required: 240 mm²
Area provided: 628.32 mm²
Percentages: (pcrit) 1.6755 %
provided: (gross) 0.83776 %
Weight provided: 4.9323 kg/m

Shear links:
Size: 8 mm
Spacing: 180 mm
Number of legs: 2
Weight (approx.): 1.7448 kg/m

Cover to all steel: 40 mm
Total weight of steel (approx.): 18.237 kg/m

satisfies:
1. Ultimate limit-state
2. Service limit-state of cracking in flexure
3. Temperature-cracking requirements

Calculated

Flexure: In tension face: between bars 0.17814 mm
at corners 0.18556 mm
Flexure: In side of beam 0.11014 mm
Temperature change: In tension face 0.039789 mm
In compression face 0.074604 mm
Location: Ex1 Combined compression & bending on substantial section

Analysis of rectangular beam section to EC2 Part 3 and/or Part 1-1

Section under consideration is assumed to be subjected primarily to bending.

\[ b = \text{breadth of section} \]
\[ d = \text{effective depth to tension reinforcement} \]
\[ d' = \text{depth to compression reinforcement} \]

Design partial safety factor \( g_{ams} = 1.15 \) (reinforcement)

Design BM before redistribution \( M_{Ed'} = 980 \text{ kNm} \)

Design BM after redistribution \( M_{Ed} = 980 \text{ kNm} \)

Service moment (quasi-permanent) \( M_{y'} = 686 \text{ kNm} \)

Maximum permissible crack width \( w_k = 0.2 \text{ mm} \)

Shear force due to ultimate loads \( V_{Ed} = 450 \text{ kN} \)

Ultimate axial load (-ve tension) \( N_{Ed'} = 400 \text{ kN} \)

Service axial load (-ve tension) \( N' = 280 \text{ kN} \)

Characteristic steel strength \( f_{yk} = 500 \text{ N/mm}^2 \)

Overall depth of concrete section \( h = 1100 \text{ mm} \)

Breadth of concrete section \( b = 800 \text{ mm} \)

Spacing of bars to sides of beam \( spacs = 140 \text{ mm} \)

Diameter of tension bars \( diat = 25 \text{ mm} \)

Number of tension bars \( ntb = 9 \)

Maximum dia.of coarse aggregate \( hagg = 20 \text{ mm} \)

Diameter of shear links \( dial = 10 \text{ mm} \)

Characteristic strength of links \( f_{yv} = 500 \text{ N/mm}^2 \)

Chosen effective depth of section \( d = 1037.5 \text{ mm} \)

Diameter of compression bars \( diac = 25 \text{ mm} \)

Number of compression bars \( ncb = 4 \)

Chosen depth to compression bars \( d' = 62.5 \text{ mm} \)

Temp. fall: Hydrat. peak & ambient \( T_1 = 20 \text{ °C} \)

Seasonal variation in temperature \( T_2 = 15 \text{ °C} \)

Design shear stress \( v_{Ed} = V_{Ed} \times 10^3/(0.9 \times b \times d) = 0.6024 \text{ N/mm}^2 \)

Concrete strut capacity \( v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 3.083 \text{ N/mm}^2 \)

Angle of inclination of strut \( \theta' = 22^\circ \)

Number of shear legs \( nsl = 4 \)

Chosen spacing \( s = 300 \text{ mm} \)

**SHEAR SUMMARY**

- **Design shear force**: 450 kN
- **Design shear stress**: 0.6024 N/mm²
- **Concrete strut capacity**: 3.083 N/mm²
- **Area/spacing ratio**: 0.6773
- **Diameter of links**: 10 mm
- **Area of links**: 314 mm²
- **Maximum spacing**: 463 mm
- **Actual spacing**: 300 mm
Shrinkage & thermal cracking caused by edge restrained to movement

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is negative). Coefficient of thermal expansion $a_{Tc} = 12 \, \mu e/°C$

The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement:
- Shrinkage strain (short-term) $e_{cas} = 13.6 \, \mu e$
- Shrinkage strain (long-term) $e_{cal} = 45 \, \mu e$

The short-term crack width 0.0187 mm is less than the permissible value of 0.2 mm hence satisfactory.

The long-term crack width 0.0854 mm is less than the permissible value of 0.2 mm hence satisfactory.

(B) Compression reinforcement:
The short-term crack width 0.0324 mm is less than the permissible value of 0.2 mm hence satisfactory.

The long-term crack width 0.1479 mm is less than the permissible value of 0.2 mm hence satisfactory.

<table>
<thead>
<tr>
<th>REINFORCEMENT</th>
<th>Tension steel:</th>
<th>Diameter 25 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Number</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>84.38 mm</td>
</tr>
<tr>
<td></td>
<td>Area required</td>
<td>2287 mm²</td>
</tr>
<tr>
<td></td>
<td>Area provided</td>
<td>4418 mm²</td>
</tr>
<tr>
<td></td>
<td>Percentages provided</td>
<td>0.502 %</td>
</tr>
<tr>
<td></td>
<td>Weight provided</td>
<td>34.68 kg/m</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Compression steel:</th>
<th>Diameter 25 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number</td>
<td>4</td>
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<tr>
<td>Spacing</td>
<td>225 mm</td>
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<tr>
<td>Area required</td>
<td>0 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>1963 mm²</td>
</tr>
<tr>
<td>Percentages provided</td>
<td>0.2231 %</td>
</tr>
<tr>
<td>Weight provided</td>
<td>15.41 kg/m</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Beam side faces:</th>
<th>Diameter 12 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number (total)</td>
<td>10</td>
</tr>
<tr>
<td>Spacing</td>
<td>140 mm</td>
</tr>
<tr>
<td>Weight provided</td>
<td>8.878 kg/m</td>
</tr>
</tbody>
</table>
Shear links: Diameter 10 mm  
Spacing 300 mm  
Number of legs 4  
Weight (approx.) 13.36 kg/m  
Cover to all steel 40 mm  
Total weight of steel (approx.) 72.33 kg/m  

satisfies:  
1. Ultimate limit-state  
2. Service limit-state of cracking in flexure  
3. Temperature-cracking requirements  
4. Fire-resistance requirements  

Calculated crack widths:  
Flexure: In tension face: between bars 0.1792 mm  
Temperature change: Short-term (tension face) 0.0187 mm  
Long-term 0.0854 mm  
Temperature change: Short-term (compr. face) 0.0324 mm  
Long-term 0.1479 mm
Analysis of rectangular beam section to EC2 Part 3 and/or Part 1-1

Section under consideration is assumed to be subjected primarily to bending.

b = breadth of section
d = effective depth to tension reinforcement
d' = depth to compression reinforcement

Design partial safety factor \( \gamma_{ams} = 1.15 \) (reinforcement)
Design BM before redistribution \( M_{Ed'} = 1000 \text{ kNm} \)
Design BM after redistribution \( M_{Ed} = 1000 \text{ kNm} \)
Service moment (quasi-permanent) \( M_y' = 700 \text{ kNm} \)
Maximum permissible crack width \( w_k = 0.2 \text{ mm} \)
Shear force due to ultimate loads \( V_{Ed} = 800 \text{ kN} \)
Ultimate axial load (−ve tension) \( N_{Ed'} = -600 \text{ kN} \)
Service axial load (−ve tension) \( N' = -420 \text{ kN} \)
Characteristic steel strength \( f_yk = 500 \text{ N/mm}^2 \)
Overall depth of concrete section \( h = 1100 \text{ mm} \)
Breadth of concrete section \( b = 800 \text{ mm} \)
Spacing of bars to sides of beam \( \text{spacs} = 140 \text{ mm} \)
Diameter of tension bars \( d\text{iat} = 25 \text{ mm} \)
Number of tension bars \( n\text{tb} = 10 \)
Maximum dia.of coarse aggregate \( h\text{agg} = 20 \text{ mm} \)
Diameter of shear links \( d\text{ial} = 8 \text{ mm} \)
Characteristic strength of links \( f_{yv} = 500 \text{ N/mm}^2 \)
Chosen effective depth of section \( d = 1039.5 \text{ mm} \)
Diameter of compression bars \( d\text{iac} = 25 \text{ mm} \)
Number of compression bars \( n\text{cb} = 4 \)
Chosen depth to compression bars \( d' = 60.5 \text{ mm} \)
Temp. fall: Hydrat. peak & ambient \( T_1 = 20 \text{ °C} \)
Seasonal variation in temperature \( T_2 = 10 \text{ °C} \)
Design shear stress \( v_{Ed} = V_{Ed} \times 10^3 / (0.9 \times b \times d) = 1.069 \text{ N/mm}^2 \)
Concrete strut capacity \( v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 3.274 \text{ N/mm}^2 \)
Angle of inclination of strut \( \theta' = 22^\circ \)
Number of shear legs \( n\text{sl} = 4 \)
Chosen spacing \( s = 250 \text{ mm} \)

**SHEAR SUMMARY**

- Design shear force: 800 kN
- Design shear stress: 1.069 N/mm²
- Concrete strut capacity: 3.274 N/mm²
- Area/spacing ratio: 0.7867
- Diameter of links: 8 mm
- Area of links: 201 mm²
- Maximum spacing: 255 mm
- Actual spacing: 250 mm
Shrinkage & thermal cracking caused by edge restrained to movement

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is -\infty ).

Coefficient of thermal expansion \( a_{TC} = 12 \, \mu e/\degree C \)

The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement:
Shrinkage strain (short-term) \( e_{cas} = 15 \, \mu e \)
Shrinkage strain (long-term) \( e_{cal} = 50 \, \mu e \)

The short-term crack width 0.0167 mm is less than the permissible value of 0.2 mm hence satisfactory.

The long-term crack width 0.0469 mm is less than the permissible value of 0.2 mm hence satisfactory.

(B) Compression reinforcement:

The short-term crack width 0.0308 mm is less than the permissible value of 0.2 mm hence satisfactory.

The long-term crack width 0.0864 mm is less than the permissible value of 0.2 mm hence satisfactory.

WARNING:

Spacing of bars is too great to satisfy service limit-state conditions.
Location: Ex3 Combined tension & bending, as Ex2 but greater tension

Analysis of rectangular beam section to EC2 Part 3 and/or Part 1-1

![Beam section diagram]

Section under consideration is assumed to be subjected primarily to bending.

\[ b = \text{breadth of section} \]
\[ d = \text{effective depth to tension reinforcement} \]
\[ d' = \text{depth to compression reinforcement} \]

Design partial safety factor \( g_{ams} = 1.15 \) (reinforcement)

Design BM before redistribution \( M_{Ed'} = 1000 \text{ kNm} \)

Design BM after redistribution \( M_{Ed} = 1000 \text{ kNm} \)

Service moment (quasi-permanent) \( M_y' = 700 \text{ kNm} \)

Maximum permissible crack width \( w_k = 0.2 \text{ mm} \)

Shear force due to ultimate loads \( V_{Ed} = 800 \text{ kN} \)

Ultimate axial load (-ve tension) \( N_{Ed'} = -1100 \text{ kN} \)

Service axial load (-ve tension) \( N' = -770 \text{ kN} \)

Characteristic steel strength \( f_{yk} = 500 \text{ N/mm}^2 \)

Overall depth of concrete section \( h = 1100 \text{ mm} \)

Breadth of concrete section \( b = 800 \text{ mm} \)

Spacing of bars to sides of beam \( \text{spacs} = 140 \text{ mm} \)

Diameter of tension bars \( \text{diat} = 25 \text{ mm} \)

Number of tension bars \( n_{tb} = 10 \)

Maximum dia.of coarse aggregate \( h_{agg} = 20 \text{ mm} \)

Diameter of shear links \( \text{dial} = 8 \text{ mm} \)

Characteristic strength of links \( f_{vy} = 500 \text{ N/mm}^2 \)

Chosen effective depth of section \( d = 1039.5 \text{ mm} \)

Diameter of compression bars \( \text{diac} = 25 \text{ mm} \)

Number of compression bars \( n_{cb} = 4 \)

Chosen depth to compression bars \( d' = 60.5 \text{ mm} \)

Temp.fall: Hydrat.peak & ambient \( T_1 = 20 \text{ °C} \)

Seasonal variation in temperature \( T_2 = 10 \text{ °C} \)

Design shear stress \( v_{Ed} = V_{Ed} \times 10^3/(0.9 \times b \times w \times d) = 1.069 \text{ N/mm}^2 \)

Concrete strut capacity \( v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 3.274 \text{ N/mm}^2 \)

Angle of inclination of strut \( \theta' = 22^\circ \)

Number of shear legs \( n_{sl} = 4 \)

Chosen spacing \( s = 250 \text{ mm} \)

**SHEAR SUMMARY**

- **Design shear force**: 800 kN
- **Design shear stress**: 1.069 N/mm²
- **Concrete strut capacity**: 3.274 N/mm²
- **Area/spacing ratio**: 0.7867
- **Diameter of links**: 8 mm
- **Area of links**: 201 mm²
- **Maximum spacing**: 255 mm
- **Actual spacing**: 250 mm
Shrinkage & thermal cracking caused by edge restrained to movement

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is \(-\infty\)).

Coefficient of thermal expansion \(a_{Tc}=12 \, \mu\varepsilon/\degree C\)

The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement:

Shrinkage strain (short-term) \(e_{cas}=15 \, \mu\varepsilon\)
Shrinkage strain (long-term) \(e_{cal}=50 \, \mu\varepsilon\)

The short-term crack width 0.0167 mm is less than the permissible value of 0.2 mm hence satisfactory.

The long-term crack width 0.0469 mm is less than the permissible value of 0.2 mm hence satisfactory.

(B) Compression reinforcement:

The short-term crack width 0.0308 mm is less than the permissible value of 0.2 mm hence satisfactory.

The long-term crack width 0.0864 mm is less than the permissible value of 0.2 mm hence satisfactory.

WARNING:
Spacing of bars is too great to satisfy service limit-state conditions.
Location: Ex4 Shallow beam (Bending only)

Analysis of rectangular beam section to EC2 Part 3 and/or Part 1-1

Section under consideration is assumed to be subjected primarily to bending.

\[ b = \text{breadth of section} \]
\[ d = \text{effective depth to tension reinforcement} \]
\[ d' = \text{depth to compression reinforcement} \]

Design partial safety factor \( g_{am}=1.15 \) (reinforcement)
Design BM before redistribution \( M_{Ed}'=45 \text{ kNm} \)
Design BM after redistribution \( M_{Ed}=45 \text{ kNm} \)
Service moment (quasi-permanent) \( M_{y}'=31.5 \text{ kNm} \)
Maximum permissible crack width \( w_k=0.2 \text{ mm} \)
Shear force due to ultimate loads \( V_{Ed}=50 \text{ kN} \)
Ultimate axial load (-ve tension) \( N_{Ed}'=0 \text{ kN} \)
Service axial load (-ve tension) \( N'=0 \text{ kN} \)
Characteristic steel strength \( f_{yk}=500 \text{ N/mm}^2 \)
Overall depth of concrete section \( h=250 \text{ mm} \)
Breadth of concrete section \( b=300 \text{ mm} \)
Diameter of tension bars \( d_{iat}=25 \text{ mm} \)
Number of tension bars \( n_{tb}=3 \)
Maximum dia.of coarse aggregate \( h_{agg}=20 \text{ mm} \)
Diameter of shear links \( d_{ial}=8 \text{ mm} \)
Characteristic strength of links \( f_{yv}=500 \text{ N/mm}^2 \)
Chosen effective depth of section \( d=179.5 \text{ mm} \)
Diameter of compression bars \( d_{iac}=20 \text{ mm} \)
Number of compression bars \( n_{cb}=2 \)
Chosen depth to compression bars \( d'=68 \text{ mm} \)
Temp.fall: Hydrat.peak & ambient \( T_1=20 ^\circ \text{C} \)
Seasonal variation in temperature \( T_2=5 ^\circ \text{C} \)
Design shear stress \( v_{Ed}=V_{Ed} \times 10^3/(0.9 \times b \times d)=1.032 \text{ N/mm}^2 \)
Concrete strut capacity \( v_{Rdm}=0.124 \times f_{ck} \times (1-f_{ck}/250)=3.083 \text{ N/mm}^2 \)
Angle of inclination of strut \( \theta'=22 ^\circ \)
Number of shear legs \( n_{sl}=2 \)
Chosen spacing \( s=125 \text{ mm} \)

**SHEAR SUMMARY**

| Design shear force | 50 kN |
| Design shear stress | 1.032 N/mm² |
| Concrete strut capacity | 3.083 N/mm² |
| Area-spacing ratio | 0.2847 |
| Diameter of links | 8 mm |
| Area of links | 100 mm² |
| Maximum spacing | 134.6 mm |
| (based on 0.75d) | |
| Actual spacing | 125 mm |
Shrinkage & thermal cracking caused by edge restrained to movement

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is $-\infty$).

Coefficient of thermal expansion $a_{Tc}=12 \, \mu e/\degree C$

The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement:
Shrinkage strain (short-term) $\varepsilon_{cas}=13.6 \, \mu e$
Shrinkage strain (long-term) $\varepsilon_{cal}=45 \, \mu e$

The short-term crack width 0.0189 mm is less than the permissible value of 0.2 mm hence satisfactory.
The long-term crack width 0.0472 mm is less than the permissible value of 0.2 mm hence satisfactory.

(B) Compression reinforcement:
The short-term crack width 0.0275 mm is less than the permissible value of 0.2 mm hence satisfactory.
The long-term crack width 0.0688 mm is less than the permissible value of 0.2 mm hence satisfactory.

WARNING:
Spacing of bars is too great to satisfy service limit-state conditions.
Location: Ex5 From RC Design to EC2 by B Mosley p.150, edge restraint

Analysis of rectangular beam section to EC2 Part 3 and/or Part 1-1

Section under consideration is assumed to be subjected primarily to bending.

\[
b = \text{breadth of section} \\
d = \text{effective depth to tension reinforcement} \\
d' = \text{depth to compression reinforcement}
\]

Design partial safety factor \( \gamma_{ams} = 1.15 \) (reinforcement)
Design BM before redistribution \( M_{Ed}' = 780 \text{ kNm} \)
Design BM after redistribution \( M_{Ed} = 780 \text{ kNm} \)
Service moment (quasi-permanent) \( M_{y}' = 650 \text{ kNm} \)
Maximum permissible crack width \( w_k = 0.295 \text{ mm} \)
Shear force due to ultimate loads \( V_{Ed} = 100 \text{ kN} \)
Ultimate axial load (-ve tension) \( N_{Ed}' = 0 \text{ kN} \)
Service axial load (-ve tension) \( N' = 0 \text{ kN} \)
Characteristic steel strength \( f_{yk} = 500 \text{ N/mm}^2 \)
Overall depth of concrete section \( h = 1000 \text{ mm} \)
Breadth of concrete section \( b = 400 \text{ mm} \)
Diameter of tension bars \( d_{it} = 40 \text{ mm} \)
Number of tension bars \( n_{tb} = 3 \)
Maximum dia. of coarse aggregate \( h_{agg} = 20 \text{ mm} \)
Diameter of shear links \( d_{ial} = 10 \text{ mm} \)
Characteristic strength of links \( f_{yv} = 500 \text{ N/mm}^2 \)
Chosen effective depth of section \( d = 930 \text{ mm} \)
Diameter of compression bars \( d_{ic} = 25 \text{ mm} \)
Number of compression bars \( n_{cb} = 3 \)
Chosen depth to compression bars \( d' = 62.5 \text{ mm} \)
Temp. fall: Hydrat. peak & ambient \( T_1 = 39 \text{ °C} \)
Seasonal variation in temperature \( T_2 = 20 \text{ °C} \)
Design shear stress \( v_{Ed} = V_{Ed} \times \frac{10^3}{0.9 \times b \times d} = 0.2987 \text{ N/mm}^2 \)
Concrete strut capacity \( v_{Rdm} = 0.124 \times f_{ck} \times (1 - f_{ck}/250) = 2.79 \text{ N/mm}^2 \)
Angle of inclination of strut \( \theta' = 22° \)
Number of shear legs \( n_{sl} = 2 \)
Chosen spacing \( s = 300 \text{ mm} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Design shear force</th>
<th>100 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear stress</td>
<td>0.2987 N/mm²</td>
</tr>
<tr>
<td>Concrete strut capacity</td>
<td>2.79 N/mm²</td>
</tr>
<tr>
<td>Area-spacing ratio</td>
<td>0.32</td>
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<tr>
<td>Diameter of links</td>
<td>10 mm</td>
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<tr>
<td>Area of links</td>
<td>157 mm²</td>
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<tr>
<td>Maximum spacing</td>
<td>490 mm</td>
</tr>
<tr>
<td>Actual spacing</td>
<td>300 mm</td>
</tr>
</tbody>
</table>
Shrinkage & thermal cracking caused by edge restrained to movement

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is \( -\infty \)).

Coefficient of thermal expansion \( a_{Tc}=12 \, \mu \varepsilon/°C \)

The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement:
Shrinkage strain (short-term) \( e_{cas}=11.5 \, \mu \varepsilon \)
Shrinkage strain (long-term) \( e_{cal}=37.5 \, \mu \varepsilon \)

The short-term crack width 0.0511 mm is less than the permissible value of 0.295 mm hence satisfactory.
The long-term crack width 0.1274 mm is less than the permissible value of 0.295 mm hence satisfactory.

(B) Compression reinforcement:

The short-term crack width 0.0641 mm is less than the permissible value of 0.295 mm hence satisfactory.
The long-term crack width 0.16 mm is less than the permissible value of 0.295 mm hence satisfactory.

WARNING:
Spacing of bars is too great to satisfy service limit-state conditions.
Location: Ex6 From RC Design to EC2 by B Mosley p.150, end restraint

Analysis of rectangular beam section to EC2 Part 3 and/or Part 1-1

Section under consideration is assumed to be subjected primarily to bending.

- $b =$ breadth of section
- $d =$ effective depth to tension reinforcement
- $d' =$ depth to compression reinforcement

Design partial safety factor \( g_{ams}=1.15 \) (reinforcement)

Design BM before redistribution \( M_{Ed'}=780 \text{ kNm} \)

Design BM after redistribution \( M_{Ed}=780 \text{ kNm} \)

Service moment (quasi-permanent) \( M_{y'}=650 \text{ kNm} \)

Maximum permissible crack width \( w_k=0.295 \text{ mm} \)

Shear force due to ultimate loads \( V_{Ed}=100 \text{ kN} \)

Ultimate axial load (-ve tension) \( N_{Ed'}=0 \text{ kN} \)

Service axial load (-ve tension) \( N'=0 \text{ kN} \)

Characteristic steel strength \( f_{yk}=500 \text{ N/mm}^2 \)

Overall depth of concrete section \( h=1000 \text{ mm} \)

Breadth of concrete section \( b=400 \text{ mm} \)

Diameter of tension bars \( d_{iat}=40 \text{ mm} \)

Number of tension bars \( n_{tb}=3 \)

Maximum dia. of coarse aggregate \( h_{agg}=20 \text{ mm} \)

 Diameter of shear links \( d_{ial}=10 \text{ mm} \)

Characteristic strength of links \( f_{yv}=500 \text{ N/mm}^2 \)

Chosen effective depth of section \( d=930 \text{ mm} \)

Diameter of compression bars \( d_{iac}=25 \text{ mm} \)

Number of compression bars \( n_{cb}=3 \)

Chosen depth to compression bars \( d'=62.5 \text{ mm} \)

Temp. fall: Hydrat. peak & ambient \( T_1=39 \text{ °C} \)

Seasonal variation in temperature \( T_2=20 \text{ °C} \)

Design shear stress \( v_{Ed}=V_{Ed} \times 10^3/(0.9 \times b \times w_d)=0.2987 \text{ N/mm}^2 \)

Concrete strut capacity \( v_{Rdm}=0.124 \times f_{ck} \times (1-f_{ck}/250)=2.79 \text{ N/mm}^2 \)

Angle of inclination of strut \( \theta'=22^\circ \)

Number of shear legs \( n_{sl}=2 \)

Chosen spacing \( s=250 \text{ mm} \)

### SHEAR SUMMARY

- **Design shear force** 100 kN
- **Design shear stress** 0.2987 N/mm²
- **Concrete strut capacity** 2.79 N/mm²
- **Area-spacing ratio** 0.32
- **Diameter of links** 10 mm
- **Area of links** 157 mm²
- **Maximum spacing** 490 mm
- **Actual spacing** 250 mm
Shrinkage & thermal cracking caused by end restrained to movement

For cracking due to restraint to movement the whole section will be considered to be in tension (i.e. the neutral axis depth is $-\infty$). Coefficient of thermal expansion $a_{Tc}=12 \, \mu e/\circ C$

The area of tension and compression reinforcement provided exceeds the minimum area required to control thermal and shrinkage cracking. Hence satisfactory.

(A) Tension reinforcement:
The short-term crack width 0.0869 mm is less than the permissible value of 0.295 mm hence satisfactory.
The long-term crack width 0.1477 mm is less than the permissible value of 0.295 mm hence satisfactory.

(B) Compression reinforcement:
The long-term crack width 0.1856 mm is less than the permissible value of 0.295 mm hence satisfactory.

WARNING:
Spacing of bars is too great to satisfy service limit-state conditions.
Location: Two way spanning wall slab with hydrostatic pressure

Free top & fixed at the other three sides

Coefficients are taken from Table 3 of 'Rectangular Concrete Tanks' published by Portland Cement Association (PCA) and reproduced by British Cement Association

Vertical height of wall      a=2 m
Horizontal length of wall    b=3 m
Density of fluid             w=10 kN/m³

BENDING MOMENTS FOR VERTICAL REINFORCEMENT (kNm/m)
(minus sign indicates tension on loaded side)

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BENDING MOMENTS FOR HORIZONTAL REINFORCEMENT (kNm/m)
(minus sign indicates tension on loaded side)

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## SHEARS (kN/m & kN for totals)
(minus sign indicates reaction acts in direction of load)

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<td>16.2</td>
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</table>
Location: Ex1 - Buried concrete (drainage works designated mix)

Concrete specification to BS8500-1:2006

- Sulphate value: SO4=680 mg/l
- pH value: pH=7
- Magnesium value: Mg=0 mg/l

Designated concrete specification

- Designated concrete from Table A.13: GEN1
- Minimum strength class Table A.14: C8/10
- Maximum aggregate size: 20 mm
- Target density: NWC density
Location: Ex2 - Buried concrete (drainage works designated mix)

Concrete specification to BS8500-1:2006

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<th>Parameter</th>
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<td>pH value</td>
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<tr>
<td>Magnesium value</td>
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Designated concrete specification
Location: Ex3 - Buried concrete (drainage works designed concrete)

Concrete specification to BS8500-1:2006

Sulphate value  
SO₄=680 mg/l

pH value  
pH=7

Magnesium value  
Mg=0 mg/l

Concrete exposure class

Designed concrete specification
Location: Ex4 - General applications

Concrete specification to BS8500-1:2006

Designated concrete specification

- Concrete designation from Table A.13: GEN0
- Minimum strength class from Table A.14: C6/8
- Maximum aggregate size: 20 mm
- Target density: NWC density
**Location:** Ex5 - Ground floor

**Concrete specification to BS8500-1:2006**

**Designated concrete specification**

<table>
<thead>
<tr>
<th>Concrete designation from Table A.13</th>
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<tr>
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<td>C40/50</td>
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<tr>
<td>Maximum aggregate size</td>
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<tr>
<td>Target density</td>
<td>NWC density</td>
</tr>
</tbody>
</table>
Location: Ex6 - Paving

Concrete specification to BS8500-1:2006

Designated concrete specification

Concrete designation from Table A.13: PAV2
Minimum strength class from Table A.14: C28/35
Maximum aggregate size: 20 mm
Target density: NWC density
Location: Ex7 - Non-buried concrete

Concrete specification to BS8500-1:2006

Concrete exposure class

Designated concrete specification
Location: Ex8 - Non-buried concrete (designed concrete)

Concrete specification to BS8500-1:2006

Concrete exposure class

Designed concrete specification
Location: Ex9 - Buried concrete (designated mix)

Concrete specification to BS8500-1:2006

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<thead>
<tr>
<th>Designated concrete specification</th>
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<tbody>
<tr>
<td>Designated concrete from Table A.13</td>
<td>FND4</td>
</tr>
<tr>
<td>Minimum strength class Table A.14</td>
<td>C25/30</td>
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<tr>
<td>Maximum aggregate size</td>
<td>20 mm</td>
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<tr>
<td>Target density</td>
<td>NWC density</td>
</tr>
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</table>
Location: Ex10 - Buried concrete (designated mix)

Concrete specification to BS8500-1:2006

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<th>Specification</th>
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<tbody>
<tr>
<td>Sulphate value</td>
<td>SO4=1200 mg/l</td>
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<tr>
<td>pH value</td>
<td>pH=7</td>
</tr>
<tr>
<td>Magnesium value</td>
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</table>

Designated concrete specification

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<thead>
<tr>
<th>Specification</th>
<th>Value</th>
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<tbody>
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<td>Concrete designation</td>
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<tr>
<td>Minimum strength class</td>
<td>C25/30</td>
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<td>Maximum aggregate size</td>
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<tr>
<td>Target density</td>
<td>NWC density</td>
</tr>
</tbody>
</table>
Location: Ex11 - Buried concrete (designated mix)

Concrete specification to BS8500-1:2006

- Sulphate value: SO4=1200 mg/l
- pH value: pH=7
- Magnesium value: Mgc=0 mg/l
Location: Ex12 - Non-buried concrete (designed concrete)

Concrete specification to BS8500-1:2006

Concrete exposure class

Designed concrete specification
Location: Ex13 - Non-buried concrete (designed concrete)

Concrete specification to BS8500-1:2006

Concrete exposure class

Designed concrete specification
All bars test
Location: Example A.1 in 'Reinf. Concrete Design to BS8110' by Allen

Properties of transformed sections of reinforced concrete

Formulae for transformed section taken from Table 136 in 10th edition of 'Reinforced Concrete Designer's Handbook' by Reynolds and Steedman.

Rectangular cracked section (and flanged section where x ≤ hf )

\[ b \]
\[ d' \]
\[ d \]
\[ A's = \text{area of steel in compression} \]
\[ As = \text{area of steel in tension} \]

Width of section \( b = 1000 \) mm
Modular ratio \( \alpha_e = 7 \)
Tension reinforcement:
Depth to centroid of steel \( d = 124 \) mm
Size of bars \( \text{diam} = 12 \) mm
Spacing of bars \( \text{spct} = 200 \) mm
Area of tension steel \( A_s = \text{diam}^2 \pi/4 \times b / \text{spct} = 565.49 \) mm\(^2\)
Compression reinforcement:
Depth to centroid of steel \( d' = 25 \) mm
Area of compression steel \( A's = 0 \) mm\(^2\)
Depth to neutral axis \( x = d \times \text{SQR}(c_3^2 + 2 \times (c_2 + c_1 \times d'/d)) - d \times c_3 \)
\( = 27.623 \) mm
Second moment of area \( I_e = c_5 \times (c_4^3 / 3 + c_2 \times (1 - c_4)^2 + c_1 \times c_6^2) \)
\( = 43.793E6 \) mm\(^4\)
Location: Pier section at foundation

Elastic biaxial bending of a rectangular RC section

Units used in the calculations:
Input   Dimensions and bar diameters   mm
Forces                         kN
Bending moments                kNm
Output  Dimensions and co-ordinates    mm
Stresses                       N/mm²

Sign Convention for loading
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

![Diagram showing Face Numbers and Co-ordinate Axes](image)

Length of Faces 1 and 3  1000 mm
Length of Faces 2 and 4  500 mm

<table>
<thead>
<tr>
<th>Face 1</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
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</thead>
<tbody>
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<td>Face 4</td>
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<td>60</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>12</td>
<td>206</td>
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</table>

<table>
<thead>
<tr>
<th>Face 2</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
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</thead>
<tbody>
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<td>Face 1</td>
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<tr>
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<td></td>
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</tr>
<tr>
<td>1</td>
<td>3</td>
<td>12</td>
<td>155</td>
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</tbody>
</table>
**R.Concrete: Eurocode 2, BS8007, BS8666, analysis**  
**Biaxial bending of rectangular section**  
**Made by:** IFB  
**Date:** 02/12/19  
**Ref No:** SC193

---

### Face 3

<table>
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<td>Face 2</td>
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### Face 4

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<td>155</td>
</tr>
</tbody>
</table>

**Modular ratio**  
**mr=15**
Plot of Section Details
Loading Case 1

Axial Load
Axial load fixed in position.

x co-ordinate   xp(1)=600 mm
y co-ordinate   yp(1)=300 mm

Applied moments

About x axis    Amx(1)=-200 kNm
About y axis    Amy(1)=300 kNm
Plot of Cracked Section

Cracked section
Section is cracked

Neutral axis line

Face 3  x=26.075  y=500  
Face 2  x=1000  y=128.29

Centroid of stressed section  X=640.51  Y=351

Design bending moments at centroid of stressed section

About x axis
\[ D_{mx} = -P(z) \times (y_{p(z)} - Y)/1000 + A_{mx(z)} \]
\[ D_{mx} = -149 \text{ kNm} \]

About y axis
\[ D_{my} = P(z) \times (x_{p(z)} - X)/1000 + A_{my(z)} \]
\[ D_{my} = 259.49 \text{ kNm} \]

Concrete stresses

Corner  Co-ordinates  Stress  
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<tr>
<th>x</th>
<th>y</th>
<th>(N/mm²)</th>
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<tbody>
<tr>
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<td>1000</td>
<td>500</td>
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</table>

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
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<tr>
<th>Bar No.</th>
<th>Face</th>
<th>Layer</th>
<th>Co-ordinates x (mm)</th>
<th>Co-ordinates y (mm)</th>
<th>Diameter (mm)</th>
<th>Stress (N/mm²)</th>
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Number of iterations 7
Location: Uncracked section

Elastic biaxial bending of a rectangular RC section

Units used in the calculations:
Input  Dimensions and bar diameters  mm
       Forces  kN
       Bending moments  kNm
Output  Dimensions and co-ordinates  mm
       Stresses  N/mm²

Sign Convention for loading
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

[Diagram of Face Numbers and Co-ordinate Axes]

Length of Faces 1 and 3  300 mm
Length of Faces 2 and 4  800 mm

<table>
<thead>
<tr>
<th>Face 1</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer</td>
<td></td>
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<td>Face 4</td>
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</tbody>
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<th>Number of bars</th>
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<th>Edge distances to centre of bars</th>
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<td>5</td>
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<td>Number of bars</td>
<td>Bar diameter</td>
<td>Edge distances to centre of bars</td>
</tr>
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</tr>
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<td>1</td>
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<td>65</td>
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</tbody>
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<table>
<thead>
<tr>
<th>Face 4</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer</td>
<td></td>
<td></td>
<td>Face 3</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>12</td>
<td>140</td>
</tr>
</tbody>
</table>

Modular ratio mr=15
Plot of Section Details

+ + + + + + + +
+ + +
+ +
+ + + + + + + +
Loading Case 1

Axial Load

Axial load fixed in position.

\[ P(1) = 1000 \text{ kN} \]

x co-ordinate

\[ x_p(1) = 155 \text{ mm} \]

y co-ordinate

\[ y_p(1) = 450 \text{ mm} \]

Applied moments

About x axis

\[ A_{mx}(1) = 0 \text{ kNm} \]

About y axis

\[ A_{my}(1) = 0 \text{ kNm} \]

All of section in compression

Centroid of stressed section

\( X = 150 \)
\( Y = 400 \)

Design bending moments at centroid of stressed section

About x axis

\[ D_{mx} = -P(z) \frac{(y_p(z) - Y)}{1000} + A_{mx}(z) = -50 \text{ kNm} \]

About y axis

\[ D_{my} = P(z) \frac{(x_p(z) - X)}{1000} + A_{my}(z) = 5 \text{ kNm} \]

Concrete stresses

<table>
<thead>
<tr>
<th>Corner</th>
<th>Co-ordinates</th>
<th>Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>300</td>
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<tr>
<td>3</td>
<td>300</td>
<td>800</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>800</td>
</tr>
</tbody>
</table>

Maximum and minimum reinforcement bar stresses.

Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Face</th>
<th>Layer</th>
<th>Co-ordinates x (mm)</th>
<th>Co-ordinates y (mm)</th>
<th>Diameter (mm)</th>
<th>Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>1</td>
<td>1</td>
<td>65</td>
<td>65</td>
<td>12</td>
<td>35.597</td>
</tr>
<tr>
<td>Maximum</td>
<td>13</td>
<td>3</td>
<td>235</td>
<td>735</td>
<td>12</td>
<td>76.135</td>
</tr>
</tbody>
</table>
Location: Fully cracked section

Elastic biaxial bending of a rectangular RC section

Units used in the calculations:
Input
Dimensions and bar diameters mm
Forces kN
Bending moments kNm
Output
Dimensions and co-ordinates mm
Stresses N/mm²

Sign Convention for loading
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

![Face Numbers and Co-ordinate Axes]

Length of Faces 1 and 3 300 mm
Length of Faces 2 and 4 800 mm

<table>
<thead>
<tr>
<th>Layer</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
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<td>4</td>
<td>12</td>
<td>65</td>
</tr>
</tbody>
</table>

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</thead>
<tbody>
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<tr>
<td>1</td>
<td>5</td>
<td>12</td>
<td>140</td>
</tr>
</tbody>
</table>
### Table 1: Bar Distribution

<table>
<thead>
<tr>
<th>Layer</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
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</tr>
<tr>
<td>1</td>
<td>4</td>
<td>12</td>
<td>65</td>
</tr>
</tbody>
</table>

### Table 2: Bar Distribution

<table>
<thead>
<tr>
<th>Layer</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
<tr>
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</tr>
<tr>
<td>1</td>
<td>5</td>
<td>12</td>
<td>140</td>
</tr>
</tbody>
</table>

**Modular ratio**  
$m_r = 15$
Plot of Section Details
Loading Case 1

Axial Load

P(1)=-500 kN

Axial load fixed in position.

x co-ordinate

xp(1)=155 mm

y co-ordinate

yp(1)=450 mm

Applied moments

About x axis

Amx(1)=0 kNm

About y axis

Amy(1)=0 kNm

All of section in tension

Centroid of stressed section

X=150        Y=400

Design bending moments at centroid of stressed section

About x axis

Dmx=-P(z)*(yp(z)-Y)/1000+Amx(z)=25 kNm

About y axis

Dmy=P(z)*(xp(z)-X)/1000+Amy(z)=-2.5 kNm

Maximum and minimum reinforcement bar stresses.

Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Face</th>
<th>Layer</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td></td>
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<tr>
<td>13</td>
<td>3</td>
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<td>235</td>
<td>735</td>
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</tr>
<tr>
<td>Maximum</td>
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<td>1</td>
<td>1</td>
<td>1</td>
<td>65</td>
<td>65</td>
<td>-167.72</td>
</tr>
</tbody>
</table>
**Location:** Cracked section. Axial load moves to centroid.

**Elastic biaxial bending of a rectangular RC section**

Units used in the calculations:
- **Input:** Dimensions and bar diameters (mm), Forces (kN), Bending moments (kNm)
- **Output:** Dimensions and co-ordinates (mm), Stresses (N/mm²)

Sign Convention for loading:
- A positive axial load causes compression in the section.
- A positive moment is clockwise about the x or y axes.

![Face Numbers and Co-ordinate Axes](image)

Length of Faces 1 and 3: 1000 mm
Length of Faces 2 and 4: 500 mm

<table>
<thead>
<tr>
<th>Face 1</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer</td>
<td></td>
<td></td>
<td>Face 4</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>16</td>
<td>60</td>
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<tr>
<td>2</td>
<td>3</td>
<td>12</td>
<td>206</td>
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<table>
<thead>
<tr>
<th>Face 2</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer</td>
<td></td>
<td></td>
<td>Face 1</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
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<td>Face 3</td>
<td>Number of bars</td>
<td>Bar diameter</td>
<td>Edge distances to centre of bars</td>
</tr>
<tr>
<td>--------</td>
<td>----------------</td>
<td>--------------</td>
<td>---------------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Face 2</td>
</tr>
<tr>
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<table>
<thead>
<tr>
<th>Face 4</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Face 3</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>12</td>
<td>155</td>
</tr>
</tbody>
</table>

Modular ratio \( mr = 15 \)
Plot of Section Details

+ + + + +

+ + +

+ +

+ +

+ +

+ + + + +
Loading Case 1

Axial Load  \( P(1) = 1000 \text{ kN} \)
Axial load applied at centroid of stressed section.

Applied moments

About x axis  \( A_{mx}(1) = -200 \text{ kNm} \)
About y axis  \( A_{my}(1) = 300 \text{ kNm} \)
Plot of Cracked Section
Section is cracked

Neutral axis line

Face 4  x=0  y=487.34
Face 2  x=1000  y=182.08

Centroid of stressed section  X=621.09  Y=363.07

Design bending moments at centroid of stressed section

About x axis  \( D_{mx} = A_{mx}(z) = -200 \text{ kNm} \)
About y axis  \( D_{my} = A_{my}(z) = 300 \text{ kNm} \)

Concrete stresses

<table>
<thead>
<tr>
<th>Corner</th>
<th>Co-ordinates</th>
<th>Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>1000 500</td>
<td>23.331</td>
</tr>
<tr>
<td>4</td>
<td>0 500</td>
<td>0.92935</td>
</tr>
</tbody>
</table>

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Face</th>
<th>Layer</th>
<th>Co-ordinates x (mm)</th>
<th>Co-ordinates y (mm)</th>
<th>Diameter (mm)</th>
<th>Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>1</td>
<td>60</td>
<td>60</td>
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</table>

Number of iterations 11
Elastic biaxial bending of a rectangular RC section

Units used in the calculations:
Input
- Dimensions and bar diameters: mm
- Forces: kN
- Bending moments: kNm
Output
- Dimensions and co-ordinates: mm
- Stresses: N/mm²

Sign Convention for loading
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

y

Face Numbers and Co-ordinate Axes

Length of Faces 1 and 3  300 mm
Length of Faces 2 and 4  800 mm

<table>
<thead>
<tr>
<th>Face 1</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer</td>
<td></td>
<td></td>
<td>Face 4</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>12</td>
<td>65</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Face 2</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer</td>
<td></td>
<td></td>
<td>Face 1</td>
</tr>
<tr>
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<td>5</td>
<td>12</td>
<td>140</td>
</tr>
</tbody>
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### R.Concrete: Eurocode 2, BS8007, BS8666, analysis

**Biaxial bending of rectangular section**  
**Date:** 02/12/19

**Made by:** IFB  
**Ref No:** SC193

---

<table>
<thead>
<tr>
<th>Layer</th>
<th>Face 3</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
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<tbody>
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<td></td>
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<td>1</td>
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<td>12</td>
<td>140</td>
<td>65</td>
</tr>
</tbody>
</table>

**Modular ratio**  
**mr=15**
Plot of Section Details

+ + + + + + + +
+ + + + +
+ + + +
+ + + + + + + +
Loading Case 1

Axial Load  
$P(1)=1000$ kN
Axial load applied at centroid of stressed section.

Applied moments

About x axis  
$A_{mx}(1)=0$ kNm
About y axis  
$A_{my}(1)=0$ kNm

All of section in compression

Centroid of stressed section  
$x=150$  $y=400$

Design bending moments at centroid of stressed section

About x axis  
$D_{mx}=A_{mx}(z)=0$ kNm
About y axis  
$D_{my}=A_{my}(z)=0$ kNm

Concrete stresses

<table>
<thead>
<tr>
<th>Corner</th>
<th>Co-ordinates</th>
<th>Stress $(\text{N/mm}^2)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 0</td>
<td>3.7244</td>
</tr>
<tr>
<td>2</td>
<td>300 0</td>
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<tr>
<td>3</td>
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<td>3.7244</td>
</tr>
<tr>
<td>4</td>
<td>0 800</td>
<td>3.7244</td>
</tr>
</tbody>
</table>

Reinforcement bar stresses. Tension negative, Compression positive.

Compressive stress in all bars  
$55.866$ N/mm$^2$
Location: Fully cracked section. Axial load moves to centroid.

Elastic biaxial bending of a rectangular RC section

Units used in the calculations:
Input   Dimensions and bar diameters   mm
Forces                         kN
Bending moments                kNm
Output  Dimensions and co-ordinates    mm
Stresses                       N/mm²

Sign Convention for loading
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

Face Numbers and Co-ordinate Axes

Length of Faces 1 and 3  300 mm
Length of Faces 2 and 4  800 mm

<table>
<thead>
<tr>
<th>Face 1</th>
<th>Number of bars</th>
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<td>65</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Face 2</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer</td>
<td></td>
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</tr>
<tr>
<td>1</td>
<td>5</td>
<td>12</td>
<td>140</td>
</tr>
<tr>
<td>Face 3</td>
<td>Number of bars</td>
<td>Bar diameter</td>
<td>Edge distances to centre of bars</td>
</tr>
<tr>
<td>------</td>
<td>----------------</td>
<td>--------------</td>
<td>----------------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Face 2</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>12</td>
<td>65</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Face 4</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to centre of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Face 3</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>12</td>
<td>140</td>
</tr>
</tbody>
</table>

Modular ratio \( \text{mr}=15 \)
Plot of Section Details

```
+ + + + + + + +
+ + + + + +
+ + + + +
+ + + + + + + +
```
Loading Case 1

Axial Load

\[ P(1) = -500 \text{ kN} \]
Axial load applied at centroid of stressed section.

Applied moments

About x axis

\[ Amx(1) = 0 \text{ kNm} \]
About y axis

\[ Amy(1) = 0 \text{ kNm} \]

All of section in tension

Centroid of stressed section

\[ X = 150 \quad \text{Y} = 400 \]

Design bending moments at centroid of stressed section

About x axis

\[ Dmx = Amx(z) = 0 \text{ kNm} \]
About y axis

\[ Dmy = Amy(z) = 0 \text{ kNm} \]

Reinforcement bar stresses. Tension negative, Compression positive.

Tensile stress in all bars

\[ -245.61 \text{ N/mm}^2 \]
Location: Pier section at foundation

Elastic biaxial bending of a circular RC section

Units used in the calculations:

<table>
<thead>
<tr>
<th>Input</th>
<th>Dimensions and bar diameters</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forces</td>
<td></td>
<td>kN</td>
</tr>
<tr>
<td>Bending moments</td>
<td></td>
<td>kNm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Output</th>
<th>Dimensions and co-ordinates</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stresses</td>
<td></td>
<td>N/mm²</td>
</tr>
</tbody>
</table>

Sign Convention for loading

A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

A reinforcement set is defined by the following parameters:

Number of bars. Diameter of bars. Edge distance to centre of bars.
Anti-clockwise angle (θ) measured in degrees from local axis y' to locate first bar in set. Subsequent bars (if any) are positioned in an anti-clockwise sense from the first bar at equal centres.

Edge distance to centre of bar

Circular section
Diameter 600
<table>
<thead>
<tr>
<th>Set</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to bar centres</th>
<th>Start angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>16</td>
<td>56</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>12</td>
<td>56</td>
<td>45</td>
</tr>
</tbody>
</table>

Modular ratio \( mr=15 \)
Plot of Section Details
Loading Case 1

Axial Load
Axial load fixed in position.

P(1)=1000 kN

x co-ordinate
xp(1)=350 mm

y co-ordinate
yp(1)=400 mm

Applied moments

About x axis
Amx(1)=-50 kNm

About y axis
Amy(1)=75 kNm
Plot of Cracked Section

[Image of a cracked section diagram]
Section is cracked

Neutral axis line

\[
x = 59.681 \\
y = 479.41 \\
x = 519.87 \\
y = 95.977
\]

Centroid of stressed section

\[
X = 367.49 \\
Y = 380.83
\]

Design bending moments at centroid of stressed section

About x axis

\[D_{mx} = -P(z) \times (y_p(z) - Y)/1000 + A_{mx}(z)\]

\[-69.167 \text{ kNm}\]

About y axis

\[D_{my} = P(z) \times (x_p(z) - X)/1000 + A_{my}(z)\]

\[= 57.505 \text{ kNm}\]

Maximum concrete stress

<table>
<thead>
<tr>
<th>Co-ordinates</th>
<th>Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(mm) x</td>
<td>(mm) y</td>
</tr>
<tr>
<td>491.23</td>
<td>531.15</td>
</tr>
</tbody>
</table>

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar Set</th>
<th>Bar No.</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>300</td>
<td>544</td>
<td>16</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>56</td>
<td>300</td>
<td>16</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>300</td>
<td>56</td>
<td>16</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>544</td>
<td>300</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>127.47</td>
<td>472.53</td>
<td>12</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>127.47</td>
<td>127.47</td>
<td>12</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>472.53</td>
<td>127.47</td>
<td>12</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>472.53</td>
<td>472.53</td>
<td>12</td>
</tr>
</tbody>
</table>

Number of iterations 7
Location: Uncracked section

Elastic biaxial bending of a circular RC section

Units used in the calculations:

Input  Dimensions and bar diameters  mm  
Forces  kN  
Bending moments  kNm  

Output  Dimensions and co-ordinates  mm  
Stresses  N/mm²  

Sign Convention for loading

A positive axial load causes compression in the section.  
A positive moment is clockwise about the x or y axes.

A reinforcement set is defined by the following parameters:

Number of bars. Diameter of bars. Edge distance to centre of bars.  
Anti-clockwise angle (θ) measured in degrees from local axis y' to locate first bar in set. Subsequent bars (if any) are positioned in an anti-clockwise sense from the first bar at equal centres.

Edge distance to centre of bar
<table>
<thead>
<tr>
<th>Set</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to bar centres</th>
<th>Start angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8</td>
<td>20</td>
<td>70</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>16</td>
<td>100</td>
<td>45</td>
</tr>
</tbody>
</table>

Modular ratio mr=15
Plot of Section Details
Loading Case 1

Axial Load
Axial load fixed in position.

x co-ordinate                 xp(1)=350 mm
y co-ordinate                 yp(1)=400 mm

Applied moments

About x axis                   Amx(1)=-100 kNm
About y axis                   Amy(1)=150 kNm

All of section in compression

Centroid of stressed section  X=500        Y=500

Design bending moments at centroid of stressed section

About x axis                   Dmx=-P(z)*(yp(z)-Y)/1000+Amx(z)
                                =200 kNm
About y axis                   Dmy=P(z)*(xp(z)-X)/1000+Amy(z)
                                =-300 kNm

Concrete stresses
Maximum and minimum compressive stresses in concrete

<table>
<thead>
<tr>
<th>Co-ordinates</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>y</td>
</tr>
<tr>
<td>922.16</td>
<td>767.91</td>
</tr>
</tbody>
</table>

Maximum
77.836   232.09   6.9979

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar Set</th>
<th>Bar No.</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>x</td>
<td>y</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>500</td>
<td>930</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>195.94</td>
<td>804.06</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>70</td>
<td>500</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>195.94</td>
<td>195.94</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>500</td>
<td>70</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>6</td>
<td>804.06</td>
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<td>20</td>
</tr>
<tr>
<td>1</td>
<td>7</td>
<td>930</td>
<td>500</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>8</td>
<td>804.06</td>
<td>804.06</td>
<td>20</td>
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<td>2</td>
<td>1</td>
<td>217.16</td>
<td>782.84</td>
<td>16</td>
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<tr>
<td>2</td>
<td>2</td>
<td>217.16</td>
<td>217.16</td>
<td>16</td>
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<tr>
<td>2</td>
<td>3</td>
<td>782.84</td>
<td>217.16</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>782.84</td>
<td>782.84</td>
<td>16</td>
</tr>
</tbody>
</table>
Location: Fully cracked section

Elastic biaxial bending of a circular RC section

Units used in the calculations:
Input  Dimensions and bar diameters  mm
       Forcs  kN
       Bending moments  kNm
Output  Dimensions and co-ordinates  mm
       Stresses  N/mm²

Sign Convention for loading

A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

A reinforcement set is defined by the following parameters:

Number of bars. Diameter of bars. Edge distance to centre of bars.
Anti-clockwise angle (Ω) measured in degrees from local axis y' to locate first bar in set. Subsequent bars (if any) are positioned in an anti-clockwise sense from the first bar at equal centres.
<table>
<thead>
<tr>
<th>Set</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to bar centres</th>
<th>Start angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8</td>
<td>20</td>
<td>70</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>16</td>
<td>100</td>
<td>45</td>
</tr>
</tbody>
</table>

Modular ratio  

mr=15
Plot of Section Details
Loading Case 1

Axial Load

\[ P(1) = -3000 \text{ kN} \]

Axial load fixed in position.

x co-ordinate \( \text{xp}(1) = 500 \text{ mm} \)
y co-ordinate \( \text{yp}(1) = 500 \text{ mm} \)

Applied moments

About x axis

\[ \text{Amx}(1) = -100 \text{ kNm} \]

About y axis

\[ \text{Amy}(1) = 150 \text{ kNm} \]

All of section in tension

Centroid of stressed section \( X=500 \quad Y=500 \)

Design bending moments at centroid of stressed section

About x axis

\[ \text{Dmx} = -P(z) \left( \text{yp}(z) - Y \right)/1000 + \text{Amx}(z) = -100 \text{ kNm} \]

About y axis

\[ \text{Dmy} = P(z) \left( \text{xp}(z) - X \right)/1000 + \text{Amy}(z) = 150 \text{ kNm} \]

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar Set</th>
<th>Bar No.</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>500 930</td>
<td>20</td>
<td>-759.39</td>
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<tr>
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<td>2</td>
<td>195.94 804.06</td>
<td>20</td>
<td>-955.52</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>70 500</td>
<td>20</td>
<td>-1121.6</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>195.94 195.94</td>
<td>20</td>
<td>-1160.4</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>500 70</td>
<td>20</td>
<td>-1049.2</td>
</tr>
<tr>
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<td>6</td>
<td>804.06 195.94</td>
<td>20</td>
<td>-853.06</td>
</tr>
<tr>
<td>1</td>
<td>7</td>
<td>930 500</td>
<td>20</td>
<td>-686.95</td>
</tr>
<tr>
<td>1</td>
<td>8</td>
<td>804.06 804.06</td>
<td>20</td>
<td>-648.15</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>217.16 782.84</td>
<td>16</td>
<td>-951.94</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>217.16 217.16</td>
<td>16</td>
<td>-1142.6</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>782.84 217.16</td>
<td>16</td>
<td>-856.64</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>782.84 782.84</td>
<td>16</td>
<td>-666.02</td>
</tr>
</tbody>
</table>
Location: Cracked section. Axial load moves to centroid.

Elastic biaxial bending of a circular RC section

Units used in the calculations:
Input  Dimensions and bar diameters   mm
        Forces                          kN
        Bending moments                 kNm
Output Dimensions and co-ordinates   mm
        Stresses                       N/mm²

Sign Convention for loading

A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

A reinforcement set is defined by the following parameters:
Number of bars. Diameter of bars. Edge distance to centre of bars.
Anti-clockwise angle (θ) measured in degrees from local axis y’
to locate first bar in set. Subsequent bars (if any) are positioned
in an anti-clockwise sense from the first bar at equal centres.
### Table: Details of Bars

<table>
<thead>
<tr>
<th>Set</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to bar centres</th>
<th>Start angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>16</td>
<td>56</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>12</td>
<td>56</td>
<td>45</td>
</tr>
</tbody>
</table>

**Modular ratio**: $mr = 15$
Plot of Section Details
Loading Case 1

Axial Load

\[ P(1) = 1000 \text{ kN} \]
Axial load applied at centroid of stressed section.

Applied moments

About x axis

\[ A_{mx}(1) = -50 \text{ kNm} \]

About y axis

\[ A_{my}(1) = 75 \text{ kNm} \]
Plot of Cracked Section
Section is cracked

Neutral axis line

\[ \begin{align*}
x &= 120.82 \\
y &= 540.5 \\
x &= 453.29 \\
y &= 42.292
\end{align*} \]

Centroid of stressed section \( X = 387.7 \quad Y = 358.58 \)

Design bending moments at centroid of stressed section

- About x axis: \( D_{mx} = A_{mx}(z) = -50 \text{ kNm} \)
- About y axis: \( D_{my} = A_{my}(z) = 75 \text{ kNm} \)

Maximum concrete stress

\[
\begin{array}{ccc}
\text{Co-ordinates} & \text{Stress} \\
\text{x} & \text{y} & (\text{N/mm}^2) \\
(\text{mm}) & (\text{mm}) & \\
553.3 & 460.75 & 15.439 \\
\end{array}
\]

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar Set</th>
<th>Bar No.</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>300 544</td>
<td>16</td>
<td>110.84</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>56 300</td>
<td>16</td>
<td>-137.58</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>300 56</td>
<td>16</td>
<td>-88.016</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>544 300</td>
<td>16</td>
<td>160.4</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>127.47 472.53</td>
<td>12</td>
<td>-23.637</td>
</tr>
<tr>
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<td>127.47 127.47</td>
<td>12</td>
<td>-164.25</td>
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<tr>
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<td>3</td>
<td>472.53 127.47</td>
<td>12</td>
<td>46.457</td>
</tr>
<tr>
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<td>4</td>
<td>472.53 472.53</td>
<td>12</td>
<td>187.07</td>
</tr>
</tbody>
</table>

Number of iterations 21
Location: Uncracked section. Axial load moves to centroid.

Elastic biaxial bending of a circular RC section

Units used in the calculations:
Input   Dimensions and bar diameters   mm
        Forces                         kN
        Bending moments                kNm
Output  Dimensions and co-ordinates    mm
        Stresses                       N/mm²

Sign Convention for loading

A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

A reinforcement set is defined by the following parameters:

Number of bars. Diameter of bars. Edge distance to centre of bars.
Anti-clockwise angle (θ) measured in degrees from local axis y' to locate first bar in set. Subsequent bars (if any) are positioned in an anti-clockwise sense from the first bar at equal centres.

Edge distance to centre of bar

Circular section diameter 1000
<table>
<thead>
<tr>
<th>Set</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to bar centres</th>
<th>Start angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8</td>
<td>20</td>
<td>70</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>16</td>
<td>100</td>
<td>45</td>
</tr>
</tbody>
</table>

Modular ratio \( mr = 15 \)
Plot of Section Details
Loading Case 1

Axial Load $P(1)=3000$ kN
Axial load applied at centroid of stressed section.

Applied moments

About x axis $Amx(1)=-100$ kNm
About y axis $Amy(1)=150$ kNm

All of section in compression

Centroid of stressed section $X=500$ $Y=500$

Design bending moments at centroid of stressed section

About x axis $Dmx=Amx(z)=-100$ kNm
About y axis $Dmy=Amy(z)=150$ kNm

Concrete stresses

Maximum and minimum compressive stresses in concrete

<table>
<thead>
<tr>
<th>Co-ordinates</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>y</td>
</tr>
<tr>
<td>77.836</td>
<td>232.09</td>
</tr>
</tbody>
</table>

Maximum

922.16 767.91 5.3033

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar Set</th>
<th>Bar No.</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>x</td>
<td>y</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>500</td>
<td>930</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>195.94</td>
<td>804.06</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>70</td>
<td>500</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>195.94</td>
<td>195.94</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>500</td>
<td>70</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>6</td>
<td>804.06</td>
<td>195.94</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>7</td>
<td>930</td>
<td>500</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>8</td>
<td>804.06</td>
<td>804.06</td>
<td>20</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>217.16</td>
<td>782.84</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>217.16</td>
<td>217.16</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>782.84</td>
<td>217.16</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>782.84</td>
<td>782.84</td>
<td>16</td>
</tr>
</tbody>
</table>
Location: Fully cracked section. Axial load moves to centroid.

Elastic biaxial bending of a circular RC section

Units used in the calculations:

Input  Dimensions and bar diameters  mm
Forces                         kN
Bending moments                kNm

Output  Dimensions and co-ordinates  mm
Stresses                       N/mm²

Sign Convention for loading

A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

A reinforcement set is defined by the following parameters:

Number of bars. Diameter of bars. Edge distance to centre of bars.
Anti-clockwise angle (θ) measured in degrees from local axis y' to locate first bar in set. Subsequent bars (if any) are positioned in an anti-clockwise sense from the first bar at equal centres.
<table>
<thead>
<tr>
<th>Set</th>
<th>Number of bars</th>
<th>Bar diameter</th>
<th>Edge distances to bar centres</th>
<th>Start angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8</td>
<td>20</td>
<td>70</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>16</td>
<td>100</td>
<td>45</td>
</tr>
</tbody>
</table>

Modular ratio \( mr = 15 \)
Plot of Section Details
Loading Case 1

Axial Load \[ P(1) = -3000 \text{ kN} \]
Axial load applied at centroid of stressed section.

Applied moments

About x axis \[ A_{mx}(1) = -100 \text{ kNm} \]
About y axis \[ A_{my}(1) = 150 \text{ kNm} \]

All of section in tension

Centroid of stressed section \[ X = 500 \quad Y = 500 \]

Design bending moments at centroid of stressed section

About x axis \[ D_{mx} = A_{mx}(z) = -100 \text{ kNm} \]
About y axis \[ D_{my} = A_{my}(z) = 150 \text{ kNm} \]

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar Set</th>
<th>Bar No.</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>500 930</td>
<td>20</td>
<td>-759.39</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>195.94 804.06</td>
<td>20</td>
<td>-955.52</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>70 500</td>
<td>20</td>
<td>-1121.6</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>195.94 195.94</td>
<td>20</td>
<td>-1160.4</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>500 70</td>
<td>20</td>
<td>-1049.2</td>
</tr>
<tr>
<td>1</td>
<td>6</td>
<td>804.06 195.94</td>
<td>20</td>
<td>-853.06</td>
</tr>
<tr>
<td>1</td>
<td>7</td>
<td>930 500</td>
<td>20</td>
<td>-686.95</td>
</tr>
<tr>
<td>1</td>
<td>8</td>
<td>804.06 804.06</td>
<td>20</td>
<td>-648.15</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>217.16 782.84</td>
<td>16</td>
<td>-951.94</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>217.16 217.16</td>
<td>16</td>
<td>-1142.6</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>782.84 217.16</td>
<td>16</td>
<td>-856.64</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>782.84 782.84</td>
<td>16</td>
<td>-666.02</td>
</tr>
</tbody>
</table>
Location: Pier section at foundation

Elastic biaxial bending of a user defined reinforced concrete section

Units used in the calculations:

<table>
<thead>
<tr>
<th>Input</th>
<th>Dimensions and bar diameters</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forces</td>
<td></td>
<td>kN</td>
</tr>
<tr>
<td>Bending moments</td>
<td></td>
<td>kNm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Output</th>
<th>Dimensions and co-ordinates</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stresses</td>
<td></td>
<td>N/mm²</td>
</tr>
</tbody>
</table>

Sign Convention for loading
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

Section details

Number of items in section r=2

Item 1 is a polygon

Item weighting iw(1)=1

Number of corners in item n(1)=4

Item co-ordinates

Corner 1

x= ox(1)=0
y= oy(1)=0

Corner 2

x= ox(2)=1000
y= oy(2)=0

Corner 3

x= ox(3)=1000
y= oy(3)=500

Corner 4

x= ox(4)=0
y= oy(4)=500
Plot of Item Details
Item 2 is a polygon

Item weighting  \( iw(2)=-1 \)
Number of corners in item  \( n(2)=4 \)

Item co-ordinates
Corner 1  \( x= \)
          \( oy(51)=100 \)
          \( y= \)
          \( oy(51)=100 \)
Corner 2  \( x= \)
          \( ox(52)=900 \)
          \( y= \)
          \( oy(52)=100 \)
Corner 3  \( x= \)
          \( ox(53)=900 \)
          \( y= \)
          \( oy(53)=400 \)
Corner 4  \( x= \)
          \( ox(54)=100 \)
          \( y= \)
          \( oy(54)=400 \)
Plot of Item Details
### Bar reinforcement details

<table>
<thead>
<tr>
<th>No of sets of bars</th>
<th>nos=6</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bar set 1</strong></td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>na(1)=4</td>
</tr>
<tr>
<td>Co-ordinates of first bar</td>
<td>xf(1)=60, yf(1)=60</td>
</tr>
<tr>
<td>Co-ordinates of last bar</td>
<td>xl(1)=940, yl(1)=60</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>da(1)=16</td>
</tr>
<tr>
<td><strong>Bar set 2</strong></td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>na(2)=3</td>
</tr>
<tr>
<td>Co-ordinates of first bar</td>
<td>xf(2)=206, yf(2)=60</td>
</tr>
<tr>
<td>Co-ordinates of last bar</td>
<td>xl(2)=794, yl(2)=60</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>da(2)=12</td>
</tr>
<tr>
<td><strong>Bar set 3</strong></td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>na(3)=3</td>
</tr>
<tr>
<td>Co-ordinates of first bar</td>
<td>xf(3)=940, yf(3)=155</td>
</tr>
<tr>
<td>Co-ordinates of last bar</td>
<td>xl(3)=940, yl(3)=345</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>da(3)=12</td>
</tr>
<tr>
<td><strong>Bar set 4</strong></td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>na(4)=4</td>
</tr>
<tr>
<td>Co-ordinates of first bar</td>
<td>xf(4)=60, yf(4)=440</td>
</tr>
<tr>
<td>Co-ordinates of last bar</td>
<td>xl(4)=940, yl(4)=440</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>da(4)=16</td>
</tr>
<tr>
<td><strong>Bar set 5</strong></td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>na(5)=3</td>
</tr>
<tr>
<td>Co-ordinates of first bar</td>
<td>xf(5)=206, yf(5)=440</td>
</tr>
<tr>
<td>Co-ordinates of last bar</td>
<td>xl(5)=794, yl(5)=440</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>da(5)=12</td>
</tr>
<tr>
<td><strong>Bar set 6</strong></td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>na(6)=3</td>
</tr>
<tr>
<td>Co-ordinates of first bar</td>
<td>xf(6)=60, yf(6)=155</td>
</tr>
</tbody>
</table>
Co-ordinates of last bar
xl(6)=60
yl(6)=345

Diameter of bars
da(6)=12

Modular ratio
mr=15
Plot of Section Details
Loading Case 1

Axial Load
P(1)=1000 kN
Axial load fixed in position.

x co-ordinate  xp(1)=600 mm
y co-ordinate  yp(1)=300 mm

Applied moments

About x axis  \( \text{Amx}(1)=-200 \text{ kNm} \)
About y axis  \( \text{Amy}(1)=300 \text{ kNm} \)
Plot of Cracked Section
Section is cracked

Neutral axis line

Item 1
\[
\begin{align*}
  x &= 1000 \\
  y &= 67.936 \\
  x &= 17.554 \\
  y &= 500
\end{align*}
\]

Item 2
\[
\begin{align*}
  x &= 900 \\
  y &= 111.91 \\
  x &= 244.94 \\
  y &= 400
\end{align*}
\]

Centroid of stressed section  \( X = 619.43 \quad Y = 356.91 \)

Design bending moments at centroid of stressed section

About x axis  \( D_{mx} = -P(z)(y_p(z) - Y)/1000 + A_{mx}(z) = -143.09 \text{ kNm} \)

About y axis  \( D_{my} = P(z)(x_p(z) - X)/1000 + A_{my}(z) = 280.57 \text{ kNm} \)

Maximum compressive stresses in concrete

<table>
<thead>
<tr>
<th>Co-ordinates</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>y</td>
</tr>
<tr>
<td>Item 1</td>
<td></td>
</tr>
<tr>
<td>1000</td>
<td>500</td>
</tr>
<tr>
<td>Item 2</td>
<td></td>
</tr>
<tr>
<td>900</td>
<td>400</td>
</tr>
</tbody>
</table>

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Bar Set</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>60</td>
<td>60</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>353.33</td>
<td>60</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>646.67</td>
<td>60</td>
<td>16</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>940</td>
<td>60</td>
<td>16</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>206</td>
<td>60</td>
<td>12</td>
</tr>
<tr>
<td>6</td>
<td>2</td>
<td>500</td>
<td>60</td>
<td>12</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>794</td>
<td>60</td>
<td>12</td>
</tr>
<tr>
<td>8</td>
<td>3</td>
<td>940</td>
<td>155</td>
<td>12</td>
</tr>
<tr>
<td>9</td>
<td>3</td>
<td>940</td>
<td>250</td>
<td>12</td>
</tr>
<tr>
<td>10</td>
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<td>940</td>
<td>345</td>
<td>12</td>
</tr>
<tr>
<td>11</td>
<td>4</td>
<td>60</td>
<td>440</td>
<td>16</td>
</tr>
<tr>
<td>12</td>
<td>4</td>
<td>353.33</td>
<td>440</td>
<td>16</td>
</tr>
<tr>
<td>13</td>
<td>4</td>
<td>646.67</td>
<td>440</td>
<td>16</td>
</tr>
<tr>
<td>14</td>
<td>4</td>
<td>940</td>
<td>440</td>
<td>16</td>
</tr>
<tr>
<td>15</td>
<td>5</td>
<td>206</td>
<td>440</td>
<td>12</td>
</tr>
<tr>
<td>16</td>
<td>5</td>
<td>500</td>
<td>440</td>
<td>12</td>
</tr>
<tr>
<td>17</td>
<td>5</td>
<td>794</td>
<td>440</td>
<td>12</td>
</tr>
<tr>
<td>18</td>
<td>6</td>
<td>60</td>
<td>155</td>
<td>12</td>
</tr>
<tr>
<td>19</td>
<td>6</td>
<td>60</td>
<td>250</td>
<td>12</td>
</tr>
<tr>
<td>20</td>
<td>6</td>
<td>60</td>
<td>345</td>
<td>12</td>
</tr>
</tbody>
</table>
Number of iterations 7
Location: Uncracked section

Elastic biaxial bending of a user defined reinforced concrete section

Units used in the calculations:

<table>
<thead>
<tr>
<th>Input</th>
<th>Dimensions and bar diameters</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forces</td>
<td></td>
<td>kN</td>
</tr>
<tr>
<td>Bending moments</td>
<td></td>
<td>kNm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Output</th>
<th>Dimensions and co-ordinates</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stresses</td>
<td></td>
<td>N/mm²</td>
</tr>
</tbody>
</table>

Sign Convention for loading
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

Section details

Number of items in section    r=2

Item 1 is a polygon

Item weighting      iw(1)=1
Number of corners in item n(1)=4

Item co-ordinates

Corner 1   x=                  ox(1)=0
           y=                  oy(1)=0
Corner 2   x=                  ox(2)=1000
           y=                  oy(2)=0
Corner 3   x=                  ox(3)=1000
           y=                  oy(3)=500
Corner 4   x=                  ox(4)=0
           y=                  oy(4)=500
Plot of Item Details
Item 2 is a polygon

Item weighting       iw(2)=-1
Number of corners in item n(2)=4

Item co-ordinates
Corner 1           x= ox(51)=100
                      y= oy(51)=100
Corner 2           x= ox(52)=900
                      y= oy(52)=100
Corner 3           x= ox(53)=900
                      y= oy(53)=400
Corner 4           x= ox(54)=100
                      y= oy(54)=400
Plot of Item Details
Bar reinforcement details

No of sets of bars  nos=6

Bar set 1
Number of bars  na(1)=4

Co-ordinates of first bar  xf(1)=60
                       yf(1)=60
Co-ordinates of last bar  xl(1)=940
                       yl(1)=60
Diameter of bars  da(1)=16

Bar set 2
Number of bars  na(2)=3

Co-ordinates of first bar  xf(2)=206
                       yf(2)=60
Co-ordinates of last bar  xl(2)=794
                       yl(2)=60
Diameter of bars  da(2)=12

Bar set 3
Number of bars  na(3)=3

Co-ordinates of first bar  xf(3)=940
                       yf(3)=155
Co-ordinates of last bar  xl(3)=940
                       yl(3)=345
Diameter of bars  da(3)=12

Bar set 4
Number of bars  na(4)=4

Co-ordinates of first bar  xf(4)=60
                       yf(4)=440
Co-ordinates of last bar  xl(4)=940
                       yl(4)=440
Diameter of bars  da(4)=16

Bar set 5
Number of bars  na(5)=3

Co-ordinates of first bar  xf(5)=206
                       yf(5)=440
Co-ordinates of last bar  xl(5)=794
                       yl(5)=440
Diameter of bars  da(5)=12

Bar set 6
Number of bars  na(6)=3

Co-ordinates of first bar  xf(6)=60
                       yf(6)=155
Co-ordinates of last bar
xl(6)=60
yl(6)=345
Diameter of bars
da(6)=12
Modular ratio
mr=15
Plot of Section Details
Loading Case 1

Axial Load
Axial load fixed in position.

x co-ordinate xp(1)=400 mm
y co-ordinate yp(1)=300 mm

Applied moments

About x axis Amx(1)=0 kNm
About y axis Amy(1)=0 kNm

All of section in compression

Centroid of stressed section X=500 Y=250

Design bending moments at centroid of stressed section

About x axis
\[ D_{mx} = -P(z)*(yp(z)-Y)/1000 + Amx(z) \]
= -50 kNm

About y axis
\[ D_{my} = P(z)*(xp(z)-X)/1000 + Amy(z) \]
= -100 kNm

Concrete stresses

Maximum and minimum compressive stresses in concrete

<table>
<thead>
<tr>
<th>Item</th>
<th>Co-ordinates x</th>
<th>y</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1000</td>
<td>0</td>
<td>0.56021</td>
</tr>
<tr>
<td>Maximum</td>
<td>0</td>
<td>500</td>
<td>6.0729</td>
</tr>
<tr>
<td>2</td>
<td>900 100</td>
<td></td>
<td>1.3658</td>
</tr>
<tr>
<td>Maximum</td>
<td>100</td>
<td>400</td>
<td>5.2673</td>
</tr>
</tbody>
</table>

Maximum and minimum reinforcement bar stresses.
Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Bar Set</th>
<th>Co-ordinates x</th>
<th>y</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>4 1</td>
<td>940 60</td>
<td></td>
<td>16</td>
<td>15.653</td>
</tr>
<tr>
<td>Maximum</td>
<td>11 4</td>
<td>60 440</td>
<td></td>
<td>16</td>
<td>83.843</td>
</tr>
</tbody>
</table>
Location: Fully cracked section

Elastic biaxial bending of a user defined reinforced concrete section

Units used in the calculations:

<table>
<thead>
<tr>
<th>Input</th>
<th>Dimensions and bar diameters</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forces</td>
<td>kN</td>
<td></td>
</tr>
<tr>
<td>Bending moments</td>
<td>kNm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Output</th>
<th>Dimensions and co-ordinates</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stresses</td>
<td>N/mm²</td>
<td></td>
</tr>
</tbody>
</table>

Sign Convention for loading:
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

Section details

Number of items in section \( r = 2 \)

Item 1 is a polygon

Item weighting \( iw(1) = 1 \)

Number of corners in item \( n(1) = 4 \)

Item co-ordinates

Corner 1  \( x = \) ox(1) = 0  \( y = \) oy(1) = 0

Corner 2  \( x = \) ox(2) = 1000  \( y = \) oy(2) = 0

Corner 3  \( x = \) ox(3) = 1000  \( y = \) oy(3) = 500

Corner 4  \( x = \) ox(4) = 0  \( y = \) oy(4) = 500
Plot of Item Details
Item 2 is a polygon

Item weighting \( i_w(2) = -1 \)
Number of corners in item \( n(2) = 4 \)

Item co-ordinates
Corner 1  
\[
\begin{align*}
x &= o_x(51) = 100 \\
y &= o_y(51) = 100
\end{align*}
\]
Corner 2  
\[
\begin{align*}
x &= o_x(52) = 900 \\
y &= o_y(52) = 100
\end{align*}
\]
Corner 3  
\[
\begin{align*}
x &= o_x(53) = 900 \\
y &= o_y(53) = 400
\end{align*}
\]
Corner 4  
\[
\begin{align*}
x &= o_x(54) = 100 \\
y &= o_y(54) = 400
\end{align*}
\]
Bar reinforcement details

No of sets of bars  nos=6

Bar set 1
Number of bars  na(1)=4

Co-ordinates of first bar  xf(1)=60  yf(1)=60
Co-ordinates of last bar  xl(1)=940  yl(1)=60
Diameter of bars  da(1)=16

Bar set 2
Number of bars  na(2)=3

Co-ordinates of first bar  xf(2)=206  yf(2)=60
Co-ordinates of last bar  xl(2)=794  yl(2)=60
Diameter of bars  da(2)=12

Bar set 3
Number of bars  na(3)=3

Co-ordinates of first bar  xf(3)=940  yf(3)=155
Co-ordinates of last bar  xl(3)=940  yl(3)=345
Diameter of bars  da(3)=12

Bar set 4
Number of bars  na(4)=4

Co-ordinates of first bar  xf(4)=60  yf(4)=440
Co-ordinates of last bar  xl(4)=940  yl(4)=440
Diameter of bars  da(4)=16

Bar set 5
Number of bars  na(5)=3

Co-ordinates of first bar  xf(5)=206  yf(5)=440
Co-ordinates of last bar  xl(5)=794  yl(5)=440
Diameter of bars  da(5)=12

Bar set 6
Number of bars  na(6)=3

Co-ordinates of first bar  xf(6)=60  yf(6)=155
Co-ordinates of last bar  \( x_l(6)=60 \)
\( y_l(6)=345 \)

Diameter of bars  \( d_a(6)=12 \)

Modular ratio  \( m_r=15 \)
Plot of Item Details
Plot of Section Details
Loading Case 1

Axial Load

P(1)=-500 kN
Axial load applied at centroid of stressed section.

Applied moments

About x axis

Amx(1)=0 kNm

About y axis

Amy(1)=0 kNm

All of section in tension

Centroid of stressed section

X=500        Y=250

Design bending moments at centroid of stressed section

About x axis

Dmx=Amx(z)=0 kNm

About y axis

Dmy=Amy(z)=0 kNm

Reinforcement bar stresses. Tension negative, Compression positive.

Tensile stress in all bars

-168.6 N/mm²
Location: Circular section. Cracked.

Elastic biaxial bending of a user defined reinforced concrete section

Units used in the calculations:
Input   Dimensions and bar diameters   mm
        Forces                         kN
        Bending moments                kNm
Output  Dimensions and co-ordinates    mm
        Stresses                       N/mm²

Sign Convention for loading
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

Section details
Number of items in section           r=2
Item 1 is a circular section
Item weighting                       iw(1)=1
Co-ordinates of circle centre         xc(1)=300 mm
                                          yc(1)=300 mm
Diameter of circular section         cd(1)=600 mm
Plot of Item Details
Item 2 is a circular section

- Item weighting: $iw(2) = -1$
- Co-ordinates of circle centre: $xc(2) = 300$ mm, $yc(2) = 300$ mm
- Diameter of circular section: $cd(2) = 376$ mm
Plot of Item Details
Bar reinforcement details

No of sets of bars nos=8

Bar set 1
Number of bars na(1)=1
Co-ordinates of bar xf(1)=300
yf(1)=544
Diameter of bar da(1)=16

Bar set 2
Number of bars na(2)=1
Co-ordinates of bar xf(2)=56
yf(2)=300
Diameter of bar da(2)=16

Bar set 3
Number of bars na(3)=1
Co-ordinates of bar xf(3)=300
yf(3)=56
Diameter of bar da(3)=16

Bar set 4
Number of bars na(4)=1
Co-ordinates of bar xf(4)=544
yf(4)=300
Diameter of bar da(4)=16

Bar set 5
Number of bars na(5)=1
Co-ordinates of bar xf(5)=127.47
yf(5)=472.53
Diameter of bar da(5)=12

Bar set 6
Number of bars na(6)=1
Co-ordinates of bar xf(6)=127.47
yf(6)=127.47
Diameter of bar da(6)=12

Bar set 7
Number of bars na(7)=1
Co-ordinates of bar xf(7)=472.53
yf(7)=127.47
Diameter of bar da(7)=12

Bar set 8
Sample output for SCALE Proforma 195. (ans=4)              Page: 6
R.Concrete: Eurocode 2, BS8007, BS8666, analysis          Made by: IFB
Biaxial bending of user defined section                  Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.                 Ref No: SC195

Number of bars     na(8)=1
Co-ordinates of bar xf(8)=472.53  
yf(8)=472.53
Diameter of bar    da(8)=12
Modular ratio      mr=15
Plot of Item Details
Plot of Section Details
Loading Case 1

Axial Load

$P(1) = 1000 \, \text{kN}$

Axial load applied at centroid of stressed section.

Applied moments

About x axis

$Amx(1) = -200 \, \text{kNm}$

About y axis

$Amy(1) = 300 \, \text{kNm}$
Plot of Cracked Section
Section is cracked
Neutral axis line

Item 1
\[ x = 234.9 \quad y = 592.27 \]
\[ x = 544.34 \quad y = 126.85 \]

Item 2
\[ x = 474.83 \quad y = 231.41 \]
\[ x = 304.42 \quad y = 487.71 \]

Centroid of stressed section
\[ X = 427.2 \quad Y = 384.28 \]

Design bending moments at centroid of stressed section

- About x axis: \( D_{mx} = A_{mx}(z) = -200 \text{ kNm} \)
- About y axis: \( D_{my} = A_{my}(z) = 300 \text{ kNm} \)

Maximum compressive stresses in concrete

<table>
<thead>
<tr>
<th>Co-ordinates</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>y</td>
</tr>
</tbody>
</table>

- Item 1
  | Co-ordinates | Stress |
  | 559.81       | 450    | 54.013 |

- Item 2
  | Co-ordinates | Stress |
  | 462.81       | 394    | 22.534 |

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Bar Set</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>300</td>
<td>544</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>56</td>
<td>300</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>300</td>
<td>56</td>
<td>16</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>544</td>
<td>300</td>
<td>16</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>127.47</td>
<td>472.53</td>
<td>12</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>127.47</td>
<td>127.47</td>
<td>12</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>472.53</td>
<td>127.47</td>
<td>12</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>472.53</td>
<td>472.53</td>
<td>12</td>
</tr>
</tbody>
</table>

Number of iterations 7
Location: Dodecagon with circular void

Elastic biaxial bending of a user defined reinforced concrete section

Units used in the calculations:
Input  Dimensions and bar diameters  mm
Forces  kN
Bending moments  kNm
Output  Dimensions and co-ordinates  mm
Stresses  N/mm²

Sign Convention for loading
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

Section details
Number of items in section  r=2
Item 1 is a polygon

Item weighting  iw(1)=1
Number of corners in item  n(1)=12

Item co-ordinates
Corner 1  x= ox(1)=0
          y= oy(1)=0
Corner 2  x= ox(2)=50
          y= oy(2)=0
Corner 3  x= ox(3)=75
          y= oy(3)=100
Corner 4  x= ox(4)=100
          y= oy(4)=100
Corner 5  x= ox(5)=125
          y= oy(5)=0
Corner 6  x= ox(6)=175
          y= oy(6)=0
Corner 7  x= ox(7)=200
          y= oy(7)=100
Corner 8  x= ox(8)=250
          y= oy(8)=100
Corner 9  x= ox(9)=275
          y= oy(9)=0
Corner 10 x= ox(10)=350
          y= oy(10)=0
Corner 11 x= ox(11)=350
          y= oy(11)=350
Corner 12 x= ox(12)=0
          y= oy(12)=350
Plot of Item Details
Item 2 is a circular section

Item weighting  
iw(2) = -1

Co-ordinates of circle centre  
xc(2) = 150 mm  
yc(2) = 200 mm

Diameter of circular section  
cd(2) = 200 mm
Plot of Item Details
Bar reinforcement details

No of sets of bars  
nos=2

Bar set 1
Number of bars  
na(1)=2

Co-ordinates of first bar  
xf(1)=50  
yf(1)=300

Co-ordinates of last bar  
xl(1)=300  
yl(1)=300

Diameter of bars  
da(1)=12

Bar set 2
Number of bars  
na(2)=2

Co-ordinates of first bar  
xf(2)=50  
yf(2)=50

Co-ordinates of last bar  
xl(2)=300  
yl(2)=50

Diameter of bars  
da(2)=12

Modular ratio  
Mr=15
Plot of Item Details
Plot of Section Details
Loading Case 1

Axial Load
Axial load fixed in position.

x co-ordinate
xp(1)=155 mm

y co-ordinate
yp(1)=175 mm

Applied moments

About x axis
Amx(1)=300 kNm

About y axis
Amy(1)=125 kNm
Plot of Cracked Section
Section is cracked

Neutral axis line

Item 1

x=72.151  y=88.606
x=101.26   y=94.959
x=350     y=149.25
x=0       y=72.859

Item 2

x=132.41  y=101.76
x=206.91  y=118.02

Centroid of stressed section  X=200.17  Y=87.22

Design bending moments at centroid of stressed section

About x axis

\[ D_{mx} = \frac{-P(z) \cdot (y_p(z) - Y)}{1000} + A_{mx}(z) \]
= 212.22 kNm

About y axis

\[ D_{my} = \frac{P(z) \cdot (x_p(z) - X)}{1000} + A_{my}(z) \]
= 79.83 kNm

Maximum compressive stresses in concrete

<table>
<thead>
<tr>
<th>Co-ordinates</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>y</td>
</tr>
<tr>
<td>----</td>
<td>-----</td>
</tr>
<tr>
<td>Item 1</td>
<td>350</td>
</tr>
<tr>
<td>Item 2</td>
<td>175.88</td>
</tr>
</tbody>
</table>

Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Bar Set</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>50</td>
<td>300</td>
<td>12</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>300</td>
<td>300</td>
<td>12</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>50</td>
<td>50</td>
<td>12</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>300</td>
<td>50</td>
<td>12</td>
</tr>
</tbody>
</table>

Number of iterations 7
Location: Dumb-bell with circular voids

Elastic biaxial bending of a user defined reinforced concrete section

Units used in the calculations:
Input  Dimensions and bar diameters     mm
Forces                          kN
Bending moments                  kNm
Output  Dimensions and co-ordinates mm
Stresses                        N/mm²

Sign Convention for loading
A positive axial load causes compression in the section.
A positive moment is clockwise about the x or y axes.

Section details

Number of items in section    r=5

Item 1 is a polygon

Item weighting            iw(1)=1
Number of corners in item  n(1)=4

Item co-ordinates
Corner 1     x=    ox(1)=0
              y=    oy(1)=0
Corner 2     x=    ox(2)=500
              y=    oy(2)=0
Corner 3     x=    ox(3)=500
              y=    oy(3)=500
Corner 4     x=    ox(4)=0
              y=    oy(4)=500
Plot of Item Details
Item 2 is a polygon

Item weighting  \( iw(2)=1 \)
Number of corners in item  \( n(2)=4 \)

Item co-ordinates

<table>
<thead>
<tr>
<th>Corner</th>
<th>( x = )</th>
<th>( ox(51) = 1000 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( y = )</td>
<td>( oy(51) = 0 )</td>
</tr>
<tr>
<td>Corner 2</td>
<td>( x = )</td>
<td>( ox(52) = 1500 )</td>
</tr>
<tr>
<td></td>
<td>( y = )</td>
<td>( oy(52) = 0 )</td>
</tr>
<tr>
<td>Corner 3</td>
<td>( x = )</td>
<td>( ox(53) = 1500 )</td>
</tr>
<tr>
<td></td>
<td>( y = )</td>
<td>( oy(53) = 500 )</td>
</tr>
<tr>
<td>Corner 4</td>
<td>( x = )</td>
<td>( ox(54) = 1000 )</td>
</tr>
<tr>
<td></td>
<td>( y = )</td>
<td>( oy(54) = 500 )</td>
</tr>
</tbody>
</table>
Plot of Item Details
Item 3 is a polygon

Item weighting \( iw(3) = 1 \)
Number of corners in item \( n(3) = 4 \)

Item co-ordinates
Corner 1 \( x= \) ox(101)=500
\( y= \) oy(101)=150
Corner 2 \( x= \) ox(102)=1000
\( y= \) oy(102)=150
Corner 3 \( x= \) ox(103)=1000
\( y= \) oy(103)=350
Corner 4 \( x= \) ox(104)=500
\( y= \) oy(104)=350
Plot of Item Details
Item 4 is a circular section

Item weighting  $iw(4)=-1$

Co-ordinates of circle centre  $xc(4)=250 \text{ mm}$  
                           $yc(4)=250 \text{ mm}$

Diameter of circular section  $cd(4)=300 \text{ mm}$
Plot of Item Details
Item 5 is a circular section

Item weighting  \( iw(5) = -1 \)

Co-ordinates of circle centre  
\[
\begin{align*}
xc(5) &= 1250 \text{ mm} \\
yc(5) &= 250 \text{ mm}
\end{align*}
\]

Diameter of circular section  \( cd(5) = 300 \text{ mm} \)
Plot of Item Details
Bar reinforcement details

No of sets of bars  nos=5

Bar set 1
Number of bars  na(1)=2

Co-ordinates of first bar  xf(1)=50  yf(1)=50
Co-ordinates of last bar  xl(1)=450  yl(1)=50
Diameter of bars  da(1)=25

Bar set 2
Number of bars  na(2)=2

Co-ordinates of first bar  xf(2)=50  yf(2)=450
Co-ordinates of last bar  xl(2)=450  yl(2)=450
Diameter of bars  da(2)=25

Bar set 3
Number of bars  na(3)=2

Co-ordinates of first bar  xf(3)=1050  yf(3)=50
Co-ordinates of last bar  xl(3)=1450  yl(3)=50
Diameter of bars  da(3)=25

Bar set 4
Number of bars  na(4)=2

Co-ordinates of first bar  xf(4)=1050  yf(4)=450
Co-ordinates of last bar  xl(4)=1450  yl(4)=450
Diameter of bars  da(4)=25

Bar set 5
Number of bars  na(5)=2

Co-ordinates of first bar  xf(5)=550  yf(5)=250
Co-ordinates of last bar  xl(5)=950  yl(5)=250
Diameter of bars  da(5)=16
Modular ratio  mr=15
Plot of Item Details
Plot of Section Details
Loading Case 1

Axial Load
P(1)=2000 kN
Axial load fixed in position.

x co-ordinate   xp(1)=750 mm
y co-ordinate   yp(1)=250 mm

Applied moments

About x axis   Amx(1)=-500 kNm
About y axis   Amy(1)=500 kNm
Plot of Cracked Section
Section is cracked

Neutral axis line

<table>
<thead>
<tr>
<th>Item</th>
<th>x</th>
<th>y</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>500</td>
<td>279.94</td>
<td>15.347</td>
</tr>
<tr>
<td></td>
<td>1500</td>
<td>500</td>
<td>22.803</td>
</tr>
<tr>
<td></td>
<td>1000</td>
<td>350</td>
<td>8.614</td>
</tr>
<tr>
<td></td>
<td>269.58</td>
<td>398.72</td>
<td>6.5655</td>
</tr>
<tr>
<td></td>
<td>1269.6</td>
<td>398.72</td>
<td>14.022</td>
</tr>
</tbody>
</table>
Reinforcement bar stresses. Tension negative, Compression positive.

<table>
<thead>
<tr>
<th>Bar No</th>
<th>Set</th>
<th>Co-ordinates</th>
<th>Diameter</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>50 50</td>
<td>25</td>
<td>-290.88</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>450 50</td>
<td>25</td>
<td>-246.14</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>50 450</td>
<td>25</td>
<td>127.57</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>450 450</td>
<td>25</td>
<td>172.31</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>1050 50</td>
<td>25</td>
<td>-179.04</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>1450 50</td>
<td>25</td>
<td>-134.3</td>
</tr>
<tr>
<td>7</td>
<td>4</td>
<td>1050 450</td>
<td>25</td>
<td>239.42</td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>1450 450</td>
<td>25</td>
<td>284.15</td>
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<tr>
<td>9</td>
<td>5</td>
<td>550 250</td>
<td>16</td>
<td>-25.732</td>
</tr>
<tr>
<td>10</td>
<td>5</td>
<td>950 250</td>
<td>16</td>
<td>19.005</td>
</tr>
</tbody>
</table>

Number of iterations 7
Location: Beam on grid line A3

Composite steel deck floor

Internal beam with U.D.Loading

Calculations in accordance with BS 5950 Part 3 Section 3.1 'Code of practice for design of simple and continuous composite beams'- July 1990 and with 'BS 5950-1: 2000' for construction stage design.

Deck ribs transverse to beam.

Beam span \( L_1 = 9 \) m
Beam centres (floor width carried) \( C = 3 \) m
Concrete cube strength \( f_{cu} = 25 \) N/mm\(^2\)
Air dry density for concrete \( D_c = 17.6 \) kN/m\(^3\)
Freshwet density for concrete \( D'_c = 18.8 \) kN/m\(^3\)
Deck spans onto supporting beam.

305 x 102 x 33 UB.
Young's Modulus \( E = 205 \) kN/mm\(^2\)
Slab depth \( D_s = 115 \) mm
Depth of profiling \( D_p = 50 \) mm
Centres of profiles \( C_t = 206 \) mm
Width at inside of profile \( b = 78 \) mm
Width at outside of profile \( p = 103 \) mm
Profiled steel deck \( d_{cwt} = 0.12 \) kN/m\(^2\)
Reinforcement allowance \( r_{e} = 0.04 \) kN/m\(^2\)
Steel beam allowance \( s_{def} = 0.15 \) kN/m\(^2\)
Construction load allowance \( w_c = 0.5 \) kN/m\(^2\)
Other UDL on beam \( w'd_u = 0 \) kN/m
Ceiling and services \( w_c = 0.25 \) kN/m\(^2\)
Imposed \( w_i = 3 \) kN/m\(^2\)
Partitions \( w_p = 1 \) kN/m\(^2\)
Wall or other UDL \( w_d = 0 \) kN/m
Number of studs across flange \( nr = 1 \)

Check on transverse reinforcement omitted.

<table>
<thead>
<tr>
<th>COMPOSITE</th>
<th>305 x 102 x 33 UB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>with 44 No. 19 dia. stud</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>connectors @ 204 centres.</td>
</tr>
</tbody>
</table>

Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements.

Construction stage moment 111.86 kNm
Steel beam mom. capacity 170.76 kNm
Const. stage deflection 39.603 mm
Composite stage moment 287.84 kNm
Ultimate mom. capacity 359.16 kNm
Vertical shear force 127.93 kN
Ult. shear capacity 439.59 kN
Composite stage deflection 23.101 mm
Location: Long span with solid slab

Composite steel deck floor

Internal beam with U.D.Loading

Calculations in accordance with BS 5950 Part 3 Section 3.1 'Code of practice for design of simple and continuous composite beams'- July 1990 and with 'BS 5950-1: 2000' for construction stage design.

Deck ribs transverse to beam.

Beam span L1=15 m
Beam centres(floor width carried) C=3 m
Concrete cube strength fcu=25 N/mm²
Air dry density for concrete Dc=17.6 kN/m³
Freshwet density for concrete D'c=18.8 kN/m³
914 x 305 x 224 UB.
Young's Modulus E=205 kN/mm²
Slab depth Ds=150 mm
Profiled steel deck decwt=0.12 kN/m²
Reinforcement allowance reinwt=0.04 kN/m²
Steel beam allowance selfwt=0.47 kN/m²
Construction load allowance wc=0.5 kN/m²
Other UDL on beam w'du=0 kN/m
Ceiling and services wcs=0.25 kN/m²
Imposed wi=3 kN/m²
Partitions wp=1 kN/m²
Wall or other UDL wdu=0 kN/m

Number of studs across flange nr=1

Check on transverse reinforcement omitted.
| **COMPOSITE**                          | 914 x 305 x 224 UB Grade S 355 |
| **SECTION**                           | with 120 No. 19 dia. stud       |
| **SUMMARY**                           | connectors @ 125 centres.       |

Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements.

- Construction stage moment: 474.92 kNm
- Steel beam moment capacity: 3291.3 kNm
- Const. stage deflection: 8.8488 mm
- Composite stage moment: 955.7 kNm
- Ultimate moment capacity: 4690.8 kNm
- Vertical shear force: 254.85 kN
- Ult. shear capacity: 2996.4 kN
- Composite stage deflection: 5.7455 mm
Location: Example plus fire resistance M.C. method

Composite steel deck floor

Internal beam with U.D.Loading

Calculations in accordance with BS 5950 Part 3 Section 3.1 'Code of practice for design of simple and continuous composite beams' - July 1990 and with 'BS 5950-1: 2000' for construction stage design.

Deck ribs transverse to beam.

Beam span \( L_1 = 9 \) m
Beam centres (floor width carried) \( C = 3 \) m
Concrete cube strength \( f_{cu} = 25 \) N/mm\(^2\)
Air dry density for concrete \( D_c = 17.6 \) kN/m\(^3\)
Fresh wet density for concrete \( D'c = 18.8 \) kN/m\(^3\)
406 x 140 x 39 UB.
Young's Modulus \( E = 205 \) kN/mm\(^2\)
Slab depth \( D_s = 115 \) mm
Profiled steel deck \( \text{decwt} = 0.12 \) kN/m\(^2\)
Reinforcement allowance \( \text{reinwt} = 0.04 \) kN/m\(^2\)
Steel beam allowance \( \text{selfwt} = 0.15 \) kN/m\(^2\)
Construction load allowance \( w_c = 0.5 \) kN/m\(^2\)
Other UDL on beam \( w'du = 0 \) kN/m
Ceiling and services \( wcs = 0.25 \) kN/m\(^2\)
Imposed \( w_i = 3 \) kN/m\(^2\)
Partitions \( wp = 1 \) kN/m\(^2\)
Wall or other UDL \( wdu = 0 \) kN/m
Number of studs across flange \( n_r = 1 \)
Spacing of connectors \( \text{spac1} = 225 \) mm
Check on transverse reinforcement omitted.
Total permanent load \( V_{fp} = 81 \) kN
| **COMPOSITE** | 406 x 140 x 39 UB Grade S 355 |
| **SECTION**   | with 40 No. 19 dia. stud        |
| **SUMMARY**   | connectors @ 225 centres.       |

Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements.

- Construction stage moment: 129.38 kNm
- Steel beam mom. capacity: 257.02 kNm
- Const. stage deflection: 24.714 mm
- Composite stage moment: 304.25 kNm
- Ultimate mom. capacity: 470.68 kNm
- Vertical shear force: 135.22 kN
- Ult. shear capacity: 542.55 kN
- Composite stage deflection: 15.796 mm

**FIRE**
- Exposed surface area: 1208.6 mm²

**RESISTANCE**
- Area of section: 49.7 cm²

**SUMMARY**
- Section factor: 240 /m
- Section does not have an inherent fire resistance of 30 minutes.
- Fire protection should be provided.
Location: Default example plus fire resistance

Composite steel deck floor

Internal beam with U.D.Loading

Calculations in accordance with BS 5950 Part 3 Section 3.1 'Code of practice for design of simple and continuous composite beams'—July 1990 and with 'BS 5950-1: 2000' for construction stage design.

Deck ribs transverse to beam.

Beam span \( L_1 = 9 \) m
Beam centres (floor width carried) \( C = 3 \) m
Concrete cube strength \( f_{cu} = 25 \) N/mm\(^2\)
Air dry density for concrete \( D_c = 17.6 \) kN/m\(^3\)
Freshwet density for concrete \( D'_c = 18.8 \) kN/m\(^3\)
Deck spans onto supporting beam. 305 x 102 x 33 UB.
Young's Modulus \( E = 205 \) kN/mm\(^2\)
Slab depth \( D_s = 115 \) mm
Depth of profiling \( D_p = 50 \) mm
Centres of profiles \( C_t = 300 \) mm
Width at inside of profile \( b = 125 \) mm
Width at outside of profile \( p = 175 \) mm
Profiled steel deck \( d_{ecwt} = 0.12 \) kN/m\(^2\)
Reinforcement allowance \( d_{reinwt} = 0.04 \) kN/m\(^2\)
Steel beam allowance \( d_{selfwt} = 0.15 \) kN/m\(^2\)
Construction load allowance \( w_c = 0.5 \) kN/m\(^2\)
Other UDL on beam \( w'_d u = 0 \) kN/m
Ceiling and services \( w_{cs} = 0.25 \) kN/m\(^2\)
Imposed \( w_i = 3 \) kN/m\(^2\)
Partitions \( w_p = 1 \) kN/m\(^2\)
Wall or other UDL \( w_{du} = 0 \) kN/m
Number of studs across flange \( n_r = 1 \)
Spacing of connectors \( s_{apc1} = 225 \text{ mm} \)

Check on transverse reinforcement omitted.

Total permanent load \( V_{fp} = 81 \text{ kN} \)

| COMPOSITE | 305 x 102 x 33 UB Grade S 355 with 40 No. 19 dia. stud connectors @ 225 centres. |
| SECTION    | Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements. |
| SUMMARY    | |  

| FIRE | Exposed surface area 919.4 \( \text{ mm}^2 \) |
| RESISTANCE | Area of section 41.8 \( \text{ cm}^2 \) |
| SUMMARY    | Section factor 215 /m |

Load Ratio (R) 0.51253
Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided
Location: Access steel example

Composite steel deck floor

Internal beam with U.D.Loading

Calculations in accordance with BS 5950 Part 3 Section 3.1 'Code of practice for design of simple and continuous composite beams'- July 1990 and with 'BS 5950-1: 2000' for construction stage design.

Deck ribs transverse to beam.

Beam span $L_1 = 7.5 \text{ m}$
Beam centres (floor width carried) $C = 3 \text{ m}$
Concrete cube strength $f_{cu} = 25 \text{ N/mm}^2$
Air dry density for concrete $D_c = 23.6 \text{ kN/m}^3$

WARNING:
For lightweight concrete grade of concrete should be C25 to C40
For normal concrete grade of concrete should be C30 to C50
**Location: Example 1 - Internal beam with UDL**

Beam span: \( L_1 = 9 \, \text{m} \)
Beam centres (floor width carried): \( C = 3 \, \text{m} \)
Concrete density (const. stage): \( D'_c = 19 \, \text{kN/m}^3 \)
Concrete density (final stage): \( D_c = 18 \, \text{kN/m}^3 \)
Deck spans onto supporting beam and restraints compression flange.

356 x 171 x 51 UKB
Dimensions (mm): \( h = 355 \, \text{b} = 171.5 \, \text{tw} = 7.4 \, \text{tf} = 11.5 \, \text{r} = 10.2 \)
Properties (cm): \( I_y = 14100 \, I_z = 968 \, \text{W}_{ply} = 896 \, \text{W}_{plz} = 174 \, I_t = 23.8 \)
Partial safety factor (reinfnt.): \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete): \( \gamma_{mc} = 1.5 \)
Partial safety factor (steel): \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel): \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs): \( \gamma_v = 1.25 \)
Permanent load (ULS): \( \gamma_G = 1.25 \)
Variable load (ULS): \( \gamma_Q = 1.5 \)

Depth of concrete above profile: \( h_f = 80 \, \text{mm} \)
Depth of profiling: \( h_p = 50 \, \text{mm} \)
Centres of profiles: \( e_p = 206 \, \text{mm} \)
Width at inside of profile: \( b_1 = 78 \, \text{mm} \)
Width at outside of profile: \( b_2 = 103 \, \text{mm} \)
Thickness of sheeting: \( t_p = 0.75 \, \text{mm} \)
Mean width of concrete ribs: \( b_0 = 90.5 \, \text{mm} \)
Weight of steel decking: \( f_{sd} = 0.12 \, \text{kN/m}^2 \)
Variable load (const. stage): \( f_c = 0.75 \, \text{kN/m}^2 \)
Variable load (final stage): \( f_i = 3 \, \text{kN/m}^2 \)
Partition load \( f_p = 1 \) kN/m²
Ceiling and services \( f_c = 0.25 \) kN/m²
Raised floor \( f_r = 0 \) kN/m²
Wall or other UDL (final stage) \( f_w = 0 \) kN/m
Char yield strength of reinft. \( f_y = 500 \) N/mm²

| DESIGN SUMMARY | 356 x 171 x 51 UKB Grade S 355 |
| CONSTRUCTION | Steel yield strength 355 N/mm² |
| STAGE | Concrete strength 25 N/mm² |
| | Maximum moment 132 kNm |
| | Moment resistance 318 kNm |
| | Unity factor 0.414 ≤ 1 |
| | Maximum shear force 58.5 kN |
| | Shear resistance 587 kN |
| | Unity factor 0.1 ≤ 1 |
| Deflection to span ratio \((G + Q)\) 1: 313 |

| DESIGN SUMMARY | Effective width 2.25 m |
| FINAL STAGE | Stud shear resistance 61.7 kN |
| | Number of studs per rib 1 No. |
| | Number of 19 mm studs 43 No. |
| | Stud spacing 206 mm |
| Design compressive force: | Concrete 2304 kN |
| | Studs 1327 kN |
| | Shear connection ratio 0.576 |
| | Partial shear connection provided |
| | Maximum moment 289 kNm |
| | Moment resistance 519 kNm |
| | Unity factor bending 0.557 ≤ 1 |
| | Maximum shear force 129 kN |
| | Shear plastic resistance 587 kN |
| | Unity factor shear 0.219 ≤ 1 |
| Deflection to span ratio at final stage \((G + Q)\) 1: 268 |
| Slab shear stress 1.84 N/mm² |
| Slab resistance 2.51 N/mm² |
| Min transverse reinft. 169 mm²/m |
| Lightweight concrete (LWC) assumed |
Location: Example 2 - Solid slab

Beam span \( L_1 = 15 \text{ m} \)
Beam centres (floor width carried) \( C = 3 \text{ m} \)
Concrete density (const. stage) \( D_c' = 18.8 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 17.6 \text{ kN/m}^3 \)
914 x 305 x 224 UKB
Dimensions (mm): \( h = 910.4 \text{ mm}, b = 304.1 \text{ mm}, tw = 15.9 \text{ mm}, tf = 23.9 \text{ mm}, r = 19.1 \text{ mm} \)
Properties (cm): \( I_y = 376000 \text{ cm}^4, I_z = 11200 \text{ cm}^4, W_{p y} = 9540 \text{ cm}, W_{p z} = 1160 \text{ cm}, I_t = 422 \text{ cm}^4 \)
Partial safety factor (reinfnt.) \( \gamma_{m0} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{m0} = 1.0 \)
Partial safety factor (studs) \( \gamma_{mv} = 1.25 \)
Permanent load (ULS) \( \gamma_{PG} = 1.25 \)
Variable load (ULS) \( \gamma_{QG} = 1.5 \)
Variable load (const. stage) \( f_c = 0.5 \text{ kN/m}^2 \)
Variable load (final stage) \( f_i = 3 \text{ kN/m}^2 \)
Partition load \( f_p = 1 \text{ kN/m}^2 \)
Ceiling and services \( f_{cs} = 0.25 \text{ kN/m}^2 \)
Raised floor \( f_r = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( f_w = 0 \text{ kN/m} \)
Stud spacing \( \text{spac} = 200 \text{ mm} \)
Governing shear length \( h_f = 150 \text{ mm} \)
Char yield strength of reinft. \( f_y = 500 \text{ N/mm}^2 \)
DESIGN SUMMARY

914 x 305 x 224 UKB Grade S 355

CONSTRUCTION

Steel yield strength  345 N/mm²
Concrete strength    25 N/mm²
Maximum moment       474 kNm
Moment resistance    3291 kNm
Unity factor         0.144 ≤ 1
Maximum shear force  126 kN
Shear resistance     3059 kN
Unity factor         0.041 ≤ 1

Deflection to span ratio
(G + Q)              1: 1361

FINAL STAGE

Effective width       3 m
Stud shear resistance 65.8 kN
Studs across flange  in pairs
Number of 19 mm studs 150 No.
Stud spacing          200 mm

Design compressive force:
Concrete              6375 kN
Studs                 4931 kN
Shear connection ratio 0.774
Partial shear connection provided

Maximum moment       912 kNm
Moment resistance    4751 kNm
Unity factor bending 0.192 ≤ 1
Maximum shear force  243 kN
Shear plastic resistance 3059 kN
Unity factor shear   0.08 ≤ 1

Deflection to span ratio
at final stage (G + Q) 1: 1001

Slab shear stress 2.19 N/mm²
Slab resistance    2.37 N/mm²
Min transverse reinft. 377 mm²/m

Lightweight concrete (LWC) assumed
Location: Example 3 - Solid slab

Beam span L1=9 m
Beam centres (floor width carried) C=3 m
Concrete density (const.stage) D'c=18.8 kN/m³
Concrete density (final stage) Dc=17.6 kN/m³
406 x 140 x 39 UKB
Dimensions (mm): h=398 b=141.8 tw=6.4 tf=8.6 r=10.2
Properties (cm): Iy=12500 Iz=410 Wply=724 Wplz=90.8 It=10.7
Partial safety factor (reinfnt.) gams=1.15
Partial safety factor (concrete) gamc=1.5
Partial safety factor (steel) gamM0=1.0
Partial safety factor (studs) gamv=1.25
Permanent load (ULS) gamG=1.25
Variable load (ULS) gamQ=1.5
Variable load (const.stage) fc=0.5 kN/m²
Variable load (final stage) fi=3 kN/m²
Partition load fp=1 kN/m²
Ceiling and services fcs=0.25 kN/m²
Raised floor fr=0 kN/m²
Wall or other UDL (final stage) fw=0 kN/m
Stud spacing spac=300 mm
Governing shear length hf=114.25 mm
Char yield strength of reinfnt. fyk=500 N/mm²
**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>406 x 140 x 39 UKB Grade S 355</td>
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**CONSTRUCTION**

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel yield strength</td>
<td>355 N/mm²</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>25 N/mm²</td>
</tr>
<tr>
<td>Maximum moment</td>
<td>120 kNm</td>
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<tr>
<td>Moment resistance</td>
<td>257 kNm</td>
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<tr>
<td>Unity factor</td>
<td>0.466 ≤ 1</td>
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<tr>
<td>Buckling resistance</td>
<td>148 kNm</td>
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<tr>
<td>Unity factor</td>
<td>0.809 ≤ 1</td>
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<tr>
<td>Maximum shear force</td>
<td>53.2 kN</td>
</tr>
<tr>
<td>Shear resistance</td>
<td>566 kN</td>
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<tr>
<td>Unity factor</td>
<td>0.094 ≤ 1</td>
</tr>
</tbody>
</table>

**Final Stage**

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stud shear resistance</td>
<td>65.8 kN</td>
</tr>
<tr>
<td>Number of 19 mm studs</td>
<td>30 No.</td>
</tr>
<tr>
<td>Stud spacing</td>
<td>300 mm</td>
</tr>
</tbody>
</table>

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective width</td>
<td>2.25 m</td>
</tr>
</tbody>
</table>

**FINAL STAGE**

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design compressive force: Concrete</td>
<td>1764 kN</td>
</tr>
<tr>
<td>Studs</td>
<td>986 kN</td>
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<tr>
<td>Shear connection ratio</td>
<td>0.559</td>
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<tr>
<td>Partial shear connection provided</td>
<td></td>
</tr>
<tr>
<td>Maximum moment</td>
<td>280 kNm</td>
</tr>
<tr>
<td>Unity factor bending</td>
<td>0.703 ≤ 1</td>
</tr>
<tr>
<td>Maximum shear force</td>
<td>124 kN</td>
</tr>
<tr>
<td>Shear plastic resistance</td>
<td>566 kN</td>
</tr>
<tr>
<td>Unity factor shear</td>
<td>0.22 ≤ 1</td>
</tr>
</tbody>
</table>

**Deflection to span ratio**

- at final stage (G + Q) | 1: 234 |
- Maximum shear force | 124 kN |
- Shear plastic resistance | 566 kN |
- Unity factor shear | 0.22 ≤ 1 |

**Min transverse reinft.**

- Slab shear stress | 0.959 N/mm² |
- Slab resistance | 2.37 N/mm² |
- Lightweight concrete (LWC) assumed
Location: Example 4

Beam span \( L_1 = 9 \) m
Beam centres (floor width carried) \( C = 3 \) m
Concrete density (const. stage) \( D_c' = 18.8 \) kN/m\(^3\)
Concrete density (final stage) \( D_c = 17.6 \) kN/m\(^3\)
Deck spans onto supporting beam and restraints compression flange.
356 x 127 x 39 UKB
Dimensions (mm): \( h = 353.4 \), \( b = 126 \), \( t_w = 10.7 \), \( r = 10.2 \)
Properties (cm): \( I_y = 10200 \), \( I_z = 358 \), \( W_{p1y} = 659 \), \( W_{p1z} = 89.1 \), \( I_t = 15.1 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{m} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{v} = 1.25 \)
Permanent load (ULS) \( \gamma_{G} = 1.25 \)
Variable load (ULS) \( \gamma_{Q} = 1.5 \)

Depth of concrete above profile \( h_f = 80 \) mm
Depth of profiling \( h_p = 50 \) mm
Centres of profiles \( e_p = 300 \) mm
Width at inside of profile \( b_1 = 125 \) mm
Width at outside of profile \( b_2 = 175 \) mm
Thickness of sheeting \( t_p = 1.2 \) mm
Mean width of concrete ribs \( b_0 = 150 \) mm
Weight of steel decking \( f_{sd} = 0.12 \) kN/m\(^2\)
Variable load (const. stage) \( f_c = 0.5 \) kN/m\(^2\)
Variable load (final stage) \( f_i = 3 \) kN/m\(^2\)
### Partition load
- $f_p = 1 \, \text{kN/m}^2$

### Ceiling and services
- $f_{cs} = 0.25 \, \text{kN/m}^2$

### Raised floor
- $f_r = 0 \, \text{kN/m}^2$

### Wall or other UDL (final stage)
- $f_w = 0 \, \text{kN/m}$

### Char yield strength of reinft.
- $f_yk = 500 \, \text{N/mm}^2$

#### DESIGN SUMMARY
- Effective width: $2.25 \, \text{m}$
- Stud shear resistance: $65.8 \, \text{kN}$
- Number of studs per rib: 1 No.
- Number of 19 mm studs: 30 No.
- Stud spacing: $300 \, \text{mm}$

#### CONSTRUCTION
- STEEL YIELD STRENGTH: $355 \, \text{N/mm}^2$
- CONCRETE STRENGTH: $25 \, \text{N/mm}^2$
- Maximum moment: $116 \, \text{kNm}$
- Moment resistance: $234 \, \text{kNm}$
- Unity factor: $0.497 \leq 1$
- Maximum shear force: $51.7 \, \text{kN}$
- Shear resistance: $527 \, \text{kN}$
- Unity factor: $0.098 \leq 1$
- Deflection to span ratio: $1:253$

#### DESIGN SUMMARY
- Maximum moment: $277 \, \text{kNm}$
- Moment resistance: $392 \, \text{kNm}$
- Unity factor bending: $0.707 \leq 1$
- Maximum shear force: $123 \, \text{kN}$
- Shear plastic resistance: $527 \, \text{kN}$
- Unity factor shear: $0.234 \leq 1$
- Deflection to span ratio at final stage: $1:202$

#### FINAL STAGE
- Slab shear stress: $1.37 \, \text{N/mm}^2$
- Slab resistance: $2.37 \, \text{N/mm}^2$
- Min transverse reinft.: $160 \, \text{mm}^2/\text{m}$
- Lightweight concrete (LWC) assumed
Location: Example 5

Beam span \( L_1 = 7.5 \text{ m} \)
Beam centres (floor width carried) \( C = 3 \text{ m} \)
Concrete density (const. stage) \( D'_c = 19 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 18 \text{ kN/m}^3 \)
Deck spans onto supporting beam and restraints compression flange.
254 x 146 x 37 UKB
Dimensions (mm): \( h = 256 \text{ b} = 146.4 \text{ tw} = 6.3 \text{ tf} = 10.9 \text{ r} = 7.6 \)
Properties (cm): \( I_y = 5540 \text{ I}_z = 571 \text{ W}_{ply} = 483 \text{ W}_{plz} = 119 \text{ It} = 15.3 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_v = 1.25 \)
Permanent load (ULS) \( \gamma_G = 1.25 \)
Variable load (ULS) \( \gamma_Q = 1.5 \)

Depth of concrete above profile \( hf = 72 \text{ mm} \)
Depth of profiling \( hp = 58 \text{ mm} \)
Centres of profiles \( ep = 207 \text{ mm} \)
Width at inside of profile \( b_1 = 62 \text{ mm} \)
Width at outside of profile \( b_2 = 101 \text{ mm} \)
Thickness of sheeting \( tp = 0.75 \text{ mm} \)
Mean width of concrete ribs \( b_0 = 81.5 \text{ mm} \)
Weight of steel decking \( fsd = 0.12 \text{ kN/m}^2 \)
Variable load (const. stage) \( fc = 0.75 \text{ kN/m}^2 \)
Variable load (final stage) \( fi = 2.5 \text{ kN/m}^2 \)
Partition load \( fp = 0.75 \text{ kN/m}^2 \)
Ceiling and services \( f_{c,s} = 0 \text{ kN/m}^2 \)
Raised floor \( fr = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( fw = 0 \text{ kN/m} \)
Char yield strength of reinft. \( fyk = 500 \text{ N/mm}^2 \)

**DESIGN SUMMARY**

- **CONSTRUCTION**
  - Steel yield strength \( 355 \text{ N/mm}^2 \)
  - Concrete strength \( 25 \text{ N/mm}^2 \)
  - Maximum moment \( 89.7 \text{ kNm} \)
  - Moment resistance \( 171 \text{ kNm} \)
  - Unity factor \( 0.523 \leq 1 \)
  - Maximum shear force \( 47.8 \text{ kN} \)
  - Shear resistance \( 361 \text{ kN} \)
  - Unity factor \( 0.132 \leq 1 \)

- **FINAL STAGE**
  - Effective width \( 1.88 \text{ m} \)
  - Stud shear resistance \( 51.7 \text{ kN} \)
  - Number of studs per rib \( 1 \text{ No.} \)
  - Number of 19 mm studs \( 36 \text{ No.} \)
  - Stud spacing \( 207 \text{ mm} \)
  - Design compressive force:
    - Concrete \( 1676 \text{ kN} \)
    - Studs \( 931 \text{ kN} \)
    - Partial shear connection ratio \( 0.556 \)
    - Maximum moment \( 169 \text{ kNm} \)
    - Moment resistance \( 297 \text{ kNm} \)
    - Unity factor bending \( 0.568 \leq 1 \)
    - Maximum shear force \( 90 \text{ kN} \)
    - Shear plastic resistance \( 361 \text{ kN} \)
    - Unity factor shear \( 0.249 \leq 1 \)
  - Deflection to span ratio at final stage \( (G + Q) : 211 \)
  - Slab shear stress \( 1.72 \text{ N/mm}^2 \)
  - Slab resistance \( 2.51 \text{ N/mm}^2 \)
  - Min transverse reinft. \( 144 \text{ mm}^2/\text{m} \)
  - Lightweight concrete (LWC) assumed
Location: Example 6 - from SCI design guide P364, page 76

Beam span \( L_1 = 12 \text{ m} \)
Beam centres (floor width carried) \( C = 3.5 \text{ m} \)
Concrete density (const. stage) \( D'_c = 25 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 24 \text{ kN/m}^3 \)
Deck spans onto supporting beam and restraints compression flange.

533 x 165 x 75 UKB

Dimensions (mm): \( h = 529.1 \text{ b = 165.9 tw = 9.7 tf = 13.6 r = 12.7} \)

Properties (cm): \( I_y = 41100 \text{ I_z = 1040 W_ply = 1810 W_plz = 200 It = 47.9} \)

Partial safety factor (reinfnt.) \( \gamma_{ms} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_v = 1.25 \)
Permanent load (ULS) \( \gamma_G = 1.25 \)
Variable load (ULS) \( \gamma_Q = 1.5 \)

Depth of concrete above profile \( h_f = 70 \text{ mm} \)
Depth of profiling \( h_p = 60 \text{ mm} \)
Centres of profiles \( e_p = 300 \text{ mm} \)
Width at inside of profile \( b_1 = 130 \text{ mm} \)
Width at outside of profile \( b_2 = 170 \text{ mm} \)
Thickness of sheeting \( t_p = 0.9 \text{ mm} \)
Mean width of concrete ribs \( b_0 = 150 \text{ mm} \)
Weight of steel decking \( f_s = 0.1 \text{ kN/m}^2 \)
Variable load (const. stage) \( f_c = 0.75 \text{ kN/m}^2 \)
Variable load (final stage) \( f_i = 2.5 \text{ kN/m}^2 \)
<table>
<thead>
<tr>
<th>Partition load</th>
<th>( fp = 0.8 \text{ kN/m}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ceiling and services</td>
<td>( f_{cs} = 0.5 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>Raised floor</td>
<td>( fr = 0 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>Wall or other UDL (final stage)</td>
<td>( fw = 0.264 \text{ kN/m} )</td>
</tr>
<tr>
<td>Char yield strength of reinft.</td>
<td>( fyk = 500 \text{ N/mm}^2 )</td>
</tr>
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<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>533 x 165 x 75 UKB Grade S 275</th>
</tr>
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<tbody>
<tr>
<td>CONSTRUCTION</td>
<td>Steel yield strength 275 N/mm²</td>
</tr>
<tr>
<td>STAGE</td>
<td>Concrete strength 25 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum moment 292 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment resistance 498 kNm</td>
</tr>
<tr>
<td></td>
<td>Unity factor 0.587 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>Maximum shear force 97.4 kN</td>
</tr>
<tr>
<td></td>
<td>Shear resistance 871 kN</td>
</tr>
<tr>
<td></td>
<td>Unity factor 0.112 ≤ 1</td>
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<tr>
<td></td>
<td>Deflection to span ratio ((G + Q)) 1: 307</td>
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</table>

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<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Effective width 3.08 m</th>
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</thead>
<tbody>
<tr>
<td>FINAL STAGE</td>
<td>Stud shear resistance 46.8 kN</td>
</tr>
<tr>
<td></td>
<td>Number of studs per rib 2 No.</td>
</tr>
<tr>
<td></td>
<td>Number of 19 mm studs 80 No.</td>
</tr>
<tr>
<td></td>
<td>Stud spacing 300 mm</td>
</tr>
<tr>
<td></td>
<td>Design compressive force: Concrete 2618 kN</td>
</tr>
<tr>
<td></td>
<td>Studs 1872 kN</td>
</tr>
<tr>
<td></td>
<td>Shear connection ratio 0.715</td>
</tr>
<tr>
<td></td>
<td>Partial shear connection provided</td>
</tr>
<tr>
<td></td>
<td>Maximum moment 571 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment resistance 858 kNm</td>
</tr>
<tr>
<td></td>
<td>Unity factor bending 0.665 ≤ 1</td>
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<tr>
<td></td>
<td>Maximum shear force 190 kN</td>
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<tr>
<td></td>
<td>Shear plastic resistance 871 kN</td>
</tr>
<tr>
<td></td>
<td>Unity factor shear 0.218 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>Deflection to span ratio ((G + Q)) 1: 264</td>
</tr>
<tr>
<td></td>
<td>Slab shear stress 2.23 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Slab resistance 2.72 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Min transverse reinft. 179 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Normal weight concrete (NWC) assumed</td>
</tr>
</tbody>
</table>

---

SCALE 5.48
Office 1007
Proforma 210
Location: Example 7, Taken from RC Design to EC2 by B Mosley p375

Beam span \( \ell_1 = 6 \, \text{m} \)
Beam centres (floor width carried) \( C = 3 \, \text{m} \)
Concrete density (const. stage) \( D'_c = 25 \, \text{kN/m}^3 \)
Concrete density (final stage) \( D_c = 24 \, \text{kN/m}^3 \)
Deck spans onto supporting beam and restraints compression flange.

457 x 191 x 74 UKB

Dimensions (mm): \( h=457 \, \text{b}=190.4 \, \text{tw}=9 \, \text{tf}=14.5 \, r=10.2 \)
Properties (cm): \( I_y=33300 \, I_z=1670 \, W_{p|y}=1650 \, W_{p|z}=272 \, I_t=51.8 \)
Partial safety factor (reinforc.) \( \gamma_{ams}=1.15 \)
Partial safety factor (concrete) \( \gamma_{mc}=1.5 \)
Partial safety factor (steel) \( \gamma_{M0}=1.0 \)
Partial safety factor (steel) \( \gamma_{M1}=1.0 \)
Partial safety factor (studs) \( \gamma_{v}=1.25 \)
Permanent load (ULS) \( \gamma_{G} = 1.35 \)
Variable load (ULS) \( \gamma_{Q} = 1.5 \)

\[ 457 \times 191 \times 74 \, \text{UKB} \]

Dimensions (mm): \( h=457 \, b=190.4 \, tw=9 \, tf=14.5 \, r=10.2 \)
Properties (cm): \( I_{y}=33300 \, I_{z}=1670 \, W_{p|y}=1650 \, W_{p|z}=272 \, I_{t}=51.8 \)
Partial safety factor (reinforc.) \( \gamma_{ams}=1.15 \)
Partial safety factor (concrete) \( \gamma_{mc}=1.5 \)
Partial safety factor (steel) \( \gamma_{M0}=1.0 \)
Partial safety factor (steel) \( \gamma_{M1}=1.0 \)
Partial safety factor (studs) \( \gamma_{v}=1.25 \)
Permanent load (ULS) \( \gamma_{G} = 1.35 \)
Variable load (ULS) \( \gamma_{Q} = 1.5 \)

\[ 457 \times 191 \times 74 \, \text{UKB} \]
Partition load \[ fp = 1 \text{kN/m}^2 \]
Ceiling and services \[ fcs = 0 \text{kN/m}^2 \]
Raised floor \[ fr = 0 \text{kN/m}^2 \]
Wall or other UDL (final stage) \[ fw = 0 \text{kN/m} \]
Char yield strength of reinft. \[ fyk = 500 \text{N/mm}^2 \]

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>457 x 191 x 74 UKB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONSTRUCTION</td>
<td>Steel yield strength 355 N/mm²</td>
</tr>
<tr>
<td>STAGE</td>
<td>Concrete strength 25 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Maximum moment 74.7 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment resistance 586 kNm</td>
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<tr>
<td></td>
<td>Unity factor 0.128 ≤ 1</td>
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<tr>
<td></td>
<td>Maximum shear force 49.8 kN</td>
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<tr>
<td></td>
<td>Shear resistance 895 kN</td>
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<tr>
<td></td>
<td>Unity factor 0.056 ≤ 1</td>
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<tr>
<td></td>
<td>Deflection to span ratio (G + Q) 1: 2063</td>
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</table>

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Effective width 1.58 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>FINAL STAGE</td>
<td>Stud shear resistance 46.8 kN</td>
</tr>
<tr>
<td></td>
<td>Number of studs per rib 2 No.</td>
</tr>
<tr>
<td></td>
<td>Number of 19 mm studs 40 No.</td>
</tr>
<tr>
<td></td>
<td>Stud spacing 300 mm</td>
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<tr>
<td></td>
<td>Design compressive force:</td>
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<tr>
<td>Concrete</td>
<td>1913 kN</td>
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<tr>
<td>Studs</td>
<td>936 kN</td>
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<tr>
<td>Shear connection ratio</td>
<td>0.489</td>
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<td>Partial shear connection provided</td>
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</tr>
<tr>
<td>Maximum moment</td>
<td>179 kNm</td>
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<tr>
<td>Moment resistance</td>
<td>821 kNm</td>
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<tr>
<td>Unity factor bending</td>
<td>0.218 ≤ 1</td>
</tr>
<tr>
<td>Maximum shear force</td>
<td>119 kN</td>
</tr>
<tr>
<td>Shear plastic resistance</td>
<td>895 kN</td>
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<tr>
<td>Unity factor shear</td>
<td>0.133 ≤ 1</td>
</tr>
<tr>
<td>Deflection to span ratio at final stage (G + Q)</td>
<td>1: 1485</td>
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<tr>
<td>Slab shear stress</td>
<td>1.73 N/mm²</td>
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<tr>
<td>Slab resistance</td>
<td>2.72 N/mm²</td>
</tr>
<tr>
<td>Min transverse reinft.</td>
<td>180 mm²/m</td>
</tr>
<tr>
<td>Normal weight concrete (NWC) assumed</td>
<td></td>
</tr>
</tbody>
</table>
Location: Example 8 - from SCI design guide P359, page 95

Beam span $L_1=9$ m  
Beam centres (floor width carried) $C=3$ m  
Concrete density (const. stage) $D'_c=25$ kN/m$^3$  
Concrete density (final stage) $D_c=24$ kN/m$^3$  
Deck spans onto supporting beam and restraints compression flange.

406 x 140 x 46 UKB

Dimensions (mm): $h=403.2$ b=142.2 $tw=6.8$ tf=11.2 $r=10.2$

Properties (cm): $I_y=15700$ $I_z=538$ $W_{ply}=888$ $W_{plz}=118$ $I_t=19$

Partial safety factor (reinfnt.) $\gamma_{ams}=1.15$

Partial safety factor (concrete) $\gamma_{amc}=1.5$

Partial safety factor (steel) $\gamma_{amM0}=1.0$

Partial safety factor (studs) $\gamma_{amv}=1.25$

Permanent load (ULS) $\gamma_G=1.25$

Variable load (ULS) $\gamma_Q=1.5$

Depth of concrete above profile $hf=70$ mm  
Depth of profiling $hp=60$ mm  
Centres of profiles $ep=300$ mm  
Width at inside of profile $b_1=120$ mm  
Width at outside of profile $b_2=180$ mm  
Thickness of sheeting $tp=1$ mm  
Mean width of concrete ribs $b_0=150$ mm  
Weight of steel decking $f_{sd}=0.1$ kN/m$^2$  
Variable load (const. stage) $f_c=0.75$ kN/m$^2$  
Variable load (final stage) $f_i=4$ kN/m$^2$
<table>
<thead>
<tr>
<th>Partition load</th>
<th>fp=0.8 kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ceiling and services</td>
<td>fcs=0.5 kN/m²</td>
</tr>
<tr>
<td>Raised floor</td>
<td>fr=0.35 kN/m²</td>
</tr>
<tr>
<td>Wall or other UDL (final stage)</td>
<td>fw=0.54 kN/m</td>
</tr>
<tr>
<td>Char yield strength of reinft.</td>
<td>fyk=500 N/mm²</td>
</tr>
</tbody>
</table>

**DESIGN SUMMARY**

406 x 140 x 46 UKB Grade S 275

**CONSTRUCTION**

Steel yield strength 275 N/mm²
Concrete strength 25 N/mm²
Maximum moment 158 kNm
Moment resistance 244 kNm
Unity factor 0.648 ≤ 1
Maximum shear force 70.4 kN
Shear resistance 473 kN
Unity factor 0.149 ≤ 1
Deflection to span ratio
(G + Q) 1: 330

**DESIGN SUMMARY**

Effective width 2.25 m
Stud shear resistance 63.1 kN
Number of studs per rib 1 No.
Number of 19 mm studs 30 No.
Stud spacing 300 mm

**FINAL STAGE**

Design compressive force:
Concrete 1612 kN
Studs 947 kN
Shear connection ratio 0.588
Partial shear connection provided
Maximum moment 358 kNm
Moment resistance 408 kNm
Unity factor bending 0.88 ≤ 1
Maximum shear force 159 kN
Shear plastic resistance 473 kN
Unity factor shear 0.337 ≤ 1
Deflection to span ratio
at final stage (G + Q) 1: 254
Slab shear stress 1.5 N/mm²
Slab resistance 2.72 N/mm²
Min transverse reinft. 140 mm²/m
Normal weight concrete (NWC) assumed
Location: Example 9 - As Example 1 but using SCI P405 rules

Beam span: L1=9 m
Beam centres (floor width carried): C=3 m
Concrete density (const. stage): D'c=19 kN/m³
Concrete density (final stage): Dc=18 kN/m³
Deck spans onto supporting beam and restraints compression flange.

Dimensions (mm): h=355 b=171.5 tw=7.4 tf=11.5 r=10.2
Properties (cm): Iy=14100 Iz=968 Wply=896 Wplz=174 It=23.8
Partial safety factor (reinft.): gamS=1.15
Partial safety factor (concrete): gamC=1.5
Partial safety factor (steel): gamM0=1.0
Partial safety factor (studs): gamV=1.25
Permanent load (ULS): gamG=1.25
Variable load (ULS): gamQ=1.5

Depth of concrete above profile: hf=80 mm
Depth of profiling: hp=50 mm
Centres of profiles: ep=206 mm
Width at inside of profile: b1=78 mm
Width at outside of profile: b2=103 mm
Thickness of sheeting: tp=0.75 mm
Mean width of concrete ribs: b0=90.5 mm
Weight of steel decking: fsd=0.12 kN/m²
Variable load (const. stage): fc=0.75 kN/m²
Variable load (final stage): fi=3 kN/m²
Partition load \( f_p = 1 \text{ kN/m}^2 \)
Ceiling and services \( f_{cs} = 0.25 \text{ kN/m}^2 \)
Raised floor \( f_r = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( f_w = 0 \text{ kN/m} \)
Char yield strength of reinft. \( f_yk = 500 \text{ N/mm}^2 \)

**DESIGN SUMMARY**

- **CONSTRUCTION**
  - Steel yield strength \( 355 \text{ N/mm}^2 \)
  - Concrete strength \( 25 \text{ N/mm}^2 \)
  - Maximum moment \( 132 \text{ kNm} \)
  - Moment resistance \( 318 \text{ kNm} \)
  - Unity factor \( 0.414 \leq 1 \)
  - Maximum shear force \( 58.5 \text{ kN} \)
  - Shear resistance \( 587 \text{ kN} \)
  - Unity factor \( 0.1 \leq 1 \)
  - Deflection to span ratio \( (G + Q) \)

- **STAGE**
  - Effective width \( 2.25 \text{ m} \)
  - Stud shear resistance \( 61.7 \text{ kN} \)
  - Number of studs per rib \( 43 \text{ No.} \)
  - Number of 19 mm studs \( 43 \text{ No.} \)
  - Stud spacing \( 206 \text{ mm} \)

**DESIGN SUMMARY**

- **FINAL STAGE**
  - Design compressive force:
    - Concrete \( 2304 \text{ kN} \)
    - Studs \( 1327 \text{ kN} \)
  - Partial shear connection ratio \( 0.576 \)
  - Maximum moment \( 289 \text{ kNm} \)
  - Moment resistance \( 519 \text{ kNm} \)
  - Unity factor bending \( 0.557 \leq 1 \)
  - Maximum shear force \( 129 \text{ kN} \)
  - Shear plastic resistance \( 587 \text{ kN} \)
  - Unity factor shear \( 0.219 \leq 1 \)
  - Deflection to span ratio at final stage \( (G + Q) \)

  - Slab shear stress \( 1.84 \text{ N/mm}^2 \)
  - Slab resistance \( 2.51 \text{ N/mm}^2 \)
  - Min transverse reinft. \( 169 \text{ mm}^2 / \text{m} \)
  - Lightweight concrete (LWC) assumed
Location: Beam on grid line A3

Composite steel deck floor

Edge beam with U.D. Loading

Calculations in accordance with BS 5950 Part 3 Section 3.1 'Code of practice for design of simple and continuous composite beams' - July 1990 and with BS 5950-1: 2000 for construction stage design.

Deck ribs transverse to beam.

Beam span \( L_1 = 9 \) m
Beam centres \( C = 3 \) m
Concrete cube strength \( f_{cu} = 25 \) N/mm\(^2\)
Air dry density for concrete \( D_c = 17.6 \) kN/m\(^3\)
Freshwet density for concrete \( D'_c = 18.8 \) kN/m\(^3\)
Deck spans onto supporting beam.
305 x 102 x 33 UB.
Young's Modulus \( E = 205 \) kN/mm\(^2\)
Slab depth \( D_s = 115 \) mm
Depth of profiling \( D_p = 50 \) mm
Centres of profiles \( C_t = 300 \) mm
Width at inside of profile \( b = 125 \) mm
Width at outside of profile \( p = 175 \) mm
Profiled steel deck \( \text{decwt} = 0.12 \) kN/m\(^2\)
Reinforcement allowance \( \text{reinwt} = 0.04 \) kN/m\(^2\)
Steel beam allowance \( \text{selfwt} = 0.15 \) kN/m\(^2\)
Construction load allowance \( w_c = 0.5 \) kN/m\(^2\)
Other UDL on beam \( w_d' = 0 \) kN/m
Ceiling and services \( w_{cs} = 0.25 \) kN/m\(^2\)
Imposed \( w_i = 3 \) kN/m\(^2\)
Partitions \( w_p = 1 \) kN/m\(^2\)
Wall or other UDL \( w_d = 0 \) kN/m
Number of studs across flange \( nr = 1 \)
Spacing of connectors \( \text{spac1} = 400 \text{ mm} \)

Check on transverse reinforcement omitted.

| COMPOSITE | 305 x 102 x 33 UB Grade S 355 |
| SECTION   | with 24 No. 19 dia. stud connectors @ 375 centres. |
| SUMMARY   | Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements. |
| Construction stage moment | 54.718 kNm |
| Steel beam mom. capacity | 170.76 kNm |
| Const. stage deflection | 19.253 mm |
| Composite stage moment | 142.79 kNm |
| Ultimate mom. capacity | 297.92 kNm |
| Vertical shear force | 63.461 kN |
| Ult. shear capacity | 439.59 kN |
| Composite stage deflection | 14.665 mm |
Location: long span with solid slab

Composite steel deck floor

Edge beam with U.D>Loading

Calculations in accordance with BS 5950 Part 3 Section 3.1 'Code of practice for design of simple and continuous composite beams'- July 1990 and with BS 5950-1: 2000 for construction stage design.

Deck ribs transverse to beam.

Beam span
Beam centres
Concrete cube strength
Air dry density for concrete
Fresh wet density for concrete
610 x 305 x 149 UB.
Young's Modulus
Slab depth
Profiled steel deck
Reinforcement allowance
Steel beam allowance
Construction load allowance
Other UDL on beam
Ceiling and services
Imposed
Partitions
Wall or other UDL
Number of studs across flange
Percentage to be provided

Check on transverse reinforcement omitted.
COMPOSITE

SECTION

SUMMARY

610 x 305 x 149 UB Grade S 355

with 54 No. 19 dia. stud

connectors @ 277 centres.

Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements.

Construction stage moment 242.19 kNm

Steel beam mom. capacity 1583.6 kNm

Const. stage deflection 13.509 mm

Composite stage moment 482.57 kNm

Ultimate mom. capacity 2090.3 kNm

Vertical shear force 128.69 kN

Ult. shear capacity 1495.8 kN

Composite stage deflection 9.3447 mm
Location: Example 1 with fire resistance - method 1

Composite steel deck floor

Edge beam with U.D>Loading

Calculations in accordance with BS 5950 Part 3 Section 3.1 'Code of practice for design of simple and continuous composite beams' - July 1990 and with BS 5950-1: 2000 for construction stage design.

Deck ribs transverse to beam.

Beam span L1=9 m
Beam centres C=3 m
Concrete cube strength fcu=25 N/mm²
Air dry density for concrete Dc=17.6 kN/m³
Freshwet density for concrete D'c=18.8 kN/m³
Deck spans onto supporting beam. 305 x 102 x 33 UB.
Young's Modulus E=205 kN/mm²
Slab depth Ds=115 mm
Depth of profiling Dp=50 mm
Centres of profiles Ct=300 mm
Width at inside of profile b=125 mm
Width at outside of profile p=175 mm
Profiled steel deck decwt=0.12 kN/m²
Reinforcement allowance reinwt=0.04 kN/m²
Steel beam allowance selfwt=0.15 kN/m²
Construction load allowance wc=0.5 kN/m²
Other UDL on beam w'du=0 kN/m
Ceiling and services wcs=0.25 kN/m²
Imposed wi=3 kN/m²
Partitions wp=1 kN/m²
Wall or other UDL wdu=0 kN/m
Number of studs across flange     nr=1  
Spacing of connectors            spac1=400 mm

Check on transverse reinforcement omitted.

Total permanent load               Vfp=40.5 kN

| COMPOSITE                              | 305 x 102 x 33 UB Grade S 355       |
| SECTION                                 | with 24 No. 19 dia. stud             |
| SUMMARY                                  | connectors @ 375 centres.            |
|                                         | Section is satisfactory for         |
|                                         | construction, composite, elastic,   |
|                                         | plastic, moment, shear, deflection  |
|                                         | and natural frequency requirements. |
| Construction stage moment              | 54.718 kNm                           |
| Steel beam mom. capacity               | 170.76 kNm                           |
| Const. stage deflection                 | 19.253 mm                            |
| Composite stage moment                 | 142.79 kNm                           |
| Ultimate mom. capacity                  | 297.92 kNm                           |
| Vertical shear force                    | 63.461 kN                            |
| Ult. shear capacity                    | 439.59 kN                            |
| Composite stage deflection             | 14.665 mm                            |

| FIRE                                    | Exposed surface area                |
| RESISTANCE                              | 919.4 mm²                           |
| SUMMARY                                 | Area of section                     |
|                                         | 41.8 cm²                            |
| Section factor                          | 215 /m                              |
| Load Ratio (R)                          | 0.30302                             |
| Section does not have an inherent fire  |
| resistance of 30 minutes.               |
| Fire protection should be provided      |
Location: Example 1 with fire resistance - method 2

Composite steel deck floor

Edge beam with U.D. Loading

Calculations in accordance with BS 5950 Part 3 Section 3.1 'Code of practice for design of simple and continuous composite beams' - July 1990 and with BS 5950-1: 2000 for construction stage design.

Deck ribs transverse to beam.

Beam span
L1=9 m

Beam centres
C=3 m

Concrete cube strength
fcu=25 N/mm²

Air dry density for concrete
Dc=17.6 kN/m³

Freshwet density for concrete
D'c=18.8 kN/m³

Deck spans onto supporting beam. 305 x 102 x 33 UB.

Young's Modulus
E=205 kN/mm²

Slab depth
Ds=115 mm

Depth of profiling
Dp=50 mm

Centres of profiles
Ct=300 mm

Width at inside of profile
b=125 mm

Width at outside of profile
p=175 mm

Profiled steel deck
decwt=0.12 kN/m²

Reinforcement allowance
reinwt=0.04 kN/m²

Steel beam allowance
selfwt=0.15 kN/m²

Construction load allowance
wc=0.5 kN/m²

Other UDL on beam
w'du=0 kN/m

Ceiling and services
wcs=0.25 kN/m²

Imposed
wi=3 kN/m²

Partitions
wp=1 kN/m²

Wall or other UDL
wdu=0 kN/m
Number of studs across flange \( nr = 1 \)
Spacing of connectors \( \text{spac1} = 400 \text{ mm} \)

Check on transverse reinforcement omitted.

Total permanent load \( V_{fp} = 40.5 \text{ kN} \)

| COMPOSITE | 305 x 102 x 33 UB Grade S 355 |
| SECTION | with 24 No. 19 dia. stud connectors @ 375 centres. |
| SUMMARY | Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements. |

| FIRE | Exposed surface area \( 919.4 \text{ mm}^2 \) |
| RESISTANCE | Area of section \( 41.8 \text{ cm}^2 \) |
| SUMMARY | Section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided |

| Construction stage moment | 54.718 kNm |
| Steel beam mom. capacity | 170.76 kNm |
| Const. stage deflection | 19.253 mm |
| Composite stage moment | 142.79 kNm |
| Ultimate mom. capacity | 297.92 kNm |
| Vertical shear force | 63.461 kN |
| Ult. shear capacity | 439.59 kN |
| Composite stage deflection | 14.665 mm |
Location: Ex1 - Edge beam with UDL

Beam span \( L_1 = 9 \text{ m} \)
Beam centres \( C = 3 \text{ m} \)
Concrete density (const.stage) \( D_c' = 19 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 18 \text{ kN/m}^3 \)
Deck spans onto supporting beam and restraints compression flange.
305 x 165 x 46 UKB
Dimensions (mm): \( h=306.6 \) \( b=165.7 \) \( tw=6.7 \) \( tf=11.8 \) \( r=8.9 \)
Properties (cm): \( I_y=9900 \) \( I_z=896 \) \( W_{ply}=720 \) \( W_{plz}=166 \) \( t_t=22.2 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{v} = 1.25 \)
Permanent load (ULS) \( \gamma_{G} = 1.25 \)
Variable load (ULS) \( \gamma_{Q} = 1.5 \)

Depth of concrete above profile \( hf = 100 \text{ mm} \)
Depth of profiling \( hp = 50 \text{ mm} \)
Centres of profiles \( ep = 300 \text{ mm} \)
Width at inside of profile \( b_1 = 125 \text{ mm} \)
Width at outside of profile \( b_2 = 175 \text{ mm} \)
Thickness of sheeting \( tp = 1.2 \text{ mm} \)
Mean width of concrete ribs \( b_0 = 150 \text{ mm} \)
Weight of steel decking \( f_{sd} = 0.12 \text{ kN/m}^2 \)
Variable load (const.stage) \( f_c = 0.75 \text{ kN/m}^2 \)
Variable load (final stage) \( f_i = 3 \text{ kN/m}^2 \)
Partition load \( fp = 1 \) kN/m²
Ceiling and services \( fcs = 0.25 \) kN/m²
Raised floor \( fr = 0 \) kN/m²
Wall or other UDL (final stage) \( fw = 0 \) kN/m
Char yield strength of reinft. \( fyk = 500 \) N/mm²
Area of reinfntr.per shear plane \( Ae = 345 \) mm²/m

**DESIGN SUMMARY**

**CONSTRUCTION**

**STAGE**

- Steel yield strength: 355 N/mm²
- Concrete strength: 25 N/mm²
- Maximum moment: 75.1 kN/m
- Moment resistance: 256 kNm
- Unity factor: 0.294 ≤ 1
- Maximum shear force: 33.4 kN
- Shear resistance: 461 kN
- Unity factor: 0.072 ≤ 1
- Deflection to span ratio: 1: 383

**DESIGN SUMMARY**

**FINAL STAGE**

- Effective width: 1.12 m
- Stud shear resistance: 72.6 kN
- Number of studs per rib: 1 No.
- Number of 19 mm studs: 30 No.
- Stud spacing: 300 mm
- Design compressive force:
  - Concrete: 1594 kN
  - Studs: 1090 kN
- Partial shear connection provided
- Maximum moment: 154 kNm
- Moment resistance: 421 kNm
- Unity factor bending: 0.366 ≤ 1
- Maximum shear force: 68.4 kN
- Shear plastic resistance: 461 kN
- Unity factor shear: 0.148 ≤ 1
- Deflection to span ratio at final stage (G + Q): 1: 335
- Slab shear force: 242 kN/m
- Slab resistance: 242 kN/m
- Min transverse reinft.: 345 mm²/m
- Lightweight concrete (LWC) assumed
**Location: Ex2 - Solid slab**

![Diagram of composite construction](image)

- **Beam span**: $L_1 = 15\,\text{m}$
- **Beam centres**: $C = 3\,\text{m}$
- **Concrete density (const. stage)**: $D'_c = 18.8\,\text{kN/m}^3$
- **Concrete density (final stage)**: $D_c = 17.6\,\text{kN/m}^3$
- **Dimensions (mm)**: $h=612.4\,\text{mm}$, $b=304.8\,\text{mm}$, $t_w=11.8\,\text{mm}$, $t_f=19.7\,\text{mm}$, $r=16.5\,\text{mm}$
- **Properties (cm)**: $I_y=126000\,\text{cm}^4$, $I_z=9310\,\text{cm}^4$, $W_{ply}=4590\,\text{cm}^3$, $W_{plz}=937\,\text{cm}^3$, $I_t=200\,\text{cm}^4$
- **Partial safety factor (reinft.)**: $g_{ams}=1.15$
- **Partial safety factor (concrete)**: $g_{amc}=1.5$
- **Partial safety factor (steel)**: $g_{am0}=1.0$
- **Partial safety factor (studs)**: $g_{am1}=1.0$
- **Permanent load (ULS)**: $g_{amG}=1.25$
- **Variable load (ULS)**: $g_{amQ}=1.5$
- **Variable load (const. stage)**: $f_c=0.5\,\text{kN/m}^2$
- **Variable load (final stage)**: $f_i=3\,\text{kN/m}^2$
- **Partition load**: $f_p=1\,\text{kN/m}^2$
- **Ceiling and services**: $f_{cs}=0.25\,\text{kN/m}^2$
- **Rounded floor**: $f_r=0\,\text{kN/m}^2$
- **Wall or other UDL (final stage)**: $f_w=0\,\text{kN/m}$
- **Stud spacing**: $spac=300\,\text{mm}$
- **Char yield strength of reinft.**: $f_{yk}=500\,\text{N/mm}^2$
- **Governing shear length**: $h_f=150\,\text{mm}$
- **Area of reinft. per shear plane**: $A_e=768\,\text{mm}^2/\text{m}$
DESIGN SUMMARY
610 x 305 x 149 UKB Grade S 355

CONSTRUCTION
Steel yield strength 345 N/mm²
Concrete strength 25 N/mm²

STAGE
Maximum moment 250 kNm
Moment resistance 1584 kNm
Unity factor 0.158 ≤ 1
Maximum shear force 66.6 kN
Shear resistance 1568 kN
Unity factor 0.042 ≤ 1

Deflection to span ratio
(G + Q) 1: 865

DESIGN SUMMARY
Effective width 1.54 m

FINAL STAGE
Stud shear resistance 65.8 kN
Studs across flange in pairs
Number of 19 mm studs 100 No.
Stud spacing 300 mm

Design compressive force:
Concrete 3188 kN
Studs 3188 kN

Shear connection ratio 1
Full shear connection provided

Maximum moment 469 kNm
Moment resistance 2196 kNm
Unity factor bending 0.213 ≤ 1
Maximum shear force 125 kN
Shear plastic resistance 1568 kN
Unity factor shear 0.08 ≤ 1

Deflection to span ratio
at final stage (G + Q) 1: 633
Slab shear force 438 kN/m
Slab resistance 439 kN/m
Min transverse reinft. 768 mm²/m

Lightweight concrete (LWC) assumed
Location: Ex3

Beam span \( L_1 = 9 \text{ m} \)
Beam centres \( C = 3 \text{ m} \)
Concrete density (const. stage) \( D'c = 18.8 \text{ kN/m}^2 \)
Concrete density (final stage) \( Dc = 17.6 \text{ kN/m}^2 \)
Deck spans onto supporting beam and restraints compression flange.

356 x 127 x 33 UKB

Dimensions (mm): \( h = 349 \quad b = 125.4 \quad tw = 6 \quad tf = 8.5 \quad r = 10.2 \)
Properties (cm): \( I_y = 8250 \quad I_z = 280 \quad W_{ply} = 543 \quad W_{plz} = 70.3 \quad I_t = 8.79 \)
Partial safety factor (reinfnt.) \( g_{ams} = 1.15 \)
Partial safety factor (concrete) \( g_{amc} = 1.5 \)
Partial safety factor (steel) \( g_{am0} = 1.0 \quad g_{am1} = 1.0 \)
Partial safety factor (studs) \( g_{av} = 1.25 \)
Permanent load (ULS) \( g_{amG} = 1.25 \)
Variable load (ULS) \( g_{amQ} = 1.5 \)

Depth of concrete above profile \( hf = 100 \text{ mm} \)
Depth of profiling \( hp = 50 \text{ mm} \)
Centres of profiles \( ep = 300 \text{ mm} \)
Width at inside of profile \( b_1 = 125 \text{ mm} \)
Width at outside of profile \( b_2 = 175 \text{ mm} \)
Thickness of sheeting \( tp = 1.2 \text{ mm} \)
Mean width of concrete ribs \( b_0 = 150 \text{ mm} \)
Weight of steel decking \( fsd = 0.12 \text{ kN/m}^2 \)
Variable load (const. stage) \( fc = 0.5 \text{ kN/m}^2 \)
Variable load (final stage) \( fi = 3 \text{ kN/m}^2 \)
Partition load 
Ceiling and services 
Raised floor 
Wall or other UDL (final stage) 
Char yield strength of reinft. 
Area of reinfnrt.per shear plane 
Design temperature for 30 minutes temp=791.5 °C

Load factor for permanent loads 
Load factor for variable loads

| DESIGN SUMMARY | 356 x 127 x 33 UKB Grade S 355 |
| CONSTRUCTION  | Steel yield strength 355 N/mm² |
| STAGE | Concrete strength 25 N/mm² |
|          | Maximum moment 67.8 kNm |
|          | Moment resistance 193 kNm |
|          | Unity factor 0.352 ≤ 1 |
|          | Maximum shear force 30.2 kN |
|          | Shear resistance 472 kN |
|          | Unity factor 0.064 ≤ 1 |
|          | Deflection to span ratio (G + Q) 1: 350 |

| DESIGN SUMMARY | Effective width 1.12 m |
| FINAL STAGE | Stud shear resistance 65.8 kN |
|              | Number of studs per rib 1 No. |
|              | Number of 19 mm studs 30 No. |
|              | Stud spacing 300 mm |
|              | Design compressive force: |
|              | Concrete 1495 kN |
|              | Studs 986 kN |
|              | Shear connection ratio 0.66 |
|              | Partial shear connection provided |
|              | Maximum moment 148 kNm |
|              | Moment resistance 353 kNm |
|              | Unity factor bending 0.418 ≤ 1 |
|              | Maximum shear force 65.6 kN |
|              | Shear plastic resistance 472 kN |
|              | Unity factor shear 0.139 ≤ 1 |
|              | Deflection to span ratio at final stage (G + Q) 1: 288 |
|              | Slab shear force 219 kN/m |
|              | Slab resistance 219 kN/m |
|              | Min transverse reinft. 304 mm²/m |
|              | Lightweight concrete (LWC) assumed |

The steel beam does not have an inherent fire resistance of 30 minutes and therefore fire protection should be provided.
**Location: Ex4**

**Floor Plan**
(deck ribs transverse to B1)

- **Beam span**: $L_1 = 9$ m
- **Beam centres**: $C = 3$ m
- **Concrete density (const. stage)**: $D'_c = 18.8$ kN/m³
- **Concrete density (final stage)**: $D_c = 17.6$ kN/m³
- **Deck spans onto supporting beam and restraints compression flange.**
- **Dimensions (mm)**:
  - $h = 349$ mm
  - $b = 125.4$ mm
  - $tw = 6$ mm
  - $tf = 8.5$ mm
- **Properties (cm)**:
  - $I_y = 8250$ cm⁴
  - $I_z = 9090$ cm⁴
  - $W_{ply} = 543$ cm³
  - $W_{plz} = 70.3$ cm³
  - $I_t = 8.79$ cm
- **Partial safety factor (reinfnt.)**: $\gamma_{Ms} = 1.15$
- **Partial safety factor (concrete)**: $\gamma_c = 1.5$
- **Partial safety factor (steel)**: $\gamma_{M0} = 1.0$
- **Partial safety factor (studs)**: $\gamma_v = 1.25$
- **Permanent load (ULS)**: $\gamma_G = 1.25$
- **Variable load (ULS)**: $\gamma_Q = 1.5$

**Profiles**
- Depth of concrete above profile: $hf = 100$ mm
- Depth of profiling: $hp = 50$ mm
- Centres of profiles: $ep = 300$ mm
- Width at inside of profile: $b_1 = 125$ mm
- Width at outside of profile: $b_2 = 175$ mm
- Thickness of sheeting: $tp = 1.2$ mm
- Mean width of concrete ribs: $b_0 = 150$ mm
- Weight of steel decking: $fsd = 0.12$ kN/m²
- Variable load (const. stage): $fc = 0.5$ kN/m²
- Variable load (final stage): $fi = 3$ kN/m²
Partition load \( f_p = 1 \text{kN/m}^2 \)
Ceiling and services \( f_{cs} = 0.25 \text{kN/m}^2 \)
Raised floor \( f_r = 0 \text{kN/m}^2 \)
Wall or other UDL (final stage) \( f_w = 0 \text{kN/m} \)
Char yield strength of reinft. \( f_yk = 500 \text{N/mm}^2 \)
Area of reinfnnt.per shear plane \( A_e = 304 \text{mm}^2/\text{m} \)

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>356 x 127 x 33 UKB Grade S 355</th>
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**CONSTRUCTION**

| Steel yield strength | 355 \text{N/mm}^2 |
|----------------------------------|
| Concrete strength | 25 \text{N/mm}^2 |

| Maximum moment | 67.8 \text{kNm} |
|----------------------------------|
| Moment resistance | 193 \text{kNm} |
| Unity factor | 0.352 \leq 1 |

| Maximum shear force | 30.2 \text{kN} |
|----------------------------------|
| Shear resistance | 472 \text{kN} |
| Unity factor | 0.064 \leq 1 |

| Deflection to span ratio | (G + Q) 1: 350 |
|----------------------------------|

**FINAL STAGE**

| Stud shear resistance | 65.8 \text{kN} |
|----------------------------------|
| Number of studs per rib | 1 No. |
| Number of 19 mm studs | 30 No. |
| Stud spacing | 300 mm |

| Design compressive force: | 1495 \text{kN} |
|----------------------------------|
| Concrete | 986 \text{kN} |
| Studs | 0.66 |
| Shear connection ratio | 0.66 |
| Partial shear connection provided |

| Maximum moment | 148 \text{kNm} |
|----------------------------------|
| Moment resistance | 353 \text{kNm} |
| Unity factor bending | 0.418 \leq 1 |

| Maximum shear force | 65.6 \text{kN} |
|----------------------------------|
| Shear plastic resistance | 472 \text{kN} |
| Unity factor shear | 0.139 \leq 1 |

| Deflection to span ratio | at final stage (G + Q) 1: 288 |
|----------------------------------|
| Slab shear force | 219 \text{kN/m} |
| Slab resistance | 219 \text{kN/m} |
| Min transverse reinft. | 304 \text{mm}^2/\text{m} |

Lightweight concrete (LWC) assumed
Location: Ex5 - Edge beam with UDL

Beam span \( L_1 = 9 \text{ m} \)
Beam centres \( C = 3 \text{ m} \)
Concrete density (const.stage) \( D_c' = 19 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 18 \text{ kN/m}^3 \)
Deck spans onto supporting beam and restraints compression flange.
305 x 165 x 46 UKB
Dimensions (mm): \( h=306.6 \text{ b}=165.7 \text{ tw}=6.7 \text{ tf}=11.8 \text{ r}=8.9 \)
Properties (cm): \( I_y=9900 \text{ I}_z=896 \text{ W}_p=720 \text{ W}_p=166 \text{ I}_t=22.2 \)
Partial safety factor (reinfnt.) \( \gamma_{ams}=1.15 \)
Partial safety factor (concrete) \( \gamma_{mc}=1.5 \)
Partial safety factor (steel) \( \gamma_{m0}=1.0 \)
Partial safety factor (steel) \( \gamma_{m1}=1.0 \)
Partial safety factor (studs) \( \gamma_v=1.25 \)
Permanent load (ULS) \( \gamma_G=1.25 \)
Variable load (ULS) \( \gamma_Q=1.5 \)

Depth of concrete above profile \( hf=100 \text{ mm} \)
Depth of profiling \( hp=50 \text{ mm} \)
Centres of profiles \( ep=300 \text{ mm} \)
Width at inside of profile \( b_1=125 \text{ mm} \)
Width at outside of profile \( b_2=175 \text{ mm} \)
Thickness of sheeting \( tp=1.2 \text{ mm} \)
Mean width of concrete ribs \( b_0=150 \text{ mm} \)
Weight of steel decking \( fsd=0.12 \text{ kN/m}^2 \)
Variable load (const.stage) \( fc=0.75 \text{ kN/m}^2 \)
Variable load (final stage) \( fi=3 \text{ kN/m}^2 \)
### Partition load
- $f_p = 1 \text{kN/m}^2$

### Ceiling and services
- $f_{cs} = 0.25 \text{kN/m}^2$

### Raised floor
- $f_r = 0 \text{kN/m}^2$

### Wall or other UDL (final stage)
- $f_w = 0 \text{kN/m}$

### Char yield strength of reinf.
- $f_yk = 500 \text{N/mm}^2$

### Area of reinfnt. per shear plane
- $A_e = 345 \text{mm}^2/\text{m}$

#### DESIGN SUMMARY
- **Stage**: 305 x 165 x 46 UKB Grade S 355
- **Steel yield strength**: 355 N/mm$^2$
- **Concrete strength**: 25 N/mm$^2$
- **Maximum moment**: 75.1 kNm
- **Moment resistance**: 256 kNm
- **Unity factor**: 0.294 ≤ 1
- **Maximum shear force**: 33.4 kN
- **Shear resistance**: 461 kN
- **Unity factor**: 0.072 ≤ 1
- **Deflection to span ratio**: 1:383

#### DESIGN SUMMARY
- **Effective width**: 1.12 m

#### FINAL STAGE
- **Stud shear resistance**: 72.6 kN
- **Number of studs per rib**: 1 No.
- **Number of 19 mm studs**: 30 No.
- **Stud spacing**: 300 mm
- **Design compressive force**:
  - **Concrete**: 1594 kN
  - **Studs**: 1090 kN
- **Shear connection ratio**: 0.684
- **Partial shear connection provided**
- **Maximum moment**: 154 kNm
- **Moment resistance**: 421 kNm
- **Unity factor bending**: 0.366 ≤ 1
- **Maximum shear force**: 68.4 kN
- **Shear plastic resistance**: 461 kN
- **Unity factor shear**: 0.148 ≤ 1
- **Deflection to span ratio** at final stage (G + Q): 1:296
- **Slab shear force**: 242 kN/m
- **Slab resistance**: 242 kN/m
- **Min transverse reinf.**: 345 mm$^2$/m
- **Lightweight concrete (LWC) assumed**
Location: Beam on grid line A3

Beam span \( L_1 = 6 \text{ m} \)
Sum of secondary beam spans \( L_2 = 15 \text{ m} \)
Concrete cube strength \( f_{cu} = 25 \text{ N/mm}^2 \)
Air dry density for concrete \( D_c = 17.6 \text{ kN/m}^3 \)
Fresh wet density for concrete \( D'_c = 18.8 \text{ kN/m}^3 \)
Deck is parallel to supporting beam.
305 x 165 x 46 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Slab depth \( D_s = 115 \text{ mm} \)
Depth of profiling \( D_p = 50 \text{ mm} \)
Centres of profiles \( C_t = 300 \text{ mm} \)
Width at inside of profile \( b = 125 \text{ mm} \)
Width at outside of profile \( p = 175 \text{ mm} \)
Profiled steel deck \( d_{ecwt} = 0.12 \text{ kN/m}^2 \)
Reinforcement allowance \( r_{eint} = 0.04 \text{ kN/m}^2 \)
Steel beam allowance \( s_{elfwt} = 0.15 \text{ kN/m}^2 \)
Construction load allowance \( w_c = 0.5 \text{ kN/m}^2 \)
Other UDL on beam \( w'du = 0.5 \text{ kN/m} \)
Ceiling and services \( w_{cs} = 0.25 \text{ kN/m}^2 \)
Imposed \( w_i = 3 \text{ kN/m}^2 \)
Partitions \( w_p = 1 \text{ kN/m}^2 \)
Wall or other UDL \( w'du = 0.5 \text{ kN/m} \)
Number of studs across flange \( nr = 1 \)
Spacing of connectors \( \text{spac1} = 225 \text{ mm} \)

Check on transverse reinforcement omitted.

| COMPOSITE | 305 x 165 x 46 UB Grade S 355 with 28 No. 19 dia. stud connectors @ 214 centres. |
| SUMMARY    | Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements. |
| Construction stage moment | 124.74 kNm |
| Steel beam mom. capacity    | 255.6 kNm  |
| Const. stage deflection     | 10.653 mm  |
| Composite stage moment      | 320.45 kNm |
| Ultimate mom. capacity      | 397.02 kNm |
| Vertical shear force        | 107.87 kN  |
| Ult. shear capacity         | 437.55 kN  |
| Composite stage deflection  | 8.7003 mm  |
Location: long span with solid slab

Beam span: L1=15 m
Sum of secondary beam spans: L2=15 m
Concrete cube strength: f_{cu}=25 N/mm²
Air dry density for concrete: D_c=17.6 kN/m³
Fresh wet density for concrete: D'c=18.8 kN/m³
762 x 267 x 173 UB.
Young's Modulus: E=205 kN/mm²
Slab depth: D_s=150 mm
Profiled steel deck: decwt=0.12 kN/m²
Reinforcement allowance: reinwt=0.04 kN/m²
Steel beam allowance: selfwt=0.9 kN/m²
Construction load allowance: wc=0.5 kN/m²
Other UDL on beam: w'du=0 kN/m
Ceiling and services: wcs=0.25 kN/m²
Imposed: wi=3 kN/m²
Partitions: wp=1 kN/m²
Wall or other UDL: wdu=0 kN/m
Number of studs across flange $nr=2$
Percentage to be provided $per=90\%$

Check on transverse reinforcement omitted.

| COMPOSITE | 762 x 267 x 173 UB Grade S 355 |
| SECTION    | with 172 No. 19 dia. stud        |
| SUMMARY    | connectors @ 174 centres.        |
|            | Studs to be grouped in 2 's.     |
|            | Section is satisfactory for      |
|            | construction, composite, elastic,|
|            | plastic, moment, shear, deflection|
|            | and natural frequency requirements.|
|            | Construction stage moment 1314.3 kNm |
|            | Steel beam mom. capacity 2139 kNm |
|            | Const. stage deflection 36.507 mm |
|            | Composite stage moment 2516.2 kNm |
|            | Ultimate mom. capacity 3356.9 kNm |
|            | Vertical shear force 335.5 kN    |
|            | Ult. shear capacity 2256.2 kN    |
|            | Composite stage deflection 17.462 mm |
Location: Example 1 with fire resistance - method 1

Beam span \( L_1 = 6 \text{ m} \)
Sum of secondary beam spans \( L_2 = 15 \text{ m} \)
Concrete cube strength \( f_{cu} = 25 \text{ N/mm}^2 \)
Air dry density for concrete \( D_c = 17.6 \text{ kN/m}^3 \)
Fresh wet density for concrete \( D'_c = 18.8 \text{ kN/m}^3 \)
Deck is parallel to supporting beam.
305 x 165 x 46 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Slab depth \( D_s = 115 \text{ mm} \)
Depth of profiling \( D_p = 50 \text{ mm} \)
Centres of profiles \( C_t = 300 \text{ mm} \)
Width at inside of profile \( b = 125 \text{ mm} \)
Width at outside of profile \( p = 175 \text{ mm} \)
Profiled steel deck \( d_{e\text{cwt}} = 0.12 \text{ kN/m}^2 \)
Reinforcement allowance \( r_{\text{e\text{int}}} = 0.04 \text{ kN/m}^2 \)
Steel beam allowance \( s_{\text{selfwt}} = 0.15 \text{ kN/m}^2 \)
Construction load allowance \( w_c = 0.5 \text{ kN/m}^2 \)
Other UDL on beam \( w'd_{u} = 0.5 \text{ kN/m} \)
Ceiling and services \( w'_{\text{cs}} = 0.25 \text{ kN/m}^2 \)
Imposed \( w_i = 3 \text{ kN/m}^2 \)
Partitions \( w_{p} = 1 \text{ kN/m}^2 \)
Wall or other UDL \( w_{du} = 0.5 \text{ kN/m} \)
Number of studs across flange  \( nr = 1 \)
Spacing of connectors  \( \text{spac}_1 = 225 \text{ mm} \)

Check on transverse reinforcement omitted.

Total permanent load  \( V_{fp} = 67.5 \text{ kN} \)

| **COMPOSITE** | 305 x 165 x 46 UB Grade S 355 |
| **SECTION** | with 28 No. 19 dia. stud connectors @ 214 centres. |
| **SUMMARY** | Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements. |
| **FIRE** | Exposed surface area 1096.9 mm² |
| **RESISTANCE** | Area of section 58.7 cm² |
| **SUMMARY** | Section factor 185 /m |
| **Load Ratio (R)** | 0.51096 |
| Section does not have an inherent fire resistance of 30 minutes. |
| Fire protection should be provided |
**Location: Example 1 with fire resistance - method 2**

Beam span $L_1=6 \, \text{m}$

Sum of secondary beam spans $L_2=15 \, \text{m}$

Concrete cube strength $f_{cu}=25 \, \text{N/mm}^2$

Air dry density for concrete $D_c=17.6 \, \text{kN/m}^3$

Fresh wet density for concrete $D'_c=18.8 \, \text{kN/m}^3$

Deck is parallel to supporting beam.

305 x 165 x 46 UB.

Young's Modulus $E=205 \, \text{kN/mm}^2$

Slab depth $D_s=115 \, \text{mm}$

Depth of profiling $D_p=50 \, \text{mm}$

Centres of profiles $C_t=300 \, \text{mm}$

Width at inside of profile $b=125 \, \text{mm}$

Width at outside of profile $p=175 \, \text{mm}$

Profiled steel deck $\text{decwt}=0.12 \, \text{kN/m}^2$

Reinforcement allowance $\text{reinwt}=0.04 \, \text{kN/m}^2$

Steel beam allowance $\text{selfwt}=0.15 \, \text{kN/m}^2$

Construction load allowance $w_c=0.5 \, \text{kN/m}^2$

Other UDL on beam $w'_d=0.5 \, \text{kN/m}$

Ceiling and services $w_{cs}=0.25 \, \text{kN/m}^2$

Imposed $w_i=3 \, \text{kN/m}^2$

Partitions $w_p=1 \, \text{kN/m}^2$

Wall or other UDL $w_{du}=0.5 \, \text{kN/m}$
Number of studs across flange     nr=1
Spacing of connectors             spac1=225 mm

Check on transverse reinforcement omitted.

Total permanent load             Vfp=67.5 kN

COMPOSITE                      305 x 165 x 46 UB Grade S 355
SECTION                       with 28 No. 19 dia. stud
SUMMARY                      connectors @ 214 centres.
Section is satisfactory for
construction, composite, elastic,
plastic, moment, shear, deflection
and natural frequency requirements.
Construction stage moment  124.74 kNm
Steel beam mom. capacity    255.6 kNm
Const. stage deflection      10.653 mm
Composite stage moment      320.45 kNm
Ultimate mom. capacity       397.02 kNm
Vertical shear force         107.87 kN
Ult. shear capacity          437.55 kN
Composite stage deflection   8.7003 mm

FIRE                           Exposed surface area       1096.9 mm²
RESISTANCE                     Area of section            58.7 cm²
SUMMARY                       Section factor             185 /m
Section does not have an inherent
fire resistance of 30 minutes.
Fire protection should be provided
Location: Ex1 - Internal beam

Beam span \( L_1 = 6 \, \text{m} \)
Sum of secondary beam spans \( L_2 = 15 \, \text{m} \)
Concrete density (const. stage) \( D_c' = 19 \, \text{kN/m}^3 \)
Concrete density (final stage) \( D_c = 18 \, \text{kN/m}^3 \)
Deck is parallel to supporting beam and supported on secondary beams.

305 x 165 x 46 UKB

Dimensions (mm): \( h = 306.6 \, \text{b} = 165.7 \, \text{tw} = 6.7 \, \text{tf} = 11.8 \, r = 8.9 \)
Properties (cm): \( I_y = 9900 \, I_z = 896 \, W_{p_l y} = 720 \, W_{p_l z} = 166 \, I_t = 22.2 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{v} = 1.25 \)
Permanent load (ULS) \( \gamma_{G} = 1.25 \)
Variable load (ULS) \( \gamma_{Q} = 1.5 \)

Depth of concrete above profile \( h_f = 80 \, \text{mm} \)
Depth of profiling \( h_p = 50 \, \text{mm} \)
Centres of profiles \( e_p = 300 \, \text{mm} \)
Width at inside of profile \( b_{1} = 125 \, \text{mm} \)
Width at outside of profile \( b_{2} = 175 \, \text{mm} \)
Thickness of sheeting \( t_p = 1.2 \, \text{mm} \)
Mean width of concrete ribs \( b_0 = 150 \, \text{mm} \)
Weight of steel decking \( f_{sd} = 0.12 \, \text{kN/m}^2 \)
Variable load (const. stage) \( f_c = 0.75 \, \text{kN/m}^2 \)
Variable load (final stage) \( f_i = 3 \, \text{kN/m}^2 \)
## DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Component</th>
<th>Load (kN/m²)</th>
</tr>
</thead>
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<tr>
<td>Partition</td>
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<tr>
<td>Ceiling</td>
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<tr>
<td>Wall</td>
<td>0</td>
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<tr>
<td>Beam</td>
<td>0.2</td>
</tr>
</tbody>
</table>

### CONSTRUCTION

- **Steel yield strength**: 355 N/mm²
- **Concrete strength**: 25 N/mm²
- **Maximum moment**: 142 kNm
- **Moment resistance**: 256 kNm
- **Unity factor**: 0.555
- **Maximum shear force**: 48.1 kN
- **Shear resistance**: 461 kN
- **Unity factor**: 0.104

### DESIGN SUMMARY

- **Effective width**: 1.5 m
- **Stud shear resistance**: 72.6 kN
- **Number of studs per rib**: 1 No.
- **Number of 19 mm studs**: 22 No.
- **Stud spacing**: 270 mm

### FINAL STAGE

- **Design compressive force**:
  - **Concrete**: 1700 kN
  - **Studs**: 799 kN
- **Shear connection ratio**: 0.47
- **Partial shear connection provided**: True
- **Maximum moment**: 317 kNm
- **Moment resistance**: 381 kNm
- **Unity factor bending**: 0.832
- **Maximum shear force**: 106 kN
- **Shear plastic resistance**: 461 kN
- **Unity factor shear**: 0.231

### Deflection to span ratio:
- **At final stage (G + Q)**: 1: 327
- **Slab shear force**: 135 kN/m
- **Slab resistance**: 135 kN/m
- **Min transverse reinf.**: 174 mm²/m
- **Lightweight concrete (LWC) assumed**: True
Location: Ex2 - Solid slab

Beam span $L_1=15\ m$
Sum of secondary beam spans $L_2=15\ m$
Concrete density (const.stage) $D'c=19\ \text{kN/m}^3$
Concrete density (final stage) $Dc=18\ \text{kN/m}^3$
762 x 267 x 173 UKB
Dimensions (mm): $h=762.2\ b=266.7\ tw=14.3\ tf=21.6\ r=16.5$
Properties (cm): $I_y=205000\ I_z=6850\ W_{ply}=6200\ W_{plz}=807\ I_t=267$
Partial safety factor (reinfnt.) $g_{ams}=1.15$
Partial safety factor (concrete) $g_{amc}=1.5$
Partial safety factor (steel) $g_{amM0}=1.0$
Partial safety factor (steel) $g_{amM1}=1.0$
Partial safety factor (studs) $g_{amv}=1.25$
Permanent load (ULS) $g_{amG}=1.25$
Variable load (ULS) $g_{amQ}=1.5$
Variable load (const.stage) $f_c=0.5\ \text{kN/m}^2$
Variable load (final stage) $f_i=3\ \text{kN/m}^2$
Partition load $f_p=1\ \text{kN/m}^2$
Ceiling and services $f_{cs}=0.25\ \text{kN/m}^2$
Raised floor $f_r=0\ \text{kN/m}^2$
Wall or other UDL (final stage) $f_w=0\ \text{kN/m}$
Self weight of secondary beam $f_b=0.2\ \text{kN/m}$
Char yield strength of reinfnt. $f_{yk}=500\ \text{N/mm}^2$
Governing shear length $h_f=114.25\ \text{mm}$
Area of reinfnt.per shear plane $A_e=641\ \text{mm}^2/m$
<table>
<thead>
<tr>
<th><strong>DESIGN SUMMARY</strong></th>
<th>762 x 267 x 173 UKB Grade S 355</th>
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</thead>
<tbody>
<tr>
<td><strong>CONSTRUCTION</strong></td>
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</tr>
<tr>
<td><strong>STAGE</strong></td>
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<tr>
<td><strong>Steel yield strength</strong></td>
<td>345 N/mm²</td>
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<tr>
<td><strong>Concrete strength</strong></td>
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<tr>
<td><strong>Maximum moment</strong></td>
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<tr>
<td><strong>Moment resistance</strong></td>
<td>2139 kNm</td>
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<td><strong>Unity factor</strong></td>
<td>0.495 ≤ 1</td>
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<tr>
<td><strong>Buckling resistance</strong></td>
<td>1531 kNm</td>
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<tr>
<td><strong>Unity factor</strong></td>
<td>0.692 ≤ 1</td>
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<tr>
<td><strong>Maximum shear force</strong></td>
<td>149 kN</td>
</tr>
<tr>
<td><strong>Shear resistance</strong></td>
<td>2291 kN</td>
</tr>
<tr>
<td><strong>Unity factor</strong></td>
<td>0.065 ≤ 1</td>
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<tr>
<td><strong>Deflection to span ratio</strong></td>
<td>1: 410</td>
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<th>Effective width 3.75 m</th>
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<tbody>
<tr>
<td><strong>FINAL STAGE</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Stud shear resistance</strong></td>
<td>72.6 kN</td>
</tr>
<tr>
<td><strong>Number of 19 mm studs</strong></td>
<td>146 No.</td>
</tr>
<tr>
<td><strong>Stud spacing</strong></td>
<td>100 mm</td>
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<tr>
<td><strong>Design compressive force:</strong></td>
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</tr>
<tr>
<td><strong>Concrete</strong></td>
<td>7590 kN</td>
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<tr>
<td><strong>Studs</strong></td>
<td>5302 kN</td>
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<tr>
<td><strong>Shear connection ratio</strong></td>
<td>0.699</td>
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<tr>
<td><strong>Partial shear connection provided</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Maximum moment</strong></td>
<td>2233 kNm</td>
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<tr>
<td><strong>Unity factor bending</strong></td>
<td>0.687 ≤ 1</td>
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<tr>
<td><strong>Maximum shear force</strong></td>
<td>306 kN</td>
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<tr>
<td><strong>Shear plastic resistance</strong></td>
<td>2291 kN</td>
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<td><strong>Unity factor shear</strong></td>
<td>0.133 ≤ 1</td>
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<tr>
<td><strong>Deflection to span ratio at final stage (G + Q)</strong></td>
<td>1: 332</td>
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<tr>
<td><strong>Slab shear force</strong></td>
<td>363 kN/m</td>
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<tr>
<td><strong>Slab resistance</strong></td>
<td>363 kN/m</td>
</tr>
<tr>
<td><strong>Min transverse reinft.</strong></td>
<td>641 mm²/m</td>
</tr>
<tr>
<td><strong>Lightweight concrete (LWC) assumed</strong></td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex3

Beam span L1=6 m
Sum of secondary beam spans L2=15 m
Concrete density (const.stage) D'c=19 kN/m³
Concrete density (final stage) Dc=18 kN/m³
Deck is parallel to supporting beam and supported on secondary beams.
305 x 165 x 46 UKB
Dimensions (mm): h=306.6 b=165.7 tw=6.7 tf=11.8 r=8.9
Properties (cm): Iy=9900 Iz=896 Wply=720 Wplz=166 It=22.2
Partial safety factor (reinfnt.) gamM0=1.0
Partial safety factor (concrete) gamM1=1.0
Partial safety factor (steel) gamv=1.25
Permanent load (ULS) gamG=1.25
Variable load (ULS) gamQ=1.5

Depth of concrete above profile hf=80 mm
Depth of profiling hp=50 mm
Centres of profiles ep=300 mm
Width at inside of profile b1=125 mm
Width at outside of profile b2=175 mm
Thickness of sheeting tp=1.2 mm
Mean width of concrete ribs b0=150 mm
Weight of steel decking fsd=0.12 kN/m²
Variable load (const.stage) fc=0.5 kN/m²
Variable load (final stage) fi=3 kN/m²
Partition load \( fp = 1 \text{ kN/m}^2 \)
Ceiling and services \( fcs = 0.25 \text{ kN/m}^2 \)
Raised floor \( fr = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( fw = 0 \text{ kN/m} \)
Self weight of secondary beam \( fb = 0.2 \text{ kN/m} \)
Char yield strength of reinf. \( f_yk = 500 \text{ N/mm}^2 \)
Area of reinfnt.per shear plane \( Ae = 174 \text{ mm}^2 / m \)

**DESIGN SUMMARY**

**CONSTRUCTION**

Steel yield strength \( 355 \text{ N/mm}^2 \)
Concrete strength \( 25 \text{ N/mm}^2 \)
Maximum moment \( 129 \text{ kNm} \)
Moment resistance \( 256 \text{ kNm} \)
Unity factor \( 0.505 \leq 1 \)
Maximum shear force \( 43.9 \text{ kN} \)
Shear resistance \( 461 \text{ kN} \)
Unity factor \( 0.095 \leq 1 \)
Deflection to span ratio

**FINAL STAGE**

Effective width \( 1.5 \text{ m} \)
Stud shear resistance \( 72.6 \text{ kN} \)
Number of studs per rib \( 1 \text{ No.} \)
Number of 19 mm studs \( 22 \text{ No.} \)
Stud spacing \( 270 \text{ mm} \)
Design compressive force:
Concrete \( 1700 \text{ kN} \)
Studs \( 799 \text{ kN} \)
Shear connection ratio \( 0.47 \)
Partial shear connection provided
Maximum moment \( 317 \text{ kNm} \)
Moment resistance \( 381 \text{ kNm} \)
Unity factor bending \( 0.832 \leq 1 \)
Maximum shear force \( 106 \text{ kN} \)
Shear plastic resistance \( 461 \text{ kN} \)
Unity factor shear \( 0.231 \leq 1 \)
Deflection to span ratio
at final stage \( (G + Q) \) \( 1: 327 \)
Slab shear force \( 135 \text{ kN/m} \)
Slab resistance \( 135 \text{ kN/m} \)
Min transverse reinf. \( 174 \text{ mm}^2 / m \)
Lightweight concrete (LWC) assumed
**Location: Ex4**

Beam span \(L_1=6\) m

Sum of secondary beam spans \(L_2=15\) m

Concrete density (const.stage) \(D'_c=19\) kN/m\(^3\)

Concrete density (final stage) \(D_c=18\) kN/m\(^3\)

Deck is parallel to supporting beam and supported on secondary beams. 305 x 165 x 46 UKB

Dimensions (mm): \(h=306.6\) b=165.7 tw=6.7 tf=11.8 r=8.9

Properties (cm): \(I_y=9900\) \(I_z=896\) \(W_{ply}=720\) \(W_{plz}=166\) \(I_{t}=22.2\)

Partial safety factor (reinfnt.) \(\gamma_{ams}=1.15\)

Partial safety factor (concrete) \(\gamma_{amc}=1.5\)

Partial safety factor (steel) \(\gamma_{am0}=1.0\)

Partial safety factor (steel) \(\gamma_{am1}=1.0\)

Partial safety factor (studs) \(\gamma_v=1.25\)

Permanent load (ULS) \(\gamma_G=1.25\)

Variable load (ULS) \(\gamma_Q=1.5\)

Depth of concrete above profile \(hf=80\) mm

Depth of profiling \(hp=50\) mm

Centres of profiles \(ep=300\) mm

Width at inside of profile \(b_1=125\) mm

Width at outside of profile \(b_2=175\) mm

Thickness of sheeting \(tp=1.2\) mm

Mean width of concrete ribs \(b_0=150\) mm

Weight of steel decking \(fsd=0.12\) kN/m\(^2\)

Variable load (const.stage) \(fc=0.5\) kN/m\(^2\)

Variable load (final stage) \(fi=3\) kN/m\(^2\)
Partition load \( fp = 1 \text{ kN/m}^2 \)
Ceiling and services \( fcs = 0.25 \text{ kN/m}^2 \)
Raised floor \( fr = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( fw = 0 \text{ kN/m} \)
Self weight of secondary beam \( fb = 0.2 \text{ kN/m} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Area of reinft. per shear plane \( Ae = 174 \text{ mm}^2 /m \)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>305 x 165 x 46 UKB Grade S 355</th>
</tr>
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<tbody>
<tr>
<td>CONSTRUCTION</td>
<td>Steel yield strength 355 N/mm²</td>
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<tr>
<td>STAGE</td>
<td>Concrete strength 25 N/mm²</td>
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<td></td>
<td>Maximum moment 129 kNm</td>
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<td>Moment resistance 256 kNm</td>
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<td>Unity factor 0.505 ≤ 1</td>
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<td></td>
<td>Maximum shear force 43.9 kN</td>
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<tr>
<td></td>
<td>Shear resistance 461 kN</td>
</tr>
<tr>
<td></td>
<td>Unity factor 0.095 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>Deflection to span ratio (G + Q) 1: 413</td>
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</table>

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<tbody>
<tr>
<td>FINAL STAGE</td>
</tr>
<tr>
<td>Effective width 1.5 m</td>
</tr>
<tr>
<td>Stud shear resistance 72.6 kN</td>
</tr>
<tr>
<td>Number of studs per rib 1 No.</td>
</tr>
<tr>
<td>Number of 19 mm studs 22 No.</td>
</tr>
<tr>
<td>Stud spacing 270 mm</td>
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<tr>
<td>Design compressive force:</td>
</tr>
<tr>
<td>Concrete 1700 kN</td>
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<tr>
<td>Studs 799 kN</td>
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<tr>
<td>Shear connection ratio 0.47</td>
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<tr>
<td>Partial shear connection provided</td>
</tr>
<tr>
<td>Maximum moment 317 kNm</td>
</tr>
<tr>
<td>Moment resistance 381 kNm</td>
</tr>
<tr>
<td>Unity factor bending 0.832 ≤ 1</td>
</tr>
<tr>
<td>Maximum shear force 106 kN</td>
</tr>
<tr>
<td>Shear plastic resistance 461 kN</td>
</tr>
<tr>
<td>Unity factor shear 0.231 ≤ 1</td>
</tr>
<tr>
<td>Deflection to span ratio at final stage (G + Q) 1: 327</td>
</tr>
<tr>
<td>Slab shear force 135 kN/m</td>
</tr>
<tr>
<td>Slab resistance 135 kN/m</td>
</tr>
<tr>
<td>Min transverse reinft. 174 mm²/m</td>
</tr>
<tr>
<td>Lightweight concrete (LWC) assumed</td>
</tr>
</tbody>
</table>
Location: Ex5 - Internal beam

Beam span: \( L_1 = 6 \text{ m} \)
Sum of secondary beam spans: \( L_2 = 15 \text{ m} \)
Concrete density (const. stage): \( D'c = 19 \text{ kN/m}^3 \)
Concrete density (final stage): \( D_c = 18 \text{ kN/m}^3 \)
Deck is parallel to supporting beam and supported on secondary beams.

Dimensions (mm): \( h=306.6 \text{ b}=165.7 \text{ tw}=6.7 \text{ tf}=11.8 \text{ r}=8.9 \)
Properties (cm): \( I_y=9900 \text{ I}_z=896 \text{ Wp}_y=720 \text{ Wp}_z=166 \text{ It}=22.2 \)

Partial safety factor (reinfnt.) \( \gamma_{ams}=1.15 \)
Partial safety factor (concrete) \( \gamma_{mc}=1.5 \)
Partial safety factor (steel) \( \gamma_{M0}=1.0 \)
Partial safety factor (steel) \( \gamma_{M1}=1.0 \)
Partial safety factor (studs) \( \gamma_v=1.25 \)
Permanent load (ULS) \( \gamma_{G}=1.25 \)
Variable load (ULS) \( \gamma_{Q}=1.5 \)

Depth of concrete above profile \( hf=80 \text{ mm} \)
Depth of profiling \( hp=50 \text{ mm} \)
Centres of profiles \( ep=300 \text{ mm} \)
Width at inside of profile \( b_1=125 \text{ mm} \)
Width at outside of profile \( b_2=175 \text{ mm} \)
Thickness of sheeting \( tp=1.2 \text{ mm} \)
Mean width of concrete ribs \( b_0=150 \text{ mm} \)
Weight of steel decking \( fsd=0.12 \text{ kN/m}^2 \)
Variable load (const. stage) \( fc=0.75 \text{ kN/m}^2 \)
Variable load (final stage) \( fi=3 \text{ kN/m}^2 \)
Sample output for SCALE Proforma 214. (ans=5)  
Composite construction: BS5950, BS5400, Eurocode 4  
Internal beam w secondary beams to BS5950 and EC4  
Made by: IFB  
Date: 02/12/19  
Ref No: SC214 EC

Partition load \( fp = 1 \text{ kN/m}^2 \)  
Ceiling and services \( fcs = 0.25 \text{ kN/m}^2 \)  
Raised floor \( fr = 0 \text{ kN/m}^2 \)  
Wall or other UDL (final stage) \( fw = 0 \text{ kN/m} \)  
Self weight of secondary beam \( fb = 0.2 \text{ kN/m} \)  
Char yield strength of reinft. \( f_yk = 500 \text{ N/mm}^2 \)  
Area of reinft. per shear plane \( Ae = 117 \text{ mm}^2 / \text{m} \)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>305 x 165 x 46 UKB Grade S 355</th>
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<tr>
<td>CONSTRUCTION</td>
<td>Steel yield strength 355 N/mm²</td>
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<tr>
<td>STAGE</td>
<td>Concrete strength 25 N/mm²</td>
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<tr>
<td></td>
<td>Maximum moment 142 kNm</td>
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<tr>
<td></td>
<td>Moment resistance 256 kNm</td>
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<tr>
<td></td>
<td>Unity factor 0.555 ≤ 1</td>
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<tr>
<td></td>
<td>Maximum shear force 48.1 kN</td>
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<tr>
<td></td>
<td>Shear resistance 461 kN</td>
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<tr>
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<td>Unity factor 0.104 ≤ 1</td>
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<td>Deflection to span ratio (G + Q) 1: 381</td>
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<tr>
<th>DESIGN SUMMARY</th>
<th>Effective width 1.5 m</th>
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<tr>
<td>FINAL STAGE</td>
<td>Stud shear resistance 72.6 kN</td>
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<tr>
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<td>Number of 19 mm studs 18 No.</td>
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<td>Stud spacing 330 mm</td>
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<td></td>
<td>Studs 654 kN</td>
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<td>Shear connection ratio 0.385</td>
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<td></td>
<td>Maximum moment 317 kNm</td>
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<td></td>
<td>Moment resistance 371 kNm</td>
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<td>Unity factor bending 0.855 ≤ 1</td>
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<td>Maximum shear force 106 kN</td>
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<td>Shear plastic resistance 461 kN</td>
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<td>Unity factor shear 0.231 ≤ 1</td>
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<tr>
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<td>Deflection to span ratio at final stage (G + Q) 1: 256</td>
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<tr>
<td></td>
<td>Slab shear force 110 kN/m</td>
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<tr>
<td></td>
<td>Slab resistance 110 kN/m</td>
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<td>Min transverse reinft. 160 mm²/m</td>
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<td></td>
<td>Lightweight concrete (LWC) assumed</td>
</tr>
</tbody>
</table>
Location: Example 1 - beam on grid line A3

Beam span \( L_1 = 6 \text{ m} \)
Secondary beam span \( L_2 = 6 \text{ m} \)
Concrete cube strength \( f_{cu} = 25 \text{ N/mm}^2 \)
Air dry density for concrete \( D_c = 17.6 \text{ kN/m}^3 \)
Freshwet density for concrete \( D'c = 18.8 \text{ kN/m}^3 \)
Deck is parallel to supporting beam.
305 x 102 x 28 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Slab depth \( D_s = 115 \text{ mm} \)
Depth of profiling \( D_p = 50 \text{ mm} \)
Centres of profiles \( C_t = 300 \text{ mm} \)
Width at inside of profile \( b = 125 \text{ mm} \)
Width at outside of profile \( p = 175 \text{ mm} \)
Profiled steel deck \( decwt = 0.12 \text{ kN/m} \)
Reinforcement allowance \( reinwt = 0.04 \text{ kN/m} \)
Steel beam allowance \( selfwt = 0.15 \text{ kN/m} \)
Construction load allowance \( wc = 0.5 \text{ kN/m}^2 \)
Other UDL on beam \( w'du = 0.33 \text{ kN/m} \)
Ceiling and services \( wcs = 0.25 \text{ kN/m}^2 \)
Imposed \( wi = 3 \text{ kN/m}^2 \)
Partitions \( wp = 1 \text{ kN/m}^2 \)
Wall or other UDL \( wdu = 7.83 \text{ kN/m} \)
Number of studs across flange  \( n_r = 1 \)
Spacing of connectors  \( s_p = 400 \text{ mm} \)

Check on transverse reinforcement omitted.

| COMPOSITE | 305 x 102 x 28 UB Grade S 355 |
| SECTION | with 16 No. 19 dia. stud |
| SUMMARY | connectors @ 375 centres. |

Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements.

| Construction stage moment | 50.717 kNm |
| Steel beam mom. capacity | 143.07 kNm |
| Const. stage deflection | 8.1747 mm |
| Composite stage moment | 176.25 kNm |
| Ultimate mom. capacity | 238.69 kNm |
| Vertical shear force | 75.193 kN |
| Ult. shear capacity | 394.52 kN |
| Composite stage deflection | 6.2614 mm |
Location: Example 2 - long span with solid slab

Beam span                      L1=15 m
Secondary beam span            L2=6 m
Concrete cube strength         fcu=25 N/mm²
Air dry density for concrete   Dc=17.6 kN/m³
Freshwet density for concrete  D'c=18.8 kN/m³
610 x 229 x 113 UB.
Young's Modulus                E=205 kN/mm²
Slab depth                     Ds=150 mm
Profiled steel deck            decwt=0.12 kN/m²
Reinforcement allowance        reinwt=0.04 kN/m²
Steel beam allowance           selfwt=0.55 kN/m²
Construction load allowance    wc=0.5 kN/m²
Other UDL on beam              w'du=0 kN/m
Ceiling and services           wcs=0.25 kN/m²
Imposed                        wi=3 kN/m²
Partitions                     wp=1 kN/m²
Wall or other UDL              wdu=0 kN/m
Number of studs across flange  nr=1
Percentage to be provided      per=90 %

Check on transverse reinforcement omitted.
COMPOSITE
610 x 229 x 113 UB Grade S 355
 SECTION
with 90 No. 19 dia. stud connectors @ 166 centres.
 SUMMARY
Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements.
Construction stage moment 484.37 kNm
Steel beam mom. capacity 1131.6 kNm
Const. stage deflection 31.197 mm
Composite stage moment 965.15 kNm
Ultimate mom. capacity 1747.9 kNm
Vertical shear force 128.69 kN
Ult. shear capacity 1396.1 kN
Composite stage deflection 16.816 mm
Location: Example 3 - with fire resistance check

Beam span \( L_1 = 6 \text{ m} \)
Secondary beam span \( L_2 = 6 \text{ m} \)
Concrete cube strength \( f_{cu} = 25 \text{ N/mm}^2 \)
Air dry density for concrete \( D_c = 17.6 \text{ kN/m}^3 \)
Freshwet density for concrete \( D'_c = 18.8 \text{ kN/m}^3 \)
Deck is parallel to supporting beam.
305 x 102 x 28 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Slab depth \( D_s = 115 \text{ mm} \)
Depth of profiling \( D_p = 50 \text{ mm} \)
Centres of profiles \( C_t = 300 \text{ mm} \)
Width at inside of profile \( b = 125 \text{ mm} \)
Width at outside of profile \( p = 175 \text{ mm} \)
Profiled steel deck \( d_{ewt} = 0.12 \text{ kN/m} \)
Reinforcement allowance \( r_{ewt} = 0.04 \text{ kN/m} \)
Steel beam allowance \( s_{elfw} = 0.15 \text{ kN/m} \)
Construction load allowance \( w_c = 0.5 \text{ kN/m}^2 \)
Other UDL on beam \( w'_d = 0.33 \text{ kN/m} \)
Ceiling and services \( w_{cs} = 0.25 \text{ kN/m}^2 \)
Imposed \( w_i = 3 \text{ kN/m}^2 \)
Partitions \( w_p = 1 \text{ kN/m}^2 \)
Wall or other UDL \( w_{du} = 7.83 \text{ kN/m} \)

(scale=3)
Number of studs across flange \( nr=1 \)
Spacing of connectors \( spac1=400 \text{ mm} \)

Check on transverse reinforcement omitted.

Total permanent load \( V_{fp}=28 \text{ kN} \)

| COMPOSITE | 305 x 102 x 28 UB Grade S 355 with 16 No. 19 dia. stud connectors @ 375 centres. |
| SECTION | |
| SUMMARY | Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements. |
| | Construction stage moment 50.717 kNm |
| | Steel beam mom. capacity 143.07 kNm |
| | Const. stage deflection 8.1747 mm |
| | Composite stage moment 176.25 kNm |
| | Ultimate mom. capacity 238.69 kNm |
| | Vertical shear force 75.193 kN |
| | Ult. shear capacity 394.52 kN |
| | Composite stage deflection 6.2614 mm |

| FIRE | Exposed surface area 910.8 mm² |
| RESISTANCE | |
| SUMMARY | Section does not have an inherent fire resistance of 30 minutes. |
| | Fire protection should be provided |
Location: Ex1

Composite edge beam with a secondary beam at mid-span. Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel and concrete structures'. The beam is unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 6 \text{ m} \)
Secondary beam span \( L_2 = 6 \text{ m} \)
Concrete density (const. stage) \( D_c' = 19 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 18 \text{ kN/m}^3 \)
Deck is parallel to supporting beam and supported on secondary beams.
305 x 102 x 28 UKB
Dimensions (mm): \( h = 308.7 \quad b = 101.8 \quad tw = 6 \quad tf = 8.8 \quad r = 7.6 \)
Properties (cm): \( I_y = 5370 \quad I_z = 155 \quad W_{ply} = 403 \quad W_{plz} = 48.5 \quad I_t = 7.4 \)
Partial safety factor (reinfnt.) \( \gamma_{Ms} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{Mc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{v} = 1.25 \)
Permanent load (ULS) \( \gamma_{G} = 1.25 \)
Variable load (ULS) \( \gamma_{Q} = 1.5 \)

Depth of concrete above profile \( hf = 80 \text{ mm} \)
Depth of profiling \( hp = 50 \text{ mm} \)
Centres of profiles \( ep = 300 \text{ mm} \)
Width at inside of profile \( b_1 = 125 \text{ mm} \)
Width at outside of profile \( b_2 = 175 \text{ mm} \)
Thickness of sheeting \( tp = 1.2 \text{ mm} \)
Mean width of concrete ribs \( b_0 = 125 \text{ mm} \)
Weight of steel decking \( f_{sd} = 0.12 \text{ kN/m}^2 \)
Variable load (const.stage) \( f_c = 0.75 \text{ kN/m}^2 \)
Variable load (final stage) \( f_i = 3 \text{ kN/m}^2 \)
Partition load \( f_p = 1 \text{ kN/m}^2 \)
Ceiling and services \( f_{cs} = 0.25 \text{ kN/m}^2 \)
Raised floor \( f_r = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( f_w = 0 \text{ kN/m} \)
Self weight of secondary beam \( f_b = 0.2 \text{ kN/m} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Area of reinft. per shear plane \( A_e = 198 \text{ mm}^2 / \text{m} \)

**DESIGN SUMMARY**

**CONSTRUCTION**

305 x 102 x 28 UKB Grade S 355

Steel yield strength \( 355 \text{ N/mm}^2 \)
Concrete strength \( 25 \text{ N/mm}^2 \)
Maximum moment \( 57.3 \text{ kNm} \)
Moment resistance \( 143 \text{ kNm} \)
Unity factor \( 0.4 \leq 1 \)
Buckling resistance \( 95.7 \text{ kNm} \)
Unity factor \( 0.599 \leq 1 \)
Maximum shear force \( 19.6 \text{ kN} \)
Shear resistance \( 407 \text{ kN} \)
Unity factor \( 0.048 \leq 1 \)
Deflection to span ratio \( (G + Q) \)
1: 511

**FINAL STAGE**

Effective width \( 0.75 \text{ m} \)
Stud shear resistance \( 72.6 \text{ kN} \)
Number of studs per rib 1 No.
Number of 19 mm studs 12 No.
Stud spacing 500 mm

Design compressive force:
Concrete \( 850 \text{ kN} \)
Studs \( 436 \text{ kN} \)
Shear connection ratio 0.513
Partial shear connection provided
Maximum moment \( 127 \text{ kNm} \)
Moment resistance \( 226 \text{ kNm} \)
Unity factor bending \( 0.563 \leq 1 \)
Maximum shear force \( 42.9 \text{ kN} \)
Shear plastic resistance \( 407 \text{ kN} \)
Unity factor shear \( 0.106 \leq 1 \)
Deflection to span ratio at final stage \( (G + Q) \)
1: 448
Slab shear force \( 145 \text{ kN/m} \)
Slab resistance \( 145 \text{ kN/m} \)
Min transverse reinft. \( 198 \text{ mm}^2 / \text{m} \)
Lightweight concrete (LWC) assumed
Location: Ex2 - long span beam with solid slab

Composite edge beam with a secondary beam at mid-span. Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel and concrete structures'. The beam is unpropped during construction and is assumed to be simply supported.

Beam span
Secondary beam span
Concrete density (const.stage)
Concrete density (final stage)
610 x 229 x 113 UKB
Dimensions (mm): h=607.6 b=228.2 tw=11.1 tf=17.3 r=12.7
Properties (cm): Iy=87300 Iz=3430 Wply=3280 Wplz=469 It=111
Partial safety factor (reinfnt.)
Partial safety factor (concrete)
Partial safety factor (steel)
Partial safety factor (studs)
Permanent load (ULS)
Variable load (ULS)
Variable load (const.stage)
Variable load (final stage)
Partition load
Ceiling and services
Raised floor
Wall or other UDL (final stage)
Self weight of secondary beam
Char yield strength of reinft.
Governing shear length
Area of reinfmt.per shear plane

SCALE 5.48 Office 1007 Proforma 216
DESIGN SUMMARY

610 x 229 x 113 UKB Grade S 355

CONSTRUCTION

Steel yield strength 345 N/mm²
Concrete strength 25 N/mm²
Maximum moment 439 kNm
Moment resistance 1132 kNm
Unity factor 0.388 ≤ 1
Buckling resistance 707 kNm
Unity factor 0.621 ≤ 1
Maximum shear force 63.7 kN
Shear resistance 1421 kN
Unity factor 0.045 ≤ 1

Deflection to span ratio (G + Q) 1: 418

DESIGN SUMMARY

Effective width 1.88 m

FINAL STAGE

Stud shear resistance 72.6 kN
Number of 19 mm studs 78 No.
Stud spacing 190 mm

Design compressive force:
Concrete 3984 kN
Studs 2833 kN
Shear connection ratio 0.711

Partial shear connection provided

Maximum moment 908 kNm
Moment resistance 1693 kNm
Unity factor bending 0.536 ≤ 1

Maximum shear force 126 kN
Shear plastic resistance 1421 kN
Unity factor shear 0.089 ≤ 1

Deflection to span ratio at final stage (G + Q) 1: 351

Slab shear force 382 kN/m
Slab resistance 382 kN/m

Min transverse reinft. 624 mm²/m

Lightweight concrete (LWC) assumed
Composite edge beam with a secondary beam at mid-span. Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel and concrete structures'. The beam is unpropped during construction and is assumed to be simply supported.

Location: Ex3

Beam span \( L1 = 6 \) m
Secondary beam span \( L2 = 6 \) m
Concrete density (const. stage) \( D'c = 19 \) kN/m\(^3\)
Concrete density (final stage) \( Dc = 18 \) kN/m\(^3\)
Deck is parallel to supporting beam and supported on secondary beams.
305 x 102 x 28 UKB
Dimensions (mm): \( h = 308.7 \), \( b = 101.8 \), \( tw = 6 \), \( tf = 8.8 \), \( r = 7.6 \)
Properties (cm): \( Iy = 5370 \), \( Iz = 155 \), \( Wply = 403 \), \( Wplz = 48.5 \), \( It = 7.4 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{amc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{av} = 1.25 \)
Permanent load (ULS) \( \gamma_G = 1.25 \)
Variable load (ULS) \( \gamma_Q = 1.5 \)

Depth of concrete above profile \( hf = 80 \) mm
Depth of profiling \( hp = 50 \) mm
Centres of profiles \( ep = 300 \) mm
Width at inside of profile \( b1 = 125 \) mm
Width at outside of profile \( b2 = 175 \) mm
Thickness of sheeting \( tp = 1.2 \) mm
Mean width of concrete ribs \( b_0 = 125 \text{ mm} \)

Weight of steel decking \( f_{sd} = 0.12 \text{ kN/m}^2 \)

Variable load (const. stage) \( f_c = 0.5 \text{ kN/m}^2 \)

Variable load (final stage) \( f_i = 3 \text{ kN/m}^2 \)

Partition load \( f_p = 1 \text{ kN/m}^2 \)

Ceiling and services \( f_{cs} = 0.25 \text{ kN/m}^2 \)

Raised floor \( f_r = 0 \text{ kN/m}^2 \)

Wall or other UDL (final stage) \( f_w = 0 \text{ kN/m} \)

Self weight of secondary beam \( f_b = 0.2 \text{ kN/m} \)

Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)

Area of reinfnt.per shear plane \( A_e = 198 \text{ mm}^2/\text{m} \)

**DESIGN SUMMARY**

**CONSTRUCTION**

305 x 102 x 28 UKB Grade S 355

Steel yield strength 355 N/mm²

Concrete strength 25 N/mm²

Maximum moment 52.2 kNm

Moment resistance 143 kNm

Unity factor 0.365 ≤ 1

Buckling resistance 95.7 kNm

Unity factor 0.546 ≤ 1

Maximum shear force 17.9 kN

Shear resistance 407 kN

Unity factor 0.044 ≤ 1

Deflection to span ratio (G + Q) 1: 553

**FINAL STAGE**

Effective width 0.75 m

Stud shear resistance 72.6 kN

Number of studs per rib 1 No.

Number of 19 mm studs 12 No.

Stud spacing 500 mm

**DESIGN SUMMARY**

Design compressive force:

Concrete 850 kN

Studs 436 kN

Shear connection ratio 0.513

Partial shear connection provided

Maximum moment 127 kNm

Moment resistance 226 kNm

Unity factor bending 0.563 ≤ 1

Maximum shear force 42.9 kN

Shear plastic resistance 407 kN

Unity factor shear 0.106 ≤ 1

Deflection to span ratio at final stage (G + Q) 1: 448

Slab shear force 145 kN/m

Slab resistance 145 kN/m

Min transverse reinft. 198 mm²/m

Lightweight concrete (LWC) assumed
Location: Ex4 - As Ex1 but with P405 rules

Composite edge beam with a secondary beam at mid-span. Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel and concrete structures'. The beam is unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 6 \text{ m} \)
Secondary beam span \( L_2 = 6 \text{ m} \)
Concrete density (const. stage) \( D'_c = 19 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 18 \text{ kN/m}^3 \)
Deck is parallel to supporting beam and supported on secondary beams.
305 x 102 x 28 UKB
Dimensions (mm): \( h = 308.7 \), \( b = 101.8 \), \( t_w = 6 \), \( t_f = 8.8 \), \( r = 7.6 \)
Properties (cm): \( I_y = 5370 \), \( I_z = 155 \), \( W_{ply} = 403 \), \( W_{plz} = 48.5 \), \( I_t = 7.4 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{amc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{am0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{am1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{amv} = 1.25 \)
Permanent load (ULS) \( \gamma_{gm} = 1.25 \)
Variable load (ULS) \( \gamma_{q} = 1.5 \)

Depth of concrete above profile \( h_f = 80 \text{ mm} \)
Depth of profiling \( h_p = 50 \text{ mm} \)
Centres of profiles \( e_p = 300 \text{ mm} \)
Width at inside of profile \( b_1 = 125 \text{ mm} \)
Width at outside of profile \( b_2 = 175 \text{ mm} \)
Thickness of sheeting \( t_p = 1.2 \text{ mm} \)
Mean width of concrete ribs \( b_0 = 125 \text{ mm} \)

Weight of steel decking \( f_{sd} = 0.12 \text{ kN/m}^2 \)

Variable load (const. stage) \( f_c = 0.75 \text{ kN/m}^2 \)

Variable load (final stage) \( f_i = 3 \text{ kN/m}^2 \)

Partition load \( f_p = 1 \text{ kN/m}^2 \)

Ceiling and services \( f_{cs} = 0.25 \text{ kN/m}^2 \)

Raised floor \( f_r = 0 \text{ kN/m}^2 \)

Wall or other UDL (final stage) \( f_w = 0 \text{ kN/m} \)

Self weight of secondary beam \( f_b = 0.2 \text{ kN/m} \)

Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)

Area of reinfnt. per shear plane \( A_e = 87 \text{ mm}^2 / \text{m} \)

**DESIGN SUMMARY**

305 x 102 x 28 UKB Grade S 355

**CONSTRUCTION**

Steel yield strength 355 N/mm²

Concrete strength 25 N/mm²

Maximum moment 57.3 kNm

Moment resistance 143 kNm

Unity factor \( 0.4 \leq 1 \)

Buckling resistance 95.7 kNm

Unity factor \( 0.599 \leq 1 \)

Maximum shear force 19.6 kN

Shear resistance 407 kN

Unity factor \( 0.048 \leq 1 \)

Deflection to span ratio (\( G + Q \)) 1: 511

**DESIGN SUMMARY**

Effective width 0.75 m

**FINAL STAGE**

Stud shear resistance 72.6 kN

Number of studs per rib 1 No.

Number of 19 mm studs 8 No.

Stud spacing 750 mm

Design compressive force:

Concrete 850 kN

Studs 291 kN

Shear connection ratio 0.342

Partial shear connection provided

Maximum moment 127 kNm

Moment resistance 203 kNm

Unity factor bending 0.627 \( \leq 1 \)

Maximum shear force 42.9 kN

Shear plastic resistance 407 kN

Unity factor shear 0.106 \( \leq 1 \)

Deflection to span ratio at final stage (\( G + Q \)) 1: 341

Slab shear force 96.8 kN/m

Slab resistance 97 kN/m

Min transverse reinft. 160 mm²/m

Lightweight concrete (LWC) assumed
Location: Beam on gridline A3

Floor plan
(deck ribs parallel to B1)

Beam span                         L1=6 m
Sum of secondary beam spans       L2=15 m
Concrete cube strength            fcu=25 N/mm²
Air dry density for concrete      Dc=17.6 kN/m³
Freshwet density for concrete     D'c=18.8 kN/m³
Deck is parallel to supporting beam.
305 x 165 x 40 UB.
Young's Modulus                   E=205 kN/mm²
Slab depth                        Ds=115 mm
Depth of profiling                Dp=50 mm
Centres of profiles               Ct=300 mm
Width at inside of profile        b=125 mm
Width at outside of profile       p=175 mm
Profiled steel deck               decwt=0.12 kN/m²
Reinforcement allowance           reinwt=0.04 kN/m²
Steel beam allowance              selfwt=0.15 kN/m²
Construction load allowance       wc=0.5 kN/m²
Other UDL on beam                 w'du=0.5 kN/m
Ceiling and services              wcs=0.25 kN/m²
Imposed                           wi=3 kN/m²
Partitions                        wp=1 kN/m²
Wall or other UDL                 wdu=0.5 kN/m
Number of studs across flange \( nr=1 \)
Spacing of connectors \( spac1=200 \text{ mm} \)

Check on transverse reinforcement omitted.

<table>
<thead>
<tr>
<th>COMPOSITE</th>
<th>305 x 165 x 40 UB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>with 30 No. 19 dia. stud connectors @ 200 centres.</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements.</td>
</tr>
<tr>
<td>Construction stage moment</td>
<td>111.23 kNm</td>
</tr>
<tr>
<td>Steel beam mom. capacity</td>
<td>221.17 kNm</td>
</tr>
<tr>
<td>Const. stage deflection</td>
<td>13.873 mm</td>
</tr>
<tr>
<td>Composite stage moment</td>
<td>285.2 kNm</td>
</tr>
<tr>
<td>Ultimate mom. capacity</td>
<td>360.37 kNm</td>
</tr>
<tr>
<td>Vertical shear force</td>
<td>143.12 kN</td>
</tr>
<tr>
<td>Ult. shear capacity</td>
<td>387.75 kN</td>
</tr>
<tr>
<td>Composite stage deflection</td>
<td>10.757 mm</td>
</tr>
</tbody>
</table>
Location: long span with solid slab

Beam span          L1=15 m
Sum of secondary beam spans  L2=15 m
Concrete cube strength  fcu=25 N/mm²
Air dry density for concrete  Dc=17.6 kN/m³
Freshwater density for concrete  D'c=18.8 kN/m³
838 x 292 x 176 UB.
Young's Modulus  E=205 kN/mm²
Slab depth  Ds=150 mm
Profiled steel deck  decwt=0.12 kN/m²
Reinforcement allowance  reinwt=0.04 kN/m²
Steel beam allowance  selfwt=0.9 kN/m²
Construction load allowance  wc=0.5 kN/m²
Other UDL on beam  w'du=0 kN/m
Ceiling and services  wcs=0.25 kN/m²
Imposed  wi=3 kN/m²
Partitions  wp=1 kN/m²
Wall or other UDL  wdu=0 kN/m
Number of studs across flange  \( nr = 2 \)
Percentage to be provided  \( per = 90\% \)

Check on transverse reinforcement omitted.

<table>
<thead>
<tr>
<th>COMPOSITE</th>
<th>838 x 292 x 176 UB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>with 172 No. 19 dia. stud</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>connectors @ 174 centres.</td>
</tr>
</tbody>
</table>

Studs to be grouped in 2 's.
Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements.

- Construction stage moment 1168.3 kNm
- Steel beam mom. capacity 2349.5 kNm
- Const. stage deflection 34.554 mm
- Composite stage moment 2236.6 kNm
- Ultimate mom. capacity 3689.8 kNm
- Vertical shear force 447.33 kN
- Ult. shear capacity 2419.5 kN
- Composite stage deflection 16.718 mm
Location: Example 1 with fire resistance check

Beam span $L_1 = 6$ m
Sum of secondary beam spans $L_2 = 15$ m
Concrete cube strength $f_{cu} = 25$ N/mm$^2$
Air dry density for concrete $D_c = 17.6$ kN/m$^3$
Fresh wet density for concrete $D'c = 18.8$ kN/m$^3$
Deck is parallel to supporting beam.
305 x 165 x 40 UB.
Young's Modulus $E = 205$ kN/mm$^2$
Slab depth $D_s = 115$ mm
Depth of profiling $D_p = 50$ mm
Centres of profiles $C_t = 300$ mm
Width at inside of profile $b = 125$ mm
Width at outside of profile $p = 175$ mm
Profiled steel deck $d_{cwt} = 0.12$ kN/m$^2$
Reinforcement allowance $r_{inw} = 0.04$ kN/m$^2$
Steel beam allowance $s_{elfw} = 0.15$ kN/m$^2$
Construction load allowance $w_{c} = 0.5$ kN/m$^2$
Other UDL on beam $w'u_{du} = 0.5$ kN/m
Ceiling and services $w_{cs} = 0.25$ kN/m$^2$
Imposed $w_{i} = 3$ kN/m$^2$
Partitions $w_{p} = 1$ kN/m$^2$
Wall or other UDL $w'u_{du} = 0.5$ kN/m
Number of studs across flange \( nr = 1 \)
Spacing of connectors \( \text{spac}_1 = 200 \text{ mm} \)

Check on transverse reinforcement omitted.

Total permanent load \( V_{fp} = 45 \text{ kN} \)

| COMPOSITE | 305 x 165 x 40 UB Grade S 355 with 30 No. 19 dia. stud connectors @ 200 centres. |
| SECTION | |
| SUMMARY | Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements. |
| Construction stage moment | 111.23 kNm |
| Steel beam mom. capacity | 221.17 kNm |
| Const. stage deflection | 13.873 mm |
| Composite stage moment | 285.2 kNm |
| Ultimate mom. capacity | 360.37 kNm |
| Vertical shear force | 143.12 kN |
| Ult. shear capacity | 387.75 kN |
| Composite stage deflection | 10.757 mm |

| FIRE | Exposed surface area \( 1089.8 \text{ mm}^2 \) |
| RESISTANCE | |
| SUMMARY | Section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided |
| Area of section | 51.3 cm² |
| Section factor | 210 /m |
| Load Ratio (R) | 0.50107 |
Location: Ex1 - beam on grid line A3

Composite internal beam with secondary beams located at 1/3 points to BS EN 1994-1-1:2004 the Code for the 'Design of composite steel and concrete structures'. The beam is considered to be unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 6 \) m
Sum of secondary beam spans \( L_2 = 15 \) m
Concrete density (const stage) \( D'_c = 19 \) kN/m³
Concrete density (final stage) \( D_c = 18 \) kN/m³
Deck is parallel to supporting beam and supported on secondary beams.
305 x 165 x 40 UKB
Dimensions (mm): \( h = 303.4 \) b = 165 \( t_w = 6 \) tf = 10.2 \( r = 8.9 \)
Properties (cm): \( I_y = 8500 \) \( I_z = 764 \) \( W_{ply} = 623 \) \( W_{plz} = 142 \) \( I_t = 14.7 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_v = 1.25 \)
Permanent load (ULS) \( \gamma_G = 1.25 \)
Variable load (ULS) \( \gamma_Q = 1.5 \)

Depth of concrete above profile \( h_f = 80 \) mm
Depth of profiling \( h_p = 50 \) mm
Centres of profiles \( e_p = 300 \) mm
Width at inside of profile \( b_1 = 125 \) mm
Width at outside of profile \( b_2 = 175 \) mm
Thickness of sheeting \( tp = 1.2 \, \text{mm} \)
Mean width of concrete ribs \( b_0 = 125 \, \text{mm} \)
Weight of steel decking \( f_{sd} = 0.12 \, \text{kN/m}^2 \)
Variable load (const. stage) \( f_c = 0.75 \, \text{kN/m}^2 \)
Variable load (final stage) \( f_i = 3 \, \text{kN/m}^2 \)
Partition load \( f_p = 1 \, \text{kN/m}^2 \)
Ceiling and services \( f_{cs} = 0.25 \, \text{kN/m}^2 \)
Raised floor \( f_r = 0 \, \text{kN/m}^2 \)
Wall or other UDL (final stage) \( f_w = 0.5 \, \text{kN/m} \)
Self weight of secondary beam \( f_b = 0.2 \, \text{kN/m} \)
Char yield strength of reinf. \( f_yk = 500 \, \text{N/mm}^2 \)
Area of reinfnt. per shear plane \( A_e = 328 \, \text{mm}^2 \)

**DESIGN SUMMARY**
- 305 x 165 x 40 UKB Grade S 355

**CONSTRUCTION**
- Steel yield strength \( 355 \, \text{N/mm}^2 \)
- Concrete strength \( 25 \, \text{N/mm}^2 \)
- Maximum moment \( 127 \, \text{kNm} \)
- Moment resistance \( 221 \, \text{kNm} \)
- Unity factor \( 0.576 \leq 1 \)
- Maximum shear force \( 64 \, \text{kN} \)
- Shear resistance \( 411 \, \text{kN} \)
- Unity factor \( 0.156 \leq 1 \)

**DESIGN SUMMARY**
- Effective width \( 1.5 \, \text{m} \)
- Stud shear resistance \( 72.6 \, \text{kN} \)
- Number of studs per rib 1 No.
- Number of 19 mm studs see footnote
- Stud spacing between see footnote

**FINAL STAGE**
- Design compressive force:
  - Concrete \( 1700 \, \text{kN} \)
  - Studs \( 799 \, \text{kN} \)
- Shear connection ratio \( 0.47 \)
- Partial shear connection provided
- Maximum moment \( 286 \, \text{kNm} \)
- Moment resistance \( 340 \, \text{kNm} \)
- Unity factor bending \( 0.839 \leq 1 \)
- Maximum shear force \( 144 \, \text{kN} \)
- Shear plastic resistance \( 411 \, \text{kN} \)
- Unity factor shear \( 0.349 \leq 1 \)

Stud spacing between each support and 1/3 point to be 180 mm.

The number of studs required between each support & 1/3 point is 11 No. Between 1/3 points provide nominal studs.
Location: Ex2 - long span with solid slab

Composite internal beam with secondary beams located at 1/3 points to BS EN 1994-1-1:2004 the Code for the 'Design of composite steel and concrete structures'. The beam is considered to be unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 15 \text{ m} \)
Sum of secondary beam spans \( L_2 = 15 \text{ m} \)
Concrete density (const.stage) \( D'_c = 26 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 25 \text{ kN/m}^3 \)
Dimensions (mm): \( h = 834.9 \text{ b} = 291.7 \text{ tw} = 14 \text{ tf} = 18.8 \text{ r} = 17.8 \)
Properties (cm): \( I_y = 246000 \text{ Iz} = 7800 \text{ Wply} = 6810 \text{ Wplz} = 842 \text{ It} = 221 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{v} = 1.25 \)
Permanent load (ULS) \( \gamma_{G} = 1.25 \)
Variable load (ULS) \( \gamma_{Q} = 1.5 \)
Variable load (const.stage) \( f_c = 0.5 \text{ kN/m}^2 \)
Variable load (final stage) \( f_i = 3 \text{ kN/m}^2 \)
Partition load \( f_p = 1 \text{ kN/m}^2 \)
Ceiling and services \( f_{cs} = 0.25 \text{ kN/m}^2 \)
Raised floor \( f_r = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( f_w = 0.5 \text{ kN/m} \)
Self weight of secondary beam \( f_b = 0.2 \text{ kN/m} \)
Char yield strength of reinft. \( f_yk = 500 \text{ N/mm}^2 \)
Governing shear length \( h_f = 150 \text{ mm} \)
Area of reinft. per shear plane \( A_e = 968 \text{ mm}^2/m \)
DESIGN SUMMARY  
838 x 292 x 176 UKB Grade S 355

CONSTRUCTION

STAGE

Steel yield strength 345 N/mm²
Concrete strength 30 N/mm²
Maximum moment 1090 kNm
Moment resistance 2349 kNm
Unity factor 0.464 ≤ 1
Buckling resistance 2235 kNm
Unity factor 0.488 ≤ 1
Maximum shear force 222 kN
Shear resistance 2463 kN
Unity factor 0.09 ≤ 1

Deflection to span ratio

(G + Q) 1: 379

DESIGN SUMMARY

FINAL STAGE

Effective width 3.83 m

Stud shear resistance 81.7 kN
Studs across flange in pairs
Number of 19 mm studs see footnote
Stud spacing between see footnote

Design compressive force:
Concrete 7728 kN
Studs 5389 kN
Shear connection ratio 0.697

Partial shear connection provided

Maximum moment 2115 kNm
Moment resistance 3579 kNm
Unity factor bending 0.591 ≤ 1
Maximum shear force 428 kN
Shear plastic resistance 2463 kN
Unity factor shear 0.174 ≤ 1

Deflection to span ratio

at final stage (G + Q) 1: 312

Slab shear force 544 kN/m
Slab resistance 545 kN/m
Min transverse reinft. 968 mm²/m
Normal weight concrete (NWC) assumed

Stud spacing between each support and 1/3 point to be 150 mm.  
The number of studs required between each support & 1/3 point is 33 No.  Between 1/3 points provide nominal studs.
Location: Ex3

Composite internal beam with secondary beams located at 1/3 points to BS EN 1994-1-1:2004 the Code for the 'Design of composite steel and concrete structures'. The beam is considered to be unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 6 \text{ m} \)
Sum of secondary beam spans \( L_2 = 15 \text{ m} \)
Concrete density (const.stage) \( D'_c = 19 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 18 \text{ kN/m}^3 \)
Deck is parallel to supporting beam and supported on secondary beams.
305 x 165 x 40 UKB
Dimensions (mm): \( h = 303.4 \), \( b = 165 \), \( t_w = 10.2 \), \( r = 8.9 \)
Properties (cm): \( I_y = 8500 \), \( I_z = 764 \), \( W_{p_y} = 623 \), \( W_{p_z} = 142 \), \( I_t = 14.7 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{amc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{v} = 1.25 \)
Permanent load (ULS) \( \gamma_G = 1.25 \)
Variable load (ULS) \( \gamma_Q = 1.5 \)

Depth of concrete above profile \( hf = 80 \text{ mm} \)
Depth of profiling \( hp = 50 \text{ mm} \)
Centres of profiles \( ep = 300 \text{ mm} \)
Width at inside of profile \( b_1 = 125 \text{ mm} \)
Width at outside of profile \( b_2 = 175 \text{ mm} \)
<table>
<thead>
<tr>
<th>Thickness of sheeting</th>
<th>tp=1.2 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean width of concrete ribs</td>
<td>b0=125 mm</td>
</tr>
<tr>
<td>Weight of steel decking</td>
<td>fsd=0.12 kN/m²</td>
</tr>
<tr>
<td>Variable load (const.stage)</td>
<td>fc=0.5 kN/m²</td>
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<tr>
<td>Variable load (final stage)</td>
<td>fi=3 kN/m²</td>
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<tr>
<td>Partition load</td>
<td>fp=1 kN/m²</td>
</tr>
<tr>
<td>Ceiling and services</td>
<td>fcs=0.25 kN/m²</td>
</tr>
<tr>
<td>Raised floor</td>
<td>fr=0 kN/m²</td>
</tr>
<tr>
<td>Wall or other UDL (final stage)</td>
<td>fw=0.5 kN/m</td>
</tr>
<tr>
<td>Self weight of secondary beam</td>
<td>fb=0.2 kN/m</td>
</tr>
<tr>
<td>Char yield strength of reinft.</td>
<td>fyk=500 N/mm²</td>
</tr>
<tr>
<td>Area of reinfn. per shear plane</td>
<td>Ae=328 mm²/m</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>305 x 165 x 40 UKB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONSTRUCTION</td>
<td>Steel yield strength 355 N/mm²</td>
</tr>
<tr>
<td>STAGE</td>
<td>Concrete strength 25 N/mm²</td>
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<tr>
<td></td>
<td>Maximum moment 116 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment resistance 221 kNm</td>
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<tr>
<td></td>
<td>Unity factor 0.525 ≤ 1</td>
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<tr>
<td></td>
<td>Maximum shear force 58.4 kN</td>
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<td></td>
<td>Shear resistance 411 kN</td>
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<tr>
<td></td>
<td>Unity factor 0.142 ≤ 1</td>
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<td></td>
<td>Deflection to span ratio (G + Q) 1: 311</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Effective width 1.5 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>FINAL STAGE</td>
<td>Stud shear resistance 72.6 kN</td>
</tr>
<tr>
<td></td>
<td>Number of studs per rib 1 No.</td>
</tr>
<tr>
<td></td>
<td>Number of 19 mm studs see footnote</td>
</tr>
<tr>
<td></td>
<td>Stud spacing between see footnote</td>
</tr>
<tr>
<td></td>
<td>Design compressive force: Concrete 1700 kN</td>
</tr>
<tr>
<td></td>
<td>Studs 799 kN</td>
</tr>
<tr>
<td></td>
<td>Shear connection ratio 0.47</td>
</tr>
<tr>
<td></td>
<td>Partial shear connection provided</td>
</tr>
<tr>
<td></td>
<td>Maximum moment 286 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment resistance 340 kNm</td>
</tr>
<tr>
<td></td>
<td>Unity factor bending 0.839 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>Maximum shear force 144 kN</td>
</tr>
<tr>
<td></td>
<td>Shear plastic resistance 411 kN</td>
</tr>
<tr>
<td></td>
<td>Unity factor shear 0.349 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>Deflection to span ratio at final stage (G + Q) 1: 247</td>
</tr>
<tr>
<td></td>
<td>Slab shear force 202 kN/m</td>
</tr>
<tr>
<td></td>
<td>Slab resistance 202 kN/m</td>
</tr>
<tr>
<td></td>
<td>Min transverse reinft. 328 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Lightweight concrete (LWC) assumed</td>
</tr>
</tbody>
</table>

Stud spacing between each support and 1/3 point to be 180 mm. The number of studs required between each support & 1/3 point is 11 No. Between 1/3 points provide nominal studs.
Location: Ex4 – long span with metal decking

Composite internal beam with secondary beams located at 1/3 points to BS EN 1994-1-1:2004 the Code for the 'Design of composite steel and concrete structures'. The beam is considered to be unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 15 \text{ m} \)
Sum of secondary beam spans \( L_2 = 15 \text{ m} \)
Concrete density (const. stage) \( D'_c = 26 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 25 \text{ kN/m}^3 \)
Deck is parallel to supporting beam and supported on secondary beams.
838 x 292 x 176 UKB
Dimensions (mm): \( h = 834.9 \ b = 291.7 \ tw = 14 \ tf = 18.8 \ r = 17.8 \)
Properties (cm): \( I_y = 246000 \ I_z = 7800 \ W_{ply} = 6810 \ W_{plz} = 842 \ I_t = 221 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{v} = 1.25 \)
Permanent load (ULS) \( \gamma_G = 1.25 \)
Variable load (ULS) \( \gamma_Q = 1.5 \)

Depth of concrete above profile \( hf = 139 \text{ mm} \)
Depth of profiling \( hp = 40 \text{ mm} \)
Centres of profiles \( ep = 206 \text{ mm} \)
Width at inside of profile \( b_1 = 78 \text{ mm} \)
Width at outside of profile \( b_2 = 103 \text{ mm} \)
Sample output for SCALE Proforma 218. (ans=4)  
Made by: IFB  
Date: 02/12/19  
Ref No: SC218 EC

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of sheeting</td>
<td>$tp = 0.75$ mm</td>
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<tr>
<td>Mean width of concrete ribs</td>
<td>$b_0 = 90.5$ mm</td>
</tr>
<tr>
<td>Weight of steel decking</td>
<td>$fsd = 0.12$ kN/m²</td>
</tr>
<tr>
<td>Variable load (const. stage)</td>
<td>$fc = 0.5$ kN/m²</td>
</tr>
<tr>
<td>Variable load (final stage)</td>
<td>$fi = 3$ kN/m²</td>
</tr>
<tr>
<td>Partition load</td>
<td>$fp = 1$ kN/m²</td>
</tr>
<tr>
<td>Ceiling and services</td>
<td>$fcs = 0.25$ kN/m²</td>
</tr>
<tr>
<td>Raised floor</td>
<td>$fr = 0$ kN/m²</td>
</tr>
<tr>
<td>Wall or other UDL (final stage)</td>
<td>$fw = 0.5$ kN/m</td>
</tr>
<tr>
<td>Self weight of secondary beam</td>
<td>$fb = 0.2$ kN/m</td>
</tr>
<tr>
<td>Char yield strength of reinft.</td>
<td>$fy_k = 500$ N/mm²</td>
</tr>
<tr>
<td>Area of reinfnt. per shear plane</td>
<td>$A_e = 832$ mm²/m</td>
</tr>
</tbody>
</table>

**DESIGN SUMMARY**

- **Stage 1**
  - Effective width: $3.83$ m  
  - Stud shear resistance: $73.5$ kN  
  - Number of studs per rib: $2$ No.  
  - Number of $19$ mm studs: see footnote  
  - Stud spacing between: see footnote  

**DESIGN SUMMARY**

- **Stage 2**
  - Effective width: $3.83$ m  
  - Stud shear resistance: $73.5$ kN  
  - Number of studs per rib: $2$ No.  
  - Number of $19$ mm studs: see footnote  
  - Stud spacing between: see footnote  

- **Concrete**
  - Design compressive force:
    - $7728$ kN  
  - Shear connection ratio: $0.704$  

- **Studs**
  - Design compressive force:
    - $5438$ kN  

- **Shear connection**
  - Provided  

- **Max. moment**
  - $2180$ kNm  

- **Unity factor bending**
  - $0.578 \leq 1$  

- **Max. shear force**
  - $441$ kN  

- **Shear plastic resistance**
  - $2463$ kN  

- **Unity factor shear**
  - $0.179 \leq 1$  

- **Deflection to span ratio**
  - at final stage $(G + Q)$: $1: 303$  

- **Slab shear force**
  - $490$ kN/m  

- **Slab resistance**
  - $490$ kN/m  

- **Min transverse reinft.**
  - $832$ mm²/m  

- **Normal weight concrete (NWC) assumed**

Stud spacing between each support and $1/3$ point to be $150$ mm.  
The number of studs required between each support & $1/3$ point is $33$ No. Between $1/3$ points provide nominal studs.
**Location:** Ex5 - beam on grid line A3

Composite internal beam with secondary beams located at 1/3 points to BS EN 1994-1-1:2004 the Code for the 'Design of composite steel and concrete structures'. The beam is considered to be unpropped during construction and is assumed to be simply supported.

Beam span $L1=6 \text{ m}$

Sum of secondary beam spans $L2=15 \text{ m}$

Concrete density (const.stage) $D'c=19 \text{ kN/m}^3$

Concrete density (final stage) $Dc=18 \text{ kN/m}^3$

Deck is parallel to supporting beam and supported on secondary beams.

305 x 165 x 40 UKB

Dimensions (mm): $h=303.4$ $b=165$ $tw=6$ $tf=10.2$ $r=8.9$

Properties (cm): $Iy=8500$ $Iz=764$ $Wply=623$ $Wplz=142$ $It=14.7$

Partial safety factor (reinfnt.) $gam_s=1.15$

Partial safety factor (concrete) $gam_c=1.5$

Partial safety factor (steel) $gamM0=1.0$

Partial safety factor (steel) $gamM1=1.0$

Partial safety factor (studs) $gam_v=1.25$

Permanent load (ULS) $gam_G=1.25$

Variable load (ULS) $gam_Q=1.5$

Depth of concrete above profile $hf=80 \text{ mm}$

Depth of profiling $hp=50 \text{ mm}$

Centres of profiles $ep=300 \text{ mm}$

Width at inside of profile $b1=125 \text{ mm}$

Width at outside of profile $b2=175 \text{ mm}$
Thickness of sheeting \( tp = 1.2 \text{ mm} \)
Mean width of concrete ribs \( b_0 = 125 \text{ mm} \)
Weight of steel decking \( fsd = 0.12 \text{ kN/m}^2 \)
Variable load (const. stage) \( fc = 0.75 \text{ kN/m}^2 \)
Variable load (final stage) \( fi = 3 \text{ kN/m}^2 \)
Partition load \( fp = 1 \text{ kN/m}^2 \)
Ceiling and services \( fcs = 0.25 \text{ kN/m}^2 \)
Raised floor \( fr = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( fw = 0.5 \text{ kN/m} \)
Self weight of secondary beam \( fb = 0.2 \text{ kN/m} \)
Char yield strength of reinft. \( fyk = 500 \text{ N/mm}^2 \)
Area of reinfnt. per shear plane \( Ae = 244 \text{ mm}^2 / \text{m} \)

### DESIGN SUMMARY

- **CONSTRUCTION**
  - Steel yield strength \( 355 \text{ N/mm}^2 \)
  - Concrete strength \( 25 \text{ N/mm}^2 \)
  - Maximum moment \( 127 \text{ kNm} \)
  - Moment resistance \( 221 \text{ kNm} \)
  - Unity factor \( 0.576 \leq 1 \)
  - Maximum shear force \( 64 \text{ kN} \)
  - Shear resistance \( 411 \text{ kN} \)
  - Unity factor \( 0.156 \leq 1 \)

### FINAL STAGE

- **Design compressive force:**
  - Concrete \( 1700 \text{ kN} \)
  - Studs \( 654 \text{ kN} \)
  - Shear connection ratio \( 0.385 \)

- **Partial shear connection provided**
  - Maximum moment \( 286 \text{ kNm} \)
  - Moment resistance \( 326 \text{ kNm} \)
  - Unity factor bending \( 0.877 \leq 1 \)
  - Maximum shear force \( 144 \text{ kN} \)
  - Shear plastic resistance \( 411 \text{ kN} \)
  - Unity factor shear \( 0.349 \leq 1 \)

- **Deflection to span ratio**
  - at final stage \( 1: 193 \)
  - Slab shear force \( 165 \text{ kN/m} \)
  - Slab resistance \( 165 \text{ kN/m} \)
  - Min transverse reinft. \( 244 \text{ mm}^2 / \text{m} \)

### WARNING:
- Total deflection exceeds L/200 the normally acceptable limit.
Location: Beam on gridline A3

Beam span \( L_1 = 6 \text{ m} \)
Secondary beam span \( L_2 = 6 \text{ m} \)
Concrete cube strength \( f_{cu} = 25 \text{ N/mm}^2 \)
Air dry density for concrete \( D_c = 17.6 \text{ kN/m}^3 \)
Freshwet density for concrete \( D'_c = 18.8 \text{ kN/m}^3 \)
Deck is parallel to supporting beam.
305 x 102 x 28 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Slab depth \( D_s = 115 \text{ mm} \)
Depth of profiling \( D_p = 50 \text{ mm} \)
Centres of profiles \( C_t = 300 \text{ mm} \)
Width at inside of profile \( b = 125 \text{ mm} \)
Width at outside of profile \( p = 175 \text{ mm} \)
Profiled steel deck \( d_{e c w t} = 0.12 \text{ kN/m}^2 \)
Reinforcement allowance \( r_{e i n w t} = 0.04 \text{ kN/m}^2 \)
Steel beam allowance \( s_{e l f w t} = 0.15 \text{ kN/m}^2 \)
Construction load allowance \( w_c = 0.5 \text{ kN/m}^2 \)
Other UDL on beam \( w'_d u = 0.33 \text{ kN/m} \)
Ceiling and services \( w_{c s} = 0.25 \text{ kN/m}^2 \)
Imposed \( w_i = 3 \text{ kN/m}^2 \)
Partitions \( w_p = 1 \text{ kN/m}^2 \)
Wall or other UDL \( w_d u = 7.83 \text{ kN/m} \)
Number of studs across flange  \text{nr}=1
Spacing of connectors  \text{spac}=400 \text{ mm}

Check on transverse reinforcement omitted.

\begin{tabular}{ll}
\textbf{COMPOSITE} & 305 x 102 x 28 UB Grade S 355 \\
\textbf{SECTION} & with 16 No. 19 dia. stud \\
\textbf{SUMMARY} & connectors @ 375 centres. \\
& Section is satisfactory for \\
& construction, composite, elastic, \\
& plastic, moment, shear, deflection \\
& and natural frequency requirements. \\
& Construction stage moment 45.313 kNm \\
& Steel beam mom. capacity 143.07 kNm \\
& Const. stage deflection 9.055 mm \\
& Composite stage moment 162.15 kNm \\
& Ultimate mom. capacity 238.69 kNm \\
& Vertical shear force 89.296 kN \\
& Ult. shear capacity 394.52 kN \\
& Composite stage deflection 7.1117 mm \\
\end{tabular}
Location: Beam on gridline A3

Beam span \( L_1 = 6 \, \text{m} \)
Secondary beam span \( L_2 = 6 \, \text{m} \)
Concrete cube strength \( f_{cu} = 25 \, \text{N/mm}^2 \)
Air dry density for concrete \( D_c = 17.6 \, \text{kN/m}^3 \)
Freshwet density for concrete \( D'_c = 18.8 \, \text{kN/m}^3 \)
Deck is parallel to supporting beam.
305 x 102 x 28 UB.
Young's Modulus \( E = 205 \, \text{kN/mm}^2 \)
Slab depth \( D_s = 115 \, \text{mm} \)
Depth of profiling \( D_p = 50 \, \text{mm} \)
Centres of profiles \( C_t = 300 \, \text{mm} \)
Width at inside of profile \( b = 125 \, \text{mm} \)
Width at outside of profile \( p = 175 \, \text{mm} \)
Profiled steel deck \( \text{decwt} = 0.12 \, \text{kN/m}^2 \)
Reinforcement allowance \( \text{reinwt} = 0.04 \, \text{kN/m}^2 \)
Steel beam allowance \( \text{selfwt} = 0.15 \, \text{kN/m}^2 \)
Construction load allowance \( \text{wc} = 0.5 \, \text{kN/m}^2 \)
Other UDL on beam \( \text{w'du} = 0.33 \, \text{kN/m} \)
Ceiling and services \( \text{wcs} = 0.25 \, \text{kN/m}^2 \)
Imposed \( \text{wi} = 3 \, \text{kN/m}^2 \)
Partitions \( \text{wp} = 1 \, \text{kN/m}^2 \)
Wall or other UDL \( \text{wdu} = 7.83 \, \text{kN/m} \)
Number of studs across flange \( nr=1 \)
Spacing of connectors \( \text{spac1}=400 \text{ mm} \)

Check on transverse reinforcement omitted.

**COMPOSITE**

305 x 102 x 28 UB Grade S 355 with 16 No. 19 dia. stud connectors @ 375 centres.

**SUMMARY**

Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements.

Construction stage moment 45.313 kNm
Steel beam mom. capacity 143.07 kNm
Const. stage deflection 9.055 mm
Composite stage moment 162.15 kNm
Ultimate mom. capacity 238.69 kNm
Vertical shear force 89.296 kN
Ult. shear capacity 394.52 kN
Composite stage deflection 7.1117 mm
Location: example 1 with fire resistance check

---

**Beam span**
- $L_1 = 6$ m

**Secondary beam span**
- $L_2 = 6$ m

**Concrete cube strength**
- $f_{cu} = 25$ N/mm$^2$

**Air dry density for concrete**
- $D_c = 17.6$ kN/m$^3$

**Freshwet density for concrete**
- $D'c = 18.8$ kN/m$^3$

Deck is parallel to supporting beam.
305 x 102 x 28 UB.

**Young's Modulus**
- $E = 205$ kN/mm$^2$

**Slab depth**
- $D_s = 115$ mm

**Depth of profiling**
- $D_p = 50$ mm

**Centres of profiles**
- $C_t = 300$ mm

**Width at inside of profile**
- $b = 125$ mm

**Width at outside of profile**
- $p = 175$ mm

**Profiled steel deck**
- $d_{cw}t = 0.12$ kN/m$^2$

**Reinforcement allowance**
- $r_{ew}t = 0.04$ kN/m$^2$

**Steel beam allowance**
- $s_{elf}w = 0.15$ kN/m$^2$

**Construction load allowance**
- $w_c = 0.5$ kN/m$^2$

**Other UDL on beam**
- $w'd_u = 0.33$ kN/m

**Ceiling and services**
- $w_{cs} = 0.25$ kN/m$^2$

**Imposed**
- $w_i = 3$ kN/m$^2$

**Partitions**
- $w_p = 1$ kN/m$^2$

**Wall or other UDL**
- $w'd_u = 7.83$ kN/m

---
Number of studs across flange \( nr=1 \)
Spacing of connectors \( spac1=400 \) mm

Check on transverse reinforcement omitted.

Total permanent load \( Vfp=18 \) kN

| COMPOSITE | 305 x 102 x 28 UB Grade S 355 |
| SECTION   | with 16 No. 19 dia. stud connectors @ 375 centres. |
| SUMMARY   | Section is satisfactory for construction, composite, elastic, plastic, moment, shear, deflection and natural frequency requirements. |

| Construction stage moment | 45.313 kNm |
| Steel beam mom. capacity  | 143.07 kNm |
| Const. stage deflection   | 9.055 mm   |
| Composite stage moment    | 162.15 kNm |
| Ultimate mom. capacity    | 238.69 kNm |
| Vertical shear force      | 89.296 kN  |
| Ult. shear capacity       | 394.52 kN  |
| Composite stage deflection| 7.1117 mm  |

| FIRE | Exposed surface area | 910.8 mm² |
| RESISTANCE | Area of section | 35.9 cm² |
| SUMMARY | Section factor | 250 /m |
| Load Ratio (R) | 0.44644 |

Section does not have an inherent fire resistance of 30 minutes.

Fire protection should be provided.
Location: Ex1 - Beam on grid line A3

Composite edge beam with secondary beams located at 1/3 points. Design to BS EN 1994-1-1:2004 the Code for the 'Design of composite steel and concrete structures'. The beam is unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 6 \) m
Secondary beam span \( L_2 = 6 \) m
Concrete density (const. stage) \( D_c' = 19 \) kN/m³
Concrete density (final stage) \( D_C = 18 \) kN/m³
Deck is parallel to supporting beam and supported on secondary beams.
305 x 102 x 28 UKB
Dimensions (mm): \( h = 308.7 \), \( b = 101.8 \), \( t_w = 6 \), \( t_f = 8.8 \), \( r = 7.6 \)
Properties (cm): \( I_y = 5370 \), \( I_z = 155 \), \( W_{pl_y} = 403 \), \( W_{pl_z} = 48.5 \), \( I_t = 7.4 \)
Partial safety factor (reinforcement) \( \gamma_{M0} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{M1} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M2} = 1.0 \)
Partial safety factor (welding) \( \gamma_{v} = 1.25 \)
Permanent load (ULS) \( \gamma_{G} = 1.25 \)
Variable load (ULS) \( \gamma_{Q} = 1.5 \)

Depth of concrete above profile \( h_f = 80 \) mm
Depth of profiling \( h_p = 50 \) mm
Centres of profiles \( e_p = 300 \) mm
Width at inside of profile \( b_1 = 125 \) mm
Width at outside of profile \( b_2 = 175 \) mm
Thickness of sheeting \( t_p = 1.2 \) mm
Mean width of concrete ribs \( b_0 = 125 \text{ mm} \)
Weight of steel decking \( f_{sd} = 0.12 \text{ kN/m}^2 \)
Variable load (const. stage) \( f_c = 0.75 \text{ kN/m}^2 \)
Variable load (final stage) \( f_i = 3 \text{ kN/m}^2 \)
Partition load \( f_p = 1 \text{ kN/m}^2 \)
Ceiling and services \( f_{cs} = 0.25 \text{ kN/m}^2 \)
Raised floor \( f_r = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( f_w = 7.83 \text{ kN/m} \)
Self weight of secondary beam \( f_b = 0.2 \text{ kN/m} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Area of reinft./per shear plane \( A_e = 371 \text{ mm}^2 / \text{m} \)

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>STAGE</th>
<th>305 x 102 x 28 UKB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONSTRUCTION</td>
<td>Steel yield strength 355 N/mm²</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>25 N/mm²</td>
</tr>
<tr>
<td>Maximum moment</td>
<td>51.6 kNm</td>
</tr>
<tr>
<td>Moment resistance</td>
<td>143 kNm</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.361 ≤ 1</td>
</tr>
<tr>
<td>Buckling resistance</td>
<td>129 kNm</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.401 ≤ 1</td>
</tr>
<tr>
<td>Maximum shear force</td>
<td>26 kN</td>
</tr>
<tr>
<td>Shear resistance</td>
<td>407 kN</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.064 ≤ 1</td>
</tr>
</tbody>
</table>

**DESIGN SUMMARY**

| FINAL STAGE | 0.75 m |
| Stud shear resistance | 72.6 kN |
| Number of studs per rib | 1 No. |
| Number of 19 mm studs | see footnote |
| Stud spacing between | see footnote |

**Design compressive force:**

| Concrete | 850 kN |
| Studs | 436 kN |
| Shear connection ratio | 0.513 |
| Partial shear connection provided | |
| Maximum moment | 158 kNm |
| Moment resistance | 226 kNm |
| Unity factor bending | 0.698 ≤ 1 |
| Maximum shear force | 86.5 kN |
| Shear plastic resistance | 407 kN |
| Unity factor shear | 0.213 ≤ 1 |

**Deflection to span ratio**

| (G + Q) | 1: 447 |

**Stud spacing between each support and 1/3 point to be 330 mm.**
The number of studs required between each support & 1/3 point is 6 No. Between 1/3 points provide nominal studs.
Location: Ex2

Composite edge beam with secondary beams located at 1/3 points. Design to BS EN 1994-1-1:2004 the Code for the 'Design of composite steel and concrete structures'. The beam is unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 6 \) m
Secondary beam span \( L_2 = 6 \) m
Concrete density (const. stage) \( D_c' = 19 \) kN/m\(^3\)
Concrete density (final stage) \( D_c = 18 \) kN/m\(^3\)
Deck is parallel to supporting beam and supported on secondary beams.
305 x 102 x 28 UKB
Dimensions (mm): \( h = 308.7 \), \( b = 101.8 \), \( tw = 6 \), \( tf = 8.8 \), \( r = 7.6 \)
Properties (cm): \( I_y = 5370 \), \( I_z = 155 \), \( W_{ply} = 403 \), \( W_{plz} = 48.5 \), \( I_t = 7.4 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{am} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{av} = 1.25 \)
Permanent load (ULS) \( \gamma_{G} = 1.25 \)
Variable load (ULS) \( \gamma_{Q} = 1.5 \)

Depth of concrete above profile \( hf = 80 \) mm
Depth of profiling \( hp = 50 \) mm
Centres of profiles \( ep = 300 \) mm
Width at inside of profile \( b1 = 125 \) mm
Width at outside of profile \( b2 = 175 \) mm
Thickness of sheeting \( tp = 1.2 \) mm
Mean width of concrete ribs $b_0=125\,\text{mm}$
Weight of steel decking $f_{sd}=0.12\,\text{kN/m}^2$
Variable load (const. stage) $f_{c}=0.5\,\text{kN/m}^2$
Variable load (final stage) $f_i=3\,\text{kN/m}^2$
Partition load $f_p=1\,\text{kN/m}^2$
Ceiling and services $f_{cs}=0.25\,\text{kN/m}^2$
Raised floor $f_r=0\,\text{kN/m}^2$
Wall or other UDL (final stage) $f_w=7.83\,\text{kN/m}$
Self weight of secondary beam $f_b=0.2\,\text{kN/m}$
Char yield strength of reinft. $f_{yk}=500\,\text{N/mm}^2$
Area of reinfnt. per shear plane $A_e=371\,\text{mm}^2/\text{m}$

**DESIGN SUMMARY**

**305 x 102 x 28 UKB Grade S 355**

**CONSTRUCTION**

Steel yield strength $355\,\text{N/mm}^2$
Concrete strength $25\,\text{N/mm}^2$
Maximum moment $47.1\,\text{kNm}$
Moment resistance $143\,\text{kNm}$
Unity factor $0.329 \leq 1$
Buckling resistance $129\,\text{kNm}$
Unity factor $0.366 \leq 1$
Maximum shear force $23.8\,\text{kN}$
Shear resistance $407\,\text{kN}$
Unity factor $0.058 \leq 1$
Deflection to span ratio $(G + Q)$ $1:484$

**DESIGN SUMMARY**

**FINAL STAGE**

Effective width $0.75\,\text{m}$
Stud shear resistance $72.6\,\text{kN}$
Number of studs per rib 1 No.
Number of 19 mm studs see footnote
Stud spacing between see footnote
Design compressive force:
Concrete $850\,\text{kN}$
Studs $436\,\text{kN}$
Shear connection ratio $0.513$
Partial shear connection provided
Maximum moment $158\,\text{kNm}$
Moment resistance $226\,\text{kNm}$
Unity factor bending $0.698 \leq 1$
Maximum shear force $86.5\,\text{kN}$
Shear plastic resistance $407\,\text{kN}$
Unity factor shear $0.213 \leq 1$
Deflection to span ratio at final stage $(G + Q)$ $1:316$
Slab shear force $220\,\text{kN/m}$
Slab resistance $221\,\text{kN/m}$
Min transverse reinft. $371\,\text{mm}^2/\text{m}$
Lightweight concrete (LWC) assumed

Stud spacing between each support and 1/3 point to be 330 mm.
The number of studs required between each support & 1/3 point is 6 No. Between 1/3 points provide nominal studs.
Location: Ex3

Composite edge beam with secondary beams located at 1/3 points. Design to BS EN 1994-1-1:2004 the Code for the 'Design of composite steel and concrete structures'. The beam is unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 6 \text{ m} \)
Secondary beam span \( L_2 = 6 \text{ m} \)
Concrete density (const. stage) \( D'_c = 19 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 18 \text{ kN/m}^3 \)
Deck is parallel to supporting beam and supported on secondary beams.
305 x 102 x 28 UKB
Dimensions (mm): \( h = 308.7 \), \( b = 101.8 \), \( tw = 6 \), \( tf = 8.8 \), \( r = 7.6 \)
Properties (cm): \( I_y = 5370 \), \( I_z = 155 \), \( W_{ply} = 403 \), \( W_{plz} = 48.5 \), \( I_t = 7.4 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{amc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{am0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{am1} = 1.0 \)
Partial safety factor (studs) \( \gamma_{mv} = 1.25 \)
Permanent load (ULS) \( \gamma_{G} = 1.25 \)
Variable load (ULS) \( \gamma_{Q} = 1.5 \)

Depth of concrete above profile \( hf = 80 \text{ mm} \)
Depth of profiling \( hp = 50 \text{ mm} \)
Centres of profiles \( ep = 300 \text{ mm} \)
Width at inside of profile \( b_1 = 125 \text{ mm} \)
Width at outside of profile \( b_2 = 175 \text{ mm} \)
Thickness of sheeting \( tp = 1.2 \text{ mm} \)
Mean width of concrete ribs \( b_0 = 125 \text{ mm} \)
Weight of steel decking \( f_{sd} = 0.12 \text{ kN/m}^2 \)
Variable load (const. stage) \( f_c = 0.5 \text{ kN/m}^2 \)
Variable load (final stage) \( f_i = 3 \text{ kN/m}^2 \)
Partition load \( f_p = 1 \text{ kN/m}^2 \)
Ceiling and services \( f_{cs} = 0.25 \text{ kN/m}^2 \)
Raised floor \( f_r = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( f_w = 7.83 \text{ kN/m} \)
Self weight of secondary beam \( f_b = 0.2 \text{ kN/m} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Area of reinfnt. per shear plane \( A_e = 371 \text{ mm}^2/\text{m} \)

**DESIGN SUMMARY**
- Steel yield strength 355 N/mm²
- Concrete strength 25 N/mm²
- Maximum moment 47.1 kNm
- Moment resistance 143 kNm
- Unity factor 0.329 ≤ 1
- Buckling resistance 129 kNm
- Unity factor 0.366 ≤ 1
- Maximum shear force 23.8 kN
- Shear resistance 407 kN
- Unity factor 0.058 ≤ 1
- Deflection to span ratio \((G + Q)\) 1: 484

**FINAL STAGE**
- Effective width 0.75 m
- Stud shear resistance 72.6 kN
- Number of studs per rib 1 No.
- Number of 19 mm studs see footnote
- Stud spacing between see footnote
- Design compressive force:
  - Concrete 850 kN
  - Studs 436 kN
  - Shear connection ratio 0.513
  - Partial shear connection provided
  - Maximum moment 158 kNm
  - Moment resistance 226 kNm
  - Unity factor bending 0.698 ≤ 1
  - Maximum shear force 86.5 kN
  - Shear plastic resistance 407 kN
  - Unity factor shear 0.213 ≤ 1
  - Deflection to span ratio at final stage \((G + Q)\) 1: 316

Lightweight concrete (LWC) assumed

Stud spacing between each support and 1/3 point to be 330 mm. The number of studs required between each support & 1/3 point is 6 No. Between 1/3 points provide nominal studs.
Location: Ex4 - solid slab

Composite edge beam with secondary beams located at 1/3 points. Design to BS EN 1994-1-1:2004 the Code for the 'Design of composite steel and concrete structures'. The beam is unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 6 \text{ m} \)
Secondary beam span \( L_2 = 6 \text{ m} \)
Concrete density (const. stage) \( D'_c = 19 \text{ kN/m}^3 \)
Concrete density (final stage) \( D_c = 18 \text{ kN/m}^3 \)
Dimensions (mm): \( h = 308.7 \text{ b = 101.8} \text{ tw = 6} \text{ tf = 8.8} r = 7.6 \)
Properties (cm): \( I_y = 5370 \text{ I}_z = 155 \text{ W}_p\text{ly} = 403 \text{ W}_\text{plz} = 48.5 \text{ I}_t = 7.4 \)
Partial safety factor (reinfnt.) \( \gamma_{ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_c = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (studs) \( \gamma_v = 1.25 \)
Permanent load (ULS) \( \gamma_G = 1.5 \)
Variable load (ULS) \( \gamma_Q = 1.5 \)
Variable load (const. stage) \( f_c = 0.5 \text{ kN/m}^2 \)
Variable load (final stage) \( f_i = 3 \text{ kN/m}^2 \)
Partition load \( f_p = 1 \text{ kN/m}^2 \)
Ceiling and services \( f_{cs} = 0.25 \text{ kN/m}^2 \)
Raised floor \( f_r = 0 \text{ kN/m}^2 \)
Wall or other UDL (final stage) \( f_w = 7.83 \text{ kN/m} \)
Self weight of secondary beam \( f_b = 0.2 \text{ kN/m} \)
Char yield strength of reinft. \( f_y = 500 \text{ N/mm}^2 \)
Governing shear length \( h_f = 150 \text{ mm} \)
Area of reinft. per shear plane \( A_e = 505 \text{ mm}^2 / \text{m} \)
### DESIGN SUMMARY

- 305 x 102 x 28 UKB Grade S 355

### CONSTRUCTION

- Steel yield strength: 355 N/mm²
- Concrete strength: 25 N/mm²
- Maximum moment: 59.5 kNm
- Moment resistance: 143 kNm
- Unity factor: 0.416 ≤ 1
- Buckling resistance: 129 kNm
- Unity factor: 0.463 ≤ 1
- Maximum shear force: 30 kN
- Shear resistance: 407 kN
- Unity factor: 0.074 ≤ 1

### STAGE

- Deflection to span ratio (G + Q): 1:380

### DESIGN SUMMARY

- Effective width: 0.75 m

### FINAL STAGE

- Stud shear resistance: 72.6 kN
- Number of 19 mm studs: see footnote
- Stud spacing between: see footnote

### Design compressive force:

- Concrete: 1275 kN
- Studs: 654 kN
- Partial shear connection ratio: 0.513

### Design shear force:

- Maximum moment: 170 kNm
- Unity factor bending: 0.708 ≤ 1
- Maximum shear force: 92.8 kN
- Shear plastic resistance: 407 kN
- Unity factor shear: 0.228 ≤ 1

- Deflection to span ratio at final stage (G + Q): 1:283
- Slab shear force: 330 kN/m
- Slab resistance: 331 kN/m
- Min transverse reinft.: 505 mm²/m

Lightweight concrete (LWC) assumed

Stud spacing between each support and 1/3 point to be 220 mm.
The number of studs required between each support & 1/3 point
is 9 No. Between 1/3 points provide nominal studs.
Location: Ex5 - Beam on grid line A3

Composite edge beam with secondary beams located at 1/3 points. Design to BS EN 1994-1-1:2004 the Code for the 'Design of composite steel and concrete structures'. The beam is unpropped during construction and is assumed to be simply supported.

Beam span \( L_1 = 6 \) m
Secondary beam span \( L_2 = 6 \) m
Concrete density (const. stage) \( D'_c = 19 \) kN/m³
Concrete density (final stage) \( D_c = 18 \) kN/m³
Deck is parallel to supporting beam and supported on secondary beams.

305 x 102 x 28 UKB

Dimensions (mm): \( h = 308.7 \) \( b = 101.8 \) \( t_w = 6 \) \( t_f = 8.8 \) \( r = 7.6 \)
Properties (cm): \( I_y = 5370 \) \( I_z = 155 \) \( W_{ply} = 403 \) \( W_{plz} = 48.5 \) \( I_t = 7.4 \)
Partial safety factor (reinfnt.) \( \gamma_{Ams} = 1.15 \)
Partial safety factor (concrete) \( \gamma_{Mc} = 1.5 \)
Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
Partial safety factor (studs) \( \gamma_v = 1.25 \)
Permanent load (ULS) \( \gamma_G = 1.25 \)
Variable load (ULS) \( \gamma_Q = 1.5 \)

Depth of concrete above profile \( h_f = 80 \) mm
Depth of profiling \( h_p = 50 \) mm
Centres of profiles \( e_p = 300 \) mm
Width at inside of profile \( b_1 = 125 \) mm
Width at outside of profile \( b_2 = 175 \) mm
Thickness of sheeting \( t_p = 1.2 \) mm
<table>
<thead>
<tr>
<th>Mean width of concrete ribs</th>
<th>b0=125 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of steel decking</td>
<td>fsd=0.12 kN/m²</td>
</tr>
<tr>
<td>Variable load (const.stage)</td>
<td>fc=0.75 kN/m²</td>
</tr>
<tr>
<td>Variable load (final stage)</td>
<td>fi=3 kN/m²</td>
</tr>
<tr>
<td>Partition load</td>
<td>fp=1 kN/m²</td>
</tr>
<tr>
<td>Ceiling and services</td>
<td>fcs=0.25 kN/m²</td>
</tr>
<tr>
<td>Raised floor</td>
<td>fr=0 kN/m²</td>
</tr>
<tr>
<td>Wall or other UDL (final stage)</td>
<td>fw=7.83 kN/m</td>
</tr>
<tr>
<td>Self weight of secondary beam</td>
<td>fb=0.2 kN/m</td>
</tr>
<tr>
<td>Char yield strength of reinft.</td>
<td>fyk=500 N/mm²</td>
</tr>
<tr>
<td>Area of reinfnt.per shear plane</td>
<td>Ae=198 mm²/m</td>
</tr>
</tbody>
</table>

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>305 x 102 x 28 UKB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel yield strength</td>
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<tr>
<td>Concrete strength</td>
</tr>
<tr>
<td>Maximum moment</td>
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<tr>
<td>Moment resistance</td>
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<tr>
<td>Unity factor</td>
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<tr>
<td>Buckling resistance</td>
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<tr>
<td>Unity factor</td>
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<tr>
<td>Maximum shear force</td>
</tr>
<tr>
<td>Shear resistance</td>
</tr>
<tr>
<td>Unity factor</td>
</tr>
</tbody>
</table>

**CONSTRUCTION STAGE**

| (G + Q) | 1: 447 |
|---------------------------------|
| Effective width                  | 0.75 m  |
| Stud shear resistance            | 72.6 kN  |
| Number of studs per rib          | 1 No.   |
| Number of 19 mm studs            | see footnote |
| Stud spacing between             | see footnote |

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Final Stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compressive force</td>
</tr>
<tr>
<td>Studs</td>
</tr>
<tr>
<td>Shear connection ratio</td>
</tr>
<tr>
<td>Partial shear connection provided</td>
</tr>
<tr>
<td>Maximum moment</td>
</tr>
<tr>
<td>Moment resistance</td>
</tr>
<tr>
<td>Unity factor bending</td>
</tr>
<tr>
<td>Maximum shear force</td>
</tr>
<tr>
<td>Shear plastic resistance</td>
</tr>
<tr>
<td>Unity factor shear</td>
</tr>
<tr>
<td>Deflection to span ratio at final stage (G + Q)</td>
</tr>
<tr>
<td>Slab shear force</td>
</tr>
<tr>
<td>Slab resistance</td>
</tr>
<tr>
<td>Min transverse reinft.</td>
</tr>
<tr>
<td>Lightweight concrete (LWC) assumed</td>
</tr>
</tbody>
</table>

Stud spacing between each support and 1/3 point to be 500 mm. The number of studs required between each support & 1/3 point is 4 No. Between 1/3 points provide nominal studs.
Lateral restraint from steel decking - construction phase

The following calculations are in accordance with EC3 Parts 1-1, 1-3 and SCI P360 entitled 'Stability of Steel Beams and Columns'. The objective is to investigate whether during the construction phase (prior to concrete hardening), the steel decking offers sufficient lateral restraint to the beams such that the beams may be considered as fully restraint against lateral torsional buckling.

Design assumptions:

- the steel decking is perpendicular to the beam span and is attached at every rib to the top flange of beams
- the composite floor system comprises trapezoidal steel decking and an in-situ cast concrete
- more than two equally spaced beams are present.

Beam span L=8000 mm
Length of sheeting ⊥ to beams br=6000 mm (overall length)
Steel decking thickness t=1 mm
Decking profile height hw=60 mm

254 x 102 x 22 UKB
Dimensions (mm): h=254 b=101.6 tw=5.7 tf=6.8 r=7.6
Properties (cm): Iy=2840 Iz=119 Wply=259 Wplz=37.3 It=4.15

Available sheeting stiffness - BS EN 1993-1-3, Clause 10.1.1(10)

The shear stiffness of trapezoidal sheeting is given by:

\[ S = 1000 \times (t)^3 \times (50 + 10 \times (br) \times 0.3333) \times s / (hw \times 1000) \]
\[ = 11583 \text{ kN per unit length} \]

As \( S \geq S_{req} \) (11583 kN \( \geq 4960.4 \) kN), the floor beams may be considered to be laterally restrained by the sheeting during construction phase and therefore the beam may be assumed to be laterally restrained.
UNIVERSAL BEAM

254 x 102 x 22 UKB

Beam span 8000 mm

Length of sheeting \( \perp \) to beams 6000 mm

DESIGN

Beam spacing 3000 mm

SUMMARY

Decking thickness 1 mm

Decking profile height 60 mm

Required sheeting stiffness 4960 kN

Available sheeting stiffness 11583 kN

NOTE: The steel decking is perpendicular to the beam span and is attached at every rib to the top flange of beams. The composite floor system comprises trapezoidal steel decking and an in-situ cast concrete.
Location: First floor - Small UB

Composite steel deck floor

<table>
<thead>
<tr>
<th>Section properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Ds               Calculations in accordance with the 'Design recommendations for composite floors and beams using steel decks' published by the Steel Construction Institute in October 83.</td>
</tr>
<tr>
<td>Db</td>
</tr>
</tbody>
</table>

Effective width of comp. flange be=1500 mm
Concrete cube strength fcu=25 N/mm²
Air dry density for concrete Dc=17.6 kN/m³
Modular ratio for steel/concrete ALPe=21
305 x 102 x 33 UB.
Young's Modulus E=205 kN/mm²
Slab depth Ds=115 mm
Depth of profiling Dp=50 mm

SECTION 305 x 102 x 33 UB Grade S 355
SUMMARY
Slab depth 115 mm
Depth of profiling 50 mm
Effective width of comp. flange 1500 mm
Concrete cube strength 25 N/mm²
Modular ratio for steel/concrete 21
Ultimate moment of resistance 321.47 kNm
Web shear capacity 439.59 kN
2nd mom of area of comp.section 19212 cm⁴
Elastic moduli of composite section
Bottom of steel beam flange 681.19 cm³
Top of concrete slab 27699 cm³
Location: Small UB with metal decking

Composite steel deck floor

Section properties

Effective width of comp. flange  beff=1500 mm
Concrete density (final stage)  Dc=18 kN/m³
Dimensions (mm): h=312.7 b=102.4 tw=6.6 tf=10.8 r=7.6
Properties (cm): Iy=6500 Iz=194 Wply=481 Wplz=60 It=12.2
Depth of concrete above profile  hf=65 mm
Depth of profiling             hp=50 mm

SECTION  305 x 102 x 33 UB Grade S 355
SUMMARY
Overall slab depth 115 mm
Depth of profiling 50 mm
Effective width of comp.flange 1500 mm
Char.cylinder comp.strength 25 N/mm²
Modular ratio for steel/concrete 13.96
Plastic moment of resistance 343.31 kNm
Shear plastic resistance 451.65 kN
2nd mom of area of comp.section 21664 cm⁴
Elastic moduli of comp.section:
(1) Bottom of steel beam flange 708.51 cm³
(2) Top of concrete slab 24804 cm³
Lightweight concrete (LWC) assumed
Location: Example 1 - Universal beam example

Calculations in accordance with BS5950 Part 3 Section 3.1 the 'Code of practice for design of simple and continuous composite beams' July 1990.

Beam is an internal beam.
Factored B.M. in compos. stage \( M_{\text{com}} = 320 \text{ kNm} \)
Design ultimate shear \( V_r = 200 \text{ kN} \)
Beam span \( L_1 = 6 \text{ m} \)
Beam centres \( L_2 = 3 \text{ m} \)
Concrete cube strength \( f_{\text{cu}} = 25 \text{ N/mm}^2 \)
Air dry density for concrete \( D_c = 17.6 \text{ kN/m}^3 \)
Fresh wet density for concrete \( D'c = 18.8 \text{ kN/m}^3 \)

Deck spans onto supporting beam. Modular ratio for steel/concrete \( A \lambda L_{\text{pe}} = 15 \)
305 x 165 x 40 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Slab depth \( D_s = 115 \text{ mm} \)
Centres of profiles \( C_t = 300 \text{ mm} \)
Depth of profiling \( D_p = 50 \text{ mm} \)
Width at inside of profile \( b = 125 \text{ mm} \)
Width at outside of profile \( p = 175 \text{ mm} \)
Composite beam deflection (SLS)  DELf=6 mm  
Number of studs across flange  nr=1  
Percentage to be provided  per=80 %  

COMPOSITE  305 x 165 x 40 UB Grade S 355  
SECTION  with 28 No. 19 dia. stud connectors  
SUMMARY  @ 214 centres.  
Section is satisfactory for  
plastic moment, shear, deflection  
requirements.  
Steel beam mom. capacity  221.17 kNm  
Composite stage moment  320 kNm  
Ultimate mom. capacity  355.94 kNm  
Vertical shear force  200 kN  
Ult. shear capacity  387.75 kN  
Composite stage deflection 6.5962 mm
Location: Example 2 - RHS example

Calculations in accordance with BS5950 Part3 Section3.1 the 'Code of practice for design of simple and continuous composite beams' July 1990.

Beam is an internal beam.
Factored B.M. in compos.stage \( M_{com}=320 \text{kNm} \)
Design ultimate shear \( V_r=200 \text{kN} \)
Beam span \( L_1=6 \text{ m} \)
Beam centres \( L_2=3 \text{ m} \)
Concrete cube strength \( f_{cu}=25 \text{ N/mm}^2 \)
Air dry density for concrete \( D_c=17.6 \text{ kN/m}^3 \)
Freshwet density for concrete \( D'_c=18.8 \text{ kN/m}^3 \)
Modular ratio for steel/concrete \( A_{LPe}=15 \)
300 x 200 x 10 RHS - Hot finished.
Properties (cm): \( A=94.9 \text{ cm}^2 \), \( r_x=11.2 \text{ cm} \), \( Z_x=788 \text{ cm} \), \( S_x=956 \text{ cm} \), \( I_x=11800 \text{ cm}^4 \)
\( J=12900 \text{ cm} \), \( C=1020 \text{ cm} \), \( Z_y=628 \text{ cm} \), \( S_y=721 \text{ cm} \), \( I_y=6280 \text{ cm}^4 \), \( r_y=8.13 \text{ cm} \)
Young's Modulus \( E=205 \text{ kN/mm}^2 \)
Composite beam deflection (SLS)  \( \text{DELF} = 6 \text{ mm} \)
Number of studs across flange  \( n_r = 1 \)
Percentage to be provided  \( \text{per} = 80\% \)

**COMPOSITE**

Hot finished RHS to BS EN 10210

**SECTION**

300 x 200 x 10 Grade S 355

with 62 No. 19 dia. stud connectors

@ 96 centres.

**SUMMARY**

Section is satisfactory for plastic moment, shear, deflection requirements.

Steel beam mom. capacity  335.69 kNm
Composite stage moment  320 kNm
Ultimate mom. capacity  684.7 kNm
Vertical shear force  200 kN
Ult. shear capacity  1212.8 kN
Composite stage deflection  6.9059 mm
Location: Example 3 - Channel example

Calculations in accordance with BS5950 Part3 Section3.1 the 'Code of practice for design of simple and continuous composite beams' July 1990.

Beam is an internal beam.
Factored B.M. in compos.stage \( M_{\text{com}} = 320 \text{ kNm} \)
Design ultimate shear \( V_r = 200 \text{ kN} \)
Beam span \( L_1 = 6 \text{ m} \)
Beam centres \( L_2 = 3 \text{ m} \)
Concrete cube strength \( f_{\text{cu}} = 25 \text{ N/mm}^2 \)
Air dry density for concrete \( D_c = 17.6 \text{ kN/m}^3 \)
Freshet density for concrete \( D'_c = 18.8 \text{ kN/m}^3 \)
Modular ratio for steel/concrete \( A_{\text{LP}e} = 15 \)
305 x 89 Tapered Channel.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Slab depth \( D_s = 150 \text{ mm} \)
Composite beam deflection (SLS) \( \text{DELF} = 6 \text{ mm} \)
Number of studs across flange \( nr = 1 \)
Percentage to be provided \( \text{per} = 80 \% \)

<table>
<thead>
<tr>
<th>COMPOSITE</th>
<th>305 x 89 TFC Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>with 46 No. 19 dia. stud connectors @ 130 centres.</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Section is satisfactory for plastic moment, shear, deflection requirements.</td>
</tr>
<tr>
<td>Steel beam mom. capacity</td>
<td>197.85 kNm</td>
</tr>
<tr>
<td>Composite stage moment</td>
<td>320 kNm</td>
</tr>
<tr>
<td>Ultimate mom. capacity</td>
<td>450.21 kNm</td>
</tr>
<tr>
<td>Vertical shear force</td>
<td>200 kN</td>
</tr>
<tr>
<td>Ult. shear capacity</td>
<td>662.21 kN</td>
</tr>
<tr>
<td>Composite stage deflection</td>
<td>7.0506 mm</td>
</tr>
</tbody>
</table>
Location: Ex1 - Universal beam example with metal decking

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

The beam under consideration is an internal beam.

Design B.M. (compos.stage) MyEd=320 kNm
Design shear force (comp.stage) VcEd=200 kN
Concrete density (const.stage) D'c=19 kN/m³
Concrete density (final stage) Dc=18 kN/m³

Floor Plan

Beam span L1=6 m
Beam cross-centres C=3 m
Deck spans onto the supporting internal beam under consideration.

305 x 165 x 40 UKB
Dimensions (mm): h=303.4 b=165 tw=6 tf=10.2 r=8.9
Properties (cm): Iy=8500 Iz=764 Wply=623 Wplz=142 It=14.7

Material partial factors

Partial safety factor (reinfnt.) gams=1.15
Partial safety factor (concrete) gamc=1.5
Partial safety factor (steel) gamM0=1.0
Partial safety factor (steel) gamM1=1.0
Partial safety factor (studs) gamv=1.25
Concrete slab details

Depth of concrete above profile \( hf = 80 \text{ mm} \)
Depth of profiling \( hp = 50 \text{ mm} \)
Centres of profiles \( ep = 300 \text{ mm} \)
Width at inside of profile \( b_1 = 125 \text{ mm} \)
Width at outside of profile \( b_2 = 175 \text{ mm} \)
Thickness of sheeting \( tp = 1.2 \text{ mm} \)
Mean width of concrete ribs \( b_0 = 150 \text{ mm} \)

Composite section – elastic section properties

Inertia of combined section (steel units)
\( I_{na} = 27727 \text{ cm}^4 \)

Composite beam deflection (SLS)
\( dv = 6 \text{ mm} \)

Plastic moment of resistance
\( M_{pl} = M_{plc} + M_{plf} + M_{plb} + M_{plw} + M_{pll} = 364.44 \text{ kNm} \)

Shear plastic resistance
\( V_{plEd} = V_{plRd} = 411.3 \text{ kN} \)

Char yield strength of reinft.
\( f_{yk} = 500 \text{ N/mm}^2 \)
<table>
<thead>
<tr>
<th>COMPOSITE</th>
<th>305 x 165 x 40 UKB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>with 40 No. 19 dia. stud connectors</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>@ 300 centres.</td>
</tr>
<tr>
<td></td>
<td>Studs to be grouped in 2 's.</td>
</tr>
<tr>
<td></td>
<td>Section is satisfactory for plastic moment, shear &amp; deflection requirements.</td>
</tr>
<tr>
<td>Steel beam mom.capacity</td>
<td>221.17 kNm</td>
</tr>
<tr>
<td>Composite stage moment</td>
<td>320 kNm</td>
</tr>
<tr>
<td>Plastic moment resistance</td>
<td>364.44 kNm</td>
</tr>
<tr>
<td>Vertical shear resistance</td>
<td>200 kN</td>
</tr>
<tr>
<td>Design shear capacity</td>
<td>411.3 kN</td>
</tr>
<tr>
<td>Long.shear stress</td>
<td>2.1791 N/mm²</td>
</tr>
<tr>
<td>Long.shear resistance</td>
<td>2.5116 N/mm²</td>
</tr>
<tr>
<td>Min transverse reinfnt.</td>
<td>199.91 mm²/mm</td>
</tr>
<tr>
<td>Variable load deflection</td>
<td>6 mm</td>
</tr>
<tr>
<td>Lightweight concrete (LWC)</td>
<td>assumed</td>
</tr>
</tbody>
</table>

Lightweight concrete (LWC) assumed
Location: Ex2 - RHS example with solid slab

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

The beam under consideration is an internal beam.

Design B.M. (compos.stage) MyEd=320 kNm
Design shear force (comp.stage) VcEd=200 kN
Concrete density (const.stage) D'c=19 kN/m³
Concrete density (final stage) Dc=18 kN/m³
Beam span L1=6 m
300 x 200 x 10 RHS - Hot finished.
Properties (cm): A=94.9 iz=8.13 Wely=788 Wply=956 Iy=11800
                     It=12900 Wt=1020 Welz=628 Wplz=721 Iz=6280 iy=11.2

Material partial factors

Partial safety factor (reinfnt.) gamrs=1.15
Partial safety factor (concrete) gamc=1.5
Partial safety factor (steel) gamM0=1.0
Partial safety factor (steel) gamM1=1.0
Partial safety factor (studs) gamv=1.25

Concrete slab details
Composite section – elastic section properties

Inertia of combined section (steel units)

Composite beam deflection (SLS)

Plastic moment of resistance

Shear plastic resistance

Governing shear length

Char yield strength of reinft.

Hot finished RHS to BS EN 10210

COMPOSITE

SECTION

SUMMARY

Section is satisfactory for plastic moment, shear & deflection requirements.

Steel beam mom. capacity

Composite stage moment

Plastic moment resistance

Vertical shear resistance

Design shear capacity

Long. shear stress

Long. shear resistance

Min transverse reinft.

Variable load deflection

Lightweight concrete (LWC) assumed
Location: Ex3 - Channel example with solid slab

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

The beam under consideration is an internal beam.
Design B.M. (compos.stage) MyEd=320 kNm
Design shear force (comp.stage) VcEd=200 kN
Concrete density (const.stage) D'c=19 kN/m³
Concrete density (final stage) Dc=18 kN/m³
Beam span L1=6 m
300 x 100 UKPFC
Dimensions (mm): h=300 b=100 tw=9 tf=16.5 r=15
Properties (cm): Iy=8230 Iz=568 Wply=641 Wplz=148 It=36.8

Material partial factors

Partial safety factor (reinfnt.) gams=1.15
Partial safety factor (concrete) gamc=1.5
Partial safety factor (steel) gamM0=1.0
Partial safety factor (steel) gamM1=1.0
Partial safety factor (studs) gamv=1.25

Concrete slab details
Composite section - elastic section properties

\[\begin{align*}
\text{Steel deck} & \quad \text{Steel section is UKPFC} \\
\text{tf} & \quad \text{ha} \\
\text{tw} & \quad \text{h} \\
\text{beff} & \quad \text{Inertia of combined section (steel units)}
\end{align*}\]

\[I_n = 32830 \text{ cm}^4\]

Composite beam deflection (SLS) \[d_v = 6 \text{ mm}\]

Plastic moment of resistance \[M_p = M_{plc} + M_{plf} + M_{plb} + M_{plw} + M_{pll} = 351.81 \text{ kNm}\]

Shear plastic resistance \[V_{plEd} = V_{plRd} = 576.84 \text{ kN}\]

Governing shear length \[h_f = 114.25 \text{ mm}\]

Char yield strength of reinft. \[f_{yk} = 500 \text{ N/mm}^2\]

**COMPOSITE** 300 x 100 UKPFC Grade S 355

**SECTION** with 24 No. 19 dia. stud connectors @ 250 centres.

**SUMMARY** Section is satisfactory for plastic moment, shear & deflection requirements.

Steel beam mom. capacity 221.15 kNm

Composite stage moment 320 kNm

Plastic moment resistance 351.81 kNm

Vertical shear resistance 200 kN

Design shear capacity 576.84 kN

Long. shear stress 1.2715 N/mm²

Long. shear resistance 2.5116 N/mm²

Min transverse reinfnt. 228.5 mm²/mm

Variable load deflection 6 mm

Lightweight concrete (LWC) assumed
Location: Ex4 - UB with decking & one secondary beam @ mid-span

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

The beam under consideration is an internal beam.
Design B.M. (compos.stage) MyEd=320 kNm
Design shear force (comp.stage) VcEd=200 kN
Concrete density (const.stage) D'c=19 kN/m³
Concrete density (final stage) Dc=18 kN/m³
Deck is parallel to the supporting beam.

Deck ribs are parallel to B1
B1 = beam being considered
L2 B2 = secondary beam
C = L1/2

Floor Plan

Beam span L1=6 m
Sum of secondary beam spans L2=6 m
305 x 165 x 40 UKB
Dimensions (mm): h=303.4 b=165 tw=6 tf=10.2 r=8.9
Properties (cm): Iy=8500 Iz=764 Wply=623 Wplz=142 It=14.7

Material partial factors

Partial safety factor (reinfnt.) gams=1.15
Partial safety factor (concrete) gamc=1.5
Partial safety factor (steel) gamM0=1.0
Partial safety factor (steel) gamM1=1.0
Partial safety factor (studs) gamv=1.25
Concrete slab details

- Depth of concrete above profile: $h_f = 80$ mm
- Depth of profiling: $h_p = 50$ mm
- Centres of profiles: $e_p = 300$ mm
- Width at inside of profile: $b_1 = 125$ mm
- Width at outside of profile: $b_2 = 175$ mm
- Thickness of sheeting: $t_p = 1.2$ mm
- Mean width of concrete ribs: $b_0 = 150$ mm
- Shear force at maximum BM: $V = 200$ kN

Composite section – elastic section properties

- Inertia of combined section (steel units): $I_{na} = 27727$ cm$^4$
- Composite beam deflection (SLS): $d_v = 6$ mm
- Plastic moment of resistance: $M_{pl} = M_{plc} + M_{plf} + M_{plb} + M_{plw} + M_{pll}$
  $= 340.37$ kNm
- Shear plastic resistance: $V_{plEd} = V_{plRd} = 411.3$ kN
- Char yield strength of reinft.: $f_{yk} = 500$ N/mm$^2$
- Area of reinft. per shear plane: $A_e = 174$ mm$^2$/m
<table>
<thead>
<tr>
<th>COMPOSITE</th>
<th>305 x 165 x 40 UKB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>with 22 No. 19 dia. stud connectors</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>@ 270 centres.</td>
</tr>
<tr>
<td></td>
<td>Section is satisfactory for plastic moment, shear &amp; deflection requirements.</td>
</tr>
<tr>
<td></td>
<td>Steel beam mom.capacity</td>
</tr>
<tr>
<td></td>
<td>Composite stage moment</td>
</tr>
<tr>
<td></td>
<td>Plastic moment resistance</td>
</tr>
<tr>
<td></td>
<td>Vertical shear resistance</td>
</tr>
<tr>
<td></td>
<td>Design shear capacity</td>
</tr>
<tr>
<td></td>
<td>Long.shear force</td>
</tr>
<tr>
<td></td>
<td>Long.shear resistance</td>
</tr>
<tr>
<td></td>
<td>Min transverse reinfnt.</td>
</tr>
<tr>
<td></td>
<td>Variable load deflection</td>
</tr>
<tr>
<td></td>
<td>Lightweight concrete (LWC) assumed</td>
</tr>
</tbody>
</table>
Location: Ex5 - UB with decking & secondary beams @ 1/3 points

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

The beam under consideration is an internal beam.

Design B.M. (compos.stage) MyEd=320 kNm
Design shear force (comp.stage) VcEd=200 kN
Concrete density (const.stage) D'c=19 kN/m³
Concrete density (final stage) Dc=18 kN/m³
Deck is parallel to the supporting beam.

Deck ribs are parallel to B1
B1 = beam being considered
B2 = secondary beam
C = L1/3

Floor plan

Beam span L1=6 m
Sum of secondary beam spans L2=6 m
305 x 165 x 40 UKB
Dimensions (mm): h=303.4 b=165 tw=6 tf=10.2 r=8.9
Properties (cm): Iy=8500 Iz=764 Wply=623 Wplz=142 It=14.7
Material partial factors

Partial safety factor (reinfnt.) $g_{ams}=1.15$
Partial safety factor (concrete) $g_{amc}=1.5$
Partial safety factor (steel) $g_{amM0}=1.0$
Partial safety factor (steel) $g_{amM1}=1.0$
Partial safety factor (studs) $g_{amv}=1.25$

Concrete slab details

Depth of concrete above profile $h_f=80$ mm
Depth of profiling $h_p=50$ mm
Centres of profiles $e_p=300$ mm
Width at inside of profile $b_1=125$ mm
Width at outside of profile $b_2=175$ mm
Thickness of sheeting $t_p=1.2$ mm
Mean width of concrete ribs $b_0=150$ mm
Shear force at maximum BM $V=200$ kN

Composite section – elastic section properties

Inertia of combined section (steel units) $I_{na}=27727$ cm$^4$
Composite beam deflection (SLS) $d_v=6$ mm
Plastic moment of resistance $M_{pl}=M_{plc}+M_{plf}+M_{plb}+M_{plw}+M_{pll}$
$=340.37$ kNm
Shear plastic resistance $V_{plEd}=V_{plRd}=411.3$ kN
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Area of reinft. per shear plane \( A_e = 328 \text{ mm}^2/\text{m} \)

| COMPOSITE | 305 x 165 x 40 UKB Grade S 355 |
| SECTION | with 11 No. 19 dia. stud connectors |
| SUMMARY | @ 180 centres between each support and third point, plus nominal studs between third points. |
| SUMMARY | Section is satisfactory for plastic moment, shear & deflection requirements. |
| Steel beam mom. capacity | 221.17 kNm |
| Composite stage moment | 320 kNm |
| Plastic moment resistance | 340.37 kNm |
| Vertical shear resistance | 200 kN |
| Design shear capacity | 411.3 kN |
| Long. shear force | 201.77 kN/m |
| Long. shear resistance | 201.81 kN/m |
| Min transverse reinft. | 328 mm$^2$/mm |
| Variable load deflection | 6 mm |
| Lightweight concrete (LWC) assumed |
Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

The beam under consideration is an internal beam.
Design B.M. (compos.stage) MyEd=320 kNm
Design shear force (comp.stage) VcEd=200 kN
Concrete density (const.stage) D'c=19 kN/m³
Concrete density (final stage) Dc=18 kN/m³
Deck is parallel to the supporting beam.

dock ribs are parallel to B1
B1 = beam being considered
L2 B2 = secondary beam
C1 = shorter distance from point of max moment to support

Floor Plan

Beam span L1=6 m
Sum of secondary beam spans L2=6 m
Shorter distance from supp.to PL C1=2 m
305 x 165 x 40 UKB
Dimensions (mm): h=303.4 b=165 tw=6 tf=10.2 r=8.9
Properties (cm): Iy=8500 Iz=764 Wply=623 Wplz=142 It=14.7
Material partial factors

- Partial safety factor (reinforcement) \( \gamma_{amus} = 1.15 \)
- Partial safety factor (concrete) \( \gamma_{c} = 1.5 \)
- Partial safety factor (steel) \( \gamma_{M0} = 1.0 \)
- Partial safety factor (steel) \( \gamma_{M1} = 1.0 \)
- Partial safety factor (studs) \( \gamma_{v} = 1.25 \)

Concrete slab details

Depth of concrete above profile \( h_f = 80 \text{ mm} \)
Depth of profiling \( h_p = 50 \text{ mm} \)
Centres of profiles \( e_p = 300 \text{ mm} \)
Width at inside of profile \( b_1 = 125 \text{ mm} \)
Width at outside of profile \( b_2 = 175 \text{ mm} \)
Thickness of sheeting \( t_p = 1.2 \text{ mm} \)
Mean width of concrete ribs \( b_0 = 150 \text{ mm} \)
Shear force at maximum BM \( V = 200 \text{ kN} \)

Composite section - elastic section properties

Inertia of combined section (steel units) \( I_n = 27727 \text{ cm}^4 \)
Composite beam deflection (SLS) \( d_v = 6 \text{ mm} \)
Plastic moment of resistance \( M_{pl} = M_{plc} + M_{plf} + M_{plb} + M_{plw} + M_{pll} = 340.37 \text{ kNm} \)
Shear plastic resistance \( V_{plEd} = V_{plRd} = 411.3 \text{ kN} \)
Char yield strength of reinft. $f_{yk}=500$ N/mm²  
Area of reinft. per shear plane $A_e=328$ mm²/m

<table>
<thead>
<tr>
<th>COMPOSITE</th>
<th>305 x 165 x 40 UKB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>with 22 No. 19 dia. stud connectors</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>(11 No. @ 180 centres over the shortest distance and 11 No. @ 360 centres over the longest distance).</td>
</tr>
<tr>
<td>Section is satisfactory for plastic moment, shear &amp; deflection requirements.</td>
<td></td>
</tr>
<tr>
<td>Steel beam mom. capacity</td>
<td>221.17 kNm</td>
</tr>
<tr>
<td>Composite stage moment</td>
<td>320 kNm</td>
</tr>
<tr>
<td>Plastic moment resistance</td>
<td>340.37 kNm</td>
</tr>
<tr>
<td>Vertical shear resistance</td>
<td>200 kN</td>
</tr>
<tr>
<td>Design shear capacity</td>
<td>411.3 kN</td>
</tr>
<tr>
<td>Long. shear force</td>
<td>201.77 kN/m</td>
</tr>
<tr>
<td>Long. shear resistance</td>
<td>201.81 kN/m</td>
</tr>
<tr>
<td>Min transverse reinft.</td>
<td>328 mm²/mm</td>
</tr>
<tr>
<td>Variable load deflection</td>
<td>6 mm</td>
</tr>
<tr>
<td>Lightweight concrete (LWC) assumed</td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex7 - UB with decking & two sec beams not @ 1/3 points

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

![Diagram of composite construction](image)

The beam under consideration is an internal beam.

- Design B.M. (compos.stage) $\text{MyEd}=320$ kNm
- Design shear force (comp.stage) $\text{VcEd}=200$ kN
- Concrete density (const.stage) $\text{D'c}=19$ kN/m$^3$
- Concrete density (final stage) $\text{Dc}=18$ kN/m$^3$
- Deck is parallel to the supporting beam.

Deck ribs are parallel to $B_1$

- $B_1$ = beam being considered
- $B_2$ = secondary beam
- $C_1$ = distance from point of max moment to support

Floor plan

- Beam span $L_1=6$ m
- Sum of secondary beam spans $L_2=6$ m
- Distance from supp.to point load $C_1=2$ m
- Dimensions (mm): $h=303.4$ b=165 $tw=6$ $tf=10.2$ $r=8.9$
- Properties (cm): $I_y=8500$ $I_z=764$ $W_{ply}=623$ $W_{plz}=142$ $I_t=14.7$
Material partial factors

Partial safety factor (reinforcement) \( g_{ams} = 1.15 \)
Partial safety factor (concrete) \( g_{mc} = 1.5 \)
Partial safety factor (steel) \( g_{M0} = 1.0 \)
Partial safety factor (steel) \( g_{M1} = 1.0 \)
Partial safety factor (studs) \( g_{v} = 1.25 \)

Concrete slab details

Depth of concrete above profile \( h_{f} = 80 \text{ mm} \)
Depth of profiling \( h_{p} = 50 \text{ mm} \)
Centres of profiles \( e_{p} = 300 \text{ mm} \)
Width at inside of profile \( b_{1} = 125 \text{ mm} \)
Width at outside of profile \( b_{2} = 175 \text{ mm} \)
Thickness of sheeting \( t_{p} = 1.2 \text{ mm} \)
Mean width of concrete ribs \( b_{0} = 150 \text{ mm} \)
Shear force at maximum BM \( V = 200 \text{ kN} \)

Composite section - elastic section properties

Inertia of combined section (steel units) \( I_{na} = 27727 \text{ cm}^4 \)
Composite beam deflection (SLS) \( d_{v} = 6 \text{ mm} \)
Plastic moment of resistance \( M_{pl} = M_{plc} + M_{plf} + M_{plb} + M_{plw} + M_{pll} = 340.37 \text{ kNm} \)
Shear plastic resistance \( V_{plEd} = V_{plRd} = 411.3 \text{ kN} \)
Char yield strength of reinfnt. \( f_y = 500 \text{ N/mm}^2 \)
Area of reinfnt.per shear plane \( A_e = 328 \text{ mm}^2 / \text{m} \)

**COMPOSITE**

305 x 165 x 40 UKB Grade S 355

**SECTION**

with 11 No. 19 dia. stud connectors

**SUMMARY**

@ 180 centres between each support and third point, plus nominal studs between third points.

Section is satisfactory for plastic moment, shear & deflection requirements.

Steel beam mom.capacity \( 221.17 \text{ kNm} \)
Composite stage moment \( 320 \text{ kNm} \)
Plastic moment resistance \( 340.37 \text{ kNm} \)
Vertical shear resistance \( 200 \text{ kN} \)
Design shear capacity \( 411.3 \text{ kN} \)
Long.shear force \( 201.77 \text{ kN/m} \)
Long.shear resistance \( 201.81 \text{ kN/m} \)
Min transverse reinfnt. \( 328 \text{ mm}^2 / \text{mm} \)
Variable load deflection \( 6 \text{ mm} \)
Lightweight concrete (LWC) assumed
Location: Ex8 - Edge beam with two sec.beams not @ 1/3 points

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

The beam under consideration is an edge beam.
Design B.M. (compos.stage) \( \text{MyEd}=320 \text{ kNm} \)
Design shear force (comp.stage) \( \text{VcEd}=200 \text{ kN} \)
Concrete density (const.stage) \( \text{D'}c=19 \text{ kN/m}^3 \)
Concrete density (final stage) \( \text{D}c=18 \text{ kN/m}^3 \)
Deck is parallel to the supporting beam.

Floor plan

Beam span \( \text{L1}=6 \text{ m} \)
Secondary beam span \( \text{L2}=6 \text{ m} \)
Distance from supp.to point load \( \text{C1}=2 \text{ m} \)
305 x 165 x 54 UKB
Dimensions (mm): \( \text{h}=310.4 \text{ b}=166.9 \text{ tw}=7.9 \text{ tf}=13.7 \text{ r}=8.9 \)
Properties (cm): \( \text{Iy}=11700 \text{ Iz}=1060 \text{ Wply}=846 \text{ Wplz}=196 \text{ It}=34.8 \)
Material partial factors

Partial safety factor (reinforcement) $\gamma_{ams}=1.15$
Partial safety factor (concrete) $\gamma_{mc}=1.5$
Partial safety factor (steel) $\gamma_{M0}=1.0$
Partial safety factor (steel) $\gamma_{M1}=1.0$
Partial safety factor (studs) $\gamma_{v}=1.25$

Concrete slab details

Depth of concrete above profile $h_f=80$ mm
Depth of profiling $h_p=50$ mm
Centres of profiles $e_p=300$ mm
Width at inside of profile $b_1=125$ mm
Width at outside of profile $b_2=175$ mm
Thickness of sheeting $t_p=1.2$ mm
Mean width of concrete ribs $b_0=150$ mm
Shear force at maximum BM $V=200$ kN

Composite section - elastic section properties

Inertia of combined section (steel units) $I_n=27834$ cm$^4$
Composite beam deflection (SLS) $d_v=6$ mm
Plastic moment of resistance $M_{pl}=M_{plc}+M_{pltf}+M_{plwa}+M_{plwb}+M_{plll} =392.54$ kNm
Shear plastic resistance $V_{plEd}=V_{plRd}=544.99$ kN
Char yield strength of reinfnt. \( f_{yk} = 500 \text{ N/mm}^2 \)
Area of reinfnt. per shear plane \( A_e = 371 \text{ mm}^2/\text{m} \)

| COMPOSITE | 305 x 165 x 54 UKB Grade S 355 |
| SECTION    | with 6 No. 19 dia. stud connectors |
| SUMMARY    | @ 330 centres between each support and third point, plus nominal studs between third points. |
|            | Section is satisfactory for plastic moment, shear & deflection requirements. |
| Steel beam mom. capacity | 300.33 kNm |
| Composite stage moment | 320 kNm |
| Plastic moment resistance | 392.54 kNm |
| Vertical shear resistance | 200 kN |
| Design shear capacity | 544.99 kN |
| Long. shear force | 220.11 kN/m |
| Long. shear resistance | 220.5 kN/m |
| Min transverse reinfnt. | 371 mm$^2$/mm |
| Variable load deflection | 6 mm |
| Lightweight concrete (LWC) assumed |
Location: Ex9 - Universal beam with Solid slab

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

The beam under consideration is an internal beam.

Design B.M. (compos.stage) MyEd=8500 kNm
Design shear force (comp.stage) VcEd=1312 kN
Concrete density (const.stage) D'c=25 kN/m³
Concrete density (final stage) Dc=24 kN/m³
Beam span L1=34.5 m
Dimensions (mm): h=1036.1 b=308.5 tw=30 tf=54.1 r=30
Properties (cm): Iy=1.02E6 Iz=26700 Wply=23200 Wplz=2800 It=4300

Material partial factors

Partial safety factor (reinfnt.) gams=1.15
Partial safety factor (concrete) gamc=1.5
Partial safety factor (steel) gamM0=1.0
Partial safety factor (steel) gamM1=1.0
Partial safety factor (studs) gamv=1.25

Concrete slab details
Composite section – elastic section properties

Inertia of combined section (steel units) \( I_{na} = 2.3438 \times 10^6 \text{ cm}^4 \)
Composite beam deflection (SLS) \( d_v = 52 \text{ mm} \)
Plastic moment of resistance \( M_{pl} = M_{plc} + M_{plf} + M_{plb} + M_{plw} + M_{pll} = 12719 \text{ kNm} \)
Shear plastic resistance \( V_{plEd} = V_{plRd} = 6477.2 \text{ kN} \)
Governing shear length \( h_f = 154.25 \text{ mm} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)

COMPOSITE
SECTION
SUMMARY
1016 x 305 x 487 UKB Grade S 355
with 418 No. 19 dia. stud connectors
@ 160 centres.
Studs to be grouped in 2 's.
Section is satisfactory for plastic moment, shear & deflection requirements.
Steel beam mom. capacity \( 7772 \text{ kNm} \)
Composite stage moment \( 8500 \text{ kNm} \)
Plastic moment resistance \( 12719 \text{ kNm} \)
Vertical shear resistance \( 1312 \text{ kN} \)
Design shear capacity \( 6477.2 \text{ kN} \)
Long shear stress \( 3.1945 \text{ N/mm}^2 \)
Long shear resistance \( 4.5618 \text{ N/mm}^2 \)
Min transverse reinfnt. \( 565.06 \text{ mm}^2/\text{mm} \)
Variable load deflection \( 52 \text{ mm} \)
Normal weight concrete (NWC) assumed
Location: Ex10 - RHS example with solid slab using P405 rules

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

The beam under consideration is an internal beam.
Design B.M. (compos.stage)        MyEd=320 kNm
Design shear force (comp.stage)   VcEd=200 kN
Concrete density (const.stage)    D'c=19 kN/m³
Concrete density (final stage)    Dc=18 kN/m³
Beam span                         L1=6 m
300 x 200 x 10 RHS - Hot finished.
Properties (cm): A=94.9 iz=8.13 Wely=788 Wply=956 Iy=11800
It=12900 Wt=1020 Welz=628 Wplz=721 Iz=6280 iy=11.2

Material partial factors

Partial safety factor (reinfnt.)  gams=1.15
Partial safety factor (concrete) gamc=1.5
Partial safety factor (steel)    gamM0=1.0
Partial safety factor (steel)    gamM1=1.0
Partial safety factor (studs)    gamv=1.25

Concrete slab details
Composite section – elastic section properties

\[
\begin{array}{c}
\text{Steel deck} \\
\text{tw} \\
\text{h} \\
\text{tf} \\
\text{b} \\
\end{array}
\]

Steel section is RHS

\[\text{ha } \text{tw} = \text{tf} = t \]
\[\text{hf} = h\]

Inertia of combined section (steel units)
\[I_{na} = 45061 \text{ cm}^4\]

Composite beam deflection (SLS)
\[dv = 6 \text{ mm}\]

**Section chosen is considered to be suitable.**

Plastic moment of resistance
\[M_{pl} = M_{plc} + M_{pltf} + M_{plwa} + M_{plwb} + M_{plll}\]
\[= 492.92 \text{ kNm}\]

Shear plastic resistance
\[V_{plEd} = V_{plRd} = 1167 \text{ kN}\]

Governing shear length
\[h_f = 114.25 \text{ mm}\]

Char yield strength of reinft.
\[f_yk = 500 \text{ N/mm}^2\]

Total steel beam deflection
\[d_s = 22 \text{ mm}\]

Total composite beam deflection
\[d_c = 10 \text{ mm}\]

Permanent load deflection
\[d_g = 6 \text{ mm}\]

**COMPOSITE**

300 x 200 x 10 Grade S 355

**SECTION**

with 26 No. 19 dia. stud connectors

@ 230 centres.

**SUMMARY**

Section is satisfactory for plastic moment, shear & deflection requirements.

Steel beam mom.capacity
\[339.38 \text{ kNm}\]

Composite stage moment
\[320 \text{ kNm}\]

Plastic moment resistance
\[492.92 \text{ kNm}\]

Vertical shear resistance
\[200 \text{ kN}\]

Design shear capacity
\[1167 \text{ kN}\]

Long.shear stress
\[1.3775 \text{ N/mm}^2\]

Long.shear resistance
\[2.5116 \text{ N/mm}^2\]

Min transverse reinft.
\[228.5 \text{ mm}^2/\text{mm}\]

Deflection to span ratio
at final stage (G + Q)
1: 296

Lightweight concrete (LWC) assumed
**Location: Ex11 - As Ex6 but using SCI P405 rules**

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

![Diagram of composite construction]

The beam under consideration is an internal beam.

- **Design B.M. (compos. stage)**: \( \text{MyEd}=320 \text{ kNm} \)
- **Design shear force (comp. stage)**: \( \text{VcEd}=200 \text{ kN} \)
- **Concrete density (const. stage)**: \( \text{D'c}=19 \text{ kN/m}^3 \)
- **Concrete density (final stage)**: \( \text{Dc}=18 \text{ kN/m}^3 \)

Deck is parallel to the supporting beam.

- **C1** = shorter distance from point of max moment to support
- **L1** = beam span
- **L2** = sum of secondary beam spans
- **B1** = beam being considered
- **B2** = secondary beam

**Floor Plan**

- Beam span: \( L_1 = 6 \text{ m} \)
- Sum of secondary beam spans: \( L_2 = 6 \text{ m} \)
- Shorter distance from supp. to PL: \( C_1 = 2 \text{ m} \)
- Dimensions (mm): \( h=303.4 \text{ b}=165 \text{ tw}=6 \text{ tf}=10.2 \text{ r}=8.9 \)
- Properties (cm): \( I_y=8500 \text{ I_z}=764 \text{ Wp}_y=623 \text{ Wp}_z=142 \text{ It}=14.7 \)
Material partial factors

Partial safety factor (reinforcement) \( g_{ams} = 1.15 \)
Partial safety factor (concrete) \( g_{amc} = 1.5 \)
Partial safety factor (steel) \( g_{am0} = 1.0 \)
Partial safety factor (steel) \( g_{am1} = 1.0 \)
Partial safety factor (studs) \( g_{amv} = 1.25 \)

Concrete slab details

\[
\begin{align*}
\text{Depth of concrete above profile} & \quad hf = 80 \text{ mm} \\
\text{Depth of profiling} & \quad hp = 50 \text{ mm} \\
\text{Centres of profiles} & \quad ep = 300 \text{ mm} \\
\text{Width at inside of profile} & \quad b_1 = 125 \text{ mm} \\
\text{Width at outside of profile} & \quad b_2 = 175 \text{ mm} \\
\text{Thickness of sheeting} & \quad tp = 1.2 \text{ mm} \\
\text{Mean width of concrete ribs} & \quad b_0 = 150 \text{ mm} \\
\text{Shear force at maximum BM} & \quad V = 200 \text{ kN}
\end{align*}
\]

Composite section – elastic section properties

\[
\begin{align*}
\text{Inertia of combined section (steel units)} & \quad I_{na} = 27727 \text{ cm}^4 \\
\text{Composite beam deflection (SLS)} & \quad dv = 6 \text{ mm} \\
\text{Section chosen is considered to be suitable.} \\
\text{Plastic moment of resistance} & \quad M_{pl} = M_{plc} + M_{plf} + M_{plb} + M_{plw} + M_{plll} = 333.19 \text{ kNm}
\end{align*}
\]
Shear plastic resistance \( V_{plEd} = V_{plRd} = 411.3 \text{ kN} \)
Char yield strength of reinf. \( f_{yk} = 500 \text{ N/mm}^2 \)
Area of reinf. per shear plane \( A_e = 282 \text{ mm}^2 / \text{m} \)

Total steel beam deflection \( d_s = 22 \text{ mm} \)
Total composite beam deflection \( d_c = 10 \text{ mm} \)
Permanent load deflection \( d_g = 5 \text{ mm} \)

**COMPOSITE**

305 x 165 x 40 UKB Grade S 355

**SECTION**

with 20 No. 19 dia. stud connectors

(10 No. @ 200 centres over the shortest distance and 10 No. @ 400 centres over the longest distance).

Section is satisfactory for plastic moment, shear & deflection requirements.

**SUMMARY**

Steel beam moment capacity 221.17 kNm
Composite stage moment 320 kNm
Plastic moment resistance 333.19 kNm
Vertical shear resistance 200 kN
Design shear capacity 411.3 kN
Long. shear force 181.59 kN/m
Long. shear resistance 181.81 kN/m
Min transverse reinf. 282 mm²/mm

Deflection to span ratio at final stage \((G + Q)\) 1: 351
Lightweight concrete (LWC) assumed
Location: Ex12 - As Ex2 but with P405 rules

Calculations are in accordance with BS EN 1994-1-1:2004 the Code of practice for the 'Design of composite steel & concrete structures'.

Steel deck

/ Steel section is RHS

ha tw = tf = t

tf b

The beam under consideration is an internal beam.
Design B.M. (compos.stage) MyEd=320 kNm
Design shear force (comp.stage) VcEd=200 kN
Concrete density (const.stage) D'c=19 kN/m^3
Concrete density (final stage) Dc=18 kN/m^3
Beam span L1=6 m
300 x 200 x 10 RHS - Hot finished.
Properties (cm): A=94.9 iz=8.13 Wely=788 Wply=956 Iy=11800
It=12900 Wt=1020 Welz=628 Wplz=721 Iz=6280 iy=11.2

Material partial factors

Partial safety factor (reinfnt.) gams=1.15
Partial safety factor (concrete) gamc=1.5
Partial safety factor (steel) gamM0=1.0
Partial safety factor (steel) gamM1=1.0
Partial safety factor (studs) gamv=1.25

Concrete slab details
Composite section - elastic section properties

Steel deck

Steel section is RHS

\[ \text{Steel section is RHS} \]

\[ \text{ha} \quad \text{tw} = \text{tf} = t \]

\[ \text{hf} = h \]

Inertia of combined section (steel units)

\[ I_{na} = 45061 \, \text{cm}^4 \]

Composite beam deflection (SLS)

\[ dv = 6 \, \text{mm} \]

Section chosen is considered to be suitable.

Plastic moment of resistance

\[ M_{pl} = M_{plc} + M_{plf} + M_{plwa} + M_{plwb} + M_{plll} \]

\[ = 522.6 \, \text{kNm} \]

Shear plastic resistance

\[ V_{plEd} = V_{plRd} = 1167 \, \text{kN} \]

Governing shear length

\[ hf = 114.25 \, \text{mm} \]

Char yield strength of reinf.

\[ f_y = 500 \, \text{N/mm}^2 \]

Total steel beam deflection

\[ ds = 22 \, \text{mm} \]

Total composite beam deflection

\[ dc = 10 \, \text{mm} \]

Permanent load deflection

\[ dg = 5 \, \text{mm} \]

COMPOSITE

300 x 200 x 10 Grade S 355

SECTION

with 42 No. 19 dia. stud connectors

@ 140 centres.

SUMMARY

Section is satisfactory for plastic moment, shear & deflection requirements.

Steel beam mom.capacity 339.38 kNm

Composite stage moment 320 kNm

Plastic moment resistance 522.6 kNm

Vertical shear resistance 200 kN

Design shear capacity 1167 kN

Long.shear stress 2.2252 N/mm²

Long.shear resistance 2.5116 N/mm²

Min transverse reinfnt. 291.53 mm²/mm

Deflection to span ratio at final stage (G + Q) 1: 330

Lightweight concrete (LWC) assumed
Location: Ex1 - Inner beam

Differential temperature effects in composite bridge decks to

BD 37/01

Concrete slab depth  
Depth of surfacing  
Value of Young's modulus  
Coefficient of expansion  
Value of Young's modulus  
Coefficient of expansion  
Number of cross-sections  
Breadth at top  
Breadth at bottom  
Depth  
Material (1=Steel, 2=Concrete)  
Breadth at top  
Breadth at bottom  
Depth  
Material (1=Steel, 2=Concrete)  
Breadth at top  
Breadth at bottom  
Depth  
Material (1=Steel, 2=Concrete)  
Breadth at top  
Breadth at bottom  
Depth  
Material (1=Steel, 2=Concrete)  

Positive temperature calculations

Temperature gradient – increments of depth (Figure 9, BD 37/01)
Depth h1  
Depth h2  
Depth h3  
Temperature values (Figure 9 & Table 23 in BD 37/01):
Temperature T1  
Temperature T2
## Positive temperature diagram

### STRESSES (N/mm²) (tension is positive)

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-5.0592</td>
<td>1.7891</td>
<td>0.92511</td>
<td>-2.345</td>
</tr>
<tr>
<td>132</td>
<td>-1.488</td>
<td>1.7891</td>
<td>0.53013</td>
<td>0.83121</td>
</tr>
<tr>
<td>220</td>
<td>-1.2197</td>
<td>1.7891</td>
<td>0.26682</td>
<td>0.83622</td>
</tr>
<tr>
<td>220</td>
<td>-8.0656</td>
<td>11.831</td>
<td>1.7644</td>
<td>5.5298</td>
</tr>
<tr>
<td>256.6</td>
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<td>1140.5</td>
<td>0</td>
<td>11.831</td>
<td>-16.45</td>
<td>-4.6187</td>
</tr>
</tbody>
</table>

Member length L=10 m
Load factor for temperature gamma=1

### Plane frame analysis

Axial and bending effects can be analysed in a plane frame model.

Bending effects for hogging releasing moment:

\[
\theta = \frac{RM \cdot L}{E \cdot I}
\]

End 1 \( \theta \) deflected shape of released member

\( * * \) tangent to deflected shape at member ends

Calculate the angle (2*\( \theta \)) in radians

where 2*\( \theta \) = \( \frac{RM \cdot L}{E \cdot I} \)
NL-STRESS will apply the opposite of the releasing moment (RM) to the released member and rotate it by $2\theta$. The member is then clamped in the frame and let go.

Member distortion ($2\theta$) \[ mdfmp = \frac{(RM \cdot L)}{EI} = -0.96523E-3 \text{ radians} \]

Value of distortion for input to NL-STRESS \[ disfmp = mdfmp \cdot gamma = -0.96523E-3 \text{ radians} \]

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[ \text{<members>} \text{ ROTATION Z } -0.96523E-3 \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

Longitudinal effects for tensile releasing force:

Released member

\[ dL = \frac{(RF \cdot L)}{(A \cdot E)} \text{ in metres} \]

NL-STRESS will apply the opposite of the releasing force (RF) to the released member and compress it by $dL$. The member is then clamped in the frame and let go.

Member distortion \[ mdffp = \frac{(RF \cdot L) \cdot 1000}{EQ1} = 0.57712E-3 \text{ m} \]

Value of distortion for input to NL-STRESS \[ disffp = mdffp \cdot gamma = 0.57712E-3 \text{ m} \]

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[ \text{<members>} \text{ DISTORTION X } 0.57712E-3 \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

**Reverse temperature calculations**

Temperature gradient – increments of depth (Figure 9, BD 37/01)

| Depth h1 | hn1 = $0.6 \cdot h = 132 \text{ mm} $ |
| Depth h2 | hn2 = $0.4 \cdot h = 88 \text{ mm} $ |
| Depth h3 | hn3 = $400 \text{ mm} $ |
| Depth h4 | hn4 = tsd-h-400 = 520.5 \text{ mm} |

Temperature values (Figure 9 & Table 23 in BD 37/01):

| Temperature T1 | $TR1=\text{TABLE 2 for } h=220, \text{ ds}=100$ |-3.8 °C |
| Temperature T2 | $TR2=-8 \text{ °C} $ |
| Temperature T3 | $TR3=-8 \text{ °C} $ |
Reverse temperature diagram

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
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<td>-0.92855</td>
<td>0.80109</td>
<td>1.2861</td>
</tr>
<tr>
<td>132</td>
<td>0</td>
<td>-0.92855</td>
<td>0.45907</td>
<td>-0.46948</td>
</tr>
<tr>
<td>220</td>
<td>0</td>
<td>-0.92855</td>
<td>0.23105</td>
<td>-0.6975</td>
</tr>
<tr>
<td>220</td>
<td>0</td>
<td>-6.1404</td>
<td>1.5279</td>
<td>-4.6125</td>
</tr>
<tr>
<td>256.6</td>
<td>1.8007</td>
<td>-6.1404</td>
<td>0.90079</td>
<td>-3.4389</td>
</tr>
<tr>
<td>620</td>
<td>19.68</td>
<td>-6.1404</td>
<td>-5.326</td>
<td>8.2136</td>
</tr>
<tr>
<td>1103.9</td>
<td>19.68</td>
<td>-6.1404</td>
<td>-13.617</td>
<td>-0.077837</td>
</tr>
<tr>
<td>1140.5</td>
<td>19.68</td>
<td>-6.1404</td>
<td>-14.245</td>
<td>-0.70497</td>
</tr>
</tbody>
</table>

Member length: \( L = 10 \text{ m} \)

Load factor for temperature: \( \gamma = 1 \)

**Plane frame analysis**

Axial and bending effects can be analysed in a plane frame model.

Bending effects for hogging releasing moment:

\[
\text{Calculate the angle } 2\theta \text{ in radians where } 2\theta = \frac{RM \cdot L}{E \cdot I} \]

---

 SCALE 5.48                     Office 1007                 Proforma 225
NL-STRESS will apply the opposite of the releasing moment (RM) to the released member and rotate it by 2*theta. The member is then clamped in the frame and let go.

Member distortion (2*theta) \[ \text{mdfmp} = \frac{(RM \times L)}{EI} = -0.83584E-3 \text{ radians} \]

Value of distortion for input to NL-STRESS
\[ \text{disfmp} = \text{mdfmp} \times \gamma = -0.83584E-3 \text{ radians} \]

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[ \text{members} \text{ ROTATION Z } -0.83584E-3 \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

Longitudinal effects for compressive releasing force:

Released member

\[ \frac{\text{dL}}{\text{L}} = \frac{(RF \times L)}{(A \times E)} \text{ in metres} \]

NL-STRESS will apply the opposite of the releasing force (RF) to the released member and extend it by dL. The member is then clamped in the frame and let go.

Member distortion \[ \text{mdffr} = \frac{(RF \times L)}{EQ1} = -0.29953E-3 \text{ m} \]

Value of distortion for input to NL-STRESS
\[ \text{disffr} = \text{mdffr} \times \gamma = -0.29953E-3 \text{ m} \]

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[ \text{members} \text{ DISTORTION X } -0.29953E-3 \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.
Location: Ex2 - Inner beam

**Differential temperature effects in composite bridge decks to BD 37/01**

- **Concrete slab depth**: h=200 mm
- **Depth of surfacing**: ds=150 mm
- **Number of cross-sections**: ncs=7
- **Breadth at top**: xbt(11)=1000 mm
- **Breadth at bottom**: xbb(11)=800 mm
- **Depth**: xd(11)=125 mm
- **Material (1=Steel, 2=Concrete)**: mi(11)=2
- **Young's modulus**: E(11)=31000 N/mm²
- **Coefficient of expansion**: coe(11)=12E-6 /°C
- **Breadth at top**: xbt(21)=800 mm
- **Breadth at bottom**: xbb(21)=800 mm
- **Depth**: xd(21)=75 mm
- **Material (1=Steel, 2=Concrete)**: mi(21)=2
- **Young's modulus**: E(21)=25000 N/mm²
- **Coefficient of expansion**: coe(21)=10E-6 /°C
- **Breadth at top**: xbt(31)=800 mm
- **Breadth at bottom**: xbb(31)=800 mm
- **Depth**: xd(31)=15 mm
- **Material (1=Steel, 2=Concrete)**: mi(31)=1
- **Young's modulus**: E(31)=205000 N/mm²
- **Coefficient of expansion**: coe(31)=12E-6 /°C
- **Breadth at top**: xbt(41)=200 mm
- **Breadth at bottom**: xbb(41)=15 mm
- **Depth**: xd(41)=50 mm
- **Material (1=Steel, 2=Concrete)**: mi(41)=1
- **Young's modulus**: E(41)=195000 N/mm²
- **Coefficient of expansion**: coe(41)=11E-6 /°C
- **Breadth at top**: xbt(51)=15 mm
- **Breadth at bottom**: xbb(51)=15 mm
- **Depth**: xd(51)=600 mm
- **Material (1=Steel, 2=Concrete)**: mi(51)=1
- **Young's modulus**: E(51)=190000 N/mm²
- **Coefficient of expansion**: coe(51)=10E-6 /°C
- **Breadth at top**: xbt(61)=15 mm
Positive temperature calculations

Temperature gradient – increments of depth (Figure 9, BD 37/01)

Temperature values (Figure 9 & Table 23 in BD 37/01):

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>STRESSES (N/mm²)</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-3.906</td>
<td>0.99741</td>
<td>1.1522</td>
<td>-1.7564</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>-1.488</td>
<td>0.99741</td>
<td>0.8523</td>
<td>0.36171</td>
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</tr>
<tr>
<td>125</td>
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<td>0.99741</td>
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</tr>
<tr>
<td>125</td>
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<td>0.80437</td>
<td>0.67726</td>
<td>0.49205</td>
<td></td>
</tr>
<tr>
<td>200</td>
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<td>0.52612</td>
<td>0.49715</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>-8.2</td>
<td>6.5958</td>
<td>4.3142</td>
<td>2.71</td>
<td></td>
</tr>
<tr>
<td>215</td>
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<td></td>
</tr>
<tr>
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<td>3.2601</td>
<td></td>
</tr>
<tr>
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</tr>
<tr>
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<td>3.003</td>
<td>3.812</td>
<td></td>
</tr>
<tr>
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<td>0</td>
<td>6.1132</td>
<td>-2.1278</td>
<td>3.9854</td>
<td></td>
</tr>
</tbody>
</table>

Positive temperature diagram
Plane frame analysis

Axial and bending effects can be analysed in a plane frame model.

Bending effects for hogging releasing moment:

\[
\theta = \frac{RM \times L}{EI}
\]

End 1 \[\theta\] End 2

\[<---L---->\]

* * deflected shape of released member
**** tangent to deflected shape at member ends

Calculate the angle \(2\theta\) in radians where

\[
2\theta = \frac{RM \times L}{EI}
\]

NL-STRESS will apply the opposite of the releasing moment \(RM\) to the released member and rotate it by \(2\theta\). The member is then clamped in the frame and let go.

Member distortion \(2\theta\) \[mdfmp=(RM \times L)/EI=-0.8061E-3\] radians

Value of distortion for input to NL-STRESS \[disfmp=mdfmp \times \gamma=-0.8061E-3\] radians

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[<\text{members}> \text{ ROTATION } Z -0.8061E-3\]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.
Longitudinal effects for tensile releasing force:

Released member

\[ \frac{dL}{L} \approx \frac{RF}{E} \]

Calculate the extension of the member \( dL = \frac{RF \cdot L}{A \cdot E} \) in metres.

NL-STRESS will apply the opposite of the releasing force (RF) to the released member and compress it by \( dL \). The member is then clamped in the frame and let go.

Member distortion

\[ mdffp = \frac{RF \cdot L}{E} \cdot 1000 = 0.32175E-3 \text{ m} \]

Value of distortion for input to NL-STRESS

\[ disffp = mdffp \cdot \gamma = 0.32175E-3 \text{ m} \]

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format:

\[ <\text{members}> \text{ DISTORTION X } 0.32175E-3 \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

**Reverse temperature calculations**

Temperature gradient - increments of depth (Figure 9, BD 37/01):

Depth h1 \( hn1 = 0.6 \cdot h = 120 \text{ mm} \)
Depth h2 \( hn2 = 0.4 \cdot h = 80 \text{ mm} \)
Depth h3 \( hn3 = 400 \text{ mm} \)
Depth h4 \( hn4 = tsd - h - 400 = 355 \text{ mm} \)

Temperature values (Figure 9 & Table 23 in BD 37/01):

Temperature T1 \[ TR1 = \text{TABLE 2 for } h = 200, \ ds = 150 \]
\[ = -2.3 \degree C \]

Temperature T2 \[ TR2 = -8 \degree C \]

Temperature T3 \[ TR3 = -8 \degree C \]

Reverse temperature diagram
### STRESSES (N/mm²) (tension is positive)

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.8556</td>
<td>-1.5148</td>
<td>1.808</td>
<td>1.1488</td>
</tr>
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<td>125</td>
<td>0</td>
<td>-1.5148</td>
<td>1.3178</td>
<td>-0.19699</td>
</tr>
<tr>
<td>125</td>
<td>0</td>
<td>-1.2216</td>
<td>1.0628</td>
<td>-0.15886</td>
</tr>
<tr>
<td>200</td>
<td>0</td>
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<td>0.8256</td>
<td>-0.39604</td>
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<td>200</td>
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<td>0.6435</td>
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<td>6.0697</td>
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<td>4.8364</td>
<td>-1.9039</td>
</tr>
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<td>-9.2844</td>
<td>4.7123</td>
<td>-2.1021</td>
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<td>600</td>
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</tr>
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<td>-9.708</td>
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<tr>
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<td>0.84836</td>
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<td>-10.262</td>
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<tr>
<td>940</td>
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<td>955</td>
<td>28.8</td>
<td>-11.728</td>
<td>-14.995</td>
<td>2.0773</td>
</tr>
</tbody>
</table>

**Member length**  
L = 10 m  

**Load factor for temperature**  
\[
\gamma = 1
\]

### Plane frame analysis

Axial and bending effects can be analysed in a plane frame model. Bending effects for hogging releasing moment:

\[
2\theta = \frac{RM \times L}{EI}
\]

* * deflected shape of released member  
**** tangent to deflected shape at member ends

Calculate the angle (2*theta) in radians where

\[
2\theta = \frac{RM \times L}{EI}
\]
NL-STRESS will apply the opposite of the releasing moment (RM) to the released member and rotate it by 2*theta. The member is then clamped in the frame and let go.

Member distortion (2*theta) \( \text{mdfmp} = \frac{\text{RM} \times \text{L}}{\text{EI}} = -0.0012649 \text{ radians} \)

Value of distortion for input to NL-STRESS
\( \text{disfmp} = \text{mdfmp} \times \gamma = -0.0012649 \text{ radians} \)

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[ \langle \text{members} \rangle \ \text{ROTATION} \ Z \ -0.0012649 \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

Longitudinal effects for compressive releasing force:

Released member

\[ \langle _____ \ L \ _____ \rangle \]

\[ \langle -dL \rangle \]

\[ *- - - . \]

Calculate the contraction of the member \( dL = \frac{\text{RF} \times \text{L}}{\text{A} \times \text{E}} \) in metres

NL-STRESS will apply the opposite of the releasing force (RF) to the released member and extend it by \( dL \). The member is then clamped in the frame and let go.

Member distortion \( \text{mdffr} = \frac{\text{RF} \times \text{L} \times 1000}{\text{EQ1}} = -0.48865 \times 10^{-3} \text{ m} \)

Value of distortion for input to NL-STRESS
\( \text{disffr} = \text{mdffr} \times \gamma = -0.48865 \times 10^{-3} \text{ m} \)

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[ \langle \text{members} \rangle \ \text{DISTORTION} \ X \ -0.48865 \times 10^{-3} \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.
Location: Ex1 - Inner beam

Differential temperature effects in composite bridge decks

Concrete slab depth \( h = 220 \text{ mm} \)
Depth of surfacing \( ds = 100 \text{ mm} \)
Value of Young's modulus \( EMC = 31000 \text{ N/mm}^2 \)
Coefficient of expansion \( \alpha_{cs} = 12 \times 10^{-6} \text{ /} °C \)
Value of Young's modulus \( EMS = 210000 \text{ N/mm}^2 \)
Coefficient of expansion \( \alpha_{s} = 12 \times 10^{-6} \text{ /} °C \)
Number of cross-sections \( n_{cs} = 4 \)
Breadth at top \( x_{bt}(11) = 2744 \text{ mm} \)
Breadth at bottom \( x_{bb}(11) = 2744 \text{ mm} \)
Depth \( x_{d}(11) = 220 \text{ mm} \)
Material (1=Steel, 2=Concrete) \( m_{i}(11) = 2 \)
Breadth at top \( x_{bt}(21) = 420.5 \text{ mm} \)
Breadth at bottom \( x_{bb}(21) = 420.5 \text{ mm} \)
Depth \( x_{d}(21) = 36.6 \text{ mm} \)
Material (1=Steel, 2=Concrete) \( m_{i}(21) = 1 \)
Breadth at top \( x_{bt}(31) = 21.5 \text{ mm} \)
Breadth at bottom \( x_{bb}(31) = 21.5 \text{ mm} \)
Depth \( x_{d}(31) = 847.3 \text{ mm} \)
Material (1=Steel, 2=Concrete) \( m_{i}(31) = 1 \)
Breadth at top \( x_{bt}(41) = 420.5 \text{ mm} \)
Breadth at bottom \( x_{bb}(41) = 420.5 \text{ mm} \)
Depth \( x_{d}(41) = 36.6 \text{ mm} \)
Material (1=Steel, 2=Concrete) \( m_{i}(41) = 1 \)

Positive temperature calculations

Temperature gradient – increments of depth (Figure 6.2b, EN 1991-1-5)
Depth \( h_1 \) \( h_{p1} = 0.6 \times h = 132 \text{ mm} \)
Depth \( h_2 \) \( h_{p2} = 0.4 \times h + 400 = 488 \text{ mm} \)
Depth \( h_3 \) \( h_{p3} = tsd - 400 - h = 520.5 \text{ mm} \)
Temperature values (Figure 6.2b & Table B.2 in EN 1991-1-5):
Temperature \( T_1 \) \( T_{P1} = \text{TABLE } 1 \text{ for } h=220, ds=100 = 13.6 °C \)
Temperature \( T_2 \) \( T_{P2} = 4 °C \)
Positive temperature diagram

STRESSES (N/mm²) (tension is positive)

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-5.0592</td>
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<td>0.93114</td>
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<td>-4.6842</td>
</tr>
</tbody>
</table>

Member length L=10 m
Load factor for temperature gamma=1

Plane frame analysis

Axial and bending effects can be analysed in a plane frame model.
Bending effects for hogging releasing moment:

Calculate the angle (2*theta) in radians
where 2*theta = (RM*L)/(E*I)
NL-STRESS will apply the opposite of the releasing moment (RM) to the released member and rotate it by 2*theta. The member is then clamped in the frame and let go.

Member distortion (2*theta)  \[ \text{mdfmp}=\frac{(RM*L)}{EI}=-0.96178\times10^{-3} \text{ radians} \]

Value of distortion for input to NL-STRESS

\[ \text{disfmp}=\text{mdfmp}\times\gamma=-0.96178\times10^{-3} \text{ radians} \]

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[ \langle\text{members}\rangle \text{ ROTATION Z } -0.96178\times10^{-3} \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

Longitudinal effects for tensile releasing force:

Released member

\[ \text{dL}=(\text{RF}L)/(A\times E) \text{ in metres} \]

\[ \langle\text{members}\rangle \text{ DISTORTION X } 0.57348\times10^{-3} \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

**Reverse temperature calculations**

Temperature gradient – increments of depth (Figure 6.2b, EN 1991-1-5)

<table>
<thead>
<tr>
<th>Depth</th>
<th>( h_n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h_1 )</td>
<td>0.6( h )=132 mm</td>
</tr>
<tr>
<td>( h_2 )</td>
<td>0.4( h )=88 mm</td>
</tr>
<tr>
<td>( h_3 )</td>
<td>( h_3=400 ) mm</td>
</tr>
<tr>
<td>( h_4 )</td>
<td>( h_4=\text{tsd}-h-400=520.5 ) mm</td>
</tr>
</tbody>
</table>

Temperature values (Figure 6.2b & Table B.2 in EN 1991-1-5):

<table>
<thead>
<tr>
<th>Temperature</th>
<th>( T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_1 )</td>
<td>( T_1=\text{TABLE 2 for } h=220, \text{ ds}=100 )</td>
</tr>
<tr>
<td>( T_2 )</td>
<td>( -8 ) °C</td>
</tr>
<tr>
<td>( T_3 )</td>
<td>( -8 ) °C</td>
</tr>
</tbody>
</table>
Reverse temperature diagram

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
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<tr>
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<td>0</td>
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<tr>
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<td>-4.7206</td>
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<td>0.97966</td>
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<td>20.16</td>
<td>-6.3439</td>
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<td>-0.74895</td>
</tr>
</tbody>
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Member length L=10 m
Load factor for temperature gamma=1

**Plane frame analysis**

Axial and bending effects can be analysed in a plane frame model.

Bending effects for hogging releasing moment:

\[
\theta = \frac{2RM}{EI}
\]

Where \( \theta \) is the angle in radians.

Calculate the angle \( 2\theta \) in radians where \( 2\theta = \frac{RM \times L}{E \times I} \).
NL-STRESS will apply the opposite of the releasing moment (RM) to the released member and rotate it by 2*theta. The member is then clamped in the frame and let go.

Member distortion (2*theta) $mdfmp = \frac{RM \times L}{EI} = -0.83745 \times 10^{-3}$ radians

Value of distortion for input to NL-STRESS $disfmp = mdfmp \times \gamma = -0.83745 \times 10^{-3}$ radians

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

```
<members> ROTATION Z -0.83745E-3
```

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

Longitudinal effects for compressive releasing force:

Released member

```
<--dL--> L -->
```

Calculate the contraction of the member $dL = \frac{RF \times L}{A \times E}$ in metres

NL-STRESS will apply the opposite of the releasing force (RF) to the released member and extend it by $dL$. The member is then clamped in the frame and let go.

Member distortion $mdffr = \frac{RF \times L \times 1000}{EQ1} = -0.30209 \times 10^{-3}$ m

Value of distortion for input to NL-STRESS $disffr = mdffr \times \gamma = -0.30209 \times 10^{-3}$ m

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

```
<members> DISTORTION X -0.30209E-3
```

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.
Location: Ex2 - Inner beam

Differential temperature effects in composite bridge decks

Calculations in accordance with BS EN 1994-2:2005, NA to BS EN 1994-2:2005 and also BS EN 1991-1-5:2003 (reference Clause 5.4.2.5 "Temperature effects").

Deck on Truss or Plate Girders

Concrete slab depth $h=200$ mm
Depth of surfacing $ds=150$ mm
Number of cross-sections $ncs=7$
Breadth at top $xbt(11)=1000$ mm
Breadth at bottom $xbb(11)=800$ mm
Depth $xd(11)=125$ mm
Material (1=Steel, 2=Concrete) $mi(11)=2$
Young's modulus $E(11)=31000$ N/mm²
Coefficient of expansion $coe(11)=12E-6$ /°C
Breadth at top $xbt(21)=800$ mm
Breadth at bottom $xbb(21)=800$ mm
Depth $xd(21)=75$ mm
Material (1=Steel, 2=Concrete) $mi(21)=2$
Young's modulus $E(21)=25000$ N/mm²
Coefficient of expansion $coe(21)=10E-6$ /°C
Breadth at top $xbt(31)=800$ mm
Breadth at bottom $xbb(31)=800$ mm
Depth $xd(31)=15$ mm
Material (1=Steel, 2=Concrete) $mi(31)=1$
Young's modulus $E(31)=210000$ N/mm²
Coefficient of expansion $coe(31)=12E-6$ /°C
Breadth at top $xbt(41)=200$ mm
Breadth at bottom $xbb(41)=15$ mm
Depth $xd(41)=50$ mm
Material (1=Steel, 2=Concrete) $mi(41)=1$
Young's modulus $E(41)=195000$ N/mm²
Coefficient of expansion $coe(41)=11E-6$ /°C
Breadth at top $xbt(51)=15$ mm
Breadth at bottom $xbb(51)=15$ mm
Depth $xd(51)=600$ mm
Material (1=Steel, 2=Concrete) $mi(51)=1$
Young's modulus $E(51)=190000$ N/mm²
Coefficient of expansion $coe(51)=10E-6$ /°C
Breadth at top $xbt(61)=15$ mm
Breadth at bottom $xbb(61)=400$ mm
Depth $xd(61)=75$ mm
Material (1=Steel, 2=Concrete) mi(61)=1
Young's modulus E(61)=210000 N/mm²
Coefficient of expansion coe(61)=13E-6 /°C
Breadth at top xbt(71)=600 mm
Breadth at bottom xbb(71)=600 mm
Depth xd(71)=15 mm
Material (1=Steel, 2=Concrete) mi(71)=1
Young's modulus E(71)=240000 N/mm²
Coefficient of expansion coe(71)=15E-6 /°C

Positive temperature calculations

Temperature gradient – increments of depth (Figure 6.2b, EN 1991-1-5)
Depth h1 hp1=0.6*h=120 mm
Depth h2 hp2=0.4*h+400=480 mm
Depth h3 hp3=t-400-h=355 mm

Temperature values (Figure 6.2b & Table B.2 in EN 1991-1-5):
Temperature T1 TP1=TABLE 1 for h=200, ds=150 =10.5 °C
Temperature T2 TP2=4 °C

Positive temperature diagram

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained</th>
<th>Releasing force</th>
<th>Releasing moment</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-3.906</td>
<td>0.99825</td>
<td>1.1484</td>
<td>-1.7593</td>
</tr>
<tr>
<td>120</td>
<td>-1.488</td>
<td>0.99825</td>
<td>0.84889</td>
<td>0.35915</td>
</tr>
<tr>
<td>125</td>
<td>-1.4725</td>
<td>0.99825</td>
<td>0.83641</td>
<td>0.36217</td>
</tr>
<tr>
<td>125</td>
<td>-0.98958</td>
<td>0.80504</td>
<td>0.67453</td>
<td>0.48999</td>
</tr>
<tr>
<td>200</td>
<td>-0.83333</td>
<td>0.80504</td>
<td>0.52356</td>
<td>0.49527</td>
</tr>
<tr>
<td>200</td>
<td>-8.4</td>
<td>6.7624</td>
<td>4.3979</td>
<td>2.7602</td>
</tr>
<tr>
<td>215</td>
<td>-8.085</td>
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<td>4.1443</td>
<td>2.8216</td>
</tr>
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<td>215</td>
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<td>3.8483</td>
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</tr>
<tr>
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<td>3.0632</td>
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<tr>
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<td>600</td>
<td>0</td>
<td>6.1183</td>
<td>-2.1402</td>
<td>3.9781</td>
</tr>
<tr>
<td>865</td>
<td>0</td>
<td>6.1183</td>
<td>-6.1942</td>
<td>-0.07586</td>
</tr>
</tbody>
</table>
Member length L=10 m
Load factor for temperature gamma=1

**Plane frame analysis**

Axial and bending effects can be analysed in a plane frame model.
Bending effects for hogging releasing moment:

\[
\theta \rightarrow 2\theta
\]

\[
\theta \rightarrow \theta(2\theta)
\]

End 1 \[ \rightarrow \] \[ \text{tangent to deflected shape at member ends} \]

Calculate the angle (2*theta) in radians where

\[
2\theta = \frac{RM \times L}{E \times I}
\]

NL-STRESS will apply the opposite of the releasing moment (RM) to the released member and rotate it by 2*theta. The member is then clamped in the frame and let go.

Member distortion (2*theta) \[ mdfm = \frac{RM \times L}{E \times I} = -0.80516E-3 \] radians
Value of distortion for input to NL-STRESS \[ disfmp = mdfmp \times gamma = -0.80516E-3 \] radians

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[ <\text{members}> \text{ ROTATION Z } -0.80516E-3 \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.
**Longitudinal effects for tensile releasing force:**

**Released member**

\[
\left\langle \Delta L \right\rangle = \left\langle \delta \right\rangle + L
\]

Calculate the extension of the member \( \Delta L = \frac{RF \times L}{A \times E} \) in metres

NL-STRESS will apply the opposite of the releasing force \( RF \) to the released member and compress it by \( \Delta L \). The member is then clamped in the frame and let go.

**Member distortion**

\[ mdffp = \frac{(RF \times L) \times 1000}{EQ1} = 0.32202 \times 10^{-3} \text{ m} \]

**Value of distortion for input to NL-STRESS**

\[ disffp = mdffp \times \gamma = 0.32202 \times 10^{-3} \text{ m} \]

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[ <\text{members}> \text{ DISTORTION X } 0.32202 \times 10^{-3} \]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

**Reverse temperature calculations**

**Temperature gradient - increments of depth** (Figure 6.2b, EN 1991-1-5)

- \( h_{n1} = 0.6 \times h = 120 \text{ mm} \)
- \( h_{n2} = 0.4 \times h = 80 \text{ mm} \)
- \( h_{n3} = 400 \text{ mm} \)
- \( h_{n4} = tsd - h - 400 = 355 \text{ mm} \)

**Temperature values** (Figure 6.2b & Table B.2 in EN 1991-1-5):

- Temperature T1
  \[ T_{R1} = \text{TABLE 2 for } h = 200, \ ds = 150 \]
  \[ = -2.3 \degree C \]
- Temperature T2
  \[ T_{R2} = -8 \degree C \]
- Temperature T3
  \[ T_{R3} = -8 \degree C \]

Reverse temperature diagram
Sample output for SCALE Proforma 225. (ans=2)

Composite construction: BS5950, BS5400, Eurocode 4  Made by: IFB
Temp effects in composite bridge decks-BS5400&DTp  Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.  Ref No: SC225 EC

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>STRESSES (N/mm²) (tension is positive)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Restrained</td>
</tr>
<tr>
<td>0</td>
<td>0.8556</td>
</tr>
<tr>
<td>120</td>
<td>0</td>
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<tr>
<td>125</td>
<td>0</td>
</tr>
<tr>
<td>125</td>
<td>0</td>
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<tr>
<td>200</td>
<td>0</td>
</tr>
<tr>
<td>200</td>
<td>0</td>
</tr>
<tr>
<td>215</td>
<td>0.756</td>
</tr>
<tr>
<td>215</td>
<td>0.6435</td>
</tr>
<tr>
<td>265</td>
<td>2.7885</td>
</tr>
<tr>
<td>265</td>
<td>2.47</td>
</tr>
<tr>
<td>600</td>
<td>15.2</td>
</tr>
<tr>
<td>865</td>
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<td>940</td>
<td>28.8</td>
</tr>
<tr>
<td>955</td>
<td>28.8</td>
</tr>
</tbody>
</table>

Member length L=10 m
Load factor for temperature gamma=1

**Plane frame analysis**

Axial and bending effects can be analysed in a plane frame model.
Bending effects for hogging releasing moment:

\[
\theta = \frac{RM \times L}{EI}
\]

End 1 \[\theta\] \[\theta\] End 2

* * * deflected shape of released member
\[\cdots\cdots\] tangent to deflected shape at member ends

Calculate the angle (2*\(\theta\)) in radians
where 2*\(\theta\) = (RM*L)/(E*I)
NL-STRESS will apply the opposite of the releasing moment (RM) to the released member and rotate it by 2*theta. The member is then clamped in the frame and let go.

Member distortion ( 2*theta ) \( \text{mdfmp} = \frac{\text{RM} \times \text{L}}{\text{EI}} = -0.001266 \) radians

Value of distortion for input to NL-STRESS

\( \text{disfmp} = \text{mdfmp} \times \gamma = -0.001266 \) radians

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\(<\text{members}> \text{ROTATION Z} -0.001266</\text{members}>\)

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

Longitudinal effects for compressive releasing force:

Released member

\[ \begin{array}{c}
\text{L} \\
\text{dL}
\end{array} \]

\(<- - - \)

Calculate the contraction of the member \( \text{dL} = \frac{\text{RF} \times \text{L}}{\text{A} \times \text{E}} \) in metres

NL-STRESS will apply the opposite of the releasing force (RF) to the released member and extend it by \( \text{dL} \). The member is then clamped in the frame and let go.

Member distortion

\( \text{mdffr} = \text{RF} \times \text{L} \times 1000 / \text{EQ1} = -0.48685 \times 10^{-3} \) m

Value of distortion for input to NL-STRESS

\( \text{disffr} = \text{mdffr} \times \gamma = -0.48685 \times 10^{-3} \) m

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\(<\text{members}> \text{DISTORTION X} -0.48685 \times 10^{-3}</\text{members}>\)

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.
Location: Ex1 - External beam

Shrinkage modified by creep in composite bridge decks to

BS5400-5:1979 & DOT Standard BD 16/82 (Amendment No. 1)

surfacing

---

concrete deck

Deck on Truss or Plate Girders

Short term value of modulus          EMC = 31000 N/mm²
Creep reduction factor               phic = 0.4
Shrinkage strain                    esc = -0.2E-3
Value of Young's modulus            EM = 205000 N/mm²
Number of cross-sections            ncs = 4
Breadth at top                      xbt(11) = 2650 mm
Breadth at bottom                   xbb(11) = 2650 mm
Depth                               xd(11) = 220 mm
Material (1=Steel, 2=Concrete)      mi(11) = 2
Breadth at top                      xbt(21) = 500 mm
Breadth at bottom                   xbb(21) = 500 mm
Depth                               xd(21) = 25 mm
Material (1=Steel, 2=Concrete)      mi(21) = 1
Breadth at top                      xbt(31) = 12 mm
Breadth at bottom                   xbb(31) = 12 mm
Depth                               xd(31) = 1345 mm
Material (1=Steel, 2=Concrete)      mi(31) = 1
Breadth at top                      xbt(41) = 600 mm
Breadth at bottom                   xbb(41) = 600 mm
Depth                               xd(41) = 30 mm
Material (1=Steel, 2=Concrete)      mi(41) = 1
<table>
<thead>
<tr>
<th>Depth from top</th>
<th>STRESSES (N/mm²) (tension is positive)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Depth</td>
</tr>
<tr>
<td>0</td>
<td>2.48</td>
</tr>
<tr>
<td>220</td>
<td>2.48</td>
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<tr>
<td>225</td>
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<tr>
<td>245</td>
<td>0</td>
</tr>
<tr>
<td>1590</td>
<td>0</td>
</tr>
<tr>
<td>1620</td>
<td>0</td>
</tr>
</tbody>
</table>

Member length
L=10 m
Partial load factor for shrinkage gamma=1

**Plane frame analysis**

Axial and bending effects can be analysed in a plane frame model. Bending effects for sagging releasing moment:

\[
\begin{align*}
&\text{End 1} \quad \ldots \quad \text{L} \quad \ldots \quad \text{End 2} \\
&\quad \text{(*)theta} \quad \text{theta( (*)} \\
&\quad \text{(*)} \quad \text{(*)} \quad \text{(*)} \quad \text{(*)} \\
&\quad \text{(*)} \quad \text{(*)} \quad \text{(*)} \quad \text{(*)} \\
&\quad \text{(*)} \quad \text{(*)} \quad \text{(*)} \quad 2*\text{theta} \\
&\quad \text{(*)} \quad \text{(*)} \\
&\quad \text{(*)} \quad \text{(*)} \\
&\quad \text{(*)} \quad \text{(*)} \\
&\quad \text{(*)} \\
\end{align*}
\]

* * * deflected shape of released member
* * * tangent to deflected shape at member ends

Calculate the angle \(2*\text{theta}\) in radians
where \(2*\text{theta} = \frac{\text{RM}*L}{\text{E}*\text{I}}\)

NL-STRESS will apply the opposite of the releasing moment (RM) to the released member and rotate it by \(2*\text{theta}\). The member is then clamped in the frame and let go.

Member distortion \(2*\text{theta}\)
m弥漫fr=\(\frac{\text{RM}*L}{\text{E}*\text{I}}\)=0.0010983 radians

Value of distortion for input to NL-STRESS
d弥漫fr=m弥漫fr*\text{gamma}=0.0010983 radians

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\(<\text{members}> \quad \text{ROTATION Z} \quad 0.0010983\)

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.
Longitudinal effects for compressive releasing force:

Released member \[\text{\textendash} \rightarrow \text{\textendash}\]

\[\begin{align*}
\text{\textendash} & \quad \text{\textendash} \\
\left(\begin{array}{c}
dL \\
L
\end{array}\right) & \quad \text{\textendash} \\
\end{align*}\]

\[\text{\textendash} \rightarrow \text{\textendash}\]

Calculate the contraction of the member \(dL = \frac{(RF \times L)}{(A \times E)}\) in metres

NL-STRESS will apply the opposite of the releasing force (RF) to the released member and extend it by \(dL\). The member is then clamped in the frame and let go.

Member distortion \(mdffr = \frac{(RF \times L)}{1000} \times E = -0.86111E-3 \text{ m}\)

Value of distortion for input to NL-STRESS \(disffr = mdffr \times \gamma = -0.86111E-3 \text{ m}\)

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\(<\text{members}> \text{ DISTORTION X -0.86111E-3}\)

The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
**Location:** Ex2 - Inner beam

Shrinkage modified by creep in composite bridge decks to

**BS5400-5:1979 & DOT Standard BD 16/82 (Amendment No. 1)**

![Diagram of concrete deck and box girders]

**Number of cross-sections**  
ncs = 7

<table>
<thead>
<tr>
<th>Breadth at top</th>
<th>xbt(11) = 1000 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth at bottom</td>
<td>xbb(11) = 800 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(11) = 125 mm</td>
</tr>
<tr>
<td>Material (1=Steel , 2=Concrete)</td>
<td>mi(11) = 2</td>
</tr>
<tr>
<td>Creep reduction factor</td>
<td>phi(11) = 0.3</td>
</tr>
<tr>
<td>Shrinkage strain</td>
<td>ecs(11) = -0.2E-3</td>
</tr>
<tr>
<td>Short term Young's modulus</td>
<td>ES(11) = 31000 N/mm²</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(21) = 800 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(21) = 800 mm</td>
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<td>mi(21) = 2</td>
</tr>
<tr>
<td>Creep reduction factor</td>
<td>phi(21) = 0.5</td>
</tr>
<tr>
<td>Shrinkage strain</td>
<td>ecs(21) = -0.3E-3</td>
</tr>
<tr>
<td>Short term Young's modulus</td>
<td>ES(21) = 25000 N/mm²</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(31) = 800 mm</td>
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<tr>
<td>Breadth at bottom</td>
<td>xbb(31) = 800 mm</td>
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<tr>
<td>Depth</td>
<td>xd(31) = 15 mm</td>
</tr>
<tr>
<td>Material (1=Steel , 2=Concrete)</td>
<td>mi(31) = 1</td>
</tr>
<tr>
<td>Young's modulus for steel</td>
<td>E(31) = 205000 N/mm²</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(41) = 200 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(41) = 15 mm</td>
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<td>Depth</td>
<td>xd(41) = 50 mm</td>
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<tr>
<td>Material (1=Steel , 2=Concrete)</td>
<td>mi(41) = 1</td>
</tr>
<tr>
<td>Young's modulus for steel</td>
<td>E(41) = 195000 N/mm²</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(51) = 15 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(51) = 15 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(51) = 600 mm</td>
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<tr>
<td>Material (1=Steel , 2=Concrete)</td>
<td>mi(51) = 1</td>
</tr>
<tr>
<td>Young's modulus for steel</td>
<td>E(51) = 190000 N/mm²</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(61) = 15 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(61) = 400 mm</td>
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<tr>
<td>Depth</td>
<td>xd(61) = 75 mm</td>
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<tr>
<td>Material (1=Steel , 2=Concrete)</td>
<td>mi(61) = 1</td>
</tr>
</tbody>
</table>
Sample output for SCALE Proforma 226. (ans=2)  
Page: 2
Composite construction: BS5950, BS5400, Eurocode 4  
Made by: IFB
Diff shrinkage - composite bridge decks-BS5400&DTp  
Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.  
Ref No: SC226 BS

Young's modulus for steel  
Breadth at top  
Breadth at bottom  
Depth  
Material (1=Steel, 2=Concrete)  
Young's modulus for steel

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restained force</th>
<th>Releasing moment</th>
<th>Releasing</th>
<th>Self equilibrating</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.86</td>
<td>-0.32458</td>
<td>-0.63123</td>
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</tr>
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<td>-8.9245</td>
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<td>-14.939</td>
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<td>-6.9466</td>
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<td>0.52387</td>
</tr>
<tr>
<td>940</td>
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<td>-7.3291</td>
<td>9.7697</td>
<td>2.4406</td>
</tr>
<tr>
<td>940</td>
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<td>11.165</td>
<td>2.7893</td>
</tr>
<tr>
<td>955</td>
<td>0</td>
<td>-8.3761</td>
<td>11.604</td>
<td>3.2274</td>
</tr>
</tbody>
</table>

Member length  
L=10 m  
Partial load factor for shrinkage  
gamma=1

**Plane frame analysis**

Axial and bending effects can be analysed in a plane frame model.  
Bending effects for sagging releasing moment:

\[
\theta = \frac{(RM \times L)}{(E \times I)}
\]

Where 2*theta is the angle in radians where:

Calculate the angle (2*theta) in radians

```
SCALE 5.48                     Office 1007                 Proforma 226
```
NL-STRESS will apply the opposite of the releasing moment (RM) to the released member and rotate it by 2*theta. The member is then clamped in the frame and let go.

Member distortion (2*theta) \( \text{mdmfr} = \frac{\text{RM} \times L}{EI} = 0.001217 \text{ radians} \)

Value of distortion for input to NL-STRESS
\( \text{dismfr} = \text{mdmfr} \times \gamma = 0.001217 \text{ radians} \)

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[
\langle \text{members} \rangle \text{ ROTATION Z 0.001217}
\]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

Longitudinal effects for compressive releasing force:

Released member

\[
\begin{array}{c}
\langle \text{dL} \rangle \\
\text{L} \\
\text{member distortion} \\
\text{mdffr} = \frac{(\text{RF} \times L) \times 1000}{\text{EQ1}} = -0.34901 \times 10^{-3} \text{ m} \\
\text{dissffr} = \text{mdffr} \times \gamma = -0.34901 \times 10^{-3} \text{ m}
\end{array}
\]

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[
\langle \text{members} \rangle \text{ DISTORTION X -0.34901E-3}
\]

The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
Location: Ex1 - External beam

Shrinkage modified by creep in composite bridge decks to


Deck on Truss or Plate Girders

Short term value of modulus \( EMC = 31000 \text{ N/mm}^2 \)
Ambient relative humidity \( RH = 80\% \)
Total shrinkage strain \( ecsc = -0.2 \times 10^{-3} \)
Value of Young's modulus \( EMS = 210000 \text{ N/mm}^2 \)
Number of cross-sections \( ncs = 4 \)
Breadth at top \( xbt(11) = 2650 \text{ mm} \)
Breadth at bottom \( xbb(11) = 2650 \text{ mm} \)
Depth \( xd(11) = 220 \text{ mm} \)
Material \( 1 = \text{Steel}, 2 = \text{Concrete} \) \( mi(11) = 2 \)
Total shrinkage strain \( ecs(11) = ecsc = -0.2 \times 10^{-3} \)
Breadth at top \( xbt(21) = 500 \text{ mm} \)
Breadth at bottom \( xbb(21) = 500 \text{ mm} \)
Depth \( xd(21) = 25 \text{ mm} \)
Material \( 1 = \text{Steel}, 2 = \text{Concrete} \) \( mi(21) = 1 \)
Breadth at top \( xbt(31) = 12 \text{ mm} \)
Breadth at bottom \( xbb(31) = 12 \text{ mm} \)
Depth \( xd(31) = 1345 \text{ mm} \)
Material \( 1 = \text{Steel}, 2 = \text{Concrete} \) \( mi(31) = 1 \)
Breadth at top \( xbt(41) = 600 \text{ mm} \)
Breadth at bottom \( xbb(41) = 600 \text{ mm} \)
Depth \( xd(41) = 30 \text{ mm} \)
Material \( 1 = \text{Steel}, 2 = \text{Concrete} \) \( mi(41) = 1 \)
<table>
<thead>
<tr>
<th>Depth from top</th>
<th>STRESSES (N/mm²) (tension is positive)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Restricted</td>
<td>Releasing force</td>
</tr>
<tr>
<td>0</td>
<td>2.48</td>
<td>-1.0532</td>
</tr>
<tr>
<td>220</td>
<td>2.48</td>
<td>-1.0532</td>
</tr>
<tr>
<td>220</td>
<td>0</td>
<td>-17.836</td>
</tr>
<tr>
<td>245</td>
<td>0</td>
<td>-17.836</td>
</tr>
<tr>
<td>1590</td>
<td>0</td>
<td>-17.836</td>
</tr>
<tr>
<td>1620</td>
<td>0</td>
<td>-17.836</td>
</tr>
</tbody>
</table>

Member length $L=10\,\text{m}$  
Partial load factor for shrinkage $\gamma=1$

**Plane frame analysis**

Axial and bending effects can be analysed in a plane frame model. Bending effects for sagging releasing moment:

\[\theta = \frac{RM \cdot L}{EI}\]

Where $\theta$ is the deflection, $RM$ is the releasing moment, $L$ is the member length, $E$ is the modulus of elasticity, and $I$ is the moment of inertia.

\[\theta_2 = 2\theta\]

The deflected shape of the released member is calculated, and NL-STRESS will apply the opposite of the releasing moment ($RM$) to the released member and rotate it by $2\theta$. The member is then clamped in the frame and let go.

Member distortion ($2\theta$) $mdmfr=\frac{RM \cdot L}{EI}=0.0010906$ radians

Value of distortion for input to NL-STRESS $dismfr=mdmfr \cdot \gamma=0.0010906$ radians

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[\text{<members> ROTATION Z 0.0010906}\]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.
Longitudinal effects for compressive releasing force:

Released member

\[
\begin{align*}
\text{\(dL\)} & \quad \text{\(L\)} \\
\text{\(<\rightarrow\)} & \quad \text{\(\rightarrow\)}
\end{align*}
\]

Calculate the contraction of the member \(dL = (RF \times L)/(A \times E)\) in metres

NL-STRESS will apply the opposite of the releasing force (RF) to the released member and extend it by \(dL\). The member is then clamped in the frame and let go.

Member distortion \(mdffr = (RF \times L) \times 1000/EQ1 = -0.84932E-3\) m

Value of distortion for input to NL-STRESS \(disffr = mdffr \times \gamma = -0.84932E-3\) m

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\(<\text{members}>\) DISTORTION X -0.84932E-3

The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
Location: Ex2 - Inner beam

Shrinkage modified by creep in composite bridge decks to


Deck on box girders

<table>
<thead>
<tr>
<th>Number of cross-sections</th>
<th>ncs=7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth at top</td>
<td>xbt(11)=1000 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(11)=800 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(11)=125 mm</td>
</tr>
<tr>
<td>Material (1=Steel, 2=Concrete)</td>
<td>m1(11)=2</td>
</tr>
<tr>
<td>Ambient relative humidity</td>
<td>RH=80 %</td>
</tr>
<tr>
<td>Total shrinkage strain</td>
<td>ecs(11)=-0.2E-3</td>
</tr>
<tr>
<td>Short term Young's modulus</td>
<td>ES(11)=31000 N/mm²</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(21)=800 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(21)=800 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(21)=75 mm</td>
</tr>
<tr>
<td>Material (1=Steel, 2=Concrete)</td>
<td>m1(21)=2</td>
</tr>
<tr>
<td>Ambient relative humidity</td>
<td>RH=80 %</td>
</tr>
<tr>
<td>Total shrinkage strain</td>
<td>ecs(21)=-0.3E-3</td>
</tr>
<tr>
<td>Short term Young's modulus</td>
<td>ES(21)=25000 N/mm²</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(31)=800 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(31)=800 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(31)=15 mm</td>
</tr>
<tr>
<td>Material (1=Steel, 2=Concrete)</td>
<td>m1(31)=1</td>
</tr>
<tr>
<td>Young's modulus for steel</td>
<td>E(31)=210000 N/mm²</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(41)=200 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(41)=15 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(41)=50 mm</td>
</tr>
<tr>
<td>Material (1=Steel, 2=Concrete)</td>
<td>m1(41)=1</td>
</tr>
<tr>
<td>Young's modulus for steel</td>
<td>E(41)=195000 N/mm²</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(51)=15 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(51)=15 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(51)=600 mm</td>
</tr>
<tr>
<td>Material (1=Steel, 2=Concrete)</td>
<td>m1(51)=1</td>
</tr>
<tr>
<td>Young's modulus for steel</td>
<td>E(51)=190000 N/mm²</td>
</tr>
<tr>
<td>Breadth at top</td>
<td>xbt(61)=15 mm</td>
</tr>
<tr>
<td>Breadth at bottom</td>
<td>xbb(61)=400 mm</td>
</tr>
<tr>
<td>Depth</td>
<td>xd(61)=75 mm</td>
</tr>
<tr>
<td>Material (1=Steel, 2=Concrete)</td>
<td>m1(61)=1</td>
</tr>
</tbody>
</table>
Young's modulus for steel \( E(61) = 210000 \, \text{N/mm}^2 \)

Breadth at top \( x_{bt}(71) = 600 \, \text{mm} \)

Breadth at bottom \( x_{bb}(71) = 600 \, \text{mm} \)

Depth \( x_d(71) = 15 \, \text{mm} \)

Material (1=Steel, 2=Concrete) \( m_i(71) = 1 \)

Young's modulus for steel \( E(71) = 240000 \, \text{N/mm}^2 \)

**STRESSES** \( \text{N/mm}^2 \) (tension is positive)

<table>
<thead>
<tr>
<th>Depth from top</th>
<th>Restrained force</th>
<th>Releasing moment</th>
<th>Releasing equilibrating moment</th>
<th>Self equilibrating moment</th>
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<tbody>
<tr>
<td>0</td>
<td>1.86</td>
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<td>-0.62406</td>
<td>0.91292</td>
</tr>
<tr>
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<td>1.86</td>
<td>-0.32302</td>
<td>-0.48377</td>
<td>1.0532</td>
</tr>
<tr>
<td>125</td>
<td>3.75</td>
<td>-0.43416</td>
<td>-0.65023</td>
<td>2.6656</td>
</tr>
<tr>
<td>200</td>
<td>3.75</td>
<td>-0.43416</td>
<td>-0.53709</td>
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</tr>
<tr>
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<td>-9.0231</td>
<td>-16.317</td>
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<tr>
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<td>0.48494</td>
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<td>7.8299</td>
<td>0.53598</td>
</tr>
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<td>940</td>
<td>0</td>
<td>-7.2939</td>
<td>9.7306</td>
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<tr>
<td>940</td>
<td>0</td>
<td>-8.3359</td>
<td>11.121</td>
<td>2.7848</td>
</tr>
<tr>
<td>955</td>
<td>0</td>
<td>-8.3359</td>
<td>11.555</td>
<td>3.2193</td>
</tr>
</tbody>
</table>

Member length \( L = 10 \, \text{m} \)

Partial load factor for shrinkage \( \gamma = 1 \)

**Plane frame analysis**

Axial and bending effects can be analysed in a plane frame model.

Bending effects for sagging releasing moment:

\[
\theta = \frac{RM \cdot L}{E \cdot I}
\]

End 1

\[
\theta, \theta(\ast\ast) \, \text{theta}
\]

End 2

\[
\theta, \theta(\ast\ast) \, \text{theta}
\]

\[
\text{2}\theta, \text{2}\theta(\ast\ast) \, \theta(\ast\ast)
\]

* * * deflected shape of released member
* * * tangent to deflected shape at member ends

Calculate the angle \( 2\theta \) in radians

where \( 2\theta = \frac{RM \cdot L}{E \cdot I} \)
NL-STRESS will apply the opposite of the releasing moment (RM) to the released member and rotate it by 2*theta. The member is then clamped in the frame and let go.

Member distortion (2*theta) \[
mdmfr=(RM*L)/EI=0.0012068 \text{ radians}
\]

Value of distortion for input to NL-STRESS
\[
dismfr=mdmfr*gamma=0.0012068 \text{ radians}
\]

The distortion should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[
<\text{members}> \text{ ROTATION Z 0.0012068}
\]

The equilibrating stresses should be taken into account when stresses at a section due to combinations of loading are calculated.

Longitudinal effects for compressive releasing force:

Released member \[\begin{array}{c}
\text{L} \\
\hline
<\text{dL}> \\
\end{array}\] \[\begin{array}{c}
\text{---} \\
\text{---} \\
\end{array}\]

Calculate the contraction of the member dL=(RF*L)/(A*E) in metres

NL-STRESS will apply the opposite of the releasing force (RF) to the released member and extend it by dL. The member is then clamped in the frame and let go.

Member distortion \[
mdffr=(RF*L)*1000/EQ1=-0.34733E-3 \text{ m}
\]

Value of distortion for input to NL-STRESS
\[
disffr=mdffr*gamma=-0.34733E-3 \text{ m}
\]

The value should be input into the plane frame analysis using the MEMBER DISTORTIONS option with the following format.

\[
<\text{members}> \text{ DISTORTION X -0.34733E-3}
\]

The equilibrating stresses should be taken into account when the stresses at a section due to combinations of loading are calculated.
Location: Rectangular section with axial load and bending

Rectangular timber member

subject to axial load and bending about z-z axis

Calculations in accordance with BS5268-2:2002.

Design BM about zz (positive) \( M_z = 0.5 \text{ kNm} \)
Design shear force in y direction \( V = 0 \text{ kN} \)
Design axial load (+ve compress) \( F = 5 \text{ kN} \)
Eff length for bending about zz \( L_{ez}' = 2.346 \text{ m} \)
Depth of section \( d = 97 \text{ mm} \)
Width of section \( b = 97 \text{ mm} \)
Eff length for bending about yy \( L_{ey}' = 0 \text{ m} \)
Strength class C18 to Table 8.
Timber service class adopted \( t_m\text{class}=1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1.25 \)
Depth factor \( K_7 = (300/d)^{0.11} = 1.1322 \)
Load-sharing modification factor \( K_8 = 1.0 \)
Modification factor \( K_{12} = (0.5 + (1+\eta)*C/2) - ((0.5 + (1+\eta)*C/2)^2 - C)^{0.5} = 0.4319 \)

**DESIGN**

| Depth of section | 97 mm |
| Width of section | 97 mm |

**SUMMARY**

Strength class C18 to Table 8.
Timber moisture class 1
Applied comprn stress \( 0.53141 \text{ N/mm}^2 \)
Perm iss comprn stress \( 3.8331 \text{ N/mm}^2 \)
Applied bending stress \( 3.287 \text{ N/mm}^2 \)
Perm iss bending stress \( 8.2087 \text{ N/mm}^2 \)
Interaction factor 0.55611
Location: Example 2 User defined grade stresses

Rectangular timber member

subject to axial load and

bending about z-z axis

Calculations in accordance with BS5268-2:2002.

Design BM about zz (positive) \( M_z = 1.45 \text{ kNm} \)
Design shear force in y direction \( V = 0 \text{ kN} \)
Design axial load (+ve compress) \( F = 0 \text{ kN} \)
Eff length for bending about zz \( L_{ez}' = 2.346 \text{ m} \)
Depth of section \( d = 122 \text{ mm} \)
Width of section \( b = 97 \text{ mm} \)
Eff length for bending about yy \( L_{ey}' = 1.2 \text{ m} \)
Douglas fir table 10
Bending parallel to the grain \( b_{parg} = 6.2 \text{ N/mm}^2 \)
Compress parallel to the grain \( c_{parg} = 6.6 \text{ N/mm}^2 \)
Compress perpendicular to the grain \( c_{perd} = 2.4 \text{ N/mm}^2 \)
Tension parallel to the grain \( t_{parg} = 3.7 \text{ N/mm}^2 \)
Shear parallel to the grain \( s_{parg} = 0.88 \text{ N/mm}^2 \)
Mean modulus of elasticity \( E_{mean} = 10000 \text{ N/mm}^2 \)
Minimum modulus of elasticity \( E_{min} = 7000 \text{ N/mm}^2 \)
Timber service class adopted \( t_{mclass} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1.25 \)
Depth factor \( K_7 = (300/d)^{0.11} = 1.104 \)
Load-sharing modification factor \( K_8 = 1.0 \) clause 2.10.11

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMAR</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of section</td>
<td>122 mm</td>
<td></td>
</tr>
<tr>
<td>Width of section</td>
<td>97 mm</td>
<td></td>
</tr>
<tr>
<td>Douglas fir table 10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Timber moisture class</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Applied bending stress</td>
<td>6.026 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Permiss bending stress</td>
<td>8.5563 N/mm²</td>
<td></td>
</tr>
</tbody>
</table>
Location: Example 3 TR20 grade stress

Rectangular timber member

subject to axial load and

bending about z-z axis

Calculations in accordance

Design BM about zz (positive)     \( M_z = 0 \) kNm
Design shear force in y direction \( V = 0 \) kN
Design axial load (+ve compress) \( F = 6.2 \) kN
Eff length for bending about zz \( L_{ez}' = 3.6 \) m
Depth of section \( d = 97 \) mm
Width of section \( b = 97 \) mm
Eff length for bending about yy \( L_{ey}' = 0 \) m
Strength class TR20 to Table 9.
Timber service class adopted \( tmclass = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1.75 \)
Load-sharing modification factor \( K_8 = 1.0 \)
Modification factor \( K_{12} = (0.5 + (1 + \eta) \times C/2)^2 - ((0.5 + (1 + \eta) \times C/2)^2 - C)^0.5 = 0.16348 \)

DESIGN

<table>
<thead>
<tr>
<th>Depth of section</th>
<th>97 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of section</td>
<td>97 mm</td>
</tr>
</tbody>
</table>

SUMMARY

<table>
<thead>
<tr>
<th>Strength class TR20 to Table 9.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber moisture class</td>
</tr>
<tr>
<td>Applied comprn stress</td>
</tr>
<tr>
<td>Permiss comprn stress</td>
</tr>
</tbody>
</table>
### Location: Example 4 C24 grade stress with tensile axial load

**Rectangular timber member**

Subject to axial load and bending about z-z axis

Calculations in accordance with BS5268-2:2002.

Design BM about zz (positive) \( M_z = 0.4953 \) kNm

Design shear force in y direction \( V = 0 \) kN

Design axial load (+ve compress) \( F = -4.9 \) kN

Eff length for bending about zz \( L_{ez}' = 2.346 \) m

Depth of section \( d = 97 \) mm

Width of section \( b = 97 \) mm

Eff length for bending about yy \( L_{ey}' = 0 \) m

Strength class C24 to Table 8.

Tensile area of cross section \( A_t = 7469 \) mm²

Timber service class adopted \( t_m \) class = 3

Timber stress grades and moduli adjusted as Table 16.

Duration of loading \( K_3 = 1.75 \)

Depth factor \( K_7 = (300/d)^{0.11} = 1.1322 \)

Load-sharing modification factor \( K_8 = 1.0 \)

Width modification factor \( K_{14} = (300/h)^{0.11} = 1.1322 \)

#### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Depth of section</th>
<th>97 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of section</td>
<td>97 mm</td>
</tr>
<tr>
<td>Strength class C24 to Table 8.</td>
<td></td>
</tr>
<tr>
<td>Tensile cross sec area</td>
<td>7469 mm²</td>
</tr>
<tr>
<td>Timber moisture class</td>
<td>3</td>
</tr>
<tr>
<td>Applied tension stress</td>
<td>( 0.65604 ) N/mm²</td>
</tr>
<tr>
<td>Permiss tension stress</td>
<td>( 7.1331 ) N/mm²</td>
</tr>
<tr>
<td>Applied bending stress</td>
<td>( 3.2561 ) N/mm²</td>
</tr>
<tr>
<td>Permiss bending stress</td>
<td>( 11.889 ) N/mm²</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>( 0.36586 )</td>
</tr>
</tbody>
</table>
Location: Rectangular section with axial load and bending

Rectangular timber section

With axial load and bending about y-y axis, in accordance with Eurocode 5 incorporating National Annex Amendment No.2.

Design BM about yy (positive) \( M_d = 1.83 \text{ kNm} \)
Design shear force in z direction \( V_d = 0 \text{ kN} \)
Design axial load (+ve compress) \( N_d = 45.75 \text{ kN} \)
Eff length for bending about yy \( l_{ey} = 3.75 \text{ m} \)
Depth of section \( h = 250 \text{ mm} \)
Width of section \( b = 100 \text{ mm} \)
Eff length for bending about zz \( l_{ez} = 3.75 \text{ m} \)
Timber strength class C22.
Depth factor (Clause 3.2) \( k_h = 1.0 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)
Instability factor \( k_{cy} = 1/(k_y+(k_y^2-r_{sy}^2)^{0.5}) = 0.75898 \)
Instability factor \( k_{cz} = 1/(k_z+(k_z^2-r_{sz}^2)^{0.5}) = 0.17917 \)
Shear reduction factor \( k_{sh} = 0.67 \)
Effective length about zz \( L_e = 3 \text{ m} \)

DESIGN

| Depth of section | 250 mm |
| Width of section | 100 mm |

SUMMARY

| Timber strength class | C22 |
| Timber moisture class | 2 |
| Design compr. stress | 1.83 N/mm² |
| Design compr. strength | 12.308 N/mm² |
| Design bending stress | 1.7568 N/mm² |
| Design bending strength | 13.538 N/mm² |
| Interaction factor | 0.8467 ≤ 1 |
Location: Example 2 User defined grade stresses

Rectangular timber section

With axial load and bending about y-y axis, in accordance with Eurocode 5 incorporating National Annex Amendment No.2.

Design BM about yy (positive) \( M_d = 2.0735 \text{ kNm} \)
Design shear force in z direction \( V_d = 0 \text{ kN} \)
Design axial load (+ve compress) \( N_d = 0 \text{ kN} \)
Eff length for bending about yy \( l_{ey} = 2.346 \text{ m} \)
Depth of section \( h = 122 \text{ mm} \)
Width of section \( b = 97 \text{ mm} \)
Eff length for bending about zz \( l_{ez} = 1.2 \text{ m} \)

Douglas fir
Bending parallel to the grain \( f_{mk} = 24 \text{ N/mm}^2 \)
Compress parallel to the grain \( f_{c0k} = 21 \text{ N/mm}^2 \)
Compress perp to the grain \( f_{c90k} = 2.5 \text{ N/mm}^2 \)
Tension parallel to the grain \( f_{t0k} = 14 \text{ N/mm}^2 \)
Shear parallel to the grain \( f_{vk} = 2.5 \text{ N/mm}^2 \)
Mean modulus of elasticity \( E_0\text{mean} = 10000 \text{ N/mm}^2 \)
5th percentile MOE \( E_0\text{05} = 7000 \text{ N/mm}^2 \)
Depth factor (Clause 3.2) \( k_h = \left( \frac{150}{h} \right)^{0.2} = 1.0422 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)
Shear reduction factor \( k_{sh} = 0.67 \)

DESIGN

<table>
<thead>
<tr>
<th>Summary of Design Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of section</td>
</tr>
<tr>
<td>Width of section</td>
</tr>
</tbody>
</table>

SUMMARY

<table>
<thead>
<tr>
<th>Summary of Design Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Douglas fir</td>
</tr>
<tr>
<td>Timber moisture class</td>
</tr>
<tr>
<td>Design bending stress</td>
</tr>
<tr>
<td>Design bending strength</td>
</tr>
<tr>
<td>Interaction factor</td>
</tr>
</tbody>
</table>
Location: Example 3

Rectangular timber section

With axial load and bending about y-y axis, in accordance with Eurocode 5 incorporating National Annex Amendment No.2.

Design BM about yy (positive) \( M_d = 0 \) kNm
Design shear force in z direction \( V_d = 0 \) kN
Design axial load (+ve compress) \( N_d = 8.866 \) kN
Eff length for bending about yy \( l_{ey} = 3.6 \) m
Depth of section \( h = 97 \) mm
Width of section \( b = 97 \) mm
Eff length for bending about zz \( l_{ez} = 0 \) m
Timber strength class D70.
Depth factor (Clause 3.2) \( k_h = (150/h)^{0.2} = 1.0911 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)
Instability factor \( k_{cy} = 1/(k_y + (k_y^2 - rs_{ry}^2)^{0.5}) = 0.24917 \)
Instability factor \( k_{cz} = 1/(k_z + (k_z^2 - rs_{rz}^2)^{0.5}) = 1.0638 \)
Max instability factor \( k_{cz} = 1 \)
Shear reduction factor \( k_{sh} = 0.67 \)

DESIGN

<table>
<thead>
<tr>
<th>Depth of section</th>
<th>97 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of section</td>
<td>97 mm</td>
</tr>
</tbody>
</table>

SUMMARY

- Timber strength class D70.
- Timber moisture class 1
- Design compr. stress \( 0.94229 \) N/mm²
- Design compr. strength \( 30.462 \) N/mm²
- Interaction factor \( 0.12415 \leq 1 \)
Location: Example 4 C24 grade stress with tensile axial load

Rectangular timber section

With axial load and bending about y-y axis, in accordance with Eurocode 5 incorporating National Annex Amendment No.2.

Design BM about yy (positive) \( \text{Md}=0.4953 \text{ kNm} \)
Design shear force in z direction \( \text{Vd}=0 \text{ kN} \)
Design axial load (+ve compress) \( \text{Nd}=-7.007 \text{ kN} \)
Eff length for bending about yy \( \text{ley}=2.346 \text{ m} \)
Depth of section \( h=97 \text{ mm} \)
Width of section \( b=97 \text{ mm} \)
Eff length for bending about zz \( \text{lez}=0 \text{ m} \)
Timber strength class C24.
Tensile area of cross section \( \text{At}=7469 \text{ mm}^2 \)
Depth factor (Clause 3.2) \( \text{kh}=(150/h)^{0.2}=1.0911 \)
Load-sharing modification factor \( \text{ksys}=1.0 \)
Width modification factor \( \text{kw}=(150/lardim)^{0.2}=1.0911 \)
Shear reduction factor \( \text{ksh}=0.67 \)

**DESIGN**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
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<tbody>
<tr>
<td>Depth of section</td>
<td>97 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>97 mm</td>
</tr>
</tbody>
</table>

**SUMMARY**

- Timber strength class C24.
- Tensile cross sec area: 7469 mm²
- Timber moisture class: 3
- Design tension stress: 0.93814 N/mm²
- Design tension strength: 10.953 N/mm²
- Design bending stress: 3.2561 N/mm²
- Design bending strength: 18.129 N/mm²
- Interaction factor: 0.26526 ≤ 1
Location: Glulam section with axial load & bending

Rectangular glulam member

subject to axial load and

bending about axis zz with

horizontal laminations.

Calculations in accordance with BS5268-2:2002.

Design BM about zz (+ve only) \( M_z' = 21.5 \text{ kNm} \)
Design shear force in y direction \( V' = 25 \text{ kN} \)
Design axial load (+ve compress) \( F' = 30.9 \text{ kN} \)
Eff length for bending about zz \( L_{ez}' = 4.5 \text{ m} \)
Number of laminates in section \( n_p = 10 \)
Thickness of laminates in section \( t_p = 45 \text{ mm} \)
Width of section \( b = 135 \text{ mm} \)
Eff length for bending about yy \( L_{ey}' = 0 \text{ m} \)
Strength class C30 to Table 8.
Timber service class adopted \( t_{m,\text{class}} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1.25 \)
Depth factor \( K_7 = 0.81^* (d^2 + 92300) / (d^2 + 56800) = 0.92089 \)

<table>
<thead>
<tr>
<th>Laminate Thickness</th>
<th>45 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of laminates</td>
<td>10</td>
</tr>
<tr>
<td>Depth of section</td>
<td>450 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>135 mm</td>
</tr>
</tbody>
</table>

**GRADE STRESSES**
Bending \( 11 \text{ N/mm}^2 \)
Compression \( \parallel \) to grain \( 8.6 \text{ N/mm}^2 \)
Shear \( 1.2 \text{ N/mm}^2 \)
Modulus of elasticity \( 12300 \text{ N/mm}^2 \)
Strength class C30 to Table 8.
Moisture service class 1
Dry stresses are appropriate
Applied comprn stress \( 0.50864 \text{ N/mm}^2 \)
Permiss comprn stress \( 9.3187 \text{ N/mm}^2 \)
Applied bending stress \( 4.7188 \text{ N/mm}^2 \)
Permiss bending stress \( 13.929 \text{ N/mm}^2 \)
Interaction factor \( 0.39551 \)
Applied shear stress \( 0.61728 \text{ N/mm}^2 \)
Permiss shear stress \( 2.235 \text{ N/mm}^2 \)
Location: Glulam section with bending and no axial load

Rectangular glulam member

subject to axial load and

bending about axis zz with

horizontal laminations.

Calculations in accordance with BS5268-2:2002.

Design BM about zz (+ve only) $M_z' = 25$ kNm
Design shear force in y direction $V' = 20$ kN
Design axial load (+ve compress) $F' = 0$ kN
Eff length for bending about zz $L_{ez}' = 6$ m
Number of laminates in section $n_p = 9$
Thickness of laminates in section $t_p = 45$ mm
Width of section $b = 115$ mm
Eff length for bending about yy $L_{ey}' = 1.5$ m
Whitewood SS grade
Bending parallel to the grain $b_{par} = 7.5$ N/mm²
Compress parallel to the grain $c_{par} = 7.9$ N/mm²
Compress perpendicular to the grain $c_{per} = 2.1$ N/mm²
Tension parallel to the grain $t_{par} = 4.5$ N/mm²
Shear parallel to the grain $s_{par} = 0.82$ N/mm²
Mean modulus of elasticity $E_{mean} = 10000$ N/mm²
Minimum modulus of elasticity $E_{min} = 7000$ N/mm²
Timber service class adopted $t_{mclass} = 2$
Timber service class modification factor $K_2 = 1$ as Table 16.
Duration of loading $K_3 = 1.25$
Depth factor $K_7 = 0.81 \times (d^2 + 92300) / (d^2 + 56800) = 0.94022$
Modification factor $K_{15} = 1.42$
Modification factor $K_{16} = 1.39$
Modification factor $K_{17} = 1.04$
Modification factor $K_{19} = 2.34$
Modification factor $K_{20} = 1.07$
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<tbody>
<tr>
<td>Laminate Thickness</td>
<td>45 mm</td>
</tr>
<tr>
<td>Number of laminates</td>
<td>9</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of section</td>
<td>405 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>115 mm</td>
</tr>
</tbody>
</table>

**GRADE STRESSES**

- Bending: 7.5 N/mm²
- Shear: 0.82 N/mm²
- Modulus of elasticity: 10000 N/mm²
- Whitewood SS grade: Moisture service class 2
- Dry stresses are appropriate

- Applied bending stress: 7.9521 N/mm²
- Permiss bending stress: 12.517 N/mm²
- Applied shear stress: 0.64412 N/mm²
- Permiss shear stress: 2.3985 N/mm²
Location: Glulam section bending with axial load (Class 2 Hardwood)

Rectangular glulam member
subject to axial load and
bending about axis zz with
horizontal laminations.
Calculations in accordance

Design BM about zz (+ve only) \( Mz' = 60 \text{ kNm} \)
Design shear force in y direction \( V' = 42 \text{ kN} \)
Design axial load (+ve compress) \( F' = 90 \text{ kN} \)
Eff length for bending about zz \( \text{Lez}' = 6 \text{ m} \)
Number of laminates in section \( np' = 12 \)
Thickness of laminates in section \( tp = 45 \text{ mm} \)
Width of section \( b = 90 \text{ mm} \)
Eff length for bending about yy \( \text{Ley}' = 1.5 \text{ m} \)
Strength class D70 to Table 8.
Timber service class adopted \( \text{tmclass} = 2 \)
Timber service class modification factor \( K2 = 1 \) as Table 16.
Duration of loading \( K3 = 1.5 \)
Depth factor \( K7 = 0.81^*(d^2+92300)/(d^2+56800) = 0.89253 \)

<table>
<thead>
<tr>
<th>Laminate Thickness</th>
<th>45 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of laminates</td>
<td>12</td>
</tr>
<tr>
<td>Depth of section</td>
<td>540 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>90 mm</td>
</tr>
</tbody>
</table>

GRADE STRESSES
Bending \( 23 \text{ N/mm}^2 \)
Compression // to grain \( 23 \text{ N/mm}^2 \)
Shear \( 2.6 \text{ N/mm}^2 \)
Modulus of elasticity \( 21000 \text{ N/mm}^2 \)
Strength class D70 to Table 8.
Moisture service class 2
Dry stresses are appropriate
Applied comprn stress \( 1.8519 \text{ N/mm}^2 \)
Permiss comprn stress \( 22.409 \text{ N/mm}^2 \)
Applied bending stress \( 13.717 \text{ N/mm}^2 \)
Permiss bending stress \( 33.872 \text{ N/mm}^2 \)
Interaction factor \( 0.49925 \)
Applied shear stress \( 1.2963 \text{ N/mm}^2 \)
Permiss shear stress \( 5.811 \text{ N/mm}^2 \)
Location: Glulam section bending with tensile axial load

Rectangular glulam member

subject to axial load and

bending about axis zz with

horizontal laminations.

Calculations in accordance with BS5268-2:2002.

Design BM about zz (+ve only) $M_z'=25$ kNm
Design shear force in $y$ direction $V'=25$ kN
Design axial load (+ve compress) $F'=-60$ kN
Eff length for bending about zz $L_ez'=6$ m
Number of laminates in section $n_p=10$
Thickness of laminates in section $t_p=45$ mm
Width of section $b=90$ mm
Eff length for bending about $yy$ $L_{ey} '=1.5$ m
Strength class C27 to Table 8.
Tensile area of section $A_t=40000$ mm$^2$
Timber service class adopted $tmclass=1$
Timber service class modification factor $K_2=1$ as Table 16.
Duration of loading $K_3=1.25$
Depth factor $K_7=0.81*(d^2+92300)/(d^2+56800)$
  $=0.92089$
Width modification factor $K_{14}=(300/h)^{0.11}=0.95638$

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laminate Thickness</td>
<td>45 mm</td>
</tr>
<tr>
<td>Number of laminates</td>
<td>10</td>
</tr>
<tr>
<td>Depth of section</td>
<td>450 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>90 mm</td>
</tr>
</tbody>
</table>

GRADE STRESSES

Bending 10 N/mm$^2$
Tension // to grain 6 N/mm$^2$
Shear 1.1 N/mm$^2$
Modulus of elasticity 12300 N/mm$^2$
Strength class C27 to Table 8.
Moisture service class 1
Dry stresses are appropriate
Applied tension stress 1.5 N/mm$^2$
Permiss tension stress 7.8901 N/mm$^2$
Applied bending stress 8.2305 N/mm$^2$
Permiss bending stress 12.662 N/mm$^2$
Interaction factor 0.84011
Applied shear stress 0.92593 N/mm$^2$
Permiss shear stress 2.0488 N/mm$^2$
Location: Glulam section with axial load & bending

Rectangular glulam member

Glulam is subject to axial load and bending about yy axis with horizontal laminations.

Design is in accordance on with EC5 incorporating NA Amendment No.2.

Design BM about yy (+ve only) \( M_d' = 31.175 \text{ kNm} \)
Design shear force in z direction \( V_d' = 36.25 \text{ kN} \)
Design axial load (+ve compress) \( N_d' = 44.805 \text{ kN} \)
Eff length for bending about yy \( l_{ey'} = 4.5 \text{ m} \)
Number of laminates in section \( n_p = 10 \)
Thickness of laminates in section \( t_p = 45 \text{ mm} \)
Width of section \( b = 135 \text{ mm} \)
Eff length for bending about zz \( l_{ez'} = 0 \text{ m} \)
Glulam strength class GL32h
Depth factor (Clause 3.3) \( k_h = (600/h)^{0.1} = 1.0292 \)
Instability factor \( k_{cy} = 1/(k_y + (k_y^2 - rsy^2)^{0.5}) = 0.9614 \)
Instability factor \( k_{cz} = 1/(k_z + (k_z^2 - rsrz^2)^{0.5}) = 1.0309 \)
Max instability factor \( k_{cz} = 1 \)
Shear reduction factor \( k_{sh} = 0.67 \)
Effective length about zz \( L_e = 3.6 \text{ m} \)
Interaction factor \( \text{fact}_1 = s_c0d/(k_{cy}*f_{cd}) + s_m/f_{md} = 0.30173 \)
Interaction factor \( \text{fact}_2 = (s_m/(k_{crit}*f_{md}))^2 + s_c0d/(k_{cz}*f_{cd}) = 0.30173 \)
Governing interaction factor \( \text{factor} = 0.30173 \)

<table>
<thead>
<tr>
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</thead>
<tbody>
<tr>
<td>Laminate thickness</td>
</tr>
<tr>
<td>Number of laminates</td>
</tr>
<tr>
<td>SUMMARY</td>
</tr>
<tr>
<td>Depth of section</td>
</tr>
<tr>
<td>Width of section</td>
</tr>
</tbody>
</table>

CHARACTERISTIC GRADE STRENGTHS

| Bending | 32 N/mm² |
| Compression // to grain | 32 N/mm² |
| Shear | 3.5 N/mm² |
| Modulus of elasticity | 14200 N/mm² |
| Glulam strength class GL32h |
| Moisture service class | 1 |
| Dry stresses are appropriate |
| Design comprn stress | 0.73753 N/mm² |
| Design comprn strength | 24.576 N/mm² |
| Design bending strength | 6.8422 N/mm² |
| Design bending strength | 25.293 N/mm² |
| Interaction factor | 0.30173 |
| Design shear stress | 0.89061 N/mm² |
| Design shear strength | 1.801 N/mm² |
Location: Glulam section with bending and no axial load

Rectangular Glulam column/tie with axial load

- Glulam is subject to axial load and bending about yy axis with horizontal laminations.
- Design is in accordance on with EC5 incorporating NA Amendment No.2.

Design BM about yy (+ve only) \( Md' = 49.7 \text{ kN} \)
Design shear force in z direction \( Vd' = 39.76 \text{ kN} \)
Design axial load (+ve compress) \( Nd' = 0 \text{ kN} \)
Eff length for bending about yy \( ley' = 6 \text{ m} \)
Number of laminates in section \( np = 9 \)
Thickness of laminates in section \( tp = 45 \text{ mm} \)
Width of section \( b = 115 \text{ mm} \)
Eff length for bending about zz \( lez' = 1.5 \text{ m} \)

Whitewood SS grade
- Bending parallel to the grain \( f_{mk} = 24 \text{ N/mm}^2 \)
- Compress parallel to the grain \( f_{c0k} = 21 \text{ N/mm}^2 \)
- Compress perp to the grain \( f_{c90k} = 2.5 \text{ N/mm}^2 \)
- Shear parallel to the grain \( f_{vk} = 3.5 \text{ N/mm}^2 \)
- Mean modulus of elasticity \( E_{0\text{mean}} = 11000 \text{ N/mm}^2 \)
- 5th percentile MOE \( E_{005} = 9100 \text{ N/mm}^2 \)
- Depth factor (Clause 3.3) \( kh = (600/h)^{0.1} = 1.0401 \)
- Shear reduction factor \( k_{sh} = 0.67 \)
- Effective length about zz \( Le = 4.8 \text{ m} \)

### DESIGN
- Laminate thickness \( 45 \text{ mm} \)
- Number of laminates \( 9 \)

### SUMMARY
- Depth of section \( 405 \text{ mm} \)
- Width of section \( 115 \text{ mm} \)

### CHARACTERISTIC GRADE STRENGTHS
- Bending \( 24 \text{ N/mm}^2 \)
- Shear \( 3.5 \text{ N/mm}^2 \)
- Modulus of elasticity \( 11000 \text{ N/mm}^2 \)
- Whitewood SS grade
- Moisture service class \( 2 \)
- Dry stresses are appropriate
- Design bending strength \( 15.809 \text{ N/mm}^2 \)
- Design bending strength \( 19.171 \text{ N/mm}^2 \)
- Interaction factor \( 0.82463 \)
- Design shear stress \( 1.2741 \text{ N/mm}^2 \)
- Design shear strength \( 1.801 \text{ N/mm}^2 \)
Location: Glulam section bending with axial load (Class 2 Hardwood)

Rectangular Glulam member

Glulam is subject to axial load and bending about yy axis with horizontal laminations.

Design is in accordance on with EC5 incorporating NA Amendment No.2.

Design BM about yy (+ve only) $M_d' = 85.2$ kNm
Design shear force in z direction $V_d' = 59.64$ kN
Design axial load (+ve compress) $N_d' = 127.8$ kN
Eff length for bending about yy $l_{ey}' = 6$ m
Number of laminates in section $n_p = 12$
Thickness of laminates in section $t_p = 45$ mm
Width of section $b = 90$ mm
Eff length for bending about zz $l_{ez}' = 1.5$ m
Glulam strength class GL32h
Depth factor (Clause 3.3) $k_h = (600/h)^{0.1} = 1.0106$
Instability factor $k_{cy} = 1/(k_y+(k_y^2-r_{sy}^2)^{0.5}) = 0.94783$
Instability factor $k_{cz} = 1/(k_z+(k_z^2-r_{sz}^2)^{0.5}) = 0.80174$
Shear reduction factor $k_{sh} = 0.67$
Interaction factor $fact_1 = sc0d/(k_{cy} * f_{cd}) + smd/f_{md} = 0.79749$
Interaction factor $fact_2 = (smd/(k_{crit} * f_{md}))^2 + sc0d/(k_{cz} * f_{cd}) = 0.60464$
Governing interaction factor $factor = 0.79749$

Design BM about yy (+ve only) $M_d' = 85.2$ kNm
Design shear force in z direction $V_d' = 59.64$ kN
Design axial load (+ve compress) $N_d' = 127.8$ kN
Eff length for bending about yy $l_{ey}' = 6$ m
Number of laminates in section $n_p = 12$
Thickness of laminates in section $t_p = 45$ mm
Width of section $b = 90$ mm
Eff length for bending about zz $l_{ez}' = 1.5$ m
Glulam strength class GL32h
Depth factor (Clause 3.3) $k_h = (600/h)^{0.1} = 1.0106$
Instability factor $k_{cy} = 1/(k_y+(k_y^2-r_{sy}^2)^{0.5}) = 0.94783$
Instability factor $k_{cz} = 1/(k_z+(k_z^2-r_{sz}^2)^{0.5}) = 0.80174$
Shear reduction factor $k_{sh} = 0.67$
Interaction factor $fact_1 = sc0d/(k_{cy} * f_{cd}) + smd/f_{md} = 0.79749$
Interaction factor $fact_2 = (smd/(k_{crit} * f_{md}))^2 + sc0d/(k_{cz} * f_{cd}) = 0.60464$
Governing interaction factor $factor = 0.79749$

Laminate thickness 45 mm
Number of laminates 12
Depth of section 540 mm
Width of section 90 mm

CHARACTERISTIC GRADE STRENGTHS
Bending 32 N/mm²
Compression // to grain 32 N/mm²
Shear 3.5 N/mm²
Modulus of elasticity 14200 N/mm²
Glulam strength class GL32h
Moisture service class 2
Dry stresses are appropriate
Design comprn stress 2.6296 N/mm²
Design comprn strength 27.648 N/mm²
Design bending strength 19.479 N/mm²
Design bending strength 27.941 N/mm²
Interaction factor 0.79749
Design shear stress 1.8316 N/mm²
Design shear strength 2.0261 N/mm²
**Location: Glulam section bending with tensile axial load**

**Rectangular glulam member**

Glulam is subject to axial load and bending about yy axis with horizontal laminations.

Design is in accordance on with EC5 incorporating NA Amendment No.2.

---

Design BM about yy (+ve only) \( M_d' = 35.5 \text{ kNm} \)

Design shear force in z direction \( V_d' = 35.5 \text{ kN} \)

Design axial load (+ve compress) \( N_d' = -85.2 \text{ kN} \)

Eff length for bending about yy \( l_{ey}' = 6 \text{ m} \)

Number of laminates in section \( n_p = 10 \)

Thickness of laminates in section \( t_p = 45 \text{ mm} \)

Width of section \( b = 90 \text{ mm} \)

Eff length for bending about zz \( l_{ez}' = 1.5 \text{ m} \)

Glulam strength class GL32h

Tensile area of section \( A_t = 40000 \text{ mm}^2 \)

Depth factor (Clause 3.3) \( k_h = (600/h)^{0.1} = 1.0292 \)

Width modification factor \( k_w = (600/lardim)^{0.1} = 1.0292 \)

Shear reduction factor \( k_{sh} = 0.67 \)

Interaction factor \( f_{act1} = std/ft0d+smd/fmd = 0.56733 \)

Interaction factor \( f_{act2} = (smd-std)/(k_{crit}\times fmd) = 0.37786 \)

Factor governing design \( f_{act} = 0.56733 \)

**DESIGN**

<table>
<thead>
<tr>
<th>Laminate thickness</th>
<th>45 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of laminates</td>
<td>10</td>
</tr>
<tr>
<td>Depth of section</td>
<td>450 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>90 mm</td>
</tr>
</tbody>
</table>

**CHARACTERISTIC GRADE STRENGTHS**

<table>
<thead>
<tr>
<th>Bending</th>
<th>32 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension // to grain</td>
<td>25.6 N/mm²</td>
</tr>
<tr>
<td>Shear</td>
<td>3.5 N/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>14200 N/mm²</td>
</tr>
</tbody>
</table>

Glulam strength class GL32h

Moisture service class 1

Dry stresses are appropriate

Design tensile stress \(2.13 \text{ N/mm}^2\)

Design tensile strength \(20.235 \text{ N/mm}^2\)

Design bending strength \(11.687 \text{ N/mm}^2\)

Design bending strength \(25.293 \text{ N/mm}^2\)

Interaction factor \(0.56733\)

Design shear stress \(1.3083 \text{ N/mm}^2\)

Design shear strength \(1.801 \text{ N/mm}^2\)
Location: Domestic floor joist example

These calculations follow the domestic floor joists example by V C Johnson in TRADA Design Aid DA1 and BS5268-2:2002.

The following assumptions are made in these calculations:
- that the timber has a moisture content of service class 1 or 2 (i.e. K2=1)
- the floor can adequately distribute any concentrated point load to at least two joists.
- the centres of joists do not exceed 610 mm
- that load sharing of the joists can occur & K8 = 1.1.

Effective span of joist  L=3.6 m
Centres of joists  crs=600 mm
Dead load including self weight  dead=0.34 kN/m²
Imposed udl load (on floor)  live=1.5 kN/m²
Imposed point load (on one joist)  PL=2 kN
Depth of section  d=200 mm
Width of section  b=50 mm
Bearing length  lb=100 mm
Strength class C18 to Table 8.
Duration of loading  K3=1
Depth factor  K7=(300/d)^0.11=1.0456
Load sharing (Clause 2.9)  K8=1.1
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor  K4=1.0

Joists: 200 mm x 50 mm @ 600 mm crs
Strength class C18 to Table 8.
Bending stress  6.3914 N/mm²
Permissible bending  6.671 N/mm²
Deflection  8.3371 mm
Limiting deflection  10.8 mm
Shear stress  0.35508 N/mm²
Permissible shear  0.737 N/mm²
Bearing stress  0.47344 N/mm²
Permissible bearing  2.42 N/mm²
Location: Example 2: TR26 grade stress

Domestic floor joist

These calculations follow the domestic floor joists example by V C Johnson in TRADA Design Aid DA1 and BS5268-2:2002.

The following assumptions are made in these calculations:
• that the timber has a moisture content of service class 1 or 2 (i.e. $K_2=1$
• the floor can adequately distribute any concentrated point load to at least two joists.
• the centres of joists do not exceed 610 mm
• that load sharing of the joists can occur & $K_8 = 1.1$.

Effective span of joist $L=2.7$ m
Centres of joists $crs=400$ mm
Dead load including self weight $dead=0.21$ kN/m²
Imposed udl load (on floor) $live=1.5$ kN/m²
Imposed point load (on one joist) $PL=2$ kN
Depth of section $d=193$ mm
Width of section $b=38$ mm
Bearing length $lb=75$ mm
Strength class TR26 to Table 9.
Duration of loading $K_3=1$
Depth factor $K_7=(300/d)^{0.11}=1.0497$
Load sharing (Clause 2.9) $K_8=1.1$
From BS5268-2 Table 18, bearing is > 75 mm from joist end.
Bearing modification factor $K_4=1.14$

Joists: 193 mm x 38 mm @ 400 mm crs
Strength class TR26 to Table 9.
Bending stress 6.047 N/mm²
Permissible bending 11.547 N/mm²
Deflection 3.5253 mm
Limiting deflection 8.1 mm
Shear stress 0.43225 N/mm²
Permissible shear 1.21 N/mm²
Bearing stress 0.74154 N/mm²
Permissible bearing 3.135 N/mm²

DESIGN
SUMMARY
Location: Example 3: D60 Hardwood grade stress

These calculations follow the domestic floor joists example by V C Johnson in TRADA Design Aid DA1 and BS5268-2:2002.

The following assumptions are made in these calculations:
- that the timber has a moisture content of service class 1 or 2 (i.e. K2=1)
- the floor can adequately distribute any concentrated point load to at least two joists.
- the centres of joists do not exceed 610 mm
- that load sharing of the joists can occur & K8 = 1.1.

Effective span of joist  
Centres of joists  
Dead load including self weight  
Imposed udl load (on floor)  
Imposed point load (on one joist)  
Depth of section  
Width of section  
Bearing length  
Strength class D60 to Table 8.
Duration of loading  
Depth factor  
Load sharing (Clause 2.9)  
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor  

Joists: 247 mm x 47 mm @ 600 mm crs
Strength class D60 to Table 8.
Bending stress  
Permissible bending  
Deflection  
Limiting deflection  
Shear stress  
Permissible shear  
Bearing stress  
Permissible bearing  

DESIGN
SUMMARY

SCALE 5.48  Office 1007  Proforma 252
Location: Example 4: User defined grade stresses

These calculations follow the domestic floor joists example by V C Johnson in TRADA Design Aid DA1 and BS5268-2:2002.

The following assumptions are made in these calculations:
- that the timber has a moisture content of service class 1 or 2 (i.e. K2=1)
- the floor can adequately distribute any concentrated point load to at least two joists.
- the centres of joists do not exceed 610 mm
- that load sharing of the joists can occur & K8 = 1.1.

Effective span of joist \( L = 2.8 \) m
Centres of joists \( c_{rs} = 333 \) mm
Dead load including self weight \( \text{dead} = 0.15 \) kN/m²
Imposed udl load (on floor) \( \text{live} = 1.5 \) kN/m²
Imposed point load (on one joist) \( P_L = 2 \) kN
Depth of section \( d = 125 \) mm
Width of section \( b = 75 \) mm
Bearing length \( l_b = 150 \) mm
Douglas fir table 10
Bending parallel to the grain \( b_{parg} = 6.2 \) N/mm²
Compress parallel to the grain \( c_{parg} = 6.6 \) N/mm²
Compress perpendicular to the grain \( c_{perr} = 2.4 \) N/mm²
Tension parallel to the grain \( t_{parg} = 3.7 \) N/mm²
Shear parallel to the grain \( s_{parg} = 0.88 \) N/mm²
Mean modulus of elasticity \( E_{mean} = 11000 \) N/mm²
Minimum modulus of elasticity \( E_{min} = 7000 \) N/mm²
Duration of loading \( K_3 = 1.25 \)
Depth factor \( K_7 = (300/d)^0.11 = 1.1011 \)
Load sharing (Clause 2.9) \( K_8 = 1.1 \)
Bearing modification factor \( K_4 = 1.0 \)

| Joists: 125 mm x 75 mm @ 333 mm crs |
| Douglas fir table 10 |
| Bending stress 7.4186 N/mm² |
| Permissible bending 9.3868 N/mm² |
| Deflection 7.1186 mm |
| Limiting deflection 8.4 mm |
| Shear stress 0.33119 N/mm² |
| Permissible shear 1.21 N/mm² |
| Bearing stress 0.18399 N/mm² |
| Permissible bearing 3.3 N/mm² |

SCALE 5.48 Office 1007 Proforma 252
Location: Domestic floor joist example

The following assumptions are made in these calculations:
- that the timber has a moisture content of service class 1 or 2
- the floor can adequately distribute any concentrated point load to at least two joists
- the centres of joists do not exceed 600 mm
- that load sharing of the joists can occur.

Effective span of joist \( L = 3.6 \text{ m} \)
Centres of joists \( c_{rs} = 600 \text{ mm} \)
Permanent load including s.weight \( G_k = 0.34 \text{ kN/m}^2 \)
Imposed udl load (on floor) \( Q_k = 1.5 \text{ kN/m}^2 \)
Imposed point load (on one joist) \( P_L = 2 \text{ kN} \)
Depth of section \( h = 200 \text{ mm} \)
Width of section \( b = 50 \text{ mm} \)
Bearing length \( l_b = 100 \text{ mm} \)
Timber strength class C18.
Depth factor (Clause 3.2) \( k_h = 1.0 \)
Load sharing factor \( k_{sys} = 1.1 \)
Bearing modification factor \( k_c90 = 1.5 \)
Notch factor for shear \( k_v = 1.0 \)
Shear reduction factor \( k_{sh} = 0.67 \)

Joists: 200 mm x 50 mm @ 600 mm crs
Timber strength class C18.

\[
\begin{align*}
\text{DESIGN SUMMARY} \\
\text{Bending stress} & \quad 9.4384 \text{ N/mm}^2 \\
\text{Design bending strength} & \quad 12.185 \text{ N/mm}^2 \\
\text{Combined total deflection} & \quad 10.601 \text{ mm} \\
\text{Allowable deflection} & \quad 14.4 \text{ mm} \\
\text{Shear stress} & \quad 0.52175 \text{ N/mm}^2 \\
\text{Design shear strength} & \quad 1.542 \text{ N/mm}^2 \\
\text{Bearing stress} & \quad 0.69914 \text{ N/mm}^2 \\
\text{Design bearing strength} & \quad 2.2338 \text{ N/mm}^2 
\end{align*}
\]
Location: Domestic floor joist example

The following assumptions are made in these calculations:
- that the timber has a moisture content of service class 1 or 2
- the floor can adequately distribute any concentrated point load to at least two joists
- the centres of joists do not exceed 600 mm
- that load sharing of the joists can occur.

Effective span of joist \( L = 3.8 \text{ m} \)
Centres of joists \( \text{crs} = 600 \text{ mm} \)
Permanent load including s.weight \( G_k = 0.3666 \text{ kN/m}^2 \)
Imposed udl load (on floor) \( Q_k = 1.5 \text{ kN/m}^2 \)
Imposed point load (on one joist) \( P_L = 1 \text{ kN} \)
Depth of section \( h = 225 \text{ mm} \)
Width of section \( b = 44 \text{ mm} \)
Bearing length \( l_b = 75 \text{ mm} \)
Timber strength class C22.
Depth factor (Clause 3.2) \( k_h = 1.0 \)
Load sharing factor \( k_{sys} = 1.1 \)
Bearing modification factor \( k_{c90} = 1.5 \)
Notch factor for shear \( k_{v} = 1.0 \)
Shear reduction factor \( k_{sh} = 0.67 \)

Joists: 225 mm x 44 mm @ 600 mm crs

Timber strength class C22.

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress</td>
</tr>
<tr>
<td>Design bending strength</td>
</tr>
<tr>
<td>Combined total deflection</td>
</tr>
<tr>
<td>Allowable deflection</td>
</tr>
<tr>
<td>Shear stress</td>
</tr>
<tr>
<td>Design shear strength</td>
</tr>
<tr>
<td>Bearing stress</td>
</tr>
<tr>
<td>Design bearing strength</td>
</tr>
</tbody>
</table>
Location: Example 2: D60 Hardwood grade stress

The following assumptions are made in these calculations:
- that the timber has a moisture content of service class 1 or 2
- the floor can adequately distribute any concentrated point load to at least two joists
- the centres of joists do not exceed 600 mm
- that load sharing of the joists can occur.

Effective span of joist  \( L = 5 \text{ m} \)
Centres of joists  \( c_{rs} = 600 \text{ mm} \)
Permanent load including s.weight  \( G_k = 0.44 \text{ kN/m}^2 \)
Imposed udl load (on floor)  \( Q_k = 1.5 \text{ kN/m}^2 \)
Imposed point load (on one joist)  \( P_L = 2 \text{ kN} \)
Depth of section  \( h = 247 \text{ mm} \)
Width of section  \( b = 47 \text{ mm} \)
Bearing length  \( l_b = 100 \text{ mm} \)
Timber strength class D70.
Depth factor (Clause 3.2)  \( k_h = 1.0 \)
Load sharing factor  \( k_{sys} = 1.1 \)
Bearing modification factor  \( k_c90 = 1.0 \)
Notch factor for shear  \( k_v = 1.0 \)
Shear reduction factor  \( k_{sh} = 0.67 \)

Joists: 247 mm x 47 mm @ 600 mm crs
Timber strength class D70.

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress</td>
</tr>
<tr>
<td>Design bending strength</td>
</tr>
<tr>
<td>Combined total deflection</td>
</tr>
<tr>
<td>Allowable deflection</td>
</tr>
<tr>
<td>Shear stress</td>
</tr>
<tr>
<td>Design shear strength</td>
</tr>
<tr>
<td>Bearing stress</td>
</tr>
<tr>
<td>Design bearing strength</td>
</tr>
</tbody>
</table>
**Location: Example 3: User defined grade stresses**

The following assumptions are made in these calculations:
- that the timber has a moisture content of service class 1 or 2
- the floor can adequately distribute any concentrated point load to at least two joists
- the centres of joists do not exceed 600 mm
- that load sharing of the joists can occur.

Effective span of joist \( L = 2.8 \text{ m} \)
Centres of joists \( c_{rs} = 333 \text{ mm} \)
Permanent load including s.weight \( G_k = 0.15 \text{ kN/m}^2 \)
Imposed udl load (on floor) \( Q_k = 1.5 \text{ kN/m}^2 \)
Imposed point load (on one joist) \( P_L = 2 \text{ kN} \)
Depth of section \( h = 125 \text{ mm} \)
Width of section \( b = 75 \text{ mm} \)
Bearing length \( l_b = 150 \text{ mm} \)

Softwood C24 strength class
- Bending parallel to the grain \( f_{mk} = 24 \text{ N/mm}^2 \)
- Compress parallel to the grain \( f_{c0k} = 21 \text{ N/mm}^2 \)
- Compress perp to the grain \( f_{c90k} = 2.5 \text{ N/mm}^2 \)
- Tension parallel to the grain \( f_{t0k} = 14 \text{ N/mm}^2 \)
- Shear parallel to the grain \( f_{vk} = 2.5 \text{ N/mm}^2 \)
- Mean modulus of elasticity \( E_{0\text{mean}} = 11000 \text{ N/mm}^2 \)
- 5th percentile MOE \( E_{0.05} = 7400 \text{ N/mm}^2 \)
- Depth factor (Clause 3.2) \( k_h = (150/h)^{0.2} = 1.0371 \)
- Load sharing factor \( k_{sys} = 1.1 \)
- Bearing modification factor \( k_{c90} = 1.5 \)
- Notch factor for shear \( k_v = 1.0 \)
- Shear reduction factor \( k_{sh} = 0.67 \)

**Joists: 125 mm x 75 mm @ 333 mm crs**

**Softwood C24 strength class**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress</td>
<td>11.09 N/mm²</td>
</tr>
<tr>
<td>Design bending strength</td>
<td>16.849 N/mm²</td>
</tr>
<tr>
<td>Combined total deflection</td>
<td>8.8364 mm</td>
</tr>
<tr>
<td>Allowable deflection</td>
<td>11.2 mm</td>
</tr>
<tr>
<td>Shear stress</td>
<td>0.49264 N/mm²</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>1.1338 N/mm²</td>
</tr>
<tr>
<td>Bearing stress</td>
<td>0.27506 N/mm²</td>
</tr>
<tr>
<td>Design bearing strength</td>
<td>2.5385 N/mm²</td>
</tr>
</tbody>
</table>
Location: Biaxial bending of timber section

Dead+imposed

Timber beam subject to

bi-axial bending to

BS5268-2:2002

Beam span \( L = 2.2 \text{ m} \)
Depth of section \( d = 172 \text{ mm} \)
Width of section \( b = 63 \text{ mm} \)
Strength class C18 to Table 8.
Timber service class adopted \( \text{tmclass} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.

Theta is the angle between force \( W \) and axis \( Y-Y \), and also as shown in the diagram. Load \( W \) is applied vertically and is to be resolved into components in each of the axes \( X-X \) and \( Y-Y \). Load \( W \) is applied in the vertical direction only.

Dead vertical load \( W \) \( W_{vd} = 0.44 \text{ kN/m} \)
Live vertical load \( W \) \( W_{vl} = 0.44 \text{ kN/m} \)
Angle of \( X-X \) axis to horizontal \( \text{theta} = 30^\circ \)
Axial load (+ve Compress) \( F_a^\prime = 0 \text{ kN} \)
Duration of loading \( K_3 = 1 \)
Depth factor \( K_x = (300/d)^{0.11} = 1.0631 \)
Depth factor \( K_y = 1.17 \)
Load-sharing modification factor \( K_8 = 1.0 \)

Length of bearing \( L_b = 100 \text{ mm} \)
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor \( K_4 = 1.0 \)
Member: 172 mm x 63 mm
Strength class C18 to Table 8.
Bending stress X-X  1.4843 N/mm²
Bending stress Y-Y  2.3396 N/mm²
Permiss bending X-X 6.166 N/mm²
Permiss bending Y-Y 6.786 N/mm²
Interaction factor  0.5855

DESIGN
Deflection  6.5158 mm

SUMMARY
Limiting deflection  6.6 mm
Shear stress  0.13333 N/mm²
Permissible shear  0.67 N/mm²
Wx Bearing stress  0.13307 N/mm²
Wy Bearing stress  0.02814 N/mm²
Permissible bearing  2.2 N/mm²
Location: Example 2; Table 9 grade stresses; No axial force

Beam span \( L = 2.2 \) m
Depth of section \( d = 172 \) mm
Width of section \( b = 63 \) mm
Strength class C22 to Table 8.
Timber service class adopted \( tmclass = 1 \)
Timber service class modification factor \( K2 = 1 \) as Table 16.

Load applied along local axes of member.

Dead load in direction \( P \) \( W_{xd} = 0.4 \) kN/m
Live load in direction \( P \) \( W_{xl} = 0.4 \) kN/m
Dead load in direction \( Q \) \( W_{yd} = 0.175 \) kN/m
Live load in direction \( Q \) \( W_{yl} = 0.175 \) kN/m
Axial load (+ve Compress) \( Fa' = 0 \) kN
Duration of loading \( K3 = 1 \)
Depth factor \( Kx7 = (300/d)^0.11 = 1.0631 \)
Depth factor \( Ky7 = 1.17 \)
Load-sharing modification factor \( K8 = 1.0 \)

Length of bearing \( lb = 100 \) mm
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor \( K4 = 1.0 \)
### Member: 172 mm x 63 mm

<table>
<thead>
<tr>
<th>Strength class</th>
<th>C22 to Table 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress X-X</td>
<td>1.5581 N/mm²</td>
</tr>
<tr>
<td>Bending stress Y-Y</td>
<td>1.8611 N/mm²</td>
</tr>
<tr>
<td>Permiss bending X-X</td>
<td>7.2291 N/mm²</td>
</tr>
<tr>
<td>Permiss bending Y-Y</td>
<td>7.956 N/mm²</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.44946</td>
</tr>
</tbody>
</table>

**DESIGN**

| Deflection | 4.8883 mm |

**SUMMARY**

| Limiting deflection | 6.6 mm |
| Shear stress | 0.1323 N/mm² |
| Permissible shear | 0.71 N/mm² |
| Wx Bearing stress | 0.13968 N/mm² |
| Wy Bearing stress | 0.022384 N/mm² |
| Permissible bearing | 2.3 N/mm² |
Location: Example 3; Table 9 grade stresses; Comp axial force

Timber beam subject to bi-axial bending to BS5268-2:2002

Beam span \( L = 2.2 \) m
Depth of section \( d = 172 \) mm
Width of section \( b = 63 \) mm
Strength class C22 to Table 8.
Timber service class adopted \( tmclass = 2 \)
Timber service class modification factor \( K2 = 1 \) as Table 16.

Force \( P \) acts along axis \( Y-Y \) causing local bending about \( X-X \) axis.
Force \( Q \) acts along axis \( X-X \) causing local bending about \( Y-Y \) axis.
Load applied along local axes of member.

Dead load in direction \( P \) \( W_{xd} = 0.4 \) kN/m
Live load in direction \( P \) \( W_{xl} = 0.4 \) kN/m
Dead load in direction \( Q \) \( W_{yd} = 0.175 \) kN/m
Live load in direction \( Q \) \( W_{yl} = 0.175 \) kN/m
Axial load (+ve Compress) \( F_a' = 5 \) kN
Eff length for bending about \( x-x \) \( L_{ex} = 2200 \) mm
Eff length for bending about \( y-y \) \( L_{ey} = 0 \) mm
Duration of loading \( K3 = 1 \)
Depth factor \( Kx7 = (300/d)^{0.11} = 1.0631 \)
Depth factor \( Ky7 = 1.17 \)
Load-sharing modification factor \( K8 = 1.0 \)
Modification factor \( K12 = (0.5 + (1 + \eta) \times C/2) - ((0.5 + (1 + \eta) \times C/2)^2 - C)^{0.5} = 0.7685 \)

Length of bearing \( lb = 100 \) mm
From BS5268-2 Table 18, bearing is > 75 mm from joist end.
Bearing modification factor \( K4 = 1.1 \)
Member: 172 mm x 63 mm
Strength class C22 to Table 8.
Bending stress X-X  1.5581 N/mm²
Bending stress Y-Y  1.8611 N/mm²
Permiss bending X-X 7.2291 N/mm²
Permiss bending Y-Y 7.956 N/mm²
Applied Comp Stress 0.46142 N/mm²
Permiss Compression 5.7637 N/mm²
Interaction factor 0.53695

**DESIGN**
Deflection  4.8883 mm

**SUMMARY**
Limiting deflection 6.6 mm
Shear stress 0.1323 N/mm²
Permissible shear 0.71 N/mm²
Wx Bearing stress 0.13968 N/mm²
Wy Bearing stress 0.022384 N/mm²
Permissible bearing 2.53 N/mm²
Location: Example 4; User defined grade stresses

Beam span \( L = 2.4 \text{ m} \)
Depth of section \( d = 193 \text{ mm} \)
Width of section \( b = 96 \text{ mm} \)
Whitewood SS grade

Bending parallel to the grain \( b_{parg} = 6.8 \text{ N/mm}^2 \)
Compress parallel to the grain \( c_{parg} = 7.5 \text{ N/mm}^2 \)
Compress perpendicular to the grain \( c_{perd} = 2.3 \text{ N/mm}^2 \)
Tension parallel to the grain \( t_{parg} = 3.7 \text{ N/mm}^2 \)
Shear parallel to the grain \( s_{parg} = 0.88 \text{ N/mm}^2 \)
Mean modulus of elasticity \( E_{mean} = 10000 \text{ N/mm}^2 \)
Minimum modulus of elasticity \( E_{min} = 7000 \text{ N/mm}^2 \)
Timber service class adopted \( t_{mclass} = 1 \)
Timber service class modification factor \( K2 = 1 \) as Table 16.

Theta is the angle between force \( W \) and axis \( Y-Y \), and also as shown in the diagram. Load \( W \) is applied vertically and is to be resolved into components in each of the axes \( X-X \) and \( Y-Y \).

Dead vertical load \( W \) \( W_{vd} = 1.9 \text{ kN/m} \)
Live vertical load \( W \) \( W_{vl} = 1.9 \text{ kN/m} \)
Angle of \( X-X \) axis to horizontal \( \theta = 10^\circ \)
Axial load (+ve Compress) \( F_a' = 0 \text{ kN} \)
Duration of loading \( K3 = 1 \)
Depth factor \( Kx7 = (300/d)^{0.11} = 1.0497 \)
Depth factor \( Ky7 = (300/b)^{0.11} = 1.1335 \)
Load-sharing modification factor \( K8 = 1.0 \)

Length of bearing \( b = 100 \text{ mm} \)
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor \( K4 = 1.0 \)
<table>
<thead>
<tr>
<th>Member: 193 mm x 96 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whitewood SS grade</td>
</tr>
<tr>
<td>Bending stress X-X 4.521 N/mm²</td>
</tr>
<tr>
<td>Bending stress Y-Y 1.6026 N/mm²</td>
</tr>
<tr>
<td>Permiss bending X-X 7.1381 N/mm²</td>
</tr>
<tr>
<td>Permiss bending Y-Y 7.708 N/mm²</td>
</tr>
<tr>
<td>Interaction factor 0.84128</td>
</tr>
<tr>
<td>Deflection 5.2996 mm</td>
</tr>
<tr>
<td>DESIGN</td>
</tr>
<tr>
<td>SUMMARY</td>
</tr>
<tr>
<td>Limiting deflection 7.2 mm</td>
</tr>
<tr>
<td>Shear stress 0.36733 N/mm²</td>
</tr>
<tr>
<td>Permissible shear 0.88 N/mm²</td>
</tr>
<tr>
<td>Wx Bearing stress 0.46778 N/mm²</td>
</tr>
<tr>
<td>Wy Bearing stress 0.041028 N/mm²</td>
</tr>
<tr>
<td>Permissible bearing 2.3 N/mm²</td>
</tr>
</tbody>
</table>
Location: Biaxial bending of timber section

Timber beam subject to bi-axial bending to EC5 incorporating National Annex Amendment No.2

Beam span \( L = 2.2 \text{ m} \)
Depth of section \( h = 172 \text{ mm} \)
Width of section \( b = 63 \text{ mm} \)
Bearing length \( l_b = 100 \text{ mm} \)
Timber strength class C18.

Loads \( G_k \) and \( Q_k \) are to be resolved into \( yy \) and \( zz \) components.

Permanent load \( G_k = 0.44 \text{ kN/m} \)
Variable load \( Q_k = 0.44 \text{ kN/m} \)
Angle of \( yy \) axis to horizontal \( \theta = 30^\circ \)
Permanent axial load (+ve Comp) \( N_{\text{per}} = 0 \text{ kN} \)
Variable axial load (+ve Comp) \( N_{\text{var}} = 0 \text{ kN} \)
Bearing modification factor \( k_c = 1.5 \)
Depth factor (Clause 3.3) \( k_y = 1.0 \)
Depth factor (Clause 3.2) \( k_z = (150/b)^{0.2} = 1.1895 \)
Load-sharing modification factor \( k_{\text{sys}} = 1.0 \)
Shear reduction factor \( k_{\text{sh}} = 0.67 \)
Effective length about \( zz \) \( L_e = 1.76 \text{ m} \)
Member: 172 mm x 63 mm  
Timber strength class C18.  
Bending stress yy 2.1151 N/mm²  
Bending stress zz 3.334 N/mm²  
Design bending strength yy 8.3077 N/mm²  
Design bending strength zz 9.8817 N/mm²  
Interaction factor 0.51561  
Deflection 6.038 mm  

DESIGN  
Limiting deflection 8.8 mm  

SUMMARY  
Shear stress 0.19 N/mm²  
Design shear strength 1.0514 N/mm²  
Fyd Bearing stress 0.18962 N/mm²  
Fzd Bearing stress 0.040099 N/mm²  
Design bearing strength 3.3 N/mm²
Location: Biaxial bending of timber section

Timber beam subject to bi-axial bending to EC5 incorporating National Annex Amendment No.2

Beam span L=2.2 m
Depth of section h=172 mm
Width of section b=63 mm
Bearing length lb=50 mm
Timber strength class C18.

Loads Gk and Qk are to be resolved into yy and zz components.

Permanent load Gk=0.44 kN/m
Variable load Qk=0.44 kN/m
Angle of yy axis to horizontal theta=30°
Permanent axial load (+ve Comp) Nper=0 kN
Variable axial load (+ve Comp) Nvar=0 kN
Bearing modification factor kc90=1.5
Depth factor (Clause 3.3) kyh=1.0
Depth factor (Clause 3.2) kzh=(150/b)^0.2=1.1895
Load-sharing modification factor ksys=1.0
Shear reduction factor ksh=0.67
Effective length about zz Le=1.76 m
**Member**: 172 mm x 63 mm  
Timber strength class C18.  

<table>
<thead>
<tr>
<th>Stress or Strength</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress yy</td>
<td>2.1151 N/mm²</td>
</tr>
<tr>
<td>Bending stress zz</td>
<td>3.334 N/mm²</td>
</tr>
<tr>
<td>Design bending strength yy</td>
<td>8.3077 N/mm²</td>
</tr>
<tr>
<td>Design bending strength zz</td>
<td>9.8817 N/mm²</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.51561</td>
</tr>
<tr>
<td>Deflection</td>
<td>6.038 mm</td>
</tr>
</tbody>
</table>

**DESIGN**  
Limiting deflection 8.8 mm  
Shear stress 0.19 N/mm²  
Design shear strength 1.0514 N/mm²  
Fyd Bearing stress 0.37924 N/mm²  
Fzd Bearing stress 0.080198 N/mm²  
Design bearing strength 3.3 N/mm²  

**SUMMARY**  
Shear stress 0.19 N/mm²  
Design shear strength 1.0514 N/mm²  
Fyd Bearing stress 0.37924 N/mm²  
Fzd Bearing stress 0.080198 N/mm²  
Design bearing strength 3.3 N/mm²
Location: Example 2 - No axial force

Timber beam subject to bi-axial bending to EC5 incorporating National Annex Amendment No.2

Beam span \( L = 2.2 \) m
Depth of section \( h = 172 \) mm
Width of section \( b = 63 \) mm
Bearing length \( l_b = 50 \) mm

Timber strength class C22.

Gky \( \geq \) Qky

Gkz \( \geq \) Qkz

Loads Gky and Qky act along axis \( \mathbf{zz} \) causing local bending about \( \mathbf{yy} \) axis.

Loads Gkz and Qkz act along axis \( \mathbf{yy} \) causing local bending about \( \mathbf{zz} \) axis.

Loads are applied along local axes of member.

Permanent load in direction \( \mathbf{zz} \) \( G_{ky} = 0.4 \) kN/m
Variable load in direction \( \mathbf{zz} \) \( Q_{ky} = 0.4 \) kN/m
Permanent load in direction \( \mathbf{yy} \) \( G_{kz} = 0.175 \) kN/m
Variable load in direction \( \mathbf{yy} \) \( Q_{kz} = 0.175 \) kN/m
Permanent axial load (+ve Comp) \( N_{per} = 0 \) kN
Variable axial load (+ve Comp) \( N_{var} = 0 \) kN
Bearing modification factor \( k_c90 = 1.5 \)
Depth factor (Clause 3.3) \( k_{yh} = 1.0 \)
Depth factor (Clause 3.2) \( k_{zh} = (150/b)^{0.2} = 1.1895 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)
Shear reduction factor \( k_{sh} = 0.67 \)
Effective length about \( \mathbf{zz} \) \( L_e = 1.76 \) m
Member: 172 mm x 63 mm
Timber strength class C22.

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress yy</td>
<td>2.2203 N/mm²</td>
</tr>
<tr>
<td>Bending stress zz</td>
<td>2.652 N/mm²</td>
</tr>
<tr>
<td>Design bending strength yy</td>
<td>10.154 N/mm²</td>
</tr>
<tr>
<td>Design bending strength zz</td>
<td>12.078 N/mm²</td>
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<tr>
<td>Interaction factor</td>
<td>0.37265</td>
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<tr>
<td>Deflection</td>
<td>4.4166 mm</td>
</tr>
</tbody>
</table>

**DESIGN**

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limiting deflection</td>
<td>8.8 mm</td>
</tr>
<tr>
<td>Shear stress</td>
<td>0.18853 N/mm²</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>1.1751 N/mm²</td>
</tr>
<tr>
<td>Fyd Bearing stress</td>
<td>0.3981 N/mm²</td>
</tr>
<tr>
<td>Fzd Bearing stress</td>
<td>0.063794 N/mm²</td>
</tr>
<tr>
<td>Design bearing strength</td>
<td>3.6 N/mm²</td>
</tr>
</tbody>
</table>
Location: Example 3 - Compression axial force

Timber beam subject to bi-axial bending to EC5 incorporating National Annex Amendment No.2

Beam span \( L = 2.2 \text{ m} \)
Depth of section \( h = 172 \text{ mm} \)
Width of section \( b = 63 \text{ mm} \)
Bearing length \( lb = 50 \text{ mm} \)
Timber strength class C22.

Loads \( G_{ky} \) and \( Q_{ky} \) act along axis \( zz \) causing local bending about \( yy \) axis.
Loads \( G_{kz} \) and \( Q_{kz} \) act along axis \( yy \) causing local bending about \( zz \) axis.

Loads are applied along local axes of member.

Permanent load in direction \( zz \) \( G_{ky} = 0.4 \text{ kN/m} \)
Variable load in direction \( zz \) \( Q_{ky} = 0.4 \text{ kN/m} \)
Permanent load in direction \( yy \) \( G_{kz} = 0.175 \text{ kN/m} \)
Variable load in direction \( yy \) \( Q_{kz} = 0.175 \text{ kN/m} \)
Permanent axial load (+ve Comp) \( N_{per} = 20 \text{ kN} \)
Variable axial load (+ve Comp) \( N_{var} = 15 \text{ kN} \)
Eff length for bending about \( yy \) \( l_{ey} = 2200 \text{ mm} \)
Eff length for bending about \( zz \) \( l_{ez} = 0 \text{ mm} \)
Bearing modification factor \( k_{c90} = 1.5 \)
Depth factor (Clause 3.3) \( k_{yh} = 1.0 \)
Depth factor (Clause 3.2) \( k_{zh} = (150/b)^{0.2} = 1.1895 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)
Instability factor \( k_{cy} = 1/(k_{y} + (k_{y}^2 - r_{sry}^2)^{0.5}) = 0.84165 \)
Instability factor \( k_{cz} = 1/(k_{z} + (k_{z}^2 - r_{szr}^2)^{0.5}) = 1.0638 \)
Max instability factor \( k_{cz} = 1 \)
Shear reduction factor \( k_{sh} = 0.67 \)
Sample output for SCALE Proforma 253. (ans=4) Page: 2
Timber design to BS5268 and Eurocode 5 Made by: IFB
Biaxial bending of timber section Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd. Ref No: SC253 EC

Effective length about zz Le=1.76 m

<p>| | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Member:</td>
<td>172 mm x 63 mm</td>
<td>Timber strength class C22.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bending stress yy</td>
<td>2.2203 N/mm²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bending stress zz</td>
<td>2.652 N/mm²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design bending strength yy</td>
<td>10.154 N/mm²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design bending strength zz</td>
<td>12.078 N/mm²</td>
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</tr>
<tr>
<td>Applied compression Stress</td>
<td>4.5681 N/mm²</td>
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</tr>
<tr>
<td>Design compressive strength</td>
<td>9.2308 N/mm²</td>
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<td></td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.96036</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deflection</td>
<td>4.8297 mm</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DESIGN**

| Limiting deflection | 8.8 mm |
| Shear stress        | 0.18853 N/mm² |
| Design shear strength | 1.1751 N/mm² |
| Fyd Bearing stress  | 0.3981 N/mm² |
| Fzd Bearing stress  | 0.063794 N/mm² |
| Design bearing strength | 3.6 N/mm² |

**SUMMARY**
Location: Example 4 - User defined grade stresses

Timber beam subject to bi-axial bending to EC5 incorporating National Annex Amendment No.2

Beam span \( L = 2.4 \text{ m} \)
Depth of section \( h = 193 \text{ mm} \)
Width of section \( b = 96 \text{ mm} \)
Bearing length \( l_b = 50 \text{ mm} \)

Whitewood
Bending parallel to the grain \( f_{mk} = 18 \text{ N/mm}^2 \)
Compress parallel to the grain \( f_{c0k} = 18 \text{ N/mm}^2 \)
Compress perp to the grain \( f_{c90k} = 2.2 \text{ N/mm}^2 \)
Tension parallel to the grain \( f_{t0k} = 7.2 \text{ N/mm}^2 \)
Shear parallel to the grain \( f_{vk} = 3 \text{ N/mm}^2 \)
Mean modulus of elasticity \( E_{0\text{mean}} = 9000 \text{ N/mm}^2 \)
5th percentile MOE \( E_{005} = 6000 \text{ N/mm}^2 \)

\[ G_k \quad Q_k \]

Loads \( G_k \) and \( Q_k \) are to be resolved into \( yy \) and \( zz \) components.

Permanent load \( G_k = 1.9 \text{ kN/m} \)
Variable load \( Q_k = 1.9 \text{ kN/m} \)
Angle of \( yy \) axis to horizontal \( \theta = 10^\circ \)
Permanent axial load (+ve Comp) \( N_{\text{per}} = 0 \text{ kN} \)
Variable axial load (+ve Comp) \( N_{\text{var}} = 0 \text{ kN} \)
Bearing modification factor \( k_{c90} = 1.5 \)
Depth factor (Clause 3.3) \( k_{yh} = 1.0 \)
Depth factor (Clause 3.2) \( k_{zh} = (150/b)^{0.2} = 1.0934 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)
Shear reduction factor \( k_{sh} = 0.67 \)
Effective length about zz \( L_e = 1.92 \text{ m} \)

<table>
<thead>
<tr>
<th>Member: 193 mm x 96 mm Whitewood</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress yy: 6.4424 N/mm²</td>
</tr>
<tr>
<td>Bending stress zz: 2.2838 N/mm²</td>
</tr>
<tr>
<td>Design bending strength yy: 8.3077 N/mm²</td>
</tr>
<tr>
<td>Design bending strength zz: 9.0833 N/mm²</td>
</tr>
<tr>
<td>Interaction factor: 0.95147</td>
</tr>
<tr>
<td>Deflection: 5.7295 mm</td>
</tr>
</tbody>
</table>

**DESIGN**
- Limiting deflection: 9.6 mm

**SUMMARY**
- Shear stress: 0.52345 N/mm²
- Design shear strength: 0.92769 N/mm²
- Fyd Bearing stress: 1.3332 N/mm²
- Fzd Bearing stress: 0.11693 N/mm²
- Design bearing strength: 3.3 N/mm²
Location: Example 5 - Compression axial force

Timber beam subject to bi-axial bending to EC5 incorporating National Annex Amendment No.2

Beam span \( L = 3.75 \text{ m} \)
Depth of section \( h = 250 \text{ mm} \)
Width of section \( b = 100 \text{ mm} \)
Bearing length \( l_b = 50 \text{ mm} \)
Timber strength class C22.

Loads \( G_kz \) and \( Q_kz \) act along axis \( zz \) causing local bending about \( yy \) axis.

Loads \( G_{ky} \) and \( Q_{ky} \) act along axis \( zz \) causing local bending about \( yy \) axis.

Loads are applied along local axes of member.

Permanent load in direction \( zz \) \( G_{ky} = 0.5 \text{ kN/m} \)
Variable load in direction \( zz \) \( Q_{ky} = 0.25 \text{ kN/m} \)
Permanent load in direction \( yy \) \( G_{kz} = 0 \text{ kN/m} \)
Variable load in direction \( yy \) \( Q_{kz} = 0 \text{ kN/m} \)
Permanent axial load (+ve Comp) \( N_{\text{per}} = 17 \text{ kN} \)
Variable axial load (+ve Comp) \( N_{\text{var}} = 15 \text{ kN} \)
Eff length for bending about \( yy \) \( l_{ey} = 3750 \text{ mm} \)
Eff length for bending about \( zz \) \( l_{ez} = 3750 \text{ mm} \)
Bearing modification factor \( k_{c90} = 1.5 \)
Depth factor (Clause 3.3) \( k_{yh} = 1.0 \)
Depth factor (Clause 3.2) \( k_{zh} = (150/b)^{0.2} = 1.0845 \)
Load-sharing modification factor \( k_{\text{sys}} = 1.0 \)
Instability factor \( k_{cy} = 1/(k_{y}+(k_{y}^2-r_{sry}^2)^{0.5}) = 0.75898 \)
Instability factor \( k_{cz} = 1/(k_{z}+(k_{z}^2-r_{srz}^2)^{0.5}) = 0.17917 \)
Shear reduction factor \( k_{sh} = 0.67 \)
Effective length about \( zz \) \( L_{e} = 3 \text{ m} \)
<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member:</td>
<td>250 mm x 100 mm</td>
</tr>
<tr>
<td>Timber strength class</td>
<td>C22</td>
</tr>
<tr>
<td>Bending stress yy</td>
<td>1.7719 N/mm²</td>
</tr>
<tr>
<td>Bending stress zz</td>
<td>0 N/mm²</td>
</tr>
<tr>
<td>Design bending strength yy</td>
<td>13.538 N/mm²</td>
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<tr>
<td>Design bending strength zz</td>
<td>14.682 N/mm²</td>
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<tr>
<td>Applied compression Stress</td>
<td>1.818 N/mm²</td>
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<tr>
<td>Design compressive strength</td>
<td>12.308 N/mm²</td>
</tr>
<tr>
<td>Interaction factor</td>
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<tr>
<td>Deflection</td>
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<td>DESIGN</td>
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</tr>
<tr>
<td>Limiting deflection</td>
<td>15 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td></td>
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<tr>
<td>shear stress</td>
<td>0.11754 N/mm²</td>
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<tr>
<td>Design shear strength</td>
<td>1.5668 N/mm²</td>
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<tr>
<td>Fyd Bearing stress</td>
<td>0.39375 N/mm²</td>
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<tr>
<td>Fzd Bearing stress</td>
<td>0 N/mm²</td>
</tr>
<tr>
<td>Design bearing strength</td>
<td>3.6 N/mm²</td>
</tr>
</tbody>
</table>
**Location:** Timber beam with udl & point loads

**Timber beam to BS5268-2:2002**

Simply supported beam subjected to vertical loads.

Beam span \( L = 3.6 \text{ m} \)
Distance from left support to start \( L_{au(1)} = 0 \text{ m} \)
Distance from left support to end \( L_{bu(1)} = 3.6 \text{ m} \)
Dead load (unfactored) \( G_{ku(1)} = 0.204 \text{ kN/m} \)
Imposed load (unfactored) \( Q_{ku(1)} = 0.9 \text{ kN/m} \)
Maximum span bending moment \( 1.7885 \text{ kNm} \)
Design shear force \( F_{ve'} = 1.9872 \)

**Section design parameters**

Design axial load (+ve comp) \( F_a = 3 \text{ kN} \)
Depth of section \( d = 225 \text{ mm} \)
Width of section \( b = 50 \text{ mm} \)
Eff length for bending about xx \( L_{ex} = 3600 \text{ mm} \)
Eff length for bending about yy \( L_{ey} = 0 \text{ mm} \)
Length of bearing \( l_b = 100 \text{ mm} \)
From BS5268-2 Table 18, bearing is > 75 mm from joist end.
Bearing modification factor \( K_4 = 1.1 \)
Strength class C18 to Table 8.
Timber service class adopted \( t_{m\text{class}} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1 \)
Depth factor \( K_7 = (300/d)^{0.11} = 1.0322 \)
Load-sharing modification factor \( K_8 = 1.1 \)
Modification factor \( K_{12} = (0.5 + (1 + \eta \times C/2) - ((0.5 + (1 + \eta \times C/2)^2 - C)^0.5 = 0.69057 \)
Depth of notch at support \( \text{notch} = 20 \text{ mm} \)
Extent of notch \( a_l = 100 \text{ mm} \)
\( K_5 = (d^2(he-al)+(al^2he))/he/he = 1.05 \)
Member: 225 mm x 50 mm
Strength class C18 to Table 8.
Moisture service class 1
Bending stress 4.2394 N/mm²
Permissible bending 6.5851 N/mm²
Comp Stress @ Mids. 0.26667 N/mm²
Permiss compression 5.3933 N/mm²
Interaction factor 0.70258
Comp Stress @ Supp. 0.29268 N/mm²
Deflection 5.9258 mm

DESIGN
SUMMARY
Limiting deflection 10.8 mm
Shear stress 0.29081 N/mm²
Permissible shear 0.77383 N/mm²
Bearing stress 0.39744 N/mm²
Permissible bearing 2.662 N/mm²
**Location: Example 2; No axial load**

**Timber beam to BS5268-2:2002**

Simply supported beam subjected to vertical loads.

Beam span \( L = 3.6 \) m
Distance from left support to start \( L_{au}(1) = 0 \) m
Distance from left support to end \( L_{bu}(1) = 3.6 \) m
Dead load (unfactored) \( G_{ku}(1) = 0.204 \) kN/m
Imposed load (unfactored) \( Q_{ku}(1) = 1.5 \) kN/m
Maximum span bending moment \( 2.7605 \) kNm
Design shear force \( F_{ve}' = 3.0672 \)

**Section design parameters**

Design axial load (+ve comp) \( F_{a} = 0 \) kN
Depth of section \( d = 225 \) mm
Width of section \( b = 47 \) mm
Eff length for bending about \( xx \) \( L_{ex} = 3600 \) mm
Eff length for bending about \( yy \) \( L_{ey} = 0 \) mm
Length of bearing \( L_{b} = 147 \) mm
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor \( K_{4} = 1.0 \)
Strength class TR26 to Table 9.
Timber service class adopted \( \text{tmclass} = 1 \)
Timber service class modification factor \( K_{2} = 1 \) as Table 16.
Duration of loading \( K_{3} = 1.5 \)
Depth factor \( K_{7} = (300/d)^{0.11} = 1.0322 \)
Load-sharing modification factor \( K_{8} = 1.1 \)
No notches exist at the support \( K_{5} = 1.0 \)
Member: 225 mm x 47 mm  
Strength class TR26 to Table 9.  
Moisture service class 1  

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress</td>
<td>6.961 N/mm²</td>
</tr>
<tr>
<td>Permissible bending</td>
<td>17.03 N/mm²</td>
</tr>
<tr>
<td>Deflection</td>
<td>8.0495 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>10.8 mm</td>
</tr>
<tr>
<td>Shear stress</td>
<td>0.43506 N/mm²</td>
</tr>
<tr>
<td>Permissible shear</td>
<td>1.815 N/mm²</td>
</tr>
<tr>
<td>Bearing stress</td>
<td>0.44394 N/mm²</td>
</tr>
<tr>
<td>Permissible bearing</td>
<td>4.125 N/mm²</td>
</tr>
</tbody>
</table>
Location: Example 3; Bending and tensile axial load

Timber beam to BS5268-2:2002

Simply supported beam subjected to vertical loads.

Beam span \( L=4.5 \) m
Dist. from left support to start \( L_{au}(1)=0 \) m
Distance from left support to end \( L_{bu}(1)=4.5 \) m
Dead load (unfactored) \( G_{ku}(1)=0.15 \) kN/m
Imposed load (unfactored) \( Q_{ku}(1)=1.75 \) kN/m
Maximum span bending moment \( 4.8094 \) kNm
Design shear force \( F_{ve}'=4.275 \)

Section design parameters

Design axial load (+ve comp) \( F_a=-3 \) kN
Depth of section \( d=272 \) mm
Width of section \( b=63 \) mm
Eff length for bending about xx \( L_{ex}=4.5 \) mm
Eff length for bending about yy \( L_{ey}=0 \) mm
Length of bearing \( l_b=100 \) mm
From BS5268-2 Table 18, bearing is > 75 mm from joist end.
Bearing modification factor \( K_4=1.1 \)
Strength class C24 to Table 8.
Timber service class adopted \( t_{mclass}=1 \)
Timber service class modification factor \( K_2=1 \) as Table 16.
Tensile area of cross section \( A_t=10000 \) mm²
Duration of loading \( K_3=1.5 \)
Depth factor \( K_7=(300/d)^{0.11}=1.0108 \)
Load-sharing modification factor \( K_8=1.1 \) clause 2.10.11
No notches exist at the support \( K_5=1.0 \)
Width modification factor \[ K_{14} = (300/h)^{0.11} = 1.0108 \]

<table>
<thead>
<tr>
<th>Member: 272 mm x 63 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength class C24 to Table 8.</td>
</tr>
<tr>
<td>Moisture service class 1</td>
</tr>
<tr>
<td>Bending stress 6.191 N/mm²</td>
</tr>
<tr>
<td>Permissible bending 12.509 N/mm²</td>
</tr>
<tr>
<td>Tension stress 0.3 N/mm²</td>
</tr>
<tr>
<td>Permiss tension 6.8231 N/mm²</td>
</tr>
<tr>
<td>Interaction factor 0.53889</td>
</tr>
</tbody>
</table>

**DESIGN**

| Deflection 12.355 mm |

**SUMMARY**

| Limiting deflection 13.5 mm |
| Shear stress 0.37421 N/mm² |
| Permissible shear 1.1715 N/mm² |
| Bearing stress 0.67857 N/mm² |
| Permissible bearing 4.356 N/mm² |
Location: Example 4; with User defined grade stresses

Timber beam to BS5268-2:2002

Simply supported beam subjected to vertical loads.

Beam span \( L = 3.6 \text{ m} \)
Distance from left support to start \( L_{au}(1) = 0 \text{ m} \)
Distance from left support to end \( L_{bu}(1) = 3.6 \text{ m} \)
Dead load (unfactored) \( G_{ku}(1) = 0.204 \text{ kN/m} \)
Imposed load (unfactored) \( Q_{ku}(1) = 0.9 \text{ kN/m} \)
Maximum span bending moment \( 1.7885 \text{ kNm} \)
Design shear force \( F_{ve}' = 1.9872 \)

Section design parameters

Design axial load (+ve comp) \( F_a = 3 \text{ kN} \)
Depth of section \( d = 225 \text{ mm} \)
Width of section \( b = 50 \text{ mm} \)
Eff length for bending about \( xx \) \( L_{ex} = 3600 \text{ mm} \)
Eff length for bending about \( yy \) \( L_{ey} = 0 \text{ mm} \)
Length of bearing \( l_b = 100 \text{ mm} \)
From BS5268-2 Table 18, bearing is > 75 mm from joist end.
Bearing modification factor \( K_4 = 1.1 \)
Redwood / Whitewood SS grade table 10
Bending parallel to the grain \( b_{parg} = 7.5 \text{ N/mm}^2 \)
Compress parallel to the grain \( c_{parg} = 7.9 \text{ N/mm}^2 \)
Compress perpendicular to the grain \( c_{perd} = 2.1 \text{ N/mm}^2 \)
Tension parallel to the grain \( t_{parg} = 4.5 \text{ N/mm}^2 \)
Shear parallel to the grain \( s_{parg} = 0.82 \text{ N/mm}^2 \)
Mean modulus of elasticity \( E_{mean} = 10000 \text{ N/mm}^2 \)
Minimum modulus of elasticity \( E_{min} = 7000 \text{ N/mm}^2 \)
Timber service class adopted \( t_{mclass} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Sample output for SCALE Proforma 254. (ans=4)  
Timber design to BS5268 and Eurocode 5  
Simply supported beam with general loading  
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Duration of loading K3=1
Depth factor K7=(300/d)^0.11=1.0322
Load-sharing modification factor K8=1.1
Modification factor K12=(0.5+(1+eta)*C/2)-((0.5+(1+eta)*C/2)^2
-C)^0.5=0.69566

Depth of notch at support
notch=20 mm
Extent of notch
al=100 mm
K5=(d*(he-al)+(al*he))/he/he=1.05

Member: 225 mm x 50 mm
Redwood / Whitewood SS grade table 10
Moisture service class 1
Bending stress 4.2394 N/mm²
Permissible Bending 8.5152 N/mm²
Comp Stress @ Mids. 0.26667 N/mm²
Permiss compression 6.0453 N/mm²
Interaction factor 0.5482
Comp Stress @ Supp. 0.29268 N/mm²

DEFINITION
Deflection 5.3925 mm
Limiting deflection 10.8 mm
Shear stress 0.29081 N/mm²
Permissible shear 0.94707 N/mm²
Bearing stress 0.39744 N/mm²
Permissible bearing 2.541 N/mm²
Location: Timber beam with udl & point loads

Timber beam to EC5 - incorporating national amendment No.2

Simply supported beam subject to vertical loads.

Beam span \( L = 3.6 \text{ m} \)
Dist. from left support to start \( L_{au}(1) = 0 \text{ m} \)
Distance from left support to end \( L_{bu}(1) = 3.6 \text{ m} \)
Permanent load (unfactored) \( G_{ku}(1) = 0.21996 \text{ kN/m} \)
Variable load 1 (unfactored) \( Q_{ua}(1) = 0.9 \text{ kN/m} \)

Characteristic values of actions

Permanent:
- Reaction at left hand end \( 0.39593 \text{ kN} \)
- Reaction at right hand end \( 0.39593 \text{ kN} \)
- Maximum span moment \( M_{kp} = 0.35634 \text{ kNm} \)
- Maximum shear force \( F_{kp} = 0.39593 \text{ kN} \)

Variable 1:
- Reaction at left hand end \( 1.62 \text{ kN} \)
- Reaction at right hand end \( 1.62 \text{ kN} \)
- Maximum span moment \( M_{kv1} = 1.458 \text{ kNm} \)
- Maximum shear force \( F_{kv1} = 1.62 \text{ kN} \)
- Design axial load (+ve compress) \( N_d = 0 \text{ kN} \)

Section design parameters

\[
\begin{align*}
\text{Depth of section} & = 225 \text{ mm} \\
\text{Width of section} & = 44 \text{ mm} \\
\text{Eff.length for bending about yy} & = 3800 \text{ mm} \\
\text{Eff.length for bending about zz} & = 0 \text{ mm} \\
\text{Length of bearing} & = 75 \text{ mm} \\
\text{Timber strength class C18.}
\end{align*}
\]
Depth factor (Clause 3.2)         \( kh = 1.0 \)
Load-sharing modification factor \( k_{sys} = 1.1 \)
Depth of notch at support       \( \text{notch} = 20 \text{ mm} \)
Notch is on top edge of beam    \( k_v = 1 \)
Bearing modification factor     \( k_{c90} = 1 \)
Shear reduction factor          \( k_{sh} = 0.67 \)
Effective length about \( zz \) \( Le = 2.88 \text{ m} \)

Member: 225 mm x 44 mm
Timber strength class C18.

Moisture service class         1
Design bending stress          7.1867 N/mm²
Design bending strength        8.6394 N/mm²
Deflection                     8.852 mm

Design shear stress            0.49054 N/mm²
Design shear strength          1.542 N/mm²
Design bearing stress          0.89834 N/mm²
Design bearing strength        1.4892 N/mm²
Location: Example 2; No axial load

Timber beam to EC5 - incorporating national amendment No.2

Simply supported beam subject to vertical loads.

Beam span \(L=3.6\) m
Dist. from left support to start \(L_{au(1)}=0\) m
Distance from left support to end \(L_{bu(1)}=3.6\) m
Permanent load (unfactored) \(G_{ku(1)}=0.204\) kN/m
Variable load 1 (unfactored) \(Q_{u(1)}=1.5\) kN/m

**Characteristic values of actions**

**Permanent:**
- Reaction at left hand end \(0.3672\) kN
- Reaction at right hand end \(0.3672\) kN
- Maximum span moment \(M_{kp}=0.33048\) kNm
- Maximum shear force \(F_{kp}=0.3672\) kN

**Variable 1:**
- Reaction at left hand end \(2.7\) kN
- Reaction at right hand end \(2.7\) kN
- Maximum span moment \(M_{kv1}=2.43\) kNm
- Maximum shear force \(F_{kv1}=2.7\) kN
- Design axial load (+ve compress) \(N_d=0\) kN

**Section design parameters**

- Depth of section \(h=225\) mm
- Width of section \(b=47\) mm
- Eff. length for bending about \(yy\) \(l_{ey}=3600\) mm
- Eff. length for bending about \(zz\) \(l_{ez}=0\) mm
- Length of bearing \(l_b=147\) mm
- Timber strength class D70.
<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth factor (Clause 3.2)</td>
<td>$k_h = 1.0$</td>
</tr>
<tr>
<td>Load-sharing modification factor</td>
<td>$k_{sys} = 1.0$</td>
</tr>
<tr>
<td>No notches exist at the support</td>
<td>$k_v = 1.0$</td>
</tr>
<tr>
<td>Bearing modification factor</td>
<td>$k_{c90} = 1.0$</td>
</tr>
<tr>
<td>Shear reduction factor</td>
<td>$k_{sh} = 0.67$</td>
</tr>
<tr>
<td>Effective length about zz</td>
<td>$L_e = 2.88$ m</td>
</tr>
</tbody>
</table>

**Member: 225 mm x 47 mm**
- **Timber strength class D70.**
- **Moisture service class**: 1

**DESIGN**
- **Design bending stress**: $10.317$ N/mm²
- **Design bending strength**: $25.449$ N/mm²
- **Deflection**: $5.5394$ mm

**SUMMARY**
- **Limiting deflection**: $14.4$ mm
- **Design shear stress**: $0.64158$ N/mm²
- **Design shear strength**: $2.0615$ N/mm²
- **Design bearing stress**: $0.65794$ N/mm²
- **Design bearing strength**: $7.3846$ N/mm²
Location: Example 3; Bending and tensile axial load

Timber beam to EC5 - incorporating national amendment No.2

Simply supported beam subject to vertical loads.

Beam span $L=4.5$ m
Dist. from left support to start $L_{au}(1)=0$ m
Distance from left support to end $L_{bu}(1)=4.5$ m
Permanent load (unfactored) $G_{ku}(1)=0.15$ kN/m
Variable load 1 (unfactored) $Q_{ua}(1)=1.75$ kN/m

Characteristic values of actions

Permanent:
Reaction at left hand end $0.3375$ kN
Reaction at right hand end $0.3375$ kN
Maximum span moment $M_{kp}=0.37969$ kNm
Maximum shear force $F_{kp}=0.3375$ kN
Variable 1:
Reaction at left hand end $3.9375$ kN
Reaction at right hand end $3.9375$ kN
Maximum span moment $M_{kv1}=4.4297$ kNm
Maximum shear force $F_{kv1}=3.9375$ kN
Design axial load (+ve compress) $N_d=-3$ kN

Section design parameters

Depth of section $h=272$ mm
Width of section $b=63$ mm
Eff.length for bending about yy $l_{ey}=4.5$ mm
Eff.length for bending about zz $l_{ez}=0$ mm
Length of bearing $l_b=100$ mm
Timber strength class C24.
Tensile area of cross section  \( At=10000 \text{ mm}^2 \)

Depth factor (Clause 3.2)  \( kh=1.0 \)

Load-sharing modification factor  \( ksys=1.0 \)

Width modification factor  \( kw=(150/lardim)^{0.2}=0.88778 \)

Width modification factor  \( kw=1.0 \)

No notches exist at the support  \( kv=1.0 \)

Bearing modification factor  \( kc90=1 \)

Shear reduction factor  \( ksh=0.67 \)

**Member: 272 mm x 63 mm**

**Timber strength class C24.**

<table>
<thead>
<tr>
<th>Moisture service class</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design bending stress</td>
<td>9.2132 N/mm²</td>
</tr>
<tr>
<td>Design bending strength</td>
<td>16.615 N/mm²</td>
</tr>
<tr>
<td>Design tension stress</td>
<td>0.3 N/mm²</td>
</tr>
<tr>
<td>Design tension strength</td>
<td>10.038 N/mm²</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.58438</td>
</tr>
</tbody>
</table>

**DESIGN**

| Deflection  | 11.374 mm |

**SUMMARY**

<table>
<thead>
<tr>
<th>Limiting deflection</th>
<th>18 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear stress</td>
<td>0.55412 N/mm²</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>1.8554 N/mm²</td>
</tr>
<tr>
<td>Design bearing stress</td>
<td>1.0098 N/mm²</td>
</tr>
<tr>
<td>Design bearing strength</td>
<td>1.7308 N/mm²</td>
</tr>
</tbody>
</table>
Location: Example 4; with User defined grade stresses

Timber beam to EC5 - incorporating national amendment No.2

Simply supported beam subject to vertical loads.

Beam span \( L = 3.6 \text{ m} \)
Distance from left support to start \( L_{au(1)} = 0 \text{ m} \)
Distance from left support to end \( L_{bu(1)} = 3.6 \text{ m} \)
Permanent load (unfactored) \( G_{ku(1)} = 0.204 \text{ kN/m} \)
Variable load 1 (unfactored) \( Q_{u(1)} = 0.45 \text{ kN/m} \)
Variable load 2 (unfactored) \( Q_{u(2)} = 0.45 \text{ kN/m} \)

Characteristic values of actions

Permanent:
- Reaction at left hand end: 0.3672 kN
- Reaction at right hand end: 0.3672 kN
- Maximum span moment: \( M_{kp} = 0.33048 \text{ kNm} \)
- Maximum shear force: \( F_{kp} = 0.3672 \text{ kN} \)

Variable 1:
- Reaction at left hand end: 0.81 kN
- Reaction at right hand end: 0.81 kN
- Maximum span moment: \( M_{kv1} = 0.729 \text{ kNm} \)
- Maximum shear force: \( F_{kv1} = 0.81 \text{ kN} \)

Variable 2:
- Reaction at left hand end: 0.81 kN
- Reaction at right hand end: 0.81 kN
- Maximum span moment: \( M_{kv2} = 0.729 \text{ kNm} \)
- Maximum shear force: \( F_{kv2} = 0.81 \text{ kN} \)

Design moment:
- \( M_{d1} = \gamma_{G} G_{u} M_{kp} + \gamma_{Q} Q_{u(2)} M_{kv2} = 1.5396 \text{ kNm} \)
- \( M_{d2} = \gamma_{G} G_{u} M_{kp} + \gamma_{Q} (M_{kv2} + \psi_{0} v_{1} M_{kv1}) = 2.3051 \text{ kNm} \)

Design shear force:
- \( V_{d1} = \gamma_{G} F_{kp} + \gamma_{Q} F_{kv2} = 2.5612 \text{ kN} \)
- \( V_{d2} = \gamma_{G} F_{kp} + \gamma_{Q} (F_{kv2} + \psi_{0} v_{1} F_{kv1}) = 2.5612 \text{ kN} \)

Design values - combination 2

- Design bending moment: \( M_{d} = 2.3051 \text{ kNm} \)
- Design shear force: \( V_{d} = 2.5612 \text{ kN} \)
- Load duration factor: \( k_{mod} = 0.8 \)
- Design axial load (+ve compress): \( N_{d} = 0 \text{ kN} \)
Section design parameters

- Depth of section: h = 225 mm
- Width of section: b = 50 mm
- Eff. length for bending about yy: ley = 3600 mm
- Eff. length for bending about zz: lez = 0 mm
- Length of bearing: lb = 100 mm

Redwood / Whitewood SS grade table 10
- Bending parallel to the grain: $f_{mk} = 24 \text{ N/mm}^2$
- Compress parallel to the grain: $f_{c0k} = 21 \text{ N/mm}^2$
- Compress perp to the grain: $f_{c90k} = 2.5 \text{ N/mm}^2$
- Tension parallel to the grain: $f_{t0k} = 14 \text{ N/mm}^2$
- Shear parallel to the grain: $f_{vk} = 3 \text{ N/mm}^2$
- Mean modulus of elasticity: $E_{0\text{mean}} = 11000 \text{ N/mm}^2$
- 5th percentile MOE: $E_{005} = 7400 \text{ N/mm}^2$
- Depth factor (Clause 3.2): $k_h = 1.0$
- Load-sharing modification factor: $k_{sys} = 1.0$
- Depth of notch at support: $k_v = 1$
- Notch is on top edge of beam: $k_{c90} = 1$
- Bearing modification factor: $k_{sh} = 0.67$

Member: 225 mm x 50 mm

Redwood / Whitewood SS grade table 10
- Moisture service class: 1
- Design bending stress: 5.4639 N/mm²
- Design bending strength: 14.769 N/mm²

DESIGN
- Deflection: 5.5928 mm

SUMMARY
- Limiting deflection: 14.4 mm
- Design shear stress: 0.37295 N/mm²
- Design shear strength: 1.2369 N/mm²
- Design bearing stress: 0.51224 N/mm²
- Design bearing strength: 1.5385 N/mm²

NOTE: The above values relate to combination 2 and need to be compared with those of combination 1.
**Location:** Simply supported rectangular glulam section

**Simply supported glulam beam to BS5268-2:2002**

Simply supported beam subjected to vertical loads.

Beam span \( L = 4.5 \text{ m} \)
Distance from left support to start \( L_{au(1)} = 0 \text{ m} \)
Distance from left support to end \( L_{bu(1)} = 4.5 \text{ m} \)
Dead load (unfactored) \( G_{ku(1)} = 6 \text{ kN/m} \)
Imposed load (unfactored) \( Q_{ku(1)} = 12 \text{ kN/m} \)
Maximum span bending moment \( 45.563 \text{ kNm} \)
Design shear force \( F_{ve'} = 40.5 \)

**Rectangular glulam member**

**with horizontal laminates**

Member laterally restrained in bending

Design axial load (+ve compress) \( F' = 30.9 \text{ kN} \)
Effective length for buckling about \( zz \) \( L_{ex'} = 4.5 \text{ m} \)
Effective length for buckling about \( yy \) \( L_{ey'} = 0 \text{ m} \)
Number of laminates in section \( n_p = 10 \)
Thickness of laminates in section \( t_p = 45 \text{ mm} \)
Width of section \( b = 135 \text{ mm} \)
Strength class C22 to Table 8.
Timber service class adopted \( t_m\text{class}=1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1.25 \)
Depth factor \( K_7 = 0.81 \times (d^2 + 92300)/(d^2 + 56800) = 0.92089 \)

No notches exist at the support \( K_5 = 1.0 \)
### DESIGN
- Laminate Thickness: 45 mm
- Number of laminates: 10

### SUMMARY
- Depth of section: 450 mm
- Width of section: 135 mm

Strength class C22 to Table 8.
- Moisture service class: 1
- Applied comprn @ Midsp: 0.50864 N/mm²
- Permiss comprn stress: 8.1199 N/mm²
- Applied bending stress: 10 N/mm²
- Permiss bending stress: 11.115 N/mm²
- Interact fact. @ Midsp: 0.96906
- Applied shear stress: 1 N/mm²
- Permiss shear stress: 2.0768 N/mm²
- Dead+imposed+shear Defl: 10.42 mm
- Limiting deflection: 13.5 mm

No precamber of beam is necessary.
Simply supported Glulam beam with general loading

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Ref No: SC255 BS

Simply supported Glulam beam to BS5268-2:2002

Simply supported beam subjected to vertical loads.

Beam span \( L = 6 \text{ m} \)

Distance from left support \( L_c(1) = 2 \text{ m} \)

Dead load (unfactored) \( G_{kc}(1) = 8 \text{ kN} \)

Imposed load (unfactored) \( Q_{kc}(1) = 0 \text{ kN} \)

Distance from left support \( L_c(2) = 4 \text{ m} \)

Dead load (unfactored) \( G_{kc}(2) = 8 \text{ kN} \)

Imposed load (unfactored) \( Q_{kc}(2) = 0 \text{ kN} \)

Dist. from left support to start \( L_{au}(1) = 0 \text{ m} \)

Distance from left support to end \( L_{bu}(1) = 6 \text{ m} \)

Dead load (unfactored) \( G_{ku}(1) = 5.5 \text{ kN/m} \)

Imposed load (unfactored) \( Q_{ku}(1) = 0 \text{ kN/m} \)

Maximum span bending moment \( 40.75 \text{ kNm} \)

Design shear force \( F'_v = 24.5 \text{ kN} \)

Rectangular Glulam member with horizontal laminates

Member laterally restrained in bending

Design axial load (+ve compress) \( F' = 25 \text{ kN} \)

Eff length for buckling about \( zz \) \( L_{ex}' = 6 \text{ m} \)

Eff length for buckling about \( yy \) \( L_{ey}' = 1.5 \text{ m} \)

Number of laminates in section \( n_p = 10 \)

Thickness of laminates in section \( t_p = 45 \text{ mm} \)

Width of section \( b = 115 \text{ mm} \)

Whitewood

Bending parallel to the grain \( b_{parg} = 7.5 \text{ N/mm}^2 \)

Compress parallel to the grain \( c_{parg} = 7.9 \text{ N/mm}^2 \)

Compress perpendicular to the grain \( c_{perd} = 2.1 \text{ N/mm}^2 \)

Tension parallel to the grain \( t_{parg} = 4.5 \text{ N/mm}^2 \)

Shear parallel to the grain \( s_{parg} = 0.82 \text{ N/mm}^2 \)

Mean modulus of elasticity \( E_{mean} = 10000 \text{ N/mm}^2 \)

Minimum modulus of elasticity \( E_{min} = 7000 \text{ N/mm}^2 \)
Timber service class adopted \( \text{tmclass}=1 \)

Timber service class modification factor \( K_2=1 \) as Table 16.

Duration of loading \( K_3=1.25 \)

Depth factor \( K_7=0.81 \frac{(d^2+92300)}{(d^2+56800)} =0.92089 \)

Modification factor \( K_{15}=1.39 \)

Modification factor \( K_{16}=1.39 \)

Modification factor \( K_{17}=1.04 \)

Modification factor \( K_{19}=2.34 \)

Modification factor \( K_{20}=1.07 \)

No notches exist at the support \( K_5=1.0 \)

---

**DESIGN**

<table>
<thead>
<tr>
<th>Laminate Thickness</th>
<th>45 mm</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Number of laminates</th>
<th>10</th>
</tr>
</thead>
</table>

**SUMMARY**

<table>
<thead>
<tr>
<th>Depth of section</th>
<th>450 mm</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Width of section</th>
<th>115 mm</th>
</tr>
</thead>
</table>

Whitewood

Moisture service class 1

Applied comprn @ Midsp 0.48309 N/mm²

Permiss comprn stress 7.9001 N/mm²

Applied bending stress 10.499 N/mm²

Permiss bending stress 12 N/mm²

Interact fact. @ Midsp 0.94602

Applied shear stress 0.71014 N/mm²

Permiss shear stress 2.3985 N/mm²

Dead+imposed+shear Defl 17.91 mm

Limiting deflection 18 mm

No precamber of beam is necessary
Location: Rectangular glulam section; tension and bending

Simply supported glulam beam to BS5268-2:2002

Simply supported beam subjected to vertical loads.

Beam span \( L = 6 \) m
Distance from left support \( L_c(1) = 2 \) m
Dead load (unfactored) \( G_{kc}(1) = 8 \) kN
Imposed load (unfactored) \( Q_{kc}(1) = 0 \) kN
Distance from left support \( L_c(2) = 4 \) m
Dead load (unfactored) \( G_{kc}(2) = 8 \) kN
Imposed load (unfactored) \( Q_{kc}(2) = 0 \) kN
Dist. from left support to start \( L_{au}(1) = 2 \) m
Distance from left support to end \( L_{bu}(1) = 4 \) m
Dead load (unfactored) \( G_{ku}(1) = 5.5 \) kN/m
Imposed load (unfactored) \( Q_{ku}(1) = 0 \) kN/m
Maximum span bending moment \( 29.75 \) kNm
Design shear force \( F_{ve}' = 13.5 \)

Rectangular glulam member

with horizontal laminates

Member laterally restrained

in bending

Design axial load (+ve compress) \( F' = -40 \) kN
Number of laminates in section \( n_p = 10 \)
Thickness of laminates in section \( t_p = 45 \) mm
Width of section \( b = 90 \) mm
Strength class D70 to Table 8.
Tensile area of section \( A_t = 40000 \) mm\(^2\)
Timber service class adopted \( t_m_{class} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1.25 \)
Depth factor \( K_7 = 0.81 \times (d^2 + 92300) / (d^2 + 56800) = 0.92089 \)
Width modification factor \( K_{14} = (300/h)^{0.11} = 0.95638 \)
No notches exist at the support \( K_5 = 1.0 \)
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
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<tbody>
<tr>
<td>Laminate Thickness</td>
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</tr>
<tr>
<td>Depth of section</td>
<td>450 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>90 mm</td>
</tr>
</tbody>
</table>

Strength class D70 to Table 8.
Moisture service class 1
Applied tension stress 1 N/mm²
Permiss tension stress 18.147 N/mm²
Applied bending stress 9.7942 N/mm²
Permiss bending stress 29.123 N/mm²
Interact fact. @ Midsp 0.39141
Applied shear stress 0.5 N/mm²
Permiss shear stress 4.8425 N/mm²
Dead+imposed+shear Defl 8.2184 mm
Limiting deflection 18 mm
No precamber of beam is necessary
Location: Rectangular glulam section; comp and bending

Simply supported glulam beam to BS5268-2:2002

Simply supported beam subjected to vertical loads.

Beam span \( L=6 \) m
Distance from left support \( Lc(1)=1 \) m
Dead load (unfactored) \( Gkc(1)=10 \) kN
Imposed load (unfactored) \( Qkc(1)=0 \) kN
Distance from left support \( Lc(2)=5 \) m
Dead load (unfactored) \( Gkc(2)=10 \) kN
Imposed load (unfactored) \( Qkc(2)=0 \) kN
Dist. from left support to start \( Lau(1)=0 \) m
Distance from left support to end \( Lbu(1)=6 \) m
Dead load (unfactored) \( Gku(1)=5 \) kN/m
Imposed load (unfactored) \( Qku(1)=5 \) kN/m
Maximum span bending moment \( 55 \) kNm
Design shear force \( Fve'=40 \)

Rectangular glulam member

with horizontal laminates

Member laterally restrained

in bending

Design axial load (+ve compress) \( F'=90 \) kN
Eff length for buckling about zz \( Lex'=6 \) m
Eff length for buckling about yy \( Ley'=1.5 \) m
Number of laminates in section \( np=10 \)
Thickness of laminates in section \( tp=45 \) mm
Width of section \( b=90 \) mm
Strength class D70 to Table 8.
Timber service class adopted \( tmclass=2 \)
Timber service class modification factor \( K2=1 \) as Table 16.
Duration of loading \( K3=1.25 \)
Depth factor \( K7=0.81*(d^2+92300)/(d^2+56800) =0.92089 \)
No notches exist at the support  \( K_5 = 1.0 \)

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</table>

Strength class D70 to Table 8.
Moisture service class 2
Applied comprn @ Midsp 2.2222 N/mm²
Permiss comprn stress 19.562 N/mm²
Applied bending stress 18.107 N/mm²
Permiss bending stress 29.123 N/mm²
Interact fact. @ Midsp 0.75793
Applied shear stress 1.4815 N/mm²
Permiss shear stress 4.8425 N/mm²
Dead+imposed+shear Defl 16.019 mm
Limiting deflection 18 mm
No precamber of beam is necessary
Location: Simply supported rectangular glulam section

Simply supported glulam beam to EC5

Simply supported beam subject to vertical loads.

Beam span \( L = 4.5 \text{ m} \)
Distance from left support to start \( \text{Lau}(1) = 0 \text{ m} \)
Distance from left support to end \( \text{Lbu}(1) = 4.5 \text{ m} \)
Permanent load (unfactored) \( G_{ku}(1) = 6 \text{ kN/m} \)
Variable load 1 (unfactored) \( Q_{ua}(1) = 12 \text{ kN/m} \)

Characteristic values of actions

Permanent:
Reaction at left hand end \( 13.5 \text{ kN} \)
Reaction at right hand end \( 13.5 \text{ kN} \)
Maximum span moment \( M_{kp} = 15.188 \text{ kNm} \)
Maximum shear force \( F_{kp} = 13.5 \text{ kN} \)
Variable 1:
Reaction at left hand end \( 27 \text{ kN} \)
Reaction at right hand end \( 27 \text{ kN} \)
Maximum span moment \( M_{kv1} = 30.375 \text{ kNm} \)
Maximum shear force \( F_{kv1} = 27 \text{ kN} \)
Design axial load (+ve compress) \( N_d = 43.878 \text{ kN} \)
Eff. length for buckling about \( yy \) \( 1e_y' = 4.5 \text{ m} \)
Eff. length for buckling about \( zz \) \( 1e_z' = 0 \text{ m} \)

Rectangular glulam member with horizontal laminates member laterally restrained in bending

Number of laminates in section \( n_p = 10 \)
Thickness of laminates in section \( t_p = 45 \text{ mm} \)
Width of section \( b = 135 \text{ mm} \)
Length of bearing \( l_b = 200 \text{ mm} \)
Glulam strength class GL26h
Depth factor (Clause 3.3) \( kh = (600/h)^{0.1} = 1.0292 \)
Instability factor \( kcy = 1/(ky+(ky^2-rsry^2)^{0.5}) = 0.96417 \)
Instability factor \( kcz = 1/(kz+(kz^2-rsrz^2)^{0.5}) = 1.0309 \)
Max instability factor \( kcz = 1 \)
No notches exist at the support \( kv = 1.0 \)
Bearing modification factor \( kc90 = 1.75 \)
Shear reduction factor \( ksh = 0.67 \)

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<tr>
<td>Width of section</td>
<td>135 mm</td>
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</table>

Glulam strength class GL26h
Moisture service class | 1
Design comprn @ Midsp | 0.72227 N/mm²
Design comprn strength | 19.968 N/mm²
Design bending stress | 14.5 N/mm²
Design bending strength | 20.551 N/mm²
Interact fact. @ Midsp | 0.74309
Design shear stress | 1.4428 N/mm²
Design shear strength | 1.801 N/mm²
Final Deflection | 12.131 mm
Limiting deflection | 18 mm
No precamber of beam is necessary
Simply supported glulam beam to EC5

Simply supported beam subject to vertical loads.

Beam span \( L = 6 \) m
Distance from left support \( L_c(1) = 2 \) m
Permanent load (unfactored) \( G_{kc}(1) = 8 \) kN
Variable load 1 (unfactored) \( Q_{ca}(1) = 1 \) kN
Distance from left support \( L_c(2) = 4 \) m
Permanent load (unfactored) \( G_{kc}(2) = 8 \) kN
Variable load 1 (unfactored) \( Q_{ca}(2) = 0 \) kN
Dist. from left support to start \( L_{au}(1) = 0 \) m
Distance from left support to end \( L_{bu}(1) = 6 \) m
Permanent load (unfactored) \( G_{ku}(1) = 5.56 \) kN/m
Variable load 1 (unfactored) \( Q_{ua}(1) = 0 \) kN/m

Characteristic values of actions

Permanent:
Reaction at left hand end \( 24.68 \) kN
Reaction at right hand end \( 24.68 \) kN
Maximum span moment \( M_{kp} = 41.02 \) kNm
Maximum shear force \( F_{kp} = 24.68 \) kN
Variable 1:
Reaction at left hand end \( 0.667 \) kN
Reaction at right hand end \( 0.3337 \) kN
Maximum span moment \( M_{kvp} = 1.3005 \) kNm
Maximum shear force \( F_{kp} = 0.667 \) kN
Design axial load (+ve compress) \( N_d = 35.5 \) kN
Eff. length for buckling about yy \( l_{ey}' = 1.5 \) m
Eff. length for buckling about zz \( l_{ez}' = 0 \) m

Rectangular glulam member
with horizontal laminates
Member laterally restrained
in bending

Number of laminates in section \( n_p = 10 \)
Thickness of laminates in section \( t_p = 45 \) mm
Width of section \( b = 115 \) mm
Length of bearing \( \text{lb}=200 \text{ mm} \)

Whitewood

| Bending parallel to the grain | \( f_{mk}=32 \text{ N/mm}^2 \) |
| Compress parallel to the grain | \( f_{c0k}=31 \text{ N/mm}^2 \) |
| Compress perp to the grain     | \( f_{c90k}=2.5 \text{ N/mm}^2 \) |
| Shear parallel to the grain    | \( f_{vk}=3.5 \text{ N/mm}^2 \) |
| Mean modulus of elasticity    | \( E_{0\text{mean}}=14700 \text{ N/mm}^2 \) |
| 5th percentile MOE            | \( E_{0.05}=11900 \text{ N/mm}^2 \) |
| Depth factor (Clause 3.3)     | \( k_h=(600/h)^{0.1}=1.0292 \) |
| Instability factor            | \( k_{cy}=1 \) |
| Instability factor            | \( k_{cz}=1 \) |
| No notches exist at the support | \( k_v=1.0 \) |
| Bearing modification factor   | \( k_{c90}=1 \) |
| Shear reduction factor        | \( k_{sh}=0.67 \) |
| Effective length about zz     | \( L_e=4.8 \text{ m} \) |

| Laminate Thickness | 45 mm |
| Number of laminates | 10 |
| Depth of section   | 450 mm |
| Width of section   | 115 mm |

Whitewood

| Moisture service class | 1 |
| Design comprn @ Midsp | 0.68599 \text{ N/mm}^2 |
| Design comprn strength | 23.808 \text{ N/mm}^2 |
| Design bending stress  | 14.77 \text{ N/mm}^2 |
| Design bending strength| 25.293 \text{ N/mm}^2 |
| Interact fact. @ Midsp | 0.5848 |
| Design shear stress   | 0.98979 \text{ N/mm}^2 |
| Design shear strength | 1.801 \text{ N/mm}^2 |
| Final Deflection      | 21.4 mm |
| Limiting deflection   | 24 mm |

No precamber of beam is necessary
Location: Simply supported glulam beam with axial load

Simply supported glulam beam to EC5

Simply supported beam subject to vertical loads.

Beam span \( L=3.5 \) m
Distance from left support to start \( L_{au}(1)=0 \) m
Distance from left support to end \( L_{bu}(1)=3.5 \) m
Permanent load (unfacted) \( G_{ku}(1)=0 \) kN/m
Variable load 1 (unfacted) \( Q_{ua}(1)=20 \) kN/m

Characteristic values of actions

Permanent:
Reaction at left hand end \( 0.175E-3 \) kN
Reaction at right hand end \( 0.175E-3 \) kN
Maximum span moment \( M_{kp}=0.15313E-3 \) kNm
Maximum shear force \( F_{kp}=0.175E-3 \) kN
Variable 1:
Reaction at left hand end \( 35 \) kN
Reaction at right hand end \( 35 \) kN
Maximum span moment \( M_{kv1}=30.625 \) kNm
Maximum shear force \( F_{kv1}=35 \) kN
Design axial load (+ve compress) \( N_d=77.63 \) kN
Eff.length for buckling about yy \( l_{ey}'=3.5 \) m
Eff.length for buckling about zz \( l_{ez}'=0 \) m

Rectangular glulam member
with horizontal laminates
Member laterally restrained
in bending

Number of laminates in section \( n_p=27 \)
Thickness of laminates in section \( t_p=20 \) mm
Width of section \( b=165 \) mm
Length of bearing \( l_b=300 \) mm
Glulam strength class GL28h
Depth factor (Clause 3.3)  \( kh = (600/h)^{0.1} = 1.0106 \)
Instability factor  \( kcy = 1/(ky+(ky^2-rsry^2)^{0.5}) = 0.99208 \)
Instability factor  \( kcz = 1/(kz+(kz^2-rsrz^2)^{0.5}) = 1.0309 \)
Max instability factor  \( kcz = 1 \)
No notches exist at the support  \( kv = 1.0 \)
Bearing modification factor  \( kc90 = 1.75 \)
Shear reduction factor  \( ksh = 0.67 \)
Effective length about zz  \( Le = 2.8 \text{ m} \)

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<tr>
<td>Laminate Thickness</td>
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<tr>
<td>Number of laminates</td>
<td>27</td>
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</tbody>
</table>

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<thead>
<tr>
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<tbody>
<tr>
<td>Depth of section</td>
<td>540 mm</td>
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<tr>
<td>Width of section</td>
<td>165 mm</td>
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</tbody>
</table>

Glulam strength class GL28h
Moisture service class 2
Design comprn @ Midsp 0.87127 N/mm²
Design comprn strength 21.504 N/mm²
Design bending stress 5.7286 N/mm²
Design bending strength 21.732 N/mm²
Interact fact. @ Midsp 0.30445
Design shear stress 0.87945 N/mm²
Design shear strength 1.801 N/mm²
Final Deflection 2.719 mm
Limiting deflection 14 mm
No precamber of beam is necessary
**Location:** Simply supported glulam beam with axial load

**Simply supported glulam beam to EC5**

- Beam span $L = 10\, m$
- Dist. from left support to start $L_a(1) = 0\, m$
- Distance from left support to end $L_b(1) = 10\, m$
- Permanent load (unfactored) $G_{ku}(1) = 1.94\, kN/m$
- Variable load 1 (unfactored) $Q_{ua}(1) = 2.25\, kN/m$

**Characteristic values of actions**

<table>
<thead>
<tr>
<th>Action</th>
<th>Value</th>
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<tbody>
<tr>
<td>Permanent Reaction at left hand end</td>
<td>9.7 kN</td>
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<tr>
<td>Permanent Reaction at right hand end</td>
<td>9.7 kN</td>
</tr>
<tr>
<td>Maximum span moment</td>
<td>$M_{kp} = 24.25, kNm$</td>
</tr>
<tr>
<td>Maximum shear force</td>
<td>$F_{kp} = 9.7, kN$</td>
</tr>
<tr>
<td>Variable 1 Reaction at left hand end</td>
<td>11.25 kN</td>
</tr>
<tr>
<td>Variable 1 Reaction at right hand end</td>
<td>11.25 kN</td>
</tr>
<tr>
<td>Variable 1 Maximum span moment</td>
<td>$M_{kv1} = 28.125, kNm$</td>
</tr>
<tr>
<td>Variable 1 Maximum shear force</td>
<td>$F_{kv1} = 11.25, kN$</td>
</tr>
<tr>
<td>Design axial load (+ve compress)</td>
<td>$N_d = 0, kN$</td>
</tr>
<tr>
<td>Eff. length for buckling about yy</td>
<td>$l_{ey}' = 10, m$</td>
</tr>
<tr>
<td>Eff. length for buckling about zz</td>
<td>$l_{ez}' = 0, m$</td>
</tr>
</tbody>
</table>

**Rectangular glulam member**

**with horizontal laminates**

**Member laterally restrained**

**in bending**

- Number of laminates in section $n_p = 45$
- Thickness of laminates in section $t_p = 12\, mm$
- Width of section $b = 115\, mm$
- Length of bearing $l_b = 50\, mm$
- Glulam strength class GL26h
Depth factor (Clause 3.3) $kh = (600/h)^{0.1} = 1.0106$
No notches exist at the support $kv = 1.0$
Bearing modification factor $kc90 = 4$
Shear reduction factor $ksh = 0.67$

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<td>540 mm</td>
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<td>115 mm</td>
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</table>

Glulam strength class GL26h
Moisture service class 2
Design bending stress 13.406 N/mm²
Design bending strength 22.702 N/mm²
Interact fact. @ Midsp 0.59051
Design shear stress 0.72031 N/mm²
Design shear strength 2.0261 N/mm²
Final Deflection 47.206 mm
Limiting deflection 40 mm
Precamber of beam 13.834 mm
Location: Rectangular glulam section; tension and bending (EC only)

Simply supported glulam beam to EC5

Simply supported beam subject to vertical loads.

Beam span
Distance from left support
Permanent load (unfactored)
Variable load 1 (unfactored)
Distance from left support
Permanent load (unfactored)
Variable load 1 (unfactored)
Dist. from left support to start
Distance from left support to end
Permanent load (unfactored)
Variable load 1 (unfactored)

Characteristic values of actions

Permanent:
Reaction at left hand end 18.68 kN
Reaction at right hand end 18.68 kN
Maximum span moment Mkp=32.02 kNm
Maximum shear force Fkp=18.68 kN
Variable 1:
Reaction at left hand end 0.4E-3 kN
Reaction at right hand end 0.4E-3 kN
Maximum span moment Mkv1=0.65E-3 kNm
Maximum shear force Fkv1=0.4E-3 kN
Design axial load (+ve compress) Nd=-57.2 kN
Eff.length for buckling about yy ley'=6 m
Eff.length for buckling about zz lez'=0 m

Rectangular glulam member

with horizontal laminates

Member laterally restrained

in bending

Number of laminates in section np=10
Thickness of laminates in section tp=45 mm
Width of section b=90 mm
Length of bearing $lb=200 \text{ mm}$

Glulam strength class GL32h

Tensile area of section $At=40000 \text{ mm}^2$

Depth factor (Clause 3.3) $kh=\left(\frac{600}{h}\right)^{0.1}=1.0292$

Width modification factor $kw=\left(\frac{600}{\text{lardim}}\right)^{0.1}=1.0292$

No notches exist at the support $kv=1.0$

Bearing modification factor $kc90=1.75$

Shear reduction factor $ksh=0.67$

Effective length about zz $Le=4.8 \text{ m}$

**DESIGN**
- Laminate Thickness $45 \text{ mm}$
- Number of laminates 10
- Depth of section $450 \text{ mm}$
- Width of section $90 \text{ mm}$

**SUMMARY**
- Glulam strength class GL32h
- Moisture service class 1
- Design tensile stress $1.43 \text{ N/mm}^2$
- Design tensile strength $20.235 \text{ N/mm}^2$
- Design bending stress $14.231 \text{ N/mm}^2$
- Design bending strength $25.293 \text{ N/mm}^2$
- Interact fact. @ Midsp $0.63333$
- Design shear stress $0.92938 \text{ N/mm}^2$
- Design shear strength $1.801 \text{ N/mm}^2$
- Final Deflection $21.727 \text{ mm}$
- Limiting deflection $24 \text{ mm}$
- No precamber of beam is necessary
Location: Rectangular glulam section; comp and bending (EC only)

Simply supported glulam beam to EC5

Simply supported beam subject to vertical loads.

Beam span \( L = 6 \text{ m} \)
Distance from left support \( L_{c(1)} = 1 \text{ m} \)
Permanent load \( G_{kc}(1) = 10 \text{ kN} \)
Variable load \( Q_{ca}(1) = 0 \text{ kN} \)
Distance from left support \( L_{c(2)} = 5 \text{ m} \)
Permanent load \( G_{kc}(2) = 10 \text{ kN} \)
Variable load \( Q_{ca}(2) = 0 \text{ kN} \)
Distance from left support to start \( L_{au}(1) = 0 \text{ m} \)
Distance from left support to end \( L_{bu}(1) = 6 \text{ m} \)
Permanent load \( G_{ku}(1) = 4 \text{ kN/m} \)
Variable load \( Q_{ua}(1) = 4 \text{ kN/m} \)

Characteristics of actions

Permanent:
Reaction at left hand end \( 22 \text{ kN} \)
Reaction at right hand end \( 22 \text{ kN} \)
Maximum span moment \( M_{kp} = 28 \text{ kNm} \)
Maximum shear force \( F_{kp} = 22 \text{ kN} \)
Variable 1:
Reaction at left hand end \( 12 \text{ kN} \)
Reaction at right hand end \( 12 \text{ kN} \)
Maximum span moment \( M_{kv1} = 18 \text{ kNm} \)
Maximum shear force \( F_{kv1} = 12 \text{ kN} \)
Design axial load (+ve compress) \( N_{d} = 128.7 \text{ kN} \)
Eff. length for buckling about yy \( l_{ey} = 6 \text{ m} \)
Eff. length for buckling about zz \( l_{ez} = 1.5 \text{ m} \)

Rectangular glulam member

with horizontal laminates

Member laterally restrained

in bending

Number of laminates in section \( n_{p} = 10 \)
Thickness of laminates in section \( t_{p} = 45 \text{ mm} \)
Width of section \( b = 90 \text{ mm} \)
Length of bearing \( L = 200 \text{ mm} \)

Glulam strength class GL32h

Depth factor (Clause 3.3) \( \phi_h = (600/h)^{0.1} = 1.0292 \)

Instability factor \( k_{cy} = 1/(k_y + (k_y^2 - rsry^2)^{0.5}) = 0.90933 \)

Instability factor \( k_{cz} = 1/(k_z + (k_z^2 - rsrz^2)^{0.5}) = 0.80174 \)

No notches exist at the support \( k_v = 1.0 \)

Bearing modification factor \( k_{c90} = 1.75 \)

Shear reduction factor \( k_{sh} = 0.67 \)

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Glulam strength class GL32h

Moisture service class 2

Design comprn @ Midsp 3.1778 N/mm²

Design comprn strength 24.576 N/mm²

Design bending stress 21.333 N/mm²

Design bending strength 25.293 N/mm²

Interact fact. @ Midsp 0.98564

Design shear stress 1.7579 N/mm²

Design shear strength 1.801 N/mm²

Final Deflection 31.946 mm

Limiting deflection 24 mm

Precamber of beam 11.421 mm
Sample output for SCALE Proforma 256. (ans=1) Made by: IFB
Timber design to BS5268 and Eurocode 5 Date: 02/12/19
Simply supported flitched beam with general loadin Copyright 1986-2019 Fitzroy Systems Ltd. Ref No: SC256 BS

Location: Flitched timber beam with udl & point loads

Fully restrained flitched beam to BS5268-2:2002 and BS 449:1990

Beam span L=7.5 m
Section comprises steel plates to top & bottom faces of timber.
Depth of timber section d=300 mm
Width of timber section b=100 mm
Depth of steel section ds=12 mm
Width of steel section bs=100 mm
Length of bearing lb=150 mm
Bearing modification factor K4=1.0
Strength class C22 to Table 8.
Timber service class adopted tmclass=1
Timber service class modification factor K2=1 as Table 16.
Load-sharing modification factor K8=1.0

Section properties and Modular ratio

Total Inertia about xx axis in equivalent timber
Extreme fibre is steel plate Yc=Ys=162 mm
Dist of centroid to timber edge Yn=Yt=150 mm
Z to top edge of steel plate Zc=Ixx/Yc=12.765E6 mm³

Simply supported beam subjected to vertical loads.

Beam span L=7.5 m
Dist. from left support to start Lau(1)=0 m
Distance from left support to end Lbu(1)=7.5 m
Dead load (unfactored) Gku(1)=2.25 kN/m
Imposed load (unfactored) Qku(1)=1 kN/m
Maximum span bending moment 22.852 kNm
Design shear force Fve'=12.188
Duration of loading K3=1.25
Depth factor K7=(300/d)^0.11=1
Bolt diameter bd=20 mm
No of rows of bolts to be adopted NRC=1
Spacing of bolts along beam space=250 mm
Timber characteristic density $\rho_{k1} = 340$ kg/m³
Angle between grain and load $\alpha_{p1}' = 0^\circ$

**Bolt shear capacities**

Basic loads in kN will be derived from formula given in Annex G.3
Load duration for bolted connection = Medium term.
Embedded strength for timber 1
Embedding strength $f_{hd} = 0.050 \times (1 - 0.01 \times bd) \times \rho_{k1} \times K_a = 13.6$
Embedding strength (Timb 1) $f_{hd1} = f_{hd} / ((K_{90} \times \sin(\alpha_{p1})^2 + \cos(\alpha_{p1})^2)) = 13.6$
Equations G1 to G6 in annex G in BS5268-2:2002.

Basic bolt shear capacity $basic = 1 \times G_{min} / (F_d \times K_d) = 11.356$ kN
Mod factor for moisture content $K_{56} = 1$
Mod factor for number of bolts $K_{57} = 1$

Permissible bolt load $p_{bl} = basic \times K_{46} \times K_{56} \times K_{57} = 14.195$ kN

**DESIGN**
- Modular ratio 31.538

**SUMMARY**
- Timber bending stress 1.6575 N/mm²
- Timber perm. bending 8.5 N/mm²
- Steel bending stress 56.459 N/mm²
- Steel perm. bending 180 N/mm²
- Deflection 10.6 mm
- Limiting deflection 22.5 mm
- Applied shear 12.188 kN
- Shear capacity 17.75 kN
- Bearing stress 0.81253 N/mm²
- Permissible bearing 2.875 N/mm²
- Applied load per bolt 8.6992 kN
- Permiss load per bolt 14.195 kN

Bolts are 20 mm diameter in 1 row(s) at 250 mm centres.
Location: Flitched timber beam with udl & point loads

Fully restrained flitched beam to BS5268-2:2002 and BS 449:1990

Beam span L=4.8 m
Section comprises one steel web plate with a timber member on each side.
Depth of timber section d=200 mm
Width of timber section b=50 mm
Depth of steel section ds=190 mm
Width of steel section bs=8 mm
Length of bearing lb=100 mm
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor K4=1.0
Strength class C18 to Table 8.
Timber service class adopted tmclass=1
Timber service class modification factor K2=1 as Table 16.
Load-sharing modification factor K8=1.1

Section properties and Modular ratio

Total Inertia about xx axis in equivalent timber
Extreme fibre is timber section Yc=Yt=100 mm
Dist of centroid to steel edge Yn=Ys=95 mm
Z to top edge of timber Zc=Ixx/Yc=2.229E6 mm³

Simply supported beam subjected to vertical loads.

Beam span L=4.8 m
Dist. from left support to start Lau(1)=0 m
Distance from left support to end Lbu(1)=4.8 m
Dead load (unfactored) Gku(1)=1.2 kN/m
Imposed load (unfactored) Qku(1)=0.6 kN/m
Maximum span bending moment 5.184 kNm
Design shear force Fve'=4.32

Duration of loading K3=1
Depth factor K7=(300/d)^0.11=1.0456

Bolts between timber and steel plate along length of beam

Bolt diameter bd=16 mm
Spacing of bolts along beam space=500 mm
Bolts are in double shear. Width of timber for bolt stress calculation
is the section width $b$ 
Angle between grain and load $\alpha_1' = 90$ degrees
Timber characteristic density $p_{k1} = 320$ kg/m$^3$
Basic loads in kN will be derived from formula given in Annex G.3
Load duration for bolted connection = Medium term.
Embedded strength for timber 1
Embedding strength $f_{hod} = 0.050 \times (1 - 0.01 \times bd) \times p_{k1} \times K_a = 17.78$
Embedding strength (Timb 1) $f_{hd1} = f_{hod} / \left( (K90 \times \sin(\alpha_1')^2 + \cos(\alpha_1')^2) \right) = 11.182$
Equations G7 G8 G9 G10 8945.8 8945.8 6709.3 9226.4
Min value of equations G7 to G10 $G_{\text{min}} = 6709.3$

Basic bolt shear capacity $\text{basic} = 1 \times G_{\text{min}} / (F_d \times K_d) = 4.7923$ kN
Mod factor for moisture content $K_{56} = 0.7$
Mod factor for number of bolts $K_{57} = 1$
Permissible bolt load in timber $p_{bl} = 2 \times \text{basic} \times K_{46} \times K_{56} \times K_{57} = 6.7093$ kN

Design
Member: 200 mm x 108 mm overall
Modular ratio 34.167

Summary
Timber bending stress 2.3257 N/mm$^2$
Timber perm. bending 6.671 N/mm$^2$
Steel bending stress 75.489 N/mm$^2$
Steel perm. bending 180 N/mm$^2$
Deflection 9.5335 mm
Limiting deflection 14 mm
Applied shear 4.32 kN
Shear capacity 9.8267 kN
Bearing stress 0.432 N/mm$^2$
Permissible bearing 2.42 N/mm$^2$

Bolts are 16 mm diameter at 500 mm centres (10 No total).
Edge distance in the steel to be a minimum of 32 mm.
Bolts along the beam to be staggered above and below the centre line with their centres offset from the beam centre line to suit the min bolt edge spacing of 64 mm.
The minimum No of bolts required at the bearing = 2
Location: Flitched timber beam with udl & point loads

Fully restrained flitched beam to BS5268-2:2002 and BS 449:1990

Beam span \( L = 8 \text{ m} \)
Section comprises steel plates to each side face of timber section.
Depth of timber section \( d = 400 \text{ mm} \)
Width of timber section \( b = 100 \text{ mm} \)
Depth of steel section \( d_s = 375 \text{ mm} \)
Width of steel section \( b_s = 12 \text{ mm} \)
Length of bearing \( l_b = 100 \text{ mm} \)
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor \( K_4 = 1.0 \)
Strength class C18 to Table 8.
Timber service class adopted \( \text{tmclass} = 2 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Load-sharing modification factor \( K_8 = 1.0 \)

Section properties and Modular ratio

Total Inertia about \( xx \) axis in equivalent timber
Extreme fibre is timber section \( Y_c = Y_t = 200 \text{ mm} \)
Dist. of centroid to steel edge \( Y_n = Y_s = 187.5 \text{ mm} \)
\( Z \) to top edge of timber \( Z_c = I_{xx}/Y_c = 20.684E6 \text{ mm}^3 \)

Simply supported beam subjected to vertical loads.

Beam span \( L = 8 \text{ m} \)
Distance from left support \( L_{c(1)} = 2 \text{ m} \)
Dead load (unfactored) \( G_{k_c(1)} = 6 \text{ kN} \)
Imposed load (unfactored) \( Q_{k_c(1)} = 4 \text{ kN} \)
Dist. from left support to start \( L_{a(1)} = 0 \text{ m} \)
Distance from left support to end \( L_{b(1)} = 8 \text{ m} \)
Dead load (unfactored) \( G_{k_u(1)} = 4 \text{ kN/m} \)
Imposed load (unfactored) \( Q_{k_u(1)} = 2 \text{ kN/m} \)
Maximum span bending moment 58.52 kNm
Design shear force $Fve' = 31.5$

Duration of loading $K3 = 1.25$

Depth factor $K7 = 0.81 \times (d^2 + 92300)/(d^2 + 56800) = 0.94263$

Bolts between timber and steel plate along length of beam

Bolt diameter $bd = 20$ mm
Spacing of bolts along beam $space = 500$ mm
Bolts are in double shear. Width of timber for bolt stress calculation is the section width $b$ $bb = 100$ mm
Angle between grain and load $\alpha_1' = 90$ degrees
Timber characteristic density $p_k1 = 320$ kg/m$^3$
Basic loads in kN will be derived from formula given in Annex G.3
Load duration for bolted connection = Medium term.
Embedded strength for timber 1
Embedding strength $fhod = 0.050 \times (1 - 0.01 \times bd) \times p_k1 \times K_a = 16.933$
Embedding strength (Timb 1) $fhd1 = fhod / ((K90 \times \text{SIN}(\alpha_1')^2 + \text{COS}(\alpha_1')^2) = 10.262$
Equations G1 to G6 in annex G in BS5268-2:2002.

Basic bolt shear capacity $\text{basic} = 1 \times G_{\text{min}} / (F_d \times K_d) = 6.4748$ kN
Mod factor for moisture content $K56 = 0.7$
Mod factor for number of bolts $K57 = 1$
Permissible bolt load in timber $pbl = 2 \times \text{basic} \times K46 \times K56 \times K57 = 9.0647$ kN

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<th>DESIGN</th>
<th>Modular ratio</th>
<th>34.167</th>
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<tbody>
<tr>
<td>SUMMARY</td>
<td>Timber bending stress</td>
<td>2.8292 N/mm$^2$</td>
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<td>Timber perm. bending</td>
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<td>Steel bending stress</td>
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<td>Steel perm. bending</td>
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<td></td>
<td>Deflection</td>
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<td>Limiting deflection</td>
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<td>Applied shear</td>
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<td>Shear capacity</td>
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<td>Bearing stress</td>
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<td>Permissible bearing</td>
<td>210 N/mm$^2$</td>
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<td>Plt buckling @ supp.</td>
<td>4.5652 N/mm$^2$</td>
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<td>Perm. buckling stress</td>
<td>72.922 N/mm$^2$</td>
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</table>

Bolts are 20 mm diameter at 500 mm centres (17 No total).
Edge distance in the steel to be a minimum of 40 mm.
Bolts along the beam to be staggered above and below the centre line with their centres offset from the beam centre line to suit the min bolt edge spacing of 80 mm.
The minimum No of bolts required at the bearing = 4
Location: Flitched timber beam with udl & point loads

Fully restrained flitched beam to BS5268-2:2002 and BS 449:1990

Beam span \( L = 6 \) m
Section comprises two steel web plates and 3 timber members.
Depth of timber section \( d = 400 \) mm
Width of timber section \( b = 125 \) mm
Depth of steel section \( ds = 375 \) mm
Width of steel section \( bs = 16 \) mm
Length of bearing \( lb = 150 \) mm
Bearing modification factor \( K4 = 1.0 \)
Strength class C22 to Table 8.
Timber service class adopted \( tmclass = 1 \)
Timber service class modification factor \( K2 = 1 \) as Table 16.
Load-sharing modification factor \( K8 = 1.0 \)

Section properties and Modular ratio

Total Inertia about xx axis in equivalent timber
Extreme fibre is timber section \( Yc = Yt = 200 \) mm
Dist of centroid to steel edge \( Yn = Ys = 187.5 \) mm
\( Z \) to top edge of timber \( Zc = Ixx/Yc = 32.175E6 \) mm³

Simply supported beam subjected to vertical loads.

Beam span \( L = 6 \) m
Distance from left support \( Lc(1) = 3 \) m
Dead load (unfactored) \( Gkc(1) = 15.75 \) kN
Imposed load (unfactored) \( Qkc(1) = 78.438 \) kN
Dist. from left support to start \( Lau(1) = 0 \) m
Distance from left support to end \( Lbu(1) = 6 \) m
Dead load (unfactored) \( Gku(1) = 0.96 \) kN/m
Imposed load (unfactored) \( Qku(1) = 0 \) kN/m
Maximum span bending moment \( 145.6 \) kNm
Design shear force \( Fve' = 49.974 \)
Duration of loading \( K3 = 1.25 \)
Depth factor \( K7 = 0.81 \times (d^2 + 92300)/(d^2 + 56800) = 0.94263 \)
Bolts between timber and steel plate along length of beam

- Bolt diameter: bd=16 mm
- Spacing of bolts along beam: space=600 mm
- Bolts are in double shear. Width of timber for bolt stress calculation is the section width: bb=125 mm
- Angle between grain and load: alp1'=90 degrees
- Timber characteristic density: pk1=340 kg/m³
- Basic loads in kN will be derived from formula given in Annex G.3
- Load duration for bolted connection = Long term.
- Embedding strength for timber 1: fhod=0.038*(1-0.01*bd)*pk1*Ka=14.357
- Embedding strength (Timb 1): fhdl=fhod/((K90*SIN(alp1)^2)+COS(alp1)^2)=9.0297

Equations G1 to G6 in annex G in BS5268-2:2002.

- Basic bolt shear capacity: basic=1*Gmin/(Fd*Kd)=6.1123 kN
- Mod factor for moisture content: K56=0.7
- Mod factor for number of bolts: K57=1
- Permissible bolt load in timber: pbl=4*basic*K46*K56*K57=17.114 kN

Member: 400 mm x 407 mm overall

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<th>DESIGN</th>
<th>Modular ratio: 31.538</th>
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<tbody>
<tr>
<td>SUMMARY</td>
<td>Timber bending stress: 4.5252 N/mm²</td>
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<td>Timber perm. bending: 8.0124 N/mm²</td>
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<td>Steel bending stress: 133.8 N/mm²</td>
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<td>Steel perm. bending: 180 N/mm²</td>
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<td>Limiting deflection: 18 mm</td>
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<td>Applied shear: 49.974 kN</td>
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<td></td>
<td>Shear capacity: 88.75 kN</td>
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<td>Bearing stress: 0.88843 N/mm²</td>
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<tr>
<td></td>
<td>Permissible bearing: 2.875 N/mm²</td>
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</tbody>
</table>

Bolts are 16 mm diameter at 600 mm centres (11 No total).
Edge distance in the steel to be a minimum of 32 mm.
Bolts along the beam to be staggered above and below the centre line with their centres offset from the beam centre line to suit the minimum bolt edge spacing of 64 mm.
The minimum No of bolts required at the bearing = 3
Location: Flitched timber beam with udl & point loads

Fully restrained flitched beam to BS5268-2:2002 and BS 449:1990

Beam span \( L = 7.5 \, \text{m} \)
Design BM about \( xx \) (positive) \( M' = 22.852 \, \text{kNm} \)
Design shear force in \( y \) direction \( F_{ve}' = 12.188 \, \text{kN} \)
Design axial load (+ve compress) \( F_a = 1.2 \, \text{kN} \)
Section comprises steel plates to top & bottom faces of timber.
Depth of timber section \( d = 300 \, \text{mm} \)
Width of timber section \( b = 100 \, \text{mm} \)
Depth of steel section \( d_s = 12 \, \text{mm} \)
Width of steel section \( b_s = 100 \, \text{mm} \)
Length of bearing \( l_b = 150 \, \text{mm} \)
Bearing modification factor \( K_4 = 1.0 \)
Strength class C22 to Table 8.
Timber service class adopted \( t_m\text{class} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Load-sharing modification factor \( K_8 = 1.0 \)

Section properties and Modular ratio

Total Inertia about \( xx \) axis in equivalent timber
Extreme fibre is steel plate \( Y_c = Y_s = 162 \, \text{mm} \)
Dist of centroid to timber edge \( Y_n = Y_t = 150 \, \text{mm} \)
Z to top edge of steel plate \( Z_c = \frac{I_{xx}}{Y_c} = 12.765 \times 10^6 \, \text{mm}^2 \)
Duration of loading \( K_3 = 1.25 \)
Depth factor \( K_7 = (\frac{300}{d})^{0.11} = 1 \)
Axial compression allowable stress \( P_c = 170 \, \text{N/mm}^2 \)
Eff length for bending about \( xx \) \( L_{ex}' = 4 \, \text{m} \)
Eff length for bending about \( yy \) \( L_{ey}' = 4 \, \text{m} \)
Modification factor \( K_{12} = (0.5 + (1 + \eta) \frac{C}{2}) - ((0.5 + (1 + \eta) \frac{C}{2})^2 - C)^{0.5} = 0.19717 \)
Percent proportion of load in timber gtar = 28\% 
Bolt diameter \( b_d = 20 \, \text{mm} \)
No of rows of bolts to be adopted \( N_{RC} = 1 \)
Spacing of bolts along beam \( \text{space} = 250 \, \text{mm} \)

Timber characteristic density \( p_{k1} = 340 \, \text{kg/m}^3 \)
Angle between grain and load \( \alpha_{p1}' = 0^\circ \)
Bolt shear capacities

Basic loads in kN will be derived from formula given in Annex G.3
Load duration for bolted connection = Medium term.

Embedded strength for timber 1
Embedding strength  \( fhod = 0.050 \times (1 - 0.01 \times bd) \times pk1 \times Ka = 13.6 \)
Embedding strength (Timb 1)  \( fhd1 = fhod / ((K90 \times \text{SIN}(\alpha_1)^2) \times \text{COS}(\alpha_1)^2) = 13.6 \)

Basic bolt shear capacity  \( \text{basic} = 1 \times G_{\text{min}} / (F_d \times K_d) = 11.356 \text{ kN} \)
Mod factor for moisture content  \( K56 = 1 \)
Mod factor for number of bolts  \( K57 = 1 \)

Permissible bolt load  \( \text{pbl} = \text{basic} \times K46 \times K56 \times K57 = 14.195 \text{ kN} \)

Member: 324 mm x 100 mm overall

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<tbody>
<tr>
<td>Modular ratio</td>
<td>31.538</td>
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<table>
<thead>
<tr>
<th>SUMMARY</th>
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<tbody>
<tr>
<td>Timber bending stress</td>
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<td>Steel bending stress</td>
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<td>Applied comprn stress</td>
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<td>Applied load per bolt</td>
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<td>Permiss load per bolt</td>
<td>14.195 kN</td>
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Bolts are 20 mm diameter in 1 row(s) at 250 mm centres.
The end connections are beyond the scope of this proforma.
Location: Flitched timber beam with udl & point loads

Fully restrained flitched beam to BS5268-2:2002 and BS 449:1990

Beam span \( L = 4.8 \text{ m} \)
Design BM about xx (positive) \( M' = 5.184 \text{ kNm} \)
Design shear force in y direction \( Fve' = 4.32 \text{ kN} \)
Design axial load (+ve compress) \( F_a = -10.2 \text{ kN} \)
Section comprises one steel web plate with a timber member on each side.
Depth of timber section \( d = 200 \text{ mm} \)
Width of timber section \( b = 50 \text{ mm} \)
Depth of steel section \( ds = 190 \text{ mm} \)
Width of steel section \( bs = 8 \text{ mm} \)
Length of bearing \( lb = 100 \text{ mm} \)
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor \( K_4 = 1.0 \)
Strength class C18 to Table 8.
Timber service class adopted \( tmclass = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Load-sharing modification factor \( K_8 = 1.1 \)

Section properties and Modular ratio

Total Inertia about xx axis in equivalent timber
Extremne fibre is timber section \( Y_c = Y_t = 100 \text{ mm} \)
Dist of centroid to steel edge \( Y_n = Y_s = 95 \text{ mm} \)
Z to top edge of timber \( Z_c = \frac{I_{xx}}{Y_c} = 2.229 \times 10^6 \text{ mm}^2 \)
Duration of loading \( K_3 = 1 \)
Depth factor \( K_7 = (300/d)^{0.11} = 1.0456 \)
Tensile area of timber section \( T_a = 19000 \text{ mm}^2 \)
Tensile area of steel section \( T_s = 1500 \text{ mm}^2 \)
Width modification factor \( K_{14} = (300/h)^{0.11} = 1.0456 \)
Percent proportion of load in timber \( g_{tar} = 25 \% \)

Bolts between timber and steel plate along length of beam

Bolt diameter \( bd = 16 \text{ mm} \)
Spacing of bolts along beam \( space = 600 \text{ mm} \)
Bolts are in double shear. Width of timber for bolt stress calculation
is the section width \( b = 50 \text{ mm} \)
Angle between grain and load \( \alpha_{1}' = 90 \text{ degrees} \)
Timber characteristic density \( p_k 1 = 320 \text{ kg/m}^3 \)
Basic loads in kN will be derived from formula given in Annex G.3
Load duration for bolted connection = Medium term.
Embedded strength for timber 1
Embedding strength \( f_{hod} = 0.050 \times (1-0.01 \times bd) \times pk_1 \times Ka = 17.78 \)
Embedding strength (Timb 1) \( f_{hodl} = f_{hod} / ((K90 \times \text{SIN} (\alpha_{1})^{2}) + \text{COS}(\alpha_{1})^{2}) = 11.182 \)
Equations G7  G8  G9  G10
8945.8  8945.8  6709.3  9226.4
Min value of equations G7 to G10 \( G_{min} = 6709.3 \)
Basic bolt shear capacity  
\[ \text{basic} = \frac{1 \times G_{min}}{(F_d \times K_d)} = 4.7923 \text{ kN} \]

Mod factor for moisture content  
\[ K_{56} = 0.7 \]

Mod factor for number of bolts  
\[ K_{57} = 1 \]

Permissible bolt load in timber  
\[ p_{bl} = 2 \times \text{basic} \times K_{46} \times K_{56} \times K_{57} = 6.7093 \text{ kN} \]

**DESIGN**
- Modular ratio 34.167

**SUMMARY**
- Timber bending stress 2.3257 N/mm²
- Timber perm. bending 6.671 N/mm²
- Steel bending stress 75.489 N/mm²
- Steel perm. bending 180 N/mm²
- Timber tensile X sec area 19000 mm²
- Steel tensile X sec area 1500 mm²
- Timber moisture class 1
- Applied tension stress 0.1275 N/mm²
- Permiss tension stress 4.0256 N/mm²
- Timber interaction factor 0.3803
- Steel interaction factor 0.44899
- Applied shear 4.32 kN
- Shear capacity 9.8267 kN
- Bearing stress 0.432 N/mm²
- Permissible bearing 2.42 N/mm²

Bolts are 16 mm diameter at 600 mm centres (9 No total).

Edge distance in the steel to be a minimum of 32 mm.

The end connections are beyond the scope of this proforma.
Location: Flitched timber beam with udl & point loads

Fully restrained flitched beam to BS5268-2:2002 and BS 449:1990

Beam span \( L = 4.8 \) m
Design BM about xx (positive) \( M' = 5.184 \) kNm
Design shear force in y direction \( Fve' = 4.32 \) kN
Design axial load (+ve compress) \( F_a = -10.2 \) kN
Section comprises one steel web plate with a timber member on each side.
Depth of timber section \( d = 200 \) mm
Width of timber section \( b = 50 \) mm
Depth of steel section \( ds = 190 \) mm
Width of steel section \( bs = 10 \) mm
Length of bearing \( lb = 100 \) mm
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor \( K_4 = 1.0 \)
Bending parallel to the grain \( bparg = 7.5 \) N/mm²
Compress parallel to the grain \( cparg = 7.9 \) N/mm²
Compress perpendicular to the grain \( cperd = 2.1 \) N/mm²
Tension parallel to the grain \( tparg = 4.5 \) N/mm²
Shear parallel to the grain \( sparg = 0.82 \) N/mm²
Mean modulus of elasticity \( E_{mean} = 10000 \) N/mm²
Minimum modulus of elasticity \( E_{min} = 7000 \) N/mm²
Timber service class adopted \( t_{mclass} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Load-sharing modification factor \( K_8 = 1.1 \)

Section properties and Modular ratio

Total Inertia about xx axis in equivalent timber
Extreme fibre is timber section \( Y_c = Y_t = 100 \) mm
Dist of centroid to steel edge \( Y_n = Y_s = 95 \) mm
Z to top edge of timber \( Z_c = I_{xx}/Y_c = 2.3406E6 \) mm³
Duration of loading \( K_3 = 1 \)
Depth factor \( K_7 = (300/d)^{0.11} = 1.0456 \)
Tensile area of timber section \( T_{at} = 19000 \) mm²
Tensile area of steel section \( T_{as} = 1500 \) mm²
Width modification factor \( K_{14} = (300/h)^{0.11} = 1.0456 \)
Percent proportion of load in timber \( g_{tar} = 25 \% \)

Bolts between timber and steel plate along length of beam

Bolt diameter \( bd = 16 \) mm
Spacing of bolts along beam \( space = 600 \) mm
Bolts are in double shear. Width of timber for bolt stress calculation
is the section width \( b \) \( bb = 50 \) mm
Angle between grain and load \( \alpha_{l1}' = 90 \) degrees
Characteristic density timber 1 \( p_{k1} = 420 \) kg/m³
Basic loads in kN will be derived from formula given in Annex G.3
Load duration for bolted connection = Medium term.
Embedded strength for timber 1
Embedding strength

\[ fh_{od} = 0.050 \times (1 - 0.01 \times bd) \times pk1 \times Ka = 23.336 \]

Embedding strength (Timb 1)

\[ fh_{d1} = \frac{fh_{od}}{(K90 \times \sin(alp1)^2 + \cos(alp1)^2)} = 14.677 \]


Min value of equations G7 to G10  \( G_{min} = 7886 \)

Basic bolt shear capacity

\[ basic = \frac{1 \times G_{min}}{(F_d \times K_d)} = 5.6328 \text{ kN} \]

Mod factor for moisture content  \( K_{56} = 0.7 \)

Mod factor for number of bolts  \( K_{57} = 1 \)

Permissible bolt load in timber

\[ p_{bl} = 2 \times basic \times K_{46} \times K_{56} \times K_{57} = 7.886 \text{ kN} \]

Member: 200 mm x 110 mm overall

**DESIGN**

- Modular ratio 29.286

**SUMMARY**

- Timber bending stress 2.2148 N/mm²
- Timber perm. bending 8.6263 N/mm²
- Steel bending stress 61.62 N/mm²
- Steel perm. bending 180 N/mm²
- Timber tensile X sec area 19000 mm²
- Steel tensile X sec area 1500 mm²
- Timber moisture class 1
- Applied tension stress 0.1275 N/mm²
- Permiss tension stress 5.1758 N/mm²
- Timber interaction factor 0.28139
- Steel interaction factor 0.36602
- Applied shear 4.32 kN
- Shear capacity 12.027 kN
- Bearing stress 0.432 N/mm²
- Permissible bearing 2.31 N/mm²

Bolts are 16 mm diameter at 600 mm centres (9 No total).

Edge distance in the steel to be a minimum of 32 mm.

The end connections are beyond the scope of this proforma.
Location: Flitched timber beam with udl & point loads

Fully restrained flitched beam to BS5268-2:2002 and BS 449:1990

Beam span \( L = 6 \text{ m} \)
Section comprises one steel web plate with a timber member on each side.
Depth of timber section \( d = 250 \text{ mm} \)
Width of timber section \( b = 75 \text{ mm} \)
Depth of steel section \( ds = 225 \text{ mm} \)
Width of steel section \( bs = 12 \text{ mm} \)
Length of bearing \( lb = 100 \text{ mm} \)
From BS5268-2 Table 18, bearing is < 75 mm from joist end.

Bearing Modification factor \( K_4 = 1.0 \)

WARNING: Strength class not recognised. The scr ref no SHOULD BE 16 or 17.
Location: Flitched timber beam with udl & point loads

Fully restrained flitched beam to BS5268-2:2002 and BS 449:1990

Beam span \( L = 5.5 \text{ m} \)
Design BM about xx (positive) \( M' = 9.1816 \text{ kNm} \)
Design shear force in y direction \( Fve' = 6.6776 \text{ kN} \)
Design axial load (+ve compress) \( F_a = 0 \text{ kN} \)
Section comprises one steel web plate with a timber member on each side.

- Depth of timber section \( d = 225 \text{ mm} \)
- Width of timber section \( b = 50 \text{ mm} \)
- Depth of steel section \( ds = 175 \text{ mm} \)
- Width of steel section \( bs = 10 \text{ mm} \)
- Length of bearing \( lb = 50 \text{ mm} \)

From BS5268-2 Table 18, bearing is < 75 mm from joist end.

- Bearing Modification factor \( K_4 = 1.0 \)
- Strength class C16 to Table 8.
- Timber service class adopted \( tmclass = 2 \)
- Timber service class modification factor \( K_2 = 1 \) as Table 16.
- Load-sharing modification factor \( K_8 = 1.0 \)

Section properties and Modular ratio

- Total Inertia about xx axis in equivalent timber
  \( Y_c = Y_t = 112.5 \text{ mm} \)
- Extreme fibre is timber section \( Y_n = Y_s = 87.5 \text{ mm} \)
- Z to top edge of timber \( Z_c = I_{xx}/Y_c = 2.2469 \times 10^6 \text{ mm}^2 \)
- Duration of loading \( K_3 = 1.25 \)
- Depth factor \( K_7 = (300/d)^0.11 = 1.0322 \)

Bolts between timber and steel plate along length of beam

- Bolt diameter \( bd = 16 \text{ mm} \)
- Spacing of bolts along beam \( space = 450 \text{ mm} \)
- Bolts are in double shear. Width of timber for bolt stress calculation is the section width b \( bb = 50 \text{ mm} \)
- Angle between grain and load \( \alpha_1' = 90 \text{ degrees} \)
- Timber characteristic density \( pk_1 = 310 \text{ kg/m}^3 \)
- Basic loads in kN will be derived from formula given in Annex G.3
- Load duration for bolted connection = Medium term.
- Embedded strength for timber 1
  \( fhod = 0.050 \times (1 - 0.01 \times bd) \times pk_1 \times Ka = 17.224 \)
  \( Embedding strength (Timb 1) \)
  \( fhdl = fhod/((K90 \times \sin(\alpha_1')^2) + \cos(\alpha_1')^2) = 10.833 \)

Equations

- \( G_7 = 8666.2 \)
- \( G_8 = 8666.2 \)
- \( G_9 = 6588 \)
- \( G_{10} = 9081.1 \)
- \( \text{Min value of equations } G_7 \text{ to } G_{10} \)
  \( G_{\text{min}} = 6588 \)

Basic bolt shear capacity \( basic = 1 \times G_{\text{min}}/(Fd \times K_d) = 4.7057 \text{ kN} \)
- Mod factor for moisture content \( K_56 = 1 \)
- Mod factor for number of bolts \( K_57 = 0.7 \)
Permissible bolt load in timber \[ pbl = 2 \cdot \text{basic} \cdot K46 \cdot K56 \cdot K57 = 6.588 \, \text{kN} \]

Member: 225 mm x 110 mm overall

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Modular ratio</td>
<td>35.345</td>
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<table>
<thead>
<tr>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber bending stress</td>
</tr>
<tr>
<td>Timber perm. bending</td>
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<tr>
<td>Steel bending stress</td>
</tr>
<tr>
<td>Steel perm. bending</td>
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<tr>
<td>Timber moisture class</td>
</tr>
<tr>
<td>Applied shear</td>
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<tr>
<td>Shear capacity</td>
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<tr>
<td>Bearing stress</td>
</tr>
<tr>
<td>Permissible bearing</td>
</tr>
</tbody>
</table>

Bolts are 16 mm diameter at 450 mm centres (13 No total). Edge distance in the steel to be a minimum of 32 mm. The end connections are beyond the scope of this proforma.
Location: Flitched timber beam with udl & point loads

Fully Restrained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.

Simply supported flitched beam with general loadings:

Beam span $L=7.5$ m
Dist. from left support to start $L_{au(1)}=0$ m
Distance from left support to end $L_{bu(1)}=7.5$ m
Permanent load (unfactored) $G_{ku(1)}=2.25$ kN/m
Variable load 1 (unfactored) $Q_{u(1)}=0.5$ kN/m
Variable load 2 (unfactored) $Q_{ub(1)}=0.5$ kN/m

Characteristic values of actions

Permanent:
Reaction at left hand end $8.4375$ kN
Reaction at right hand end $8.4375$ kN
Maximum span moment $M_{k_p}=15.82$ kNm
Maximum shear force $F_{k_p}=8.4375$ kN
Variable 1:
Reaction at left hand end $1.875$ kN
Reaction at right hand end $1.875$ kN
Maximum span moment $M_{k_v1}=3.5156$ kNm
Maximum shear force $F_{k_v1}=1.875$ kN
Variable 2:
Reaction at left hand end $1.875$ kN
Reaction at right hand end $1.875$ kN
Maximum span moment $M_{k_v2}=3.5156$ kNm
Maximum shear force $F_{k_v2}=1.875$ kN

Design values of combined actions for ULS

Design moment $M_d=gamG*M_{k_p}+gamQ*M_{k_v2}=26.63$ kNm
Design shear force $V_d=gamG*F_{k_p}+gamQ*F_{k_v2}=14.203$ kN

Design values - combination 1

Design bending moment $M_{d'}=26.63$ kNm
Design shear force $V_{d'}=14.203$ kN
Load duration factor $k_{mod}=0.8$
Section comprises steel plates to top & bottom faces of timber.
Depth of timber section       h=300 mm
Width of timber section      b=100 mm
Depth of steel section       ds=12 mm
Width of steel section       bs=100 mm
Length of bearing            lb=150 mm
Timber strength class C22.

Section design values

Design bending moment         Md=Md'*10^6=26.63E6 Nmm
Design shear force            Vd=Vd'*10^3=14203 N
Design axial load (+ve compress) Nd=0 kN
Depth factor (Clause 3.2)      kh=1.0
Bearing modification factor   kc90=1.5
Load-sharing modification factor ksys=1.0
Shear reduction factor        ksh=0.67
Bolt diameter                 bd=20 mm
No of rows of bolts to be adopted NRC=1
Spacing of bolts along beam   space=250 mm

Timber characteristic density pk1=340 kg/m³
Angle between grain and load  alp1'=0°

Bolt shear capacities

Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.
Char. value of yield moment   MyRk=0.3*400*b*d^2.6=289640
Char. embedment strength     fhok=0.082*(1-0.01*bd)*pk1=22.304
Denominator (expression 8.31) denom=((K90*SIN(alp1')^2) +COS(alp1')^2)=1
Embedment strength (timber 1) fh1k=fhok/denom=22.304
Embedment strength (timber 2) fh2k=22.304
Factor                       beta=fh2k/fh1k=1
Expressions (a) to (e) for fasteners in single shear refer to section 8.2.3 in EC5. Zero contribution from rope effect will be considered.
Min value of equations (a)-(e) Gmin=22184
Basic bolt shear resistance   basic=Gmin/1000=22.184 kN
Design resistance per bolt    pbl=kmod/gamM2*basic=13.651 kN
Member: 324 mm x 100 mm overall
Modular ratio: 31.343

<table>
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<tr>
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<tbody>
<tr>
<td>Timber bending stress</td>
<td>1.9423 N/mm²</td>
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<td>Timber design bending strength</td>
<td>13.538 N/mm²</td>
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<td>Steel design bending stress</td>
<td>65.749 N/mm²</td>
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<td>Steel design bending strength</td>
<td>275 N/mm²</td>
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<th>SUMMARY</th>
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<tr>
<td>Final deflection</td>
<td>11.561 mm</td>
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<td>Limiting deflection</td>
<td>30 mm</td>
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<tr>
<td>Design shear force</td>
<td>14.203 kN</td>
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<td>Design shear resistance</td>
<td>31.335 kN</td>
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<tr>
<td>Design bearing stress</td>
<td>0.94687 N/mm²</td>
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<tr>
<td>Design bearing strength</td>
<td>2.2154 N/mm²</td>
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<td>10.131 kN</td>
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<tr>
<td>Design bolt resistance</td>
<td>13.651 kN</td>
</tr>
</tbody>
</table>

Bolts are 20 mm diameter in 1 row(s) at 250 mm centres.
NOTE: The above values relate to combination 1 and need to be compared with those of combination 2.
Location: Flitched timber beam with udl & point loads

Fully Restrained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.

Simply supported beam subject to vertical loads.

Beam span \( L = 4.8 \) m
Distance from left support to start \( L_{au(1)} = 0 \) m
Distance from left support to end \( L_{bu(1)} = 4.8 \) m
Permanent load (unfactored) \( G_{ku(1)} = 1.2 \) kN/m
Variable load 1 (unfactored) \( Q_{ua(1)} = 0.6 \) kN/m

Characteristic values of actions

Permanent:
Reaction at left hand end \( 2.88 \) kN
Reaction at right hand end \( 2.88 \) kN
Maximum span moment \( M_{kp} = 3.456 \) kNm
Maximum shear force \( F_{kp} = 2.88 \) kN
Variable 1:
Reaction at left hand end \( 1.44 \) kN
Reaction at right hand end \( 1.44 \) kN
Maximum span moment \( M_{kv1} = 1.728 \) kNm
Maximum shear force \( F_{kv1} = 1.44 \) kN

Design values of combined actions for ULS

Design bending moment \( M_d' = g_{amG}M_{kp} + g_{amQ}M_{kv1} = 7.2576 \) kNm
Design shear force \( V_d' = g_{amG}F_{kp} + g_{amQ}F_{kv1} = 6.048 \) kN

Section comprises one steel web plate with a timber member on each side.

Depth of timber section \( h = 200 \) mm
Width of timber section \( b = 50 \) mm
Depth of steel section \( d_s = 190 \) mm
Width of steel section \( b_s = 8 \) mm
Length of bearing \( l_b = 100 \) mm
Timber strength class C18.

Section design values

Design bending moment \( M_d = M_d'*10^6 = 7.2576E6 \) Nmm
Design shear force \( V_d = V_d'*10^3 = 6048 \) N
Design axial load (+ve compress) \( N_d = 0 \) kN
Depth factor (Clause 3.2) \( k_h = 1.0 \)
Bearing modification factor \( k_c90 = 1.5 \)
Load-sharing modification factor \( k_{sys} = 1.1 \)
Shear reduction factor \( k_{sh} = 0.67 \)
**Bolts between timber and steel plate along length of beam**

- **Bolt diameter**: \( bd = 16 \text{ mm} \)
- **Spacing of bolts along beam**: \( space = 500 \text{ mm} \)
- **Bolts are in double shear. Width of timber for bolt stress calculation is the section width** \( b = 50 \text{ mm} \)
- **Angle between grain and load**: \( \alpha_1' = 90\text{ degrees} \)
- **Timber characteristic density**: \( p_k = 320 \text{ kg/m}^3 \)
- **Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.**
- **Char. value of yield moment**: \( M_{yRk} = 0.3 \times 400 \times bd^2.6 = 162141 \)
- **Char. embedment strength**: \( f_{hk} = 0.082 \times (1 - 0.01 \times bd) \times p_k = 22.042 \)
- **Denominator (expression 8.31)**: \( \text{denom} = ((K90 \times \sin(\alpha_1')^2 + \cos(\alpha_1')^2) = 1.59 \)
- **Embedment strength (timber 1)**: \( f_{h1k} = f_{hk} / \text{denom} = 13.863 \)
- **Embedment strength (timber 2)**: \( f_{h2k} = 13.863 \)
- **Factor**: \( \beta = f_{h2k} / f_{h1k} = 1 \)

Expressions (f) to (m) for fasteners in double shear refer to section 8.2.3 in EC5. Zero contribution from rope effect will be considered.

<table>
<thead>
<tr>
<th>Equations</th>
<th>( f )</th>
<th>( g )</th>
<th>( h )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>11090</td>
<td>8654.1</td>
<td>13793</td>
</tr>
</tbody>
</table>

Min value of equations (f)-(h): \( G_{\text{min}} = 8654.1 \)

- **Basic bolt shear resistance**: \( \text{basic} = G_{\text{min}} / 1000 = 8.6541 \text{ kN} \)
- **Design bolt resistance in timber**: \( p_{bl} = 2 \times k_{mod} / \gamma_{M2} \times \text{basic} = 7.9884 \text{ kN} \)

**Member:** 200 mm x 108 mm overall

<table>
<thead>
<tr>
<th>Modular ratio</th>
<th>35</th>
</tr>
</thead>
</table>

**DESIGN**

- **Timber bending stress**: 3.2013 N/mm²
- **Steel design bending stress**: 106.44 N/mm²

**SUMMARY**

- **Timber design bending strength**: 9.1385 N/mm²
- **Steel design bending strength**: 275 N/mm²
- **Final deflection**: 10.672 mm
- **Limiting deflection**: 19.2 mm
- **Design shear force**: 6.048 kN
- **Design Shear resistance**: 15.42 kN
- **Design bearing stress**: 0.6048 N/mm²
- **Design bearing strength**: 1.6754 N/mm²

Bolts are 16 mm diameter at 500 mm c/c (10 No total).

Edge distance in the steel to be a minimum of 32 mm.

Bolts along the beam to be staggered above and below the centre line with their centres offset from the beam centre line to suit the min bolt edge spacing of 64 mm.

The minimum No of bolts required at the bearing = 2
Location: Flitched timber beam with udl & point loads

Fully Restrained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.

---

Beam span \( L = 8 \) m
Distance from left support \( L_c(1) = 2 \) m
Permanent load (unfactored) \( G_k_c(1) = 6 \) kN
Variable load 1 (unfactored) \( Q_c_a(1) = 4 \) kN
Dist. from left support to start \( L_{au}(1) = 0 \) m
Distance from left support to end \( L_{bu}(1) = 8 \) m
Permanent load (unfactored) \( G_k_u(1) = 4 \) kN/m
Variable load 1 (unfactored) \( Q_u_a(1) = 2 \) kN/m

**Characteristic values of actions**

Permanent:
Reaction at left hand end \( 20.5 \) kN
Reaction at right hand end \( 17.5 \) kN
Maximum span moment \( M_{kp} = 38.28 \) kNm
Maximum shear force \( F_{kp} = 20.5 \) kN

Variable 1:
Reaction at left hand end \( 11 \) kN
Reaction at right hand end \( 9 \) kN
Maximum span moment \( M_{kv1} = 20.24 \) kNm
Maximum shear force \( F_{kv1} = 11 \) kN

Design values of combined actions for ULS

Design bending moment \( \frac{M_d' = \gamma_g G * M_{kp} + \gamma_q Q * M_{kv1}}{10^6} = 82.038 \) kNm
Design shear force \( \frac{V_d' = \gamma_g G * F_{kp} + \gamma_q Q * F_{kv1}}{10^3} = 44175 \) kN

Section comprises steel plates to each side face of timber section.
Depth of timber section \( h = 400 \) mm
Width of timber section \( b = 100 \) mm
Depth of steel section \( d_s = 375 \) mm
Width of steel section \( b_s = 12 \) mm
Length of bearing \( l_b = 100 \) mm
Timber strength class C18.

**Section design values**

Design bending moment \( M_d = \frac{M_d'}{10^6} = 82.038E6 \) Nmm
Design shear force \( V_d = \frac{V_d'}{10^3} = 44175 \) N
Design axial load (+ve compress) \( N_d = 0 \) kN
Depth factor (Clause 3.2) \( \gamma_h = 1.0 \)
Bearing modification factor \( k_{c90} = 1.5 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)
Sample output for SCALE Proforma 256. (ans=3) Page: 2
Timber design to BS5268 and Eurocode 5 Made by: IFB
Simply supported flitched beam with general loading Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd. Ref No: SC256 EC

Shear reduction factor $k_{sh} = 0.67$
Effective length about $zz$ $L_e = 6.4$ m

**Bolts between timber and steel plate along length of beam**

- Bolt diameter $b_d = 20$ mm
- Spacing of bolts along beam $space = 500$ mm
- Bolts are in double shear. Width of timber for bolt stress calculation is the section width $b$ $b_b = 100$ mm
- Angle between grain and load $\alpha_{l1}' = 90$ degrees
- Timber characteristic density $p_{k1} = 320$ kg/m³
- Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.

Char. value of yield moment $M_yR_k = 0.3 \times 400 \times b_d^2.6 = 289640$
Char. embedment strength $f_{hok} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k1} = 20.992$
Denominator (expression 8.31) $\text{denom} = ((K90 \times \sin(\alpha_{l1})^2) + \cos(\alpha_{l1})^2) = 1.65$
Embedment strength (timber 1) $f_{h1k} = f_{hok} / \text{denom} = 12.722$
Embedment strength (timber 2) $f_{h2k} = 12.722$
Factor $\beta = f_{h2k} / f_{h1k} = 1$

Expressions (j) to (k) for fasteners in double shear refer to section 8.2.3 in EC5. Zero contribution from rope effect will be considered.

Equations

- **j** 12722
- **k** 13962
- **1** 12722
- **m** 19745

Min value of equations (j)-(k) $G_{min} = 12722$

Basic bolt shear resistance $basic = G_{min} / 1000 = 12.722$ kN
Design bolt resistance in timber $p_{bl} = 2 \times k_{mod} / \gamma_{M2} \times basic = 15.658$ kN

**Member: 400 mm x 124 mm overall**
- **Modular ratio** 35
- **DESIGN**
  - Timber bending stress $3.8837$ N/mm²
  - Steel design bending stress $127.43$ N/mm²
- **SUMMARY**
  - Timber design bending strength $9.0365$ N/mm²
  - Steel design bending strength $275$ N/mm²
  - Final deflection $22.235$ mm
  - Limiting deflection $32$ mm
  - Design shear force $44.175$ kN
  - Design shear stress $1466.3$ kN
  - Design bearing stress $18.406$ N/mm²
  - Design bearing strength $265$ N/mm²
  - Plt buckling @ supp. $6.4022$ N/mm²
  - Design buckling strength $117.89$ N/mm²

Bolts are 20 mm diameter at 500 mm c/c (17 No total).
Edge distance in the steel to be a minimum of 40 mm.
Bolts along the beam to be staggered above and below the centre line with their centres offset from the beam centre line to suit the min bolt edge spacing of 80 mm.
The minimum No of bolts required at the bearing = 3
The steel plates are required to extend over the support and act together with the timber to resist the design bearing stress.
Location: Flitched timber beam with udl & point loads

Fully Restrained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.

Simply supported flitched beam with general loading

---

Beam span \( L = 6 \text{ m} \)
Distance from left support \( Lc(1) = 3 \text{ m} \)
Permanent load (unfactored) \( Gkc(1) = 15.75 \text{ kN} \)
Variable load 1 (unfactored) \( Qca(1) = 78.438 \text{ kN} \)
Dist. from left support to start \( Lau(1) = 0 \text{ m} \)
Distance from left support to end \( Lbu(1) = 6 \text{ m} \)
Permanent load (unfactored) \( Gku(1) = 0.96 \text{ kN/m} \)
Variable load 1 (unfactored) \( Qua(1) = 0 \text{ kN/m} \)

Characteristic values of actions

Permanent:
Reaction at left hand end \( 10.755 \text{ kN} \)
Reaction at right hand end \( 10.755 \text{ kN} \)
Maximum span moment \( Mkp = 27.945 \text{ kNm} \)
Maximum shear force \( Fkp = 10.755 \text{ kN} \)
Variable 1:
Reaction at left hand end \( 39.219 \text{ kN} \)
Reaction at right hand end \( 39.219 \text{ kN} \)
Maximum span moment \( Mkvl = 117.66 \text{ kNm} \)
Maximum shear force \( Fkv1 = 39.219 \text{ kN} \)

Design values of combined actions for ULS

Design bending moment \( Md' = \text{gamG} \cdot Mkp + \text{gamQ} \cdot Mkvl = 214.22 \text{ kNm} \)
Design shear force \( Vd' = \text{gamG} \cdot Fkp + \text{gamQ} \cdot Fkv1 = 73.348 \text{ kN} \)
Section comprises two steel web plates and 3 timber members.
Depth of timber section \( h = 400 \text{ mm} \)
Width of timber section \( b = 125 \text{ mm} \)
Depth of steel section \( ds = 375 \text{ mm} \)
Width of steel section \( bs = 16 \text{ mm} \)
Length of bearing \( lb = 150 \text{ mm} \)
Timber strength class C22.

Section design values

Design bending moment \( Md = Md' \times 10^6 = 214.22 \text{E6 Nmm} \)
Design shear force \( Vd = Vd' \times 10^3 = 73348 \text{ N} \)
Design axial load (+ve compress) \( Nd = 0 \text{ kN} \)
Depth factor (Clause 3.2) \( kh = 1.0 \)
Bearing modification factor \( kc90 = 1.5 \)
Load-sharing modification factor \( ksys = 1.0 \)
Shear reduction factor \[ k_{sh} = 0.67 \]

**Bolts between timber and steel plate along length of beam**

- **Bolt diameter**: \( bd = 16 \text{ mm} \)
- **Spacing of bolts along beam**: \( space = 600 \text{ mm} \)
- **Bolts are in double shear. Width of timber for bolt stress calculation**
  - **is the section width**: \( bb = 125 \text{ mm} \)
- **Angle between grain and load**: \( \alpha_{1}' = 90 \text{ degrees} \)
- **Timber characteristic density**: \( p_k = 340 \text{ kg/m}^3 \)
- **Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.**

**Char. value of yield moment**
\[ My_{Rk} = 0.3 \times 400 \times bd^{2.6} = 162141 \]

**Char. embedment strength**
\[ fh_{ok} = 0.082 \times (1 - 0.01 \times bd) \times p_k = 23.419 \]

**Denominator (expression 8.31)**
\[ denom = ((K90 \times \sin(\alpha_{1})^2) + \cos(\alpha_{1})^2) = 1.59 \]

**Embedment strength (timber 1)**
\[ fh_{1k} = fh_{ok} / denom = 14.729 \]

**Embedment strength (timber 2)**
\[ fh_{2k} = 14.729 \]

**Factor**
\[ \beta = fh_{2k} / fh_{1k} = 1 \]

**Expressions (a) to (e) for fasteners in single shear refer to section 8.2.3 in EC5. Zero contribution from rope effect will be considered.**

**Min value of equations (a)–(e)**
\[ G_{min} = 13998 \]

**Basic bolt shear resistance**
\[ basic = G_{min} / 1000 = 13.998 \text{ kN} \]

**Design bolt resistance in timber**
\[ p_{bl} = 4 \times k_{mod} / \gamma_{M2} \times basic = 34.456 \text{ kN} \]

**Member: 400 mm x 407 mm overall**

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Timber bending stress</th>
<th>6.863 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Timber design bending strength</td>
<td>13.538 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Steel design bending stress</td>
<td>196.47 N/mm²</td>
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<tr>
<td></td>
<td>Steel design bending strength</td>
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<tr>
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<td>Final deflection</td>
<td>17.62 mm</td>
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<tr>
<td></td>
<td>Limiting deflection</td>
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<tr>
<td></td>
<td>Design shear force</td>
<td>73.348 kN</td>
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<td>Design Shear resistance</td>
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<td>Design bearing stress</td>
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<tr>
<td></td>
<td>Design bearing strength</td>
<td>2.2154 N/mm²</td>
</tr>
</tbody>
</table>

Bolts are 16 mm diameter at 600 mm c/c (11 No total).

Edge distance in the steel to be a minimum of 32 mm.

Bolts along the beam to be staggered above and below the centre line with their centres offset from the beam centre line to suit the min bolt edge spacing of 64 mm.

The minimum No of bolts required at the bearing = 2
Location: Flitched timber beam with udl & point loads

Fully Restained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.

Beam span L=7.5 m
Design BM about yy (positive) Md'=32.45 kNm
Design shear force in z direction Vd'=17.307 kN
Design axial load (+ve compress) Nd=1.704 kN
Section comprises steel plates to top & bottom faces of timber.
Depth of timber section h=300 mm
Width of timber section b=100 mm
Depth of steel section ds=12 mm
Width of steel section bs=100 mm
Length of bearing lb=150 mm
Timber strength class C22.

Section design values

Design bending moment Md=Md'*10^6=32.45E6 Nmm
Design shear force Vd=Vd'*10^3=17307 N
Design axial load (+ve compress) Nd=1.704 kN
Depth factor (Clause 3.2) kh=1.0
Bearing modification factor kc90=1.5
Load-sharing modification factor ksys=1.0
Shear reduction factor ksh=0.67
Effective length about zz Le=6 m
Eff.length for bending about yy ley'=4 m
Eff.length for bending about zz lez'=4 m
Instability factor kcy=1/(ky+(ky^2-rsry^2)^0.5)=0.95085
Instability factor kcz=1/(kz+(kz^2-rsrz^2)^0.5)=0.15852
Percent proportion of load in timber gtar=28 %
Bolt diameter bd=20 mm
No of rows of bolts to be adopted NRC=1
Spacing of bolts along beam space=250 mm

Timber characteristic density pkl=340 kg/m³
Angle between grain and load alp1'=0°

Bolt shear capacities

Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.
Char. value of yield moment MyRk=0.3*400*bd^2.6=289640
Char. embedment strength fhok=0.082*(1-0.01*bd)*pkl=22.304
Denominator (expression 8.31) \[ \text{denom} = \frac{(K90 \cdot \sin(alp1)^2) + \cos(alp1)^2}{1} \]
Embedment strength (timber 1) \[ \text{fh1k} = \frac{fho}{\text{denom}} = 22.304 \]
Embedment strength (timber 2) \[ \text{fh2k} = 22.304 \]
Factor \[ \beta = \frac{\text{fh2k}}{\text{fh1k}} = 1 \]
Min value of equations (a)-(e) \[ G_{\text{min}} = 22184 \]
Basic bolt shear resistance \[ \text{basic} = \frac{G_{\text{min}}}{1000} = 22.184 \text{ kN} \]
Design resistance per bolt \[ p_{bl} = \frac{k_{mod}}{\gamma_{M2}} \cdot \text{basic} = 13.651 \text{ kN} \]

<table>
<thead>
<tr>
<th>Member: 324 mm x 100 mm overall</th>
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</thead>
<tbody>
<tr>
<td>Modular ratio</td>
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<tr>
<td>DESIGN</td>
<td></td>
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<tr>
<td>Timber bending stress</td>
<td>2.3668 N/mm²</td>
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<tr>
<td>SUMMARY</td>
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<td>Design compressive stress</td>
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<td>2.2154 N/mm²</td>
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<tr>
<td>Design load per bolt</td>
<td>12.344 kN</td>
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<tr>
<td>Design bolt resistance</td>
<td>13.651 kN</td>
</tr>
</tbody>
</table>

Bolts are 20 mm diameter in 1 row(s) at 250 mm centres.
The end connections are beyond the scope of this proforma.
Location: Flitched timber beam with udl & point loads

Fully Restrained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.

Beam span \( L = 4.8 \, \text{m} \)

Design BM about yy (positive) \( \text{Md}' = 7.3613 \, \text{kNm} \)

Design shear force in z direction \( \text{Vd}' = 6.1344 \, \text{kN} \)

Design axial load (+ve compress) \( \text{Nd} = -14.484 \, \text{kN} \)

Section comprises one steel web plate with a timber member on each side.

Depth of timber section \( h = 200 \, \text{mm} \)

Width of timber section \( b = 50 \, \text{mm} \)

Depth of steel section \( ds = 190 \, \text{mm} \)

Width of steel section \( bs = 8 \, \text{mm} \)

Length of bearing \( lb = 100 \, \text{mm} \)

Timber strength class C18.

Section design values

Design bending moment \( \text{Md} = \text{Md}' \times 10^6 = 7.3613 \times 10^6 \, \text{Nm} \)

Design shear force \( \text{Vd} = \text{Vd}' \times 10^3 = 6134.4 \, \text{N} \)

Design axial load (+ve compress) \( \text{Nd} = -14.484 \, \text{kN} \)

Depth factor (Clause 3.2) \( kh = 1.0 \)

Bearing modification factor \( kc90 = 1.5 \)

Load-sharing modification factor \( ksys = 1.1 \)

Shear reduction factor \( ksh = 0.67 \)

Tensile area of timber section \( Tat = 19000 \, \text{mm}^2 \)

Tensile area of steel section \( Tas = 1500 \, \text{mm}^2 \)

Width modification factor \( kw = 1.0 \)

Percent proportion of load in timber \( gtar = 25\% \)

Bolts between timber and steel plate along length of beam

Bolt diameter \( bd = 16 \, \text{mm} \)

Spacing of bolts along beam \( \text{space} = 600 \, \text{mm} \)

Bolts are in double shear. Width of timber for bolt stress calculation is the section width \( b = 50 \, \text{mm} \)

Angle between grain and load \( alp1' = 90 \, \text{degrees} \)

Timber characteristic density \( pk1 = 320 \, \text{kg/m}^3 \)

Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.

Char. value of yield moment \( \text{MyRk} = 0.3 \times 400 \times bd^2.6 = 162141 \)

Char. embedment strength \( fhok = 0.082 \times (1-0.01 \times bd) \times pk1 = 22.042 \)

Denominator (expression 8.31) \( \text{denom} = ((K90 \times \text{SIN}(alp1')^2) + \text{COS}(alp1')^2) = 1.59 \)

Embedment strength (timber 1) \( fh1k = fhok / \text{denom} = 13.863 \)

Embedment strength (timber 2) \( fh2k = 13.863 \)

Factor \( beta = fh2k / fh1k = 1 \)

Expressions (f) to (m) for fasteners in double shear refer to section 8.2.3 in EC5. Zero contribution from rope effect will be considered.

Equations

\[
\begin{array}{ccc}
 f & g & h \\
 11090 & 8654.1 & 13793 \\
\end{array}
\]

Min value of equations \((f)-(h)\) \( Gmin = 8654.1 \)
Basic bolt shear resistance  \( \text{basic}=\frac{\text{Gmin}}{1000}=8.6541 \text{ kN} \)
Design bolt resistance in timber  \( p_\text{bl}=2*\text{kmod}/\text{gamM2} * \text{basic}=7.9884 \text{ kN} \)

<table>
<thead>
<tr>
<th>Member: 200 mm x 108 mm overall</th>
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<tr>
<td>Modular ratio</td>
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<td>Timber bending stress</td>
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<tbody>
<tr>
<td>Timber design bending strength</td>
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<tr>
<td>Design Shear resistance</td>
</tr>
<tr>
<td>Design bearing stress</td>
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<tr>
<td>Design bearing strength</td>
</tr>
</tbody>
</table>

Bolts are 16 mm diameter at 600 mm c/c (9 No total).
Edge distance in the steel to be a minimum of 32 mm.
The end connections are beyond the scope of this proforma.
Location: Flitched timber beam with udl & point loads

Fully Restrained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.

Beam span \( L = 4.8 \text{ m} \)

Design BM about yy (positive) \( M_d' = 7.3613 \text{ kNm} \)

Design shear force in z direction \( V_d' = 6.1344 \text{ kN} \)

Design axial load (+ve compress) \( N_d = -14.484 \text{ kN} \)

Section comprises one steel web plate with a timber member on each side.

- Depth of timber section \( h = 200 \text{ mm} \)
- Width of timber section \( b = 50 \text{ mm} \)
- Depth of steel section \( d_s = 190 \text{ mm} \)
- Width of steel section \( b_s = 10 \text{ mm} \)
- Length of bearing \( l_b = 100 \text{ mm} \)

Bending parallel to the grain \( f_{mk} = 7.5 \text{ N/mm}^2 \)

Compress parallel to the grain \( f_{c0k} = 7.9 \text{ N/mm}^2 \)

Compress perp to the grain \( f_{c90k} = 2.1 \text{ N/mm}^2 \)

Tension parallel to the grain \( f_{t0k} = 4.5 \text{ N/mm}^2 \)

Shear parallel to the grain \( f_{vk} = 1 \text{ N/mm}^2 \)

Mean modulus of elasticity \( E_{0\text{mean}} = 10500 \text{ N/mm}^2 \)

5th percentile MOE \( E_{005} = 7000 \text{ N/mm}^2 \)

Section design values

Design bending moment \( M_d = M_d' \times 10^6 = 7.3613E6 \text{ Nmm} \)

Design shear force \( V_d = V_d' \times 10^3 = 6134.4 \text{ N} \)

Design axial load (+ve compress) \( N_d = -14.484 \text{ kN} \)

Depth factor (Clause 3.2) \( k_h = 1.0 \)

Bearing modification factor \( k_{c90} = 1.5 \)

Load-sharing modification factor \( k_{sys} = 1.1 \)

Shear reduction factor \( k_{sh} = 0.67 \)

Tensile area of timber section \( T_a = 19000 \text{ mm}^2 \)

Tensile area of steel section \( T_s = 1500 \text{ mm}^2 \)

Width modification factor \( k_w = (150/lardim)^{0.2} = 0.94409 \)

Percent proportion of load in timber \( g_tar = 25 \% \)

Bolts between timber and steel plate along length of beam

Bolt diameter \( b_d = 16 \text{ mm} \)

Spacing of bolts along beam \( s = 600 \text{ mm} \)

Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.

Char. value of yield moment \( M_{yRk} = 0.3 \times 400 \times b_d^2 \times 2.6 = 162141 \)

Char. embedment strength \( f_{hoK} = 0.082 \times (1 - 0.01 \times b_d) \times pk_1 = 28.93 \)

Denominator (expression 8.31) \( \text{denom} = ((K_{90} \times \text{SIN}(alp_1)^2) + \text{COS}(alp_1)^2) = 1.59 \)
Embedment strength (timber 1) \( fh_{1k} = \frac{fh_{ok}}{denom} = \frac{18.195}{18.195} \)
Embedment strength (timber 2) \( fh_{2k} = 18.195 \)
Factor \( \beta = \frac{fh_{2k}}{fh_{1k}} = 1 \)

Expressions (f) to (m) for fasteners in double shear refer to section 8.2.3 in EC5. Zero contribution from rope effect will be considered.

Equations:

\[
\begin{align*}
  f & = 14556 \\
  g & = 10194 \\
  h & = 15802 \\
\end{align*}
\]

Min value of equations (f)-(h) \( G_{min} = 10194 \)

Basic bolt shear resistance \( basic = \frac{G_{min}}{1000} = 10.194 \text{ kN} \)
Design bolt resistance in timber \( p_{bl} = 2 \times k_{mod} / \gamma_{M2} \times basic = 9.4098 \text{ kN} \)

<table>
<thead>
<tr>
<th>Member: 200 mm x 110 mm overall</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESIGN</td>
</tr>
<tr>
<td>Timber bending stress</td>
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<td>Timber design bending strength</td>
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<tr>
<td>Steel design bending stress</td>
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<td>Steel design bending strength</td>
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<tr>
<td>Timber tensile cross sec area</td>
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<td>Steel tensile cross sec area</td>
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<td>Design shear force</td>
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<td>Design Shear resistance</td>
</tr>
<tr>
<td>Design bearing stress</td>
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<tr>
<td>Design bearing strength</td>
</tr>
</tbody>
</table>

Bolts are 16 mm diameter at 600 mm c/c (9 No total).

Edge distance in the steel to be a minimum of 32 mm.

The end connections are beyond the scope of this proforma.
Location: Flitched timber beam with udl & point loads

Fully Restrained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.

Simply supported flitched beam with general loading

Beam span $L=6\,m$
Dist. from left support to start $Lau(1)=0\,m$
Distance from left support to end $Lbu(1)=4.8\,m$
Permanent load (unfactored) $Gku(1)=1.2\,kN/m$
Variable load 1 (unfactored) $Qua(1)=0.6\,kN/m$

Characteristic values of actions

Permanent:
- Reaction at left hand end $3.456\,kN$
- Reaction at right hand end $2.304\,kN$
- Maximum span moment $Mkp=4.968\,kNm$
- Maximum shear force $Fkp=3.456\,kN$

Variable 1:
- Reaction at left hand end $1.728\,kN$
- Reaction at right hand end $1.152\,kN$
- Maximum span moment $Mkv1=2.484\,kNm$
- Maximum shear force $Fkv1=1.728\,kN$

Design values of combined actions for ULS

Design bending moment $Md'=\gamma_g G\times Mkp+\gamma_q Q\times Mkv1=10.433\,kNm$
Design shear force $Vd'=\gamma_g G\times Fkp+\gamma_q Q\times Fkv1=7.2576\,kN$

Section comprises one steel web plate with a timber member on each side.

- Depth of timber section $h=250\,mm$
- Width of timber section $b=75\,mm$
- Depth of steel section $ds=225\,mm$
- Width of steel section $bs=12\,mm$
- Length of bearing $lb=100\,mm$

Timber strength class D40.

Section design values

Design bending moment $Md=Md'^{10^6}=10.433E6\,Nmm$
Design shear force $Vd=Vd'^{10^3}=7257.6\,N$
Design axial load (+ve compress) $Nd=0\,kN$
Depth factor (Clause 3.2) $kh=1.0$
Bearing modification factor $kc90=1.0$
Load-sharing modification factor $ksys=1.0$
Shear reduction factor $ksh=0.67$
Bolts between timber and steel plate along length of beam

Bolt diameter $bd = 16 \text{ mm}$
Spacing of bolts along beam $\text{space} = 600 \text{ mm}$
Bolts are in double shear. Width of timber for bolt stress calculation is the section width $b_b = 75 \text{ mm}$
Angle between grain and load $\alpha' = 90 \text{ degrees}$
Timber characteristic density $p_k1 = 550 \text{ kg/m}^3$
Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.
Char. value of yield moment $M_{yRk} = 0.3 \times 400 \times bd^2.6 = 162141$
Char. embedment strength $f_{hok} = 0.082 \times (1 - 0.01 \times bd) \times p_k1 = 37.884$
Denominator (expression 8.31) $\text{denom} = ((K90 \times \sin(\alpha')^2) + \cos(\alpha')^2) = 1.14$
Embedment strength (timber 1) $f_{h1k} = f_{hok}/\text{denom} = 33.232$
Embedment strength (timber 2) $f_{h2k} = 33.232$
Factor $\beta = f_{h2k}/f_{h1k} = 1$
Equations $f = 39878$, $g = 19497$, $h = 21356$
Min value of equations $(f)-(h)$ $G_{\text{min}} = 19497$

Basic bolt shear resistance $\text{basic} = G_{\text{min}}/1000 = 19.497 \text{ kN}$
Design bolt resistance in timber $p_{bl} = 2 \times k_{mod}/\gamma_{M2} \times \text{basic} = 17.997 \text{ kN}$

Member: 250 mm x 162 mm overall
Modular ratio 19.266

DESIGN
- Timber bending stress 3.1442 N/mm²
- Steel design bending stress 54.519 N/mm²

SUMMARY
- Timber design bending strength 18.462 N/mm²
- Steel design bending strength 275 N/mm²
- Final deflection 8.3954 mm
- Limiting deflection 24 mm
- Design shear force 7.2576 kN
- Design shear resistance 32.469 kN
- Design bearing stress 0.48384 N/mm²
- Design bearing strength 2.5385 N/mm²

Bolts are 16 mm diameter at 600 mm c/c (11 No total).
Edge distance in the steel to be a minimum of 32 mm.
Bolts along the beam to be staggered above and below the centre line with their centres offset from the beam centre line to suit the min bolt edge spacing of 64 mm.
The minimum No of bolts required at the bearing = 2
Location: Flitched timber beam with udl & point loads

Fully Restrained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.
Beam span \( L = 5.5 \text{ m} \)
Design BM about yy (positive) \( M_d' = 13.038 \text{ kNm} \)
Design shear force in z direction \( V_d' = 9.4822 \text{ kN} \)
Design axial load (+ve compress) \( N_d = 0 \text{ kN} \)
Section comprises one steel web plate with a timber member on each side.
Depth of timber section \( h = 225 \text{ mm} \)
Width of timber section \( b = 50 \text{ mm} \)
Depth of steel section \( d_s = 175 \text{ mm} \)
Width of steel section \( b_s = 10 \text{ mm} \)
Length of bearing \( l_b = 50 \text{ mm} \)
Timber strength class C16.

Section design values

Design bending moment \( M_d = M_d' \times 10^6 = 13.038E6 \text{ Nmm} \)
Design shear force \( V_d = V_d' \times 10^3 = 9482.2 \text{ N} \)
Design axial load (+ve compress) \( N_d = 0 \text{ kN} \)
Depth factor (Clause 3.2) \( k_h = 1.0 \)
Bearing modification factor \( k_c90 = 1.5 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)
Shear reduction factor \( k_{sh} = 0.67 \)

Bolts between timber and steel plate along length of beam

Bolt diameter \( b_d = 16 \text{ mm} \)
Spacing of bolts along beam \( \text{space} = 450 \text{ mm} \)
Bolts are in double shear. Width of timber for bolt stress calculation is the section width \( b = 50 \text{ mm} \)
Angle between grain and load \( \alpha_{l1} = 90 \text{ degrees} \)
Timber characteristic density \( p_k1 = 310 \text{ kg/m}^3 \)
Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.
Char. value of yield moment \( M_yRk = 0.3 \times 400 \times b_d^2 \times 2.6 = 162141 \)
Char. embedment strength \( f_h0k = 0.082 \times (1 - 0.01 \times b_d) \times p_k1 = 21.353 \)
Denominator (expression 8.31) \( \text{denom} = ((K90 \times \sin(\alpha_{l1})^2) + \cos(\alpha_{l1})^2) = 1.59 \)
Embedment strength (timber 1) \( f_h1k = f_h0k / \text{denom} = 13.429 \)
Embedment strength (timber 2) \( f_h2k = 13.429 \)
Factor \( \beta = f_h2k / f_h1k = 1 \)
Equations \( f \quad g \quad h \)
\[
\begin{array}{ccc}
10744 & 8497.2 & 13576 \\
\end{array}
\]
Min value of equations \( (f)-(h) \) \( G_{min} = 8497.2 \)
Basic bolt shear resistance \( \text{basic} = G_{min} / 1000 = 8.4972 \text{ kN} \)
Design bolt resistance in timber  

\[ p_{bl} = 2 \times k_{mod} / \gamma_{M2} \times basic = 10.458 \text{ kN} \]

<table>
<thead>
<tr>
<th>Member: 225 mm x 110 mm overall</th>
</tr>
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<tbody>
<tr>
<td>Modular ratio: 38.889</td>
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**DESIGN**

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<tr>
<th>Timber bending stress</th>
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**SUMMARY**

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<tr>
<th>Timber design bending strength</th>
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<tr>
<td>Steel design bending stress</td>
<td>165.17 N/mm²</td>
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<td>275 N/mm²</td>
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<tr>
<td>Timber moisture class</td>
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<tr>
<td>Design shear force</td>
<td>9.4822 kN</td>
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<tr>
<td>Design shear resistance</td>
<td>19.791 kN</td>
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<td>Design bearing stress</td>
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<td>2.0308 N/mm²</td>
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</table>

Bolts are 16 mm diameter at 450 mm c/c (13 No total).
Edge distance in the steel to be a minimum of 32 mm.
The end connections are beyond the scope of this proforma.
Location: Flitched timber beam with udl & point loads

Fully Restrained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.
Beam span \( L = 5.5 \text{ m} \)
Design BM about \( yy \) (positive) \( Md' = 13.038 \text{ kNm} \)
Design shear force in \( z \) direction \( Vd' = 17.04 \text{ kN} \)
Design axial load (+ve compress) \( Nd = 7.1 \text{ kN} \)
Section comprises one steel web plate with a timber member on each side.
Depth of timber section \( h = 225 \text{ mm} \)
Width of timber section \( b = 50 \text{ mm} \)
Depth of steel section \( ds = 175 \text{ mm} \)
Width of steel section \( bs = 10 \text{ mm} \)
Length of bearing \( lb = 50 \text{ mm} \)
Timber strength class C16.

Section design values

Design bending moment \( Md = Md' \times 10^6 = 13.038 \text{E6 Nmm} \)
Design shear force \( Vd = Vd' \times 10^3 = 17040 \text{ N} \)
Design axial load (+ve compress) \( Nd = 7.1 \text{ kN} \)
Depth factor (Clause 3.2) \( kh = 1.0 \)
Bearing modification factor \( kc90 = 1.5 \)
Load-sharing modification factor \( ksys = 1.0 \)
Shear reduction factor \( ksh = 0.67 \)
Eff.length for bending about \( yy \) \( ley' = 5.5 \text{ m} \)
Eff.length for bending about \( zz \) \( lez' = 1.5 \text{ m} \)
Instability factor \( kcy = 1/(kc+(kc^2-rsry^2)^0.5) = 0.27274 \)
Instability factor \( kcz = 1/(kz+(kz^2-rsrz^2)^0.5) = 0.34002 \)
Percent proportion of load in timber \( gtar = 32 \% \)

Bolts between timber and steel plate along length of beam

Bolt diameter \( bd = 16 \text{ mm} \)
Spacing of bolts along beam \( space = 450 \text{ mm} \)
Bolts are in double shear. Width of timber for bolt stress calculation is the section width \( b = 50 \text{ mm} \)
Angle between grain and load \( alp1' = 90 \text{ degrees} \)
Timber characteristic density \( pk1 = 310 \text{ kg/m}^3 \)
Basic loads in \( \text{ kN} \) will be derived from the expressions in Section 8 of Eurocode 5.
Char. value of yield moment \( MyRk = 0.3 \times 400 \times bd^2 \times 2.6 = 162141 \)
Char. embedment strength \( fhok = 0.082 \times (1-0.01 \times bd) \times pk1 = 21.353 \)
Denominator (expression 8.31) \( denom = ((K90 \times SIN(alp1)^2) + COS(alp1)^2) = 1.59 \)
Embedment strength (timber 1) \( fh1k = fhok / denom = 13.429 \)
Embedment strength (timber 2) \( fh2k = 13.429 \)
Factor \( beta = fh2k / fh1k = 1 \)
Equations \( f \times g - h = 10744 \times 8497.2 - 13576 \)
Min value of equations \( f-(h) = Gmin = 8497.2 \)
Basic bolt shear resistance \( basic = Gmin / 1000 = 8.4972 \text{ kN} \)
Design bolt resistance in timber  \( pbl = 2 \times kmod / \gamma M2 \times \text{basic} = 10.458 \text{ kN} \)

<table>
<thead>
<tr>
<th>Member: 225 mm x 110 mm overall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modular ratio</td>
</tr>
<tr>
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<tr>
<td>Timber bending stress</td>
</tr>
<tr>
<td>SUMMARY</td>
</tr>
<tr>
<td>Timber design bending strength</td>
</tr>
<tr>
<td>Steel design bending stress</td>
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<tr>
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</tr>
<tr>
<td>Design bearing stress</td>
</tr>
<tr>
<td>Design bearing strength</td>
</tr>
<tr>
<td>Plt buckling @ supp.</td>
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<tr>
<td>Design buckling strength</td>
</tr>
</tbody>
</table>

Bolts are 16 mm diameter at 450 mm c/c (13 No total).
Edge distance in the steel to be a minimum of 32 mm.
The end connections are beyond the scope of this proforma.
Location: Flitched timber beam with udl & point loads

Fully Restrained flitched beam to EC5 and EC3

Calculations include EC5 National Annex Amendment No.2.

Simply supported flitched beam with general loading

Beam span L= 4.8 m
Dist. from left support to start Lau(1)= 0 m
Dist. from left support to end Lbu(1)= 4.8 m
Permanent load (unfactored) Gku(1)= 1.2 kN/m
Variable load 1 (unfactored) Qua(1)= 0.6 kN/m
Variable load 2 (unfactored) Qub(1)= 0.5 kN/m

Characteristic values of actions

Permanent:
Reaction at left hand end 2.88 kN
Reaction at right hand end 2.88 kN
Maximum span moment Mkp= 3.456 kNm
Maximum shear force Fkp= 2.88 kN
Variable 1:
Reaction at left hand end 1.44 kN
Reaction at right hand end 1.44 kN
Maximum span moment Mkvp= 1.728 kNm
Maximum shear force Fkv1= 1.44 kN
Variable 2:
Reaction at left hand end 1.2 kN
Reaction at right hand end 1.2 kN
Maximum span moment Mkvp= 1.44 kNm
Maximum shear force Fkv2= 1.2 kN

Design values of combined actions for ULS

Design moment Md1=gamG*Mkp+gamQ*Mkv1= 7.2576 kNm
Design shear force Vd1=gamG*Fkp+gamQ*Fkv1= 6.048 kN

Design values - combination 1

Design bending moment Md'= 7.2576 kNm
Design shear force Vd'= 6.048 kN
Load duration factor kmod= 0.8

Section comprises one steel web plate with a timber member on each side.
Depth of timber section \( h = 200 \text{ mm} \)
Width of timber section \( b = 50 \text{ mm} \)
Depth of steel section \( d_s = 190 \text{ mm} \)
Width of steel section \( b_s = 8 \text{ mm} \)
Length of bearing \( l_b = 100 \text{ mm} \)
Timber strength class C18.
Modulus of elasticity of steel \( E_{st} = 200000 \text{ N/mm}^2 \)

### Section design values

- Design bending moment \( M_d = M_d' \times 10^6 = 7.2576E6 \text{ Nmm} \)
- Design shear force \( V_d = V_d' \times 10^3 = 6048 \text{ N} \)
- Design axial load (+ve compress) \( N_d = 0 \text{ kN} \)
- Depth factor (Clause 3.2) \( k_h = 1.0 \)
- Bearing modification factor \( k_{c90} = 1.5 \)
- Load-sharing modification factor \( k_{sys} = 1.1 \)
- Shear reduction factor \( k_{sh} = 0.67 \)
- Effective length about zz \( L_e = 3.84 \text{ m} \)

### Bolts between timber and steel plate along length of beam

- Bolt diameter \( b_d = 16 \text{ mm} \)
- Spacing of bolts along beam \( \text{space} = 500 \text{ mm} \)
- Bolts are in double shear. Width of timber for bolt stress calculation is the section width \( b = 50 \text{ mm} \)
- Angle between grain and load \( \alpha_{l1}' = 90 \text{ degrees} \)
- Timber characteristic density \( p_{k1} = 320 \text{ kg/m}^3 \)
- Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.
- Char. value of yield moment \( M_{yRk} = 0.3 \times 400 \times b_d^2 \times 2.6 = 162141 \)
- Char. embedment strength \( f_{hok} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k1} = 22.042 \)
- Denominator (expression 8.31) \( \text{denom} = (K_{90} \times \sin(\alpha_{l1})^2) + \cos(\alpha_{l1})^2 = 1.59 \)
- Embedment strength (timber 1) \( f_{h1k} = f_{hok} / \text{denom} = 13.863 \)
- Embedment strength (timber 2) \( f_{h2k} = 13.863 \)
- Factor \( \beta = f_{h2k} / f_{h1k} = 1 \)

Expressions (f) to (m) for fasteners in double shear refer to section 8.2.3 in EC5. Zero contribution from rope effect will be considered.

Equations

\[
\begin{align*}
\text{(f)} & : & 11090 \\
\text{(g)} & : & 8654.1 \\
\text{(h)} & : & 13793
\end{align*}
\]

Min value of equations (f)-(h) \( \text{Gmin} = 8654.1 \)

Basic bolt shear resistance \( \text{basic} = \text{Gmin} / 1000 = 8.6541 \text{ kN} \)
Design bolt resistance in timber  $p_b l = 2\times k_{mod}/\gamma_M^2 \times \text{basic}=11.077\ kN$

<table>
<thead>
<tr>
<th>Member: 200 mm x 108 mm overall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modular ratio</td>
</tr>
</tbody>
</table>

**DESIGN**
- Timber bending stress          | 3.3126 N/mm$^2$                |

**SUMMARY**
- Timber design bending strength| 9.0746 N/mm$^2$                |
- Steel design bending stress   | 104.9 N/mm$^2$                 |
- Steel design bending strength | 200 N/mm$^2$                   |
- Final deflection              | 12.797 mm                      |
- Limiting deflection           | 19.2 mm                        |
- Design shear force            | 6.048 kN                       |
- Design Shear resistance       | 20.56 kN                       |
- Design bearing stress         | 0.6048 N/mm$^2$                |
- Design bearing strength       | 2.2338 N/mm$^2$                |

Bolts are 16 mm diameter at 500 mm c/c (10 No total).

Edge distance in the steel to be a minimum of 32 mm.

Bolts along the beam to be staggered above and below the centre line with their centres offset from the beam centre line to suit the min bolt edge spacing of 64 mm.

The minimum No of bolts required at the bearing = 2

NOTE: The above values relate to combination 1 and need to be compared with those of combination 2.
Location: Spaced rectangular section with axial load and bending

Rectangular timber member composed of two or more equal shafts spaced apart by end and intermediate packing pieces. Member subject to axial load and bending about x-x.

Calculations to BS5268-2:2002.

Design BM about xx (positive) \( M_x' = 0.2953 \text{ kNm} \)
Design shear force in y direction \( V' = 0 \text{ kN} \)
Design axial load (+ve compress) \( F' = 4.9 \text{ kN} \)
Eff. length for bending about xx \( L_{ex} = 3500 \text{ mm} \)
Eff. length for bending about yy \( L_{ey} = 3500 \text{ mm} \)
Depth of section \( d = 175 \text{ mm} \)
Width of section \( b = 75 \text{ mm} \)
Number of shafts \( n_{os} = 2 \)
Distance between Crs of shafts \( c_{rs} = 200 \text{ mm} \)
Strength class C22 to Table 8.
Actual length of the column \( c_{oll} = 3500 \text{ mm} \)
Intermediate pack centres \( i_{pcrs} = 900 \text{ mm} \)
Timber service class adopted \( t_{mclass} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1.25 \)
Depth factor \( K_7 = (300/d)^{0.11} = 1.0611 \)
Load-sharing modification factor \( K_8 = 1.0 \)
Modification factor \[ K_{12} = (0.5 + (1 + \eta) \frac{C}{2}) - ((0.5 + (1 + \eta) \frac{C}{2})^2 - C)^{0.5} = 0.54924 \]

Actual length of end pack \[ e_{pl} = 450 \text{ mm} \]
Actual length intermediate pack \[ i_{pl} = 250 \text{ mm} \]

<table>
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<th>Depth of section 175 mm</th>
<th>Width of section 75 mm</th>
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<td>Strength class C22 to Table 8.</td>
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<tr>
<td>Number of shafts</td>
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<td></td>
</tr>
<tr>
<td>Timber moisture class</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

Applied comprn stress \[ 0.18667 \text{ N/mm}^2 \]
Permiss comprn stress \[ 5.1492 \text{ N/mm}^2 \]
Applied bending stress \[ 0.3857 \text{ N/mm}^2 \]
Permiss bending stress \[ 9.0192 \text{ N/mm}^2 \]
Interaction factor \[ 0.079514 \]

End pack shear force \[ 1.1944 \text{ kN} \]
End Pack min length \[ 450 \text{ mm} \]
Chosen end pack length \[ 450 \text{ mm} \]
Perm glue line stress \[ 0.71 \text{ N/mm}^2 \]
Actual glue line stress \[ 0.015167 \text{ N/mm}^2 \]

Intermed pack SF \[ 0.59719 \text{ kN} \]
Min Intermed pack length \[ 230 \text{ mm} \]
Chosen pack ctrs \[ 900 \text{ mm} \]
Perm max crs of packs for buckling to cl 2.11.9.2 \[ 1515.5 \text{ mm} \]
Chosen int pack length \[ 250 \text{ mm} \]
Perm glue line stress \[ 0.71 \text{ N/mm}^2 \]
Actual glue line stress \[ 0.01365 \text{ N/mm}^2 \]
Location: Spaced rectangular section with axial load and bending

Rectangular timber member composed of two or more equal shafts spaced apart by end and intermediate packing pieces. Member subject to axial load and bending about x-x.

Calculations to BS5268-2:2002.

Design BM about xx (positive) \( M_x' = 0 \) kNm
Design shear force in y direction \( V' = 2.5 \) kN
Design axial load (+ve compress) \( F' = 20 \) kN
Eff. length for bending about xx \( L_{ex} = 4000 \) mm
Eff. length for bending about yy \( L_{ey} = 4000 \) mm
Depth of section \( d = 120 \) mm
Width of section \( b = 35 \) mm
Number of shafts \( n_{os} = 2 \)
Distance between Crs of shafts \( c_{rs} = 105 \) mm
Strength class C24 to Table 8.
Actual length of the column \( c_{oll} = 4000 \) mm
Intermediate pack centres \( i_{pcrs} = 600 \) mm
Timber service class adopted \( t_{mclass} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1 \)
Load-sharing modification factor \( K_8 = 1.0 \)
Modification factor: \[ K12 = (0.5 + (1 + \eta) \times C/2) - ((0.5 + (1 + \eta) \times C/2)^2 - C)^{0.5} = 0.32483 \]

Actual length of end pack: \( epl = 400 \text{ mm} \)

Actual length intermediate pack: \( ipl = 230 \text{ mm} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARIZE</th>
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<tbody>
<tr>
<td>Depth of section</td>
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<tr>
<td>Width of section</td>
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<td>C24 to Table 8.</td>
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<tr>
<td>Applied comprn stress</td>
<td>2.381 N/mm²</td>
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<tr>
<td>Permiss comprn stress</td>
<td>2.5661 N/mm²</td>
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<tr>
<td>Applied shear stress</td>
<td>0.44643 N/mm²</td>
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<tr>
<td>Permiss shear stress</td>
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<tr>
<td>End pack shear force</td>
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<tr>
<td>End Pack min length</td>
<td>210 mm</td>
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<tr>
<td>Chosen end pack length</td>
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<tr>
<td>Perm glue line stress</td>
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<tr>
<td>Actual glue line stress</td>
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<tr>
<td>Intermed pack SF</td>
<td>2.1667 kN</td>
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<tr>
<td>Min Intermed pack length</td>
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<tr>
<td>Chosen pack ctrs</td>
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</tr>
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<td>Perm max crs of packs for buckling to cl 2.11.9.2</td>
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<tr>
<td>Chosen int pack length</td>
<td>230 mm</td>
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<td>Perm glue line stress</td>
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<tr>
<td>Actual glue line stress</td>
<td>0.078502 N/mm²</td>
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</tbody>
</table>
Location: Spaced rectangular section with axial load and bending

Rectangular timber member composed
of two or more equal shafts spaced
apart by end and intermediate
packing pieces. Member subject to
axial load and bending about x-x.

Calculations to BS5268-2:2002.

Design BM about xx (positive) \( M_x' = 0 \) kNm
Design shear force in y direction \( V' = 3.6 \) kN
Design axial load (+ve compress) \( F' = 30 \) kN
Eff. length for bending about xx \( L_{ex} = 3000 \) mm
Eff. length for bending about yy \( L_{ey} = 3000 \) mm
Depth of section \( d = 120 \) mm
Width of section \( b = 35 \) mm
Number of shafts \( nos = 3 \)
Distance between Crs of shafts \( crs = 105 \) mm
Strength class C22 to Table 8.
Actual length of the column \( coll = 3000 \) mm
Intermediate pack centres \( ipcrs = 550 \) mm
Timber service class adopted \( tmclass = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1.25 \)
Load-sharing modification factor \( K_8 = 1.0 \)
Modification factor \[ K12 = (0.5 + (1 + \eta)C/2) - ((0.5 + (1 + \eta)C/2)^2 - C)^0.5 = 0.41861 \]

**DESIGN**
- Depth of section: 120 mm
- Width of section: 35 mm

**SUMMARY**
- Strength class C22 to Table 8.
- Number of shafts: 3
- Timber moisture class: 1

- Applied comprn stress: 2.381 N/mm²
- Permiss comprn stress: 3.9245 N/mm²
- Applied shear stress: 0.42857 N/mm²
- Permiss shear stress: 0.8875 N/mm²

- End pack shear force: 4.3333 kN
- End Pack min length: 210 mm

- Intermed pack SF: 2.1667 kN
- Min Intermed pack length: 230 mm
- Chosen pack ctrs: 550 mm
- Perm max crs of packs for buckling to cl 2.11.9.2: 707.25 mm
Location: Spaced rectangular section with axial load and bending

Rectangular timber member composed of two or more equal shafts spaced apart by end and intermediate packing pieces. Member subject to axial load and bending about x-x.

Calculations to BS5268-2:2002.

Design BM about xx (positive) $M_x' = 0$ kNm
Design shear force in y direction $V' = 4.8$ kN
Design axial load (+ve compress) $F' = 40$ kN
Eff. length for bending about xx $L_{ex} = 3000$ mm
Eff. length for bending about yy $L_{ey} = 3000$ mm
Depth of section $d = 100$ mm
Width of section $b = 50$ mm
Number of shafts $n = 4$
Distance between Crs of shafts $crs = 105$ mm
Strength class TR20 to Table 9.
Actual length of the column $coll = 3000$ mm
Intermediate pack centres $ipcrs = 750$ mm
Timber service class adopted $tmclass = 1$
Timber service class modification factor $K_2 = 1$ as Table 16.
Duration of loading $K_3 = 1.25$
Load-sharing modification factor $K_8 = 1.0$
Modification factor \[ K_{12} = (0.5 + (1 + \eta) \times C/2) - ((0.5 + (1 + \eta) \times C/2)^2 - C)^{0.5} = 0.30798 \]

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<td>Applied comprn stress</td>
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<tr>
<td>Chosen pack ctrs</td>
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<tr>
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</table>
Location: Spaced rectangular section with axial load and bending

Rectangular timber member composed of two or more equal shafts spaced apart by end and intermediate packing pieces. Member subject to axial load and bending about x-x.

Calculations to BS5268-2:2002.

Design BM about xx (positive) \( M_x' = 1.3 \text{ kNm} \)
Design shear force in y direction \( V' = 2.5 \text{ kN} \)
Design axial load (+ve compress) \( F' = 15.2 \text{ kN} \)
Eff. length for bending about xx \( L_x = 3000 \text{ mm} \)
Eff. length for bending about yy \( L_y = 3000 \text{ mm} \)
Depth of section \( d = 119 \text{ mm} \)
Width of section \( b = 63 \text{ mm} \)
Number of shafts \( n_o = 2 \)
Distance between Crs of shafts \( c_{rs} = 100 \text{ mm} \)

Bending parallel to the grain \( b_{parg} = 7.5 \text{ N/mm}^2 \)
Compress parallel to the grain \( c_{parg} = 7.9 \text{ N/mm}^2 \)
Compress perpendicular to the grain \( c_{perd} = 2.1 \text{ N/mm}^2 \)
Tension parallel to the grain \( t_{parg} = 4.5 \text{ N/mm}^2 \)
Shear parallel to the grain \( s_{parg} = 0.82 \text{ N/mm}^2 \)
Mean modulus of elasticity \( E_{mean} = 10000 \text{ N/mm}^2 \)
Minimum modulus of elasticity \( E_{min} = 7000 \text{ N/mm}^2 \)
Actual length of the column \( c_{oll} = 3000 \text{ mm} \)
Intermediate pack centres \( c_{ipcrs} = 600 \text{ mm} \)
Timber service class adopted \( t_{mclass} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16.
Duration of loading \( K_3 = 1.25 \)
Depth factor \( K_7 = (300/d)^{0.11} = 1.1071 \)
Load-sharing modification factor \( K_8 = 1.0 \)
Modification factor: \( K_{12} = \left( 0.5 + (1 + \eta) \frac{C}{2} \right) - \left( \left( 0.5 + (1 + \eta) \frac{C}{2} \right)^2 - C \right)^{0.5} = 0.23023 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of section</td>
<td>119 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>63 mm</td>
</tr>
<tr>
<td>Number of shafts</td>
<td>2</td>
</tr>
<tr>
<td>Timber moisture class</td>
<td>1</td>
</tr>
<tr>
<td>Applied comprn stress</td>
<td>1.0137 N/mm²</td>
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<tr>
<td>Permiss comprn stress</td>
<td>2.2735 N/mm²</td>
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<tr>
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<td>4.3715 N/mm²</td>
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<tr>
<td>Permiss bending stress</td>
<td>10.379 N/mm²</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.90522</td>
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<tr>
<td>Applied shear stress</td>
<td>0.2501 N/mm²</td>
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<tr>
<td>Permiss shear stress</td>
<td>1.025 N/mm²</td>
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<tr>
<td>End pack shear force</td>
<td>6.2244 kN</td>
</tr>
<tr>
<td>End Pack min length</td>
<td>378 mm</td>
</tr>
<tr>
<td>Intermed pack SF</td>
<td>3.1122 kN</td>
</tr>
<tr>
<td>Min Intermed pack length</td>
<td>230 mm</td>
</tr>
<tr>
<td>Chosen pack ctrs</td>
<td>600 mm</td>
</tr>
<tr>
<td>Perm max crs of packs for buckling to cl 2.11.9.2</td>
<td>1273.1 mm</td>
</tr>
</tbody>
</table>
Location: Spaced rectangular section with axial load and bending

Rectangular timber member composed of two or more equal shafts spaced apart by end and intermediate packing pieces. Member subject to axial load and bending about y-y.

Calculations to EC5 incorporating National Annex Amendment No.2.

Design BM about yy (positive) $M_d'=0.41933\, \text{kNm}$
Design shear force in z direction $V_{dz}'=0\, \text{kN}$
Design axial load (+ve compress) $N_d'=6.958\, \text{kN}$
Eff. length for bending about yy $L_{ey}=3500\, \text{mm}$
Eff. length for bending about zz $L_{ez}=3500\, \text{mm}$
Depth of section $h=175\, \text{mm}$
Width of section $b=75\, \text{mm}$
Number of shafts $n_{os}=2$
Distance between Crs of shafts $crs=200\, \text{mm}$
Timber strength class C22.
Actual length of the column $coll=3500\, \text{mm}$
Intermediate pack centres $ipcrs=900\, \text{mm}$
Depth factor (Clause 3.2) $k_h=1.0$
Load-sharing modification factor $k_{sys}=1.0$
Factor $\eta$ from Table C.1 $\eta=1$
Max instability factor $k_c=1$
Instability factor $k_{cz}=1/(k_z+(k_z^2-rsrz^2)^{0.5})=0.73604$
Shear reduction factor $k_{sh}=0.67$
Effective length about zz $L_e=2.8\, \text{m}$
Packs for spaced columns

Fixings are required to transfer a shear force between the abutting face of the packing and one shaft.

End pack shear force \[ \text{epsf} = 1.3 \times A \times b \times sc0d/(\text{nos} \times \text{crs}) = 1696 \text{ N} \]

Actual length of end pack \[ \text{epl} = 450 \text{ mm} \]

Since \( plgs \geq gls \) \( (3.8 \geq 0.021537) \) then glue line stress on end pack is satisfactory.

Actual length intermediate pack \[ \text{ipl} = 250 \text{ mm} \]

Since \( plgs \geq gils \) \( (3.8 \geq 0.019383) \) then glue line stress on intermediate pack is satisfactory.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of section</td>
<td>175 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>75 mm</td>
</tr>
<tr>
<td>Timber strength class</td>
<td>C22.</td>
</tr>
<tr>
<td>Number of shafts</td>
<td>2</td>
</tr>
<tr>
<td>Timber moisture class</td>
<td>1</td>
</tr>
</tbody>
</table>

Design comprn stress \[ 0.26507 \text{ N/mm}^2 \]
Design comprn strength \[ 12.308 \text{ N/mm}^2 \]
Design bending stress \[ 0.5477 \text{ N/mm}^2 \]
Design bending strength \[ 13.538 \text{ N/mm}^2 \]
Interaction factor \[ 0.061992 \]

End pack shear force \[ 1.696 \text{ kN} \]
End Pack min length \[ 187.5 \text{ mm} \]
Chosen end pack length \[ 450 \text{ mm} \]
Design glue line strength \[ 3.8 \text{ N/mm}^2 \]
Design glue line stress \[ 0.021537 \text{ N/mm}^2 \]

Intermed.pack SF \[ 0.84801 \text{ kN} \]
Min intermed.pack length \[ 187.5 \text{ mm} \]
Chosen pack ctrs \[ 900 \text{ mm} \]
Perm max centres of packs for buckling \[ 1515.5 \text{ mm} \]
Chosen inter pack length \[ 250 \text{ mm} \]
Design glue line strength \[ 3.8 \text{ N/mm}^2 \]
Design glue line stress \[ 0.019383 \text{ N/mm}^2 \]
Location: Spaced rectangular section with axial load and bending

Rectangular timber member composed of two or more equal shafts spaced apart by end and intermediate packing pieces. Member subject to axial load and bending about y-y.

Calculations to EC5 incorporating National Annex Amendment No.2.

Design BM about yy (positive) $M_d' = 0$ kNm
Design shear force in z direction $V_{dz}' = 3.55$ kN
Design axial load (+ve compress) $N_d' = 25.56$ kN
Eff. length for bending about yy $L_{ey} = 4000$ mm
Eff. length for bending about zz $L_{ez} = 4000$ mm
Depth of section $h = 120$ mm
Width of section $b = 35$ mm
Number of shafts $nos = 2$
Distance between Crs of shafts $crs = 105$ mm
Timber strength class C24.
Actual length of the column $coll = 4000$ mm
Intermediate pack centres $ipcrs = 600$ mm
Depth factor (Clause 3.2) $k_h = (150/h)^{0.2} = 1.0456$
Load-sharing modification factor $k_{sys} = 1.0$
Factor $\eta$ from Table C.1 $\eta = 1$
Max instability factor $k_{cy} = 1$
Instability factor $k_{cz} = 1/(k_z+(k_z^2-r_{srz}^2)^{0.5}) = 0.33132$
Shear reduction factor $k_{sh} = 0.67$
Effective length about zz $L_e = 3.2$ m
Packs for spaced columns

Fixings are required to transfer a shear force between the abutting face of the packing and one shaft.

End pack shear force \[ \text{epsf} = 1.3A*b*sc0d/(\text{nos}*\text{crs}) = 5538 \text{ N} \]

Actual length of end pack \[ \text{epl} = 400 \text{ mm} \]

Since \( plgs \geq gls \ (4 \geq 0.11538) \) then glue line stress on end pack is satisfactory.

Actual length intermediate pack \[ \text{ipl} = 230 \text{ mm} \]

Since \( plgs \geq gils \ (4 \geq 0.10033) \) then glue line stress on intermediate pack is satisfactory.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Depth of section</th>
<th>Width of section</th>
<th>Timber strength class C24.</th>
<th>Timber moisture class</th>
<th>Number of shafts</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>120 mm</td>
<td>35 mm</td>
<td></td>
<td></td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

- Design comprn stress \[ 3.0429 \text{ N/mm}^2 \]
- Design comprn strength \[ 9.6923 \text{ N/mm}^2 \]
- Interaction factor \[ 0.94756 \]
- Design shear stress \[ 0.63393 \text{ N/mm}^2 \]
- Design shear strength \[ 1.2369 \text{ N/mm}^2 \]

- End pack shear force \[ 5.538 \text{ kN} \]
- End Pack min length \[ 105 \text{ mm} \]
- Chosen end pack length \[ 400 \text{ mm} \]
- Design glue line strength \[ 4 \text{ N/mm}^2 \]
- Design glue line stress \[ 0.11538 \text{ N/mm}^2 \]

- Intermed.pack SF \[ 2.769 \text{ kN} \]
- Min intermed.pack length \[ 105 \text{ mm} \]
- Chosen pack ctrs \[ 600 \text{ mm} \]
- Perm max centres of packs for buckling \[ 707.25 \text{ mm} \]
- Chosen inter pack length \[ 230 \text{ mm} \]
- Design glue line strength \[ 4 \text{ N/mm}^2 \]
- Design glue line stress \[ 0.10033 \text{ N/mm}^2 \]
Location: Spaced rectangular section with axial load and bending

Rectangular timber member composed of two or more equal shafts spaced apart by end and intermediate packing pieces. Member subject to axial load and bending about y-y.

Calculations to EC5 incorporating National Annex Amendment No.2.

Design BM about yy (positive) \(M_d' = 0\) kNm
Design shear force in z direction \(V_{dz}' = 5.112\) kN
Design axial load (+ve compress) \(N_d' = 42.6\) kN
Eff. length for bending about yy \(L_{ey} = 3000\) mm
Eff. length for bending about zz \(L_{ez} = 3000\) mm
Depth of section \(h = 120\) mm
Width of section \(b = 35\) mm
Number of shafts \(n_{os} = 3\)
Distance between Crs of shafts \(crs = 105\) mm
Timber strength class C22.
Actual length of the column \(coll = 3000\) mm
Intermediate pack centres \(ipcrs = 550\) mm
Depth factor (Clause 3.2) \(kh = (150/h)^{0.2} = 1.0456\)
Load-sharing modification factor \(k_{sys} = 1\)
Factor \(\eta\) from Table C.1 \(eta = 1\)
Max instability factor \(k_{cy} = 1\)
Instability factor \(k_{cz} = 1/(kz + (kz^2 - rsrz^2)^{0.5}) = 0.47729\)
Shear reduction factor \(k_{sh} = 0.67\)
Effective length about zz \(Le = 2.4\) m
Packs for spaced columns

Fixings are required to transfer a shear force between the abutting face of the packing and one shaft.

End pack shear force \( \text{epsf}=1.3\times A\times b\times sc0d/(\text{nos}\times \text{crs}) = 6153.3 \text{ N} \)

Actual length of end pack \( \text{epl}=200 \text{ mm} \)

Since \( \text{plgs} \geq \text{gils} (3.8 \geq 0.25639) \) then glue line stress on end pack is satisfactory.

Actual length intermediate pack \( \text{ipl}=250 \text{ mm} \)

Since \( \text{plgs} \geq \text{gils} (3.8 \geq 0.10256) \) then glue line stress on intermediate pack is satisfactory.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Width of section</th>
<th>Depth of section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber strength class C22.</td>
<td>35 mm</td>
<td>120 mm</td>
</tr>
<tr>
<td>Number of shafts</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Timber moisture class</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

- Design comprn stress \( 3.381 \text{ N/mm}^2 \)
- Design comprn strength \( 12.308 \text{ N/mm}^2 \)
- Interaction factor \( 0.57554 \)
- Design shear stress \( 0.60857 \text{ N/mm}^2 \)
- Design shear strength \( 1.5668 \text{ N/mm}^2 \)

- End pack shear force \( 6.1533 \text{ kN} \)
- End Pack min length \( 105 \text{ mm} \)
- Chosen end pack length \( 200 \text{ mm} \)
- Design glue line strength \( 3.8 \text{ N/mm}^2 \)
- Design glue line stress \( 0.25639 \text{ N/mm}^2 \)

- Intermed.pack SF \( 3.0767 \text{ kN} \)
- Min intermed.pack length \( 105 \text{ mm} \)
- Chosen pack ctrs \( 550 \text{ mm} \)
- Perm max centres of packs for buckling \( 707.25 \text{ mm} \)
- Chosen inter pack length \( 250 \text{ mm} \)
- Design glue line strength \( 3.8 \text{ N/mm}^2 \)
- Design glue line stress \( 0.10256 \text{ N/mm}^2 \)
Location: Spaced rectangular section with axial load and bending

Rectangular timber member composed of two or more equal shafts spaced apart by end and intermediate packing pieces. Member subject to axial load and bending about y-y.

Calculations to EC5 incorporating National Annex Amendment No.2.

Design BM about yy (positive) \( M_d' = 0 \) kNm
Design shear force in z direction \( V_{dz}' = 6.816 \) kN
Design axial load (+ve compress) \( N_d' = 45 \) kN
Eff. length for bending about yy \( L_{ey} = 3000 \) mm
Eff. length for bending about zz \( L_{ez} = 3000 \) mm
Depth of section \( h = 100 \) mm
Width of section \( b = 50 \) mm
Number of shafts \( \text{nos} = 4 \)
Distance between Crs of shafts \( \text{crs} = 105 \) mm
Timber strength class D60.
Actual length of the column \( \text{coll} = 3000 \) mm
Intermediate pack centres \( \text{ipcrs} = 750 \) mm
Depth factor (Clause 3.2) \( k_h = (150/h)^{0.2} = 1.0845 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)
Factor \( \eta \) from Table C.1 \( \eta = 2.5 \)
Max instability factor \( k_c = 1 \)
Instability factor \( k_{cz} = 1/(k_z + (k_z^2 - rsrz^2)^{0.5}) = 0.26866 \)
Shear reduction factor \( k_{sh} = 0.67 \)
Effective length about zz \( L_e = 2.4 \) m
### Packs for Spaced Columns

Fixings are required to transfer a shear force between the abutting face of the packing and one shaft.

- **End pack shear force**: \( \text{epsf} = \frac{1.3 \times A \times b \times \text{sc0d}}{\text{nos} \times \text{crs}} = 6964.3 \text{ N} \)
- **Actual length of end pack**: \( \text{epl} = 250 \text{ mm} \)
- **Actual length intermediate pack**: \( \text{ipl} = 250 \text{ mm} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of section</td>
<td>100 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>50 mm</td>
</tr>
<tr>
<td>Timber strength class</td>
<td>D60</td>
</tr>
<tr>
<td>Number of shafts</td>
<td>4</td>
</tr>
<tr>
<td>Timber moisture class</td>
<td>1</td>
</tr>
</tbody>
</table>

- **Design comprn stress**: 2.25 N/mm²
- **Design comprn strength**: 20.308 N/mm²
- **Interaction factor**: 0.41241
- **Design shear stress**: 0.5112 N/mm²
- **Design shear strength**: 1.9791 N/mm²

- **End pack shear force**: 6.9643 kN
- **End Pack min length**: 82.5 mm

- **Intermed.pack SF**: 3.4821 kN
- **Min intermed.pack length**: 82.5 mm
- **Chosen pack ctrs**: 750 mm
- **Perm max centres of packs for buckling**: 1010.4 mm
Location: Spaced rectangular section with axial load and bending

Rectangular timber member composed of two or more equal shafts spaced apart by end and intermediate packing pieces. Member subject to axial load and bending about y-y.

Calculations to EC5 incorporating National Annex Amendment No.2.

Design BM about yy (positive) \( M_d' = 1.846 \text{ kNm} \)
Design shear force in z direction \( V_{dz'} = 3.55 \text{ kN} \)
Design axial load (+ve compress) \( N_d' = 21.584 \text{ kN} \)
Eff.length for bending about yy \( L_{ey} = 3000 \text{ mm} \)
Eff.length for bending about zz \( L_{ez} = 3000 \text{ mm} \)
Depth of section \( h = 119 \text{ mm} \)
Width of section \( b = 63 \text{ mm} \)
Number of shafts \( \text{nos} = 2 \)
Distance between Crs of shafts \( \text{crs} = 100 \text{ mm} \)

Bending parallel to the grain \( f_{mk} = 24 \text{ N/mm}^2 \)
Compress parallel to the grain \( f_{c0k} = 21 \text{ N/mm}^2 \)
Compress perp to the grain \( f_{c90k} = 2.5 \text{ N/mm}^2 \)
Tension parallel to the grain \( f_{t0k} = 14 \text{ N/mm}^2 \)
Shear parallel to the grain \( f_{vk} = 2.5 \text{ N/mm}^2 \)
Mean modulus of elasticity \( E_{0\text{mean}} = 11000 \text{ N/mm}^2 \)
5th percentile MOE \( E_{0\text{05}} = 7400 \text{ N/mm}^2 \)
Actual length of the column \( c_{oll} = 3000 \text{ mm} \)
Intermediate pack centres \( i_{pcrs} = 600 \text{ mm} \)
Depth factor (Clause 3.2) \( k_h = (150/h)^{0.2} = 1.0474 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)
Factor \( \eta \) from Table C.1 \( \text{eta} = 1 \)
Max instability factor \( k_{cy} = 1 \)
Instability factor \( k_{cz} = 1/(k_z + (k_z^2 - r_{srz}^2)^{0.5}) = 0.60946 \)
Shear reduction factor \( k_{sh} = 0.67 \)
Effective length about zz \( L_{e} = 2.4 \text{ m} \)
Packs for spaced columns

Fixings are required to transfer a shear force between the abutting face of the packing and one shaft.

End pack shear force \( \text{epsf} = 1.3 \times A \times b \times s c_0d / (\text{nos} \times \text{crs}) = 8838.6 \text{ N} \)

Actual length of end pack \( epl = 400 \text{ mm} \)

Since \( \text{plgs} \geq \text{gls} \geq 2.5 \geq 0.18569 \) then glue line stress on end pack is satisfactory.

Actual length intermediate pack \( ipl = 230 \text{ mm} \)

Since \( \text{plgs} \geq \text{gils} \geq 2.5 \geq 0.16147 \) then glue line stress on intermediate pack is satisfactory.

| DESIGN | | | |
|--------|--------|--------|
| Depth of section | 119 mm |
| Width of section | 63 mm |

| SUMMARY | | | |
|---------|--------|--------|
| Number of shafts | 2 |
| Timber moisture class | 1 |
| Design comprn stress | 1.4395 N/mm² |
| Design comprn strength | 12.923 N/mm² |
| Design bending stress | 6.2075 N/mm² |
| Design bending strength | 15.469 N/mm² |
| Interaction factor | 0.51267 |
| Design shear stress | 0.35514 N/mm² |
| Design shear strength | 1.0308 N/mm² |
| End pack shear force | 8.8386 kN |
| End Pack min length | 55.5 mm |
| Chosen end pack length | 400 mm |
| Design glue line strength | 2.5 N/mm² |
| Design glue line stress | 0.18569 N/mm² |
| Intermed.pack SF | 4.4193 kN |
| Min intermed.pack length | 55.5 mm |
| Chosen pack ctrs | 600 mm |
| Perm max centres of packs for buckling | 1273.1 mm |
| Chosen inter pack length | 230 mm |
| Design glue line strength | 2.5 N/mm² |
| Design glue line stress | 0.16147 N/mm² |
Location: Ply box beams with udl & point loads

Laterally restrained glued ply or OSB box beam design to BS5268-2:2002

Depth of ply or OSB section $d=350$ mm
Nominal width of each ply web $b_p'=9.5$ mm
Depth of each timber section $d_c=66$ mm
Width of timber section $b=100$ mm

Face grain is parallel to span of the beam.
Strength class C24 to Table 8.
Plywood is Canadian Douglas Fir (unsanded) to Table 45 of BS5268.
Plywood comprises 3 plies
No. of laminates per chord $l_{am27}=2$
No. of laminates in both chords $l_{am28}=4$

Since $I_{ratio} > 10$ and $\leq 20$ (11.268 > 10 and $\leq 20$) to Clause 2.10.10 the beam should be held in line at the ends and web stiffeners should be used.

Simply supported beam subjected to vertical loads.

Beam span $L=7.5$ m
Dist. from left support to start $L_{au(1)}=0$ m
Distance from left support to end $L_{bu(1)}=7.5$ m
Dead load (unfactored) $G_{ku(1)}=0.425$ kN/m
Imposed load (unfactored) $Q_{ku(1)}=0.6$ kN/m
Maximum span bending moment $7.207$ kNm
Design shear force $F_{ve}'=3.8438$

Modification factors

Duration of loading $K_3=1.25$
Depth factor $K_7=0.81*(d^2+92300)/(d^2+56800) = 0.97037$
Width modification factor $K_{14}=(300/h)^{0.11}=0.98319$

The bending design of the beam is based on the modulus of the flanges and ignores the contribution of the web plate(s). Permissible stress to be based on the tension grade stress of the timber flanges.
Web to flange connection is to be by glueing to BS6446 requirements.

No. of glue lines  \( n_{gl}=2 \)

Plywood or OSB is to be adhered to timber using nails or staples as the bonding pressure. Mod factor  \( K_{70}=0.9 \)

Length of bearing  \( l_{b}=150 \) mm

Bearing modification factor  \( K_{4}=1.0 \)

BS5268 part 2 2002 does not give any specific guidance for the design of web stiffeners. Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur. Typically these stiffeners are at centres of between one to two times the clear depth of the web.

In this case this would be at 218 mm to 436 mm centres.

**Web splice case 1**

Splice plate half length  \( b_{s}=450 \) mm

Height of each splice plate  \( h_{s}=200 \) mm

Number of glue lines at this splice section  \( n_{sp}=1 \)

Applied shear force at splice  \( s_{fs}=1.5 \) kN

Since rollp > sigr ( 0.21938 N/mm² > 0.061296 N/mm² ) then glue line stress is within permissible stresses. Hence splice OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of ply section  350 mm</td>
<td>Overall beam width 118 mm</td>
</tr>
<tr>
<td>Applied BM 7.207 kNm</td>
<td>Moment capacity 9.5048 kNm</td>
</tr>
<tr>
<td>Deflection 19.839 mm</td>
<td>Limiting deflection 22.5 mm</td>
</tr>
<tr>
<td>Flange to web shear 0.0814 N/mm²</td>
<td>Perm rolling shear 0.21938 N/mm²</td>
</tr>
<tr>
<td>Panel shear stress 0.77249 N/mm²</td>
<td>Perm panel stress 2.15 N/mm²</td>
</tr>
<tr>
<td>Bearing reaction 3.8438 kN</td>
<td>Bearing capacity 70.488 kN</td>
</tr>
</tbody>
</table>
Location: Default case 2

Laterally restrained glued ply or OSB
web I beam design to BS5268-2:2002

Depth of ply or OSB section       \( d = 300 \text{ mm} \)
Nominal width of ply web           \( b_p' = 12 \text{ mm} \)
Depth of each timber section       \( d_c = 70 \text{ mm} \)
Width of timber section            \( b = 44 \text{ mm} \)
Face grain is perpendicular to span of the beam.
Strength class C24 to Table 8.
Plywood is Finnish Birch faced, Sanded to Table 32 of BS5268
No. of laminates per chord         \( \text{lam27}=2 \)
No. of laminates in both chords    \( \text{lam28}=4 \)
Since \( \text{Iratio} > 10 \) and \( \leq 20 \) (16.837 > 10 and \( \leq 20 \)) to Clause 2.10.10 the beam should be held in line at the ends and web stiffeners should be used.

Simply supported beam subjected to vertical loads.

Beam span                         \( L = 4 \text{ m} \)
Dist. from left support to start   \( L_{au(1)} = 0 \text{ m} \)
Distance from left support to end  \( L_{bu(1)} = 4 \text{ m} \)
Dead load (unfactored)             \( G_{ku(1)} = 1 \text{ kN/m} \)
Imposed load (unfactored)          \( Q_{ku(1)} = 1 \text{ kN/m} \)
Maximum span bending moment        4 kNm
Design shear force                 \( F_{ve'} = 4 \)

Modification factors

Duration of loading \( K_3 = 1 \)
Depth factor \( K_7 = (300/d)^{0.11}=1 \)
Width modification factor \( K_{14} = (300/h)^{0.11}=1 \)

The bending design of the beam is based on the modulus of the flanges and ignores the contribution of the web plate(s).
Permissible stress to be based on the tension grade stress of the timber flanges.
Web to flange connection is to be by glueing to BS6446 requirements.

No. of glue lines $n_{gl} = 2$

Plywood or OSB is to be adhered to timber using clamps as the bonding pressure. Mod factor $K_70 = 1.0$

Length of bearing $l_b = 150$ mm

Bearing modification factor $K_4 = 1.0$

BS5268 part 2 2002 does not give any specific guidance for the design of web stiffeners. Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur. Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 160 mm to 320 mm centres.

**Web splice case 1**

Splice plate half length $b_s = 400$ mm

Height of each splice plate $h_s = 160$ mm

Number of glue lines at this splice section $n_{sp} = 1$

Applied shear force at splice $s_{fs} = 2.7$ kN

Since $rollp > sigr$ ( $0.615$ N/mm² > $0.15746$ N/mm² ) then glue line stress is within permissible stresses. Hence splice OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of ply section</td>
<td>Overall beam width</td>
</tr>
<tr>
<td>300 mm</td>
<td>99.5 mm</td>
</tr>
<tr>
<td>Applied BM</td>
<td>4 kNm</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>5.5932 kNm</td>
</tr>
<tr>
<td>Deflection</td>
<td>7.2983 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>12 mm</td>
</tr>
<tr>
<td>Flange to web shear</td>
<td>0.10442 N/mm²</td>
</tr>
<tr>
<td>Perm rolling shear</td>
<td>0.615 N/mm²</td>
</tr>
<tr>
<td>Panel shear stress</td>
<td>1.5033 N/mm²</td>
</tr>
<tr>
<td>Perm panel stress</td>
<td>4.83 N/mm²</td>
</tr>
<tr>
<td>Bearing reaction</td>
<td>4 kN</td>
</tr>
<tr>
<td>Bearing capacity</td>
<td>59.039 kN</td>
</tr>
</tbody>
</table>
Location: Default case 3

Laterally restrained glued ply or OSB

web I beam design to BS5268-2:2002

Depth of ply or OSB section \( d = 600 \text{ mm} \)
Nominal width of ply web \( b_p' = 18.5 \text{ mm} \)
Depth of each timber section \( d_c = 47 \text{ mm} \)
Width of timber section \( b = 47 \text{ mm} \)
Face grain is parallel to span of the beam.
Strength class C18 to Table 8.
Plywood is Canadian Douglas Fir (unsanded) to Table 45 of BS5268.
Plywood comprises 6 plies
No. of laminates per chord \( \text{lam}_{27} = 2 \)
No. of laminates in both chords \( \text{lam}_{28} = 4 \)
Since \( I_{ratio} > 40 \) (88.983 > 40) to Clause 2.10.10 the beam should be fully restrained along its compression edge and web stiffeners should be used.

Simply supported beam subjected to vertical loads.

Beam span \( L = 8 \text{ m} \)
Dist. from left support to start \( L_{au(1)} = 0 \text{ m} \)
Distance from left support to end \( L_{bu(1)} = 8 \text{ m} \)
Dead load (unfactored) \( G_{ku(1)} = 0.6 \text{ kN/m} \)
Imposed load (unfactored) \( Q_{ku(1)} = 0.6 \text{ kN/m} \)
Maximum span bending moment \( 9.6 \text{ kNm} \)
Design shear force \( F_{ve'} = 4.8 \)

Modification factors

Duration of loading \( K_3 = 1.25 \)
Depth factor \( K_7 = 0.81 \times \frac{(d^2 + 92300)}{(d^2 + 56800)} \)
\( = 0.87899 \)
Width modification factor \( K_{14} = (300/h)^{0.11} = 0.92659 \)

Design is based on a composite section transformed to suit the varying Young's Modulii of the ply or OSB and timber.
Web to flange connection is to be by gluing to BS6446 requirements.

No. of glue lines \( ngl = 2 \)

Plywood or OSB is to be adhered to timber using clamps as the bonding pressure. Mod factor \( K70 = 1.0 \)

Length of bearing \( l_b = 100 \text{ mm} \)

From BS5268-2 Table 18, bearing is \(< 75 \text{ mm} \) from joist end.

Bearing Modification factor \( K4 = 1.0 \)

BS5268 part 2 2002 does not give any specific guidance for the design of web stiffeners. Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur. Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 506 mm to 1012 mm centres.

**Web splice case 1**

<table>
<thead>
<tr>
<th>Splice plate half length</th>
<th>bs = 600 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height of each splice plate</td>
<td>hs = 450 mm</td>
</tr>
<tr>
<td>Number of glue lines at this splice section</td>
<td>nsp = 2</td>
</tr>
<tr>
<td>Bending Moment at splice point</td>
<td>bms = 9.6 kNm</td>
</tr>
<tr>
<td>Applied shear force at splice</td>
<td>sfs = 4.8 kN</td>
</tr>
</tbody>
</table>

Since \( \sigma_{rlp} > \sigma_{gr} \) (0.24375 N/mm² > 0.17075 N/mm²) then glue line stress is within permissible stresses. Hence splice OK.

Since \( bssp \leq p_{ply} \) (1.1713 N/mm² ≤ 7.0125 N/mm²) then ply or OSB bending stress at centre of splice plate is within permissible stresses. Hence splice OK.

**DESIGN**

<table>
<thead>
<tr>
<th>Depth of ply section</th>
<th>600 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall beam width</td>
<td>112 mm</td>
</tr>
<tr>
<td>Applied BM</td>
<td>9.6 kNm</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>25.598 kNm</td>
</tr>
<tr>
<td>Deflection</td>
<td>12.861 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>14 mm</td>
</tr>
<tr>
<td>Flange to web shear</td>
<td>0.062306 N/mm²</td>
</tr>
<tr>
<td>Perm rolling shear</td>
<td>0.24375 N/mm²</td>
</tr>
<tr>
<td>Panel shear stress</td>
<td>0.54113 N/mm²</td>
</tr>
<tr>
<td>Perm panel stress</td>
<td>2.2125 N/mm²</td>
</tr>
<tr>
<td>Bearing reaction</td>
<td>4.8 kN</td>
</tr>
<tr>
<td>Bearing capacity</td>
<td>43.579 kN</td>
</tr>
</tbody>
</table>

**SUMMARY**

<table>
<thead>
<tr>
<th>Depth of ply section</th>
<th>600 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall beam width</td>
<td>112 mm</td>
</tr>
<tr>
<td>Applied BM</td>
<td>9.6 kNm</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>25.598 kNm</td>
</tr>
<tr>
<td>Deflection</td>
<td>12.861 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>14 mm</td>
</tr>
<tr>
<td>Flange to web shear</td>
<td>0.062306 N/mm²</td>
</tr>
<tr>
<td>Perm rolling shear</td>
<td>0.24375 N/mm²</td>
</tr>
<tr>
<td>Panel shear stress</td>
<td>0.54113 N/mm²</td>
</tr>
<tr>
<td>Perm panel stress</td>
<td>2.2125 N/mm²</td>
</tr>
<tr>
<td>Bearing reaction</td>
<td>4.8 kN</td>
</tr>
<tr>
<td>Bearing capacity</td>
<td>43.579 kN</td>
</tr>
</tbody>
</table>
Location: Default case 4

Laterally restrained nailed ply or OSB box beam design to BS5268-2:2002

Depth of ply or OSB section \( d = 450 \) mm
Nominal width of each ply web \( b_{p}' = 18.5 \) mm
Depth of each timber section \( d_{c} = 47 \) mm
Width of timber section \( b = 97 \) mm
Face grain is parallel to span of the beam.
Strength class C24 to Table 8.
Plywood is Canadian Douglas Fir (unsanded) to Table 45 of BS5268.
Plywood comprises 7 plies
No. of laminates per chord \( \text{lam}_{27} = 1 \)
No. of laminates in both chords \( \text{lam}_{28} = 2 \)
Since \( \text{I}_{\text{ratio}} > 10 \) and \( \leq 20 \) (10.553 > 10 and \( \leq 20 \)) to Clause 2.10.10 the beam should be held in line at the ends and web stiffeners should be used.

Simply supported beam subjected to vertical loads.

Beam span \( L = 10 \) m
Dist. from left support to start \( L_{\text{au}}(1) = 0 \) m
Distance from left support to end \( L_{\text{bu}}(1) = 10 \) m
Dead load (unfactored) \( G_{\text{ku}}(1) = 0.3 \) kN/m
Imposed load (unfactored) \( Q_{\text{ku}}(1) = 0.6 \) kN/m
Maximum span bending moment 11.25 kNm
Design shear force \( F_{\text{ve}}' = 4.5 \)

Modification factors

Duration of loading \( K_{3} = 1.25 \)
Depth factor \( K_{7} = 0.81 \times \frac{(d^2 + 92300)}{(d^2 + 56800)} = 0.92089 \)
Width modification factor \( K_{14} = (300/h)^{0.11} = 0.95638 \)

The bending design of the beam is based on the modulus of the flanges and ignores the contribution of the web plate(s). Permissible stress to be based on the bending grade stress of the timber flanges.
Web to flange connection is to be by nailing using ordinary round wire nails conforming to BS 1202 : Part 1. Plywood or OSB to timber connection with nails driven into side grain.

No. of nail shear surfaces \( ngl = 2 \)

Nailed joint position:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied SF at nail position</td>
<td>( sfs = 4.5 \text{ kN} )</td>
</tr>
<tr>
<td>Diameter of nails</td>
<td>( dfn = 4 \text{ mm} )</td>
</tr>
<tr>
<td>Length of nails</td>
<td>( l = 75 \text{ mm} )</td>
</tr>
<tr>
<td>Basic single shear lateral load</td>
<td>( \text{basic} = 502 \text{ N} )</td>
</tr>
<tr>
<td>Mod factor for duration of load</td>
<td>( K48 = 1 )</td>
</tr>
<tr>
<td>Mod factor for number of nails</td>
<td>( K50 = 0.9 )</td>
</tr>
<tr>
<td>Permissible nail load</td>
<td>( \text{pnl} = \text{basic} \times K48 \times K49 \times K50 = 451.8 \text{ N} )</td>
</tr>
<tr>
<td>Required centres of nails</td>
<td>( ncrs = ngl \times \text{pnl} \times Ix / (sfs \times 1000 \times fmarea) )</td>
</tr>
<tr>
<td></td>
<td>( = 141.04 \text{ mm} )</td>
</tr>
</tbody>
</table>

**Minimum nail spacings**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Edge dist ↓ to timber grain</td>
<td>( edpg = 20 \text{ mm} )</td>
</tr>
<tr>
<td>End distance // to grain</td>
<td>56 mm</td>
</tr>
<tr>
<td>Edge distance in the plywood</td>
<td>12 mm</td>
</tr>
<tr>
<td>Distance between lines of nails perpendicular to grain</td>
<td>28 mm</td>
</tr>
<tr>
<td>Distance between adjacent nails in any one line, // to grain</td>
<td>56 mm</td>
</tr>
</tbody>
</table>

To fix ply web(s) to the top and bottom chords of the beam adopt 1 row(s) of 4 mm nails 75 mm long at 141 mm centres in each chord.

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of bearing</td>
<td>( lb = 100 \text{ mm} )</td>
</tr>
<tr>
<td>From BS5268-2 Table 18, bearing is &lt; 75 mm from joist end.</td>
<td></td>
</tr>
<tr>
<td>Bearing Modification factor</td>
<td>( K4 = 1.0 )</td>
</tr>
</tbody>
</table>

BS5268 part 2 2002 does not give any specific guidance for the design of web stiffeners. Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur. Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 356 mm to 712 mm centres.

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of ply section</td>
<td>450 mm</td>
</tr>
<tr>
<td>Overall beam width</td>
<td>133 mm</td>
</tr>
<tr>
<td>Applied BM</td>
<td>11.25 kNm</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>14.27 kNm</td>
</tr>
<tr>
<td>Deflection</td>
<td>39.477 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>30 mm</td>
</tr>
<tr>
<td>Precamber of beam</td>
<td>10.436 mm</td>
</tr>
<tr>
<td>Panel shear stress</td>
<td>0.35448 N/mm²</td>
</tr>
<tr>
<td>Perm panel stress</td>
<td>2.2125 N/mm²</td>
</tr>
<tr>
<td>Bearing reaction</td>
<td>4.5 kN</td>
</tr>
<tr>
<td>Bearing capacity</td>
<td>56.091 kN</td>
</tr>
</tbody>
</table>

SCALE 5.48 Office 1007 Proforma 258
Location: Default case 5

Laterally restrained glued ply or OSB
box beam design to BS5268-2:2002

Depth of ply or OSB section \( d = 1200 \text{ mm} \)
Nominal width of each ply web \( b_{p}' = 18 \text{ mm} \)
Depth of each timber section \( d_{c} = 147 \text{ mm} \)
Width of timber section \( b = 147 \text{ mm} \)
Face grain is perpendicular to span of the beam.
Strength class C24 to Table 8.
Plywood is Finnish Birch faced, Sanded to Table 32 of BS5268
No. of laminates per chord \( \text{lam}_{27} = 3 \)
No. of laminates in both chords \( \text{lam}_{28} = 6 \)
Since \( \text{Iratio} > 40 \) \( (47.823 > 40) \) to Clause 2.10.10 the beam should be fully restrained along its compression edge and web stiffeners should be used.

Simply supported beam subjected to vertical loads.

Beam span \( L = 16 \text{ m} \)
Dist. from left support to start \( L_{au(1)} = 0 \text{ m} \)
Distance from left support to end \( L_{bu(1)} = 16 \text{ m} \)
Dead load (unfactored) \( G_{ku(1)} = 1.2 \text{ kN/m} \)
Imposed load (unfactored) \( Q_{ku(1)} = 1 \text{ kN/m} \)
Maximum span bending moment \( 70.4 \text{ kNm} \)
Design shear force \( F_{ve}' = 17.6 \)

Modification factors

Duration of loading \( K_{3} = 1.25 \)
Depth factor \( K_{7} = 0.81 \cdot (d^2 + 92300) / (d^2 + 56800) = 0.82921 \)
Width modification factor \( K_{14} = (300/h)^{0.11} = 0.85857 \)

Bending design is to be based on the full composite section and includes the contribution of the web(s).
Permissible stress to be based on the tension grade stress of the timber flanges \( \rho_{bs} = \sigma_{tad} = 4.4817 \text{ N/mm}^2 \)
Web to flange connection is to be by glueing to BS6446 requirements.

No. of glue lines \( n_{gl} = 2 \)

Plywood or OSB is to be adhered to timber using clamps as the bonding pressure. Mod factor \( K_{70} = 1.0 \)

Length of bearing \( l_b = 150 \text{ mm} \)

Bearing modification factor \( K_4 = 1.0 \)

BS5268 part 2 2002 does not give any specific guidance for the design of web stiffeners. Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur. Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 906 mm to 1812 mm centres.

**DESIGN**

- Depth of ply section: 1200 mm
- Overall beam width: 181.2 mm
- Applied BM: 70.4 kNm
- Moment capacity: 110.42 kNm
- Deflection: 24.049 mm
- Limiting deflection: 48 mm
- Flange to web shear: 0.040104 N/mm²
- Perm rolling shear: 0.615 N/mm²
- Panel shear stress: 0.5313 N/mm²
- Perm panel stress: 4.83 N/mm²
- Bearing reaction: 17.6 kN
- Bearing capacity: 82.337 kN
Location: Ply box beams with udl & point loads

Laterally restrained glued ply or OSB box beam design to BS5268-2:2002

Depth of ply or OSB section      $d=350$ mm
Actual width of each OSB web     $b_{p}^{'}=9$ mm
Depth of each timber section     $d_{c}=66$ mm
Width of timber section          $b=100$ mm
Face grain is parallel to span of the beam.
Strength class C24 to Table 8.
Oriented strand board of Grade 3 (OSB/3) to BS EN12369-1:2001 and BS EN300:1997.
Oriented strand board service class 1
No. of laminates per chord       $l_{a m 27}=2$
No. of laminates in both chords  $l_{a m 28}=4$
Since $I_{r a t i o} > 10$ and $\delta \leq 20$ (11.268 > 10 and $\delta \leq 20$) to Clause 2.10.10 the beam should be held in line at the ends and web stiffeners should be used.

Simply supported beam subjected to vertical loads.

Beam span                              $L=7.5$ m
Dist. from left support to start       $L_{u a 1}(1)=0$ m
Distance from left support to end      $L_{b u 1}(1)=7.5$ m
Dead load (unfactored)                 $G_{k u 1}(1)=0.425$ kN/m
Imposed load (unfactored)              $Q_{k u 1}(1)=0.6$ kN/m
Maximum span bending moment            $7.207$ kNm
Design shear force                     $F_{v e}^{'}=3.8438$

Modification factors

Duration of loading                  $K_{3}=1.25$
Depth factor                        $K_{7}=0.81 \times (d^{2}+92300)/(d^{2}+56800) = 0.97037$
Width modification factor           $K_{14}=(300/h)^{0.11}=0.98319$

The bending design of the beam is based on the modulus of the flanges and ignores the contribution of the web plate(s). Permissible stress to be based on the tension grade stress of the timber flanges.
Web to flange connection is to be by gluing to BS6446 requirements.

No. of glue lines \( ngl = 2 \)

Plywood or OSB is to be adhered to timber using nails or staples as the bonding pressure. Mod factor \( K70 = 0.9 \)

Length of bearing \( lb = 150 \text{ mm} \)

Bearing modification factor \( K4 = 1.0 \)

BS5268 part 2 2002 does not give any specific guidance for the design of web stiffeners. Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur. Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 218 mm to 436 mm centres.

**Web splice case 1**

Splice plate half length \( bs = 450 \text{ mm} \)

Height of each splice plate \( hs = 200 \text{ mm} \)

Number of glue lines at this splice section \( nsp = 1 \)

Applied shear force at splice \( sfs = 1.5 \text{ kN} \)

Since \( rollp > sigr (0.12821 \text{ N/mm}^2 > 0.061296 \text{ N/mm}^2) \) then glue line stress is within permissible stresses. Hence splice OK.

**DESIGN**

Depth of OSB/3 section \( 350 \text{ mm} \)

**SUMMARY**

Overall beam width \( 118 \text{ mm} \)

Applied BM \( 7.207 \text{ kNm} \)

Moment capacity \( 9.5048 \text{ kNm} \)

Deflection \( 19.324 \text{ mm} \)

Limiting deflection \( 22.5 \text{ mm} \)

Flange to web shear \( 0.0814 \text{ N/mm}^2 \)

Perm rolling shear \( 0.12821 \text{ N/mm}^2 \)

Panel shear stress \( 0.77249 \text{ N/mm}^2 \)

Perm panel stress \( 1.9373 \text{ N/mm}^2 \)

Bearing reaction \( 3.8438 \text{ kN} \)

Bearing capacity \( 67.788 \text{ kN} \)
Location: Ply box beams with udl & point loads

Laterally restrained glued ply or

OSB box beam design to EC5 with

National Annex Amendment No.2

Depth of ply or OSB section \( h=350 \text{ mm} \)
Nominal width of each ply web \( bp'=9.5 \text{ mm} \)
Depth of each timber section \( hc=66 \text{ mm} \)
Width of timber section \( b=100 \text{ mm} \)
Face grain is parallel to span of beam.
Timber strength class C24.
Plywood is Canadian Douglas Fir (unsanded).
Plywood comprises 3 plies.
Since \( \text{Iratio} > 10 \) and \( \leq 20 \) (11.268 > 10 and \( \leq 20 \)) the beam should be held in line at the ends and web stiffeners should be used.

Simply supported beam subject to vertical loads.

Beam span \( L=7.5 \text{ m} \)
Dist. from left support to start \( Lau(1)=0 \text{ m} \)
Distance from left support to end \( Lbu(1)=7.5 \text{ m} \)
Permanent load (unfactored) \( Gku(1)=0.425 \text{ kN/m} \)
Variable load 1 (unfactored) \( Qua(1)=0.6 \text{ kN/m} \)

Characteristic values of actions

Permanent:
Reaction at left hand end \( 1.5938 \text{ kN} \)
Reaction at right hand end \( 1.5938 \text{ kN} \)
Maximum span moment \( Mkp=2.9883 \text{ kNm} \)
Maximum shear force \( Fkp=1.5938 \text{ kN} \)
Variable 1:
Reaction at left hand end \( 2.25 \text{ kN} \)
Reaction at right hand end \( 2.25 \text{ kN} \)
Maximum span moment \( Mkv1=4.2188 \text{ kNm} \)
Maximum shear force \( Fkv1=2.25 \text{ kN} \)
Modification factors

Depth factor (Clause 3.2) \[ k_h = \left( \frac{150}{h_c} \right)^{0.2} = 1.1784 \]
Width modification factor \[ k_w = \left( \frac{150}{h_c'} \right)^{0.2} = 1.0845 \]
Load-sharing modification factor \[ k_{sys} = 1.0 \]

The bending design of the beam is based on the modulus of the flanges and ignores the contribution of the web plate(s).

Design strength to be based on the tension strength of the timber flanges.

Web service class \[ s_{ervc} = 2 \]
Web to flange connection is to be by gluing.
No. of glue lines \[ n_{gl} = 2 \]

Plywood or OSB is to be adhered to timber using nails or staples as the bonding pressure.

Length of bearing \[ l_b = 150 \text{ mm} \]
Bearing modification factor \[ k_{c90} = 1.5 \]

Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur.

Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 218 mm to 436 mm centres.

Web splice case 1

Splice plate half length \[ b_s = 450 \text{ mm} \]
Height of each splice plate \[ h_s = 200 \text{ mm} \]
Number of glue lines at this splice section \[ n_{sp} = 1 \]
Design shear force at splice \[ s_{fs} = 1.5 \text{ kN} \]

Since \[ p_{fv90d} > s_{igr} \] (0.36535 N/mm² > 0.061296 N/mm²) glue line design stress is within design strength. Hence splice is OK.

**DESIGN**  
Depth of ply section 350 mm  
Overall beam width 118 mm  
Design bending moment 10.362 kNm  
Design moment capacity 14.983 kNm  
Deflection 26.991 mm  
Limiting deflection 30 mm  
Flange/web shear stress 0.11704 N/mm²  
Rolling shear strength 0.36535 N/mm²  
Design bearing reaction 5.5266 kN  
Design bearing capacity 49.015 kN

**SUMMARY**  
Depth of ply section 350 mm  
Overall beam width 118 mm  
Design bending moment 10.362 kNm  
Design moment capacity 14.983 kNm  
Deflection 26.991 mm  
Limiting deflection 30 mm  
Flange/web shear stress 0.11704 N/mm²  
Rolling shear strength 0.36535 N/mm²  
Design bearing reaction 5.5266 kN  
Design bearing capacity 49.015 kN
Location: Default case 2

Laterally restrained glued ply or OSB web I beam design to EC5 with National Annex Amendment No.2

Depth of ply or OSB section \( h = 300 \text{ mm} \)
Nominal width of ply web \( b_p' = 12 \text{ mm} \)
Depth of each timber section \( h_c = 70 \text{ mm} \)
Width of timber section \( b = 44 \text{ mm} \)
Face grain is perpendicular to span of beam.
Timber strength class C24.
Plywood is Finnish Birch faced (sanded).
Since Iratio > 10 and \( ð ≤ 20 \) (16.837 > 10 and \( ð ≤ 20 \)) the beam should be held in line at the ends and web stiffeners should be used.

Simply supported beam subject to vertical loads.

Beam span \( L = 4 \text{ m} \)
Dist. from left support to start \( L_{au}(1) = 0 \text{ m} \)
Distance from left support to end \( L_{bu}(1) = 4 \text{ m} \)
Permanent load (unfactored) \( G_ku(1) = 1 \text{ kN/m} \)
Variable load 1 (unfactored) \( Q_{ua}(1) = 1 \text{ kN/m} \)

Characteristic values of actions

Permanent:
Reaction at left hand end \( 2 \text{ kN} \)
Reaction at right hand end \( 2 \text{ kN} \)
Maximum span moment \( M_{kp} = 2 \text{ kNm} \)
Maximum shear force \( F_{kp} = 2 \text{ kN} \)

Variable 1:
Reaction at left hand end \( 2 \text{ kN} \)
Reaction at right hand end \( 2 \text{ kN} \)
Maximum span moment \( M_{kv1} = 2 \text{ kNm} \)
Maximum shear force \( F_{kv1} = 2 \text{ kN} \)
Modification factors

Depth factor (Clause 3.2)  \( kh = (150/hc)^{0.2} = 1.1647 \)
Width modification factor  \( kw = (150/hc')^{0.2} = 1.1647 \)
Load-sharing modification factor  \( ksys = 1.0 \)
The bending design of the beam is based on the modulus of the flanges and ignores the contribution of the web plate(s).
Design strength to be based on the tension strength of the timber flanges.
Web service class  \( servc = 2 \)
Web to flange connection is to be by glueing.
No. of glue lines  \( ngl = 2 \)
Plywood or OSB is to be adhered to timber using clamps as the bonding pressure.
Length of bearing  \( lb = 150 \text{ mm} \)
Bearing modification factor  \( kc90 = 1.5 \)
Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur.
Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 160 mm to 320 mm centres.

Web splice case 1

Splice plate half length  \( bs = 400 \text{ mm} \)
Height of each splice plate  \( hs = 160 \text{ mm} \)
Number of glue lines at this splice section  \( nsp = 1 \)
Design shear force at splice  \( sfs = 2.7 \text{ kN} \)
Since \( pfv90d > sigr \) (0.66568 N/mm² > 0.15746 N/mm²) glue line design stress is within design strength. Hence splice is OK.

DESIGN

<table>
<thead>
<tr>
<th>Depth of ply section</th>
<th>300 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall beam width</td>
<td>99.5 mm</td>
</tr>
<tr>
<td>Design bending moment</td>
<td>5.7 kNm</td>
</tr>
<tr>
<td>Design moment capacity</td>
<td>10.182 kNm</td>
</tr>
<tr>
<td>Deflection</td>
<td>9.2122 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>16 mm</td>
</tr>
<tr>
<td>Flange/web shear stress</td>
<td>0.14879 N/mm²</td>
</tr>
<tr>
<td>Rolling shear strength</td>
<td>0.66568 N/mm²</td>
</tr>
<tr>
<td>Design bearing reaction</td>
<td>5.7 kN</td>
</tr>
<tr>
<td>Design bearing capacity</td>
<td>54.527 kN</td>
</tr>
</tbody>
</table>
Location: Default case 3

Laterally restrained glued ply or OSB web I beam design to EC5 with National Annex Amendment No.2

Depth of ply or OSB section \( h = 600 \text{ mm} \)
Nominal width of ply web \( b_{p}' = 18.5 \text{ mm} \)
Depth of each timber section \( h_c = 47 \text{ mm} \)
Width of timber section \( b = 47 \text{ mm} \)
Face grain is parallel to span of beam.
Timber strength class C18.
Plywood is Canadian Douglas Fir (unsanded).
Plywood comprises 6 plies.
Since \( \text{Iratio} > 40 \ (88.983 > 40) \) the beam should be fully restrained along its compression edge and web stiffeners should be used.

Simply supported beam subject to vertical loads.

Beam span \( L = 8 \text{ m} \)
Dist. from left support to start \( L_{au(1)} = 0 \text{ m} \)
Distance from left support to end \( L_{bu(1)} = 8 \text{ m} \)
Permanent load (unfactored) \( G_{ku(1)} = 0.6 \text{ kN/m} \)
Variable load 1 (unfactored) \( Q_{u(1)} = 0.6 \text{ kN/m} \)

Characteristic values of actions

Permanent:
Reaction at left hand end \( 2.4 \text{ kN} \)
Reaction at right hand end \( 2.4 \text{ kN} \)
Maximum span moment \( M_{kp} = 4.8 \text{ kNm} \)
Maximum shear force \( F_{kp} = 2.4 \text{ kN} \)
Variable 1:
Reaction at left hand end \( 2.4 \text{ kN} \)
Reaction at right hand end \( 2.4 \text{ kN} \)
Maximum span moment \( M_{kv1} = 4.8 \text{ kNm} \)
Maximum shear force \( F_{kv1} = 2.4 \text{ kN} \)
Modification factors

Depth factor (Clause 3.2)        \[ k_h = (150/\text{hc})^{0.2} = 1.2612 \]
Width modification factor       \[ k_w = (150/\text{hc'})^{0.2} = 1.2612 \]
Load-sharing modification factor \[ k_{sys} = 1.1 \]

Design is based on a composite section transformed to suit the varying Young's Modulii of the ply or OSB and timber.

Web service class              \[ \text{servc} = 2 \]
Web to flange connection is to be by glueing.
No. of glue lines               \[ n_{gl} = 2 \]

Plywood or OSB is to be adhered to timber using clamps as the bonding pressure.
Length of bearing               \[ l_b = 100 \text{ mm} \]
Bearing modification factor     \[ k_{c90} = 1.5 \]

Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur. Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 506 mm to 1012 mm centres.

Web splice case 1

Splice plate half length        \[ b_s = 600 \text{ mm} \]
Height of each splice plate     \[ h_s = 450 \text{ mm} \]
Number of glue lines at this splice section \[ n_{sp} = 2 \]
Bending Moment at splice point  \[ b_{ms} = 13.68 \text{ kNm} \]
Design shear force at splice    \[ s_{fs} = 6.84 \text{ kN} \]

Since pfv90d > sigr (0.63394 N/mm² > 0.24332 N/mm²) glue line design stress is within design strength. Hence splice is OK.
Since bssp ≤ fply (2.931 N/mm² ≤ 8.36 N/mm²) ply or OSB design bending stress at centre of splice plate is within the design strength. Hence splice OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of ply section</td>
<td>600 mm</td>
</tr>
<tr>
<td>Overall beam width</td>
<td>112 mm</td>
</tr>
<tr>
<td>Design bending moment</td>
<td>13.68 kNm</td>
</tr>
<tr>
<td>Design moment capacity</td>
<td>28.492 kNm</td>
</tr>
<tr>
<td>Deflection</td>
<td>14.824 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>32 mm</td>
</tr>
<tr>
<td>Flange/web shear stress</td>
<td>0.088786 N/mm²</td>
</tr>
<tr>
<td>Rolling shear strength</td>
<td>0.63394 N/mm²</td>
</tr>
<tr>
<td>Design bearing reaction</td>
<td>6.84 kN</td>
</tr>
<tr>
<td>Design bearing capacity</td>
<td>29.578 kN</td>
</tr>
</tbody>
</table>
**Location: Default case 4**

Laterally restrained nailed

ply or OSB box beam design

to EC5 incorporating National

Annex Amendment No.2

Depth of ply or OSB section  \( h=450 \text{ mm} \)
Nominal width of each ply web  \( b'=18.5 \text{ mm} \)
Depth of each timber section  \( hc=47 \text{ mm} \)
Width of timber section  \( b=97 \text{ mm} \)
Face grain is parallel to span of beam.
Timber strength class C24.
Plywood is Canadian Douglas Fir (unsanded).
Plywood comprises 7 plies.
Since \( \text{Iratio} > 10 \) and \( \leq 20 \) (10.553 > 10 and \( \leq 20 \)) the beam should be held in line at the ends and web stiffeners should be used.

Simply supported beam subject to vertical loads.

Beam span  \( L=10 \text{ m} \)
Dist. from left support to start  \( L_{au(1)}=0 \text{ m} \)
Distance from left support to end  \( L_{bu(1)}=10 \text{ m} \)
Permanent load (unfactored)  \( G_{ku(1)}=0.3 \text{ kN/m} \)
Variable load 1 (unfactored)  \( Q_{u(1)}=0.6 \text{ kN/m} \)

**Characteristic values of actions**

Permanent:
- Reaction at left hand end 1.5 kN
- Reaction at right hand end 1.5 kN
- Maximum span moment  \( M_{kp}=3.75 \text{ kNm} \)
- Maximum shear force  \( F_{kp}=1.5 \text{ kN} \)

Variable 1:
- Reaction at left hand end 3 kN
- Reaction at right hand end 3 kN
- Maximum span moment  \( M_{kv1}=7.5 \text{ kNm} \)
- Maximum shear force  \( F_{kv1}=3 \text{ kN} \)
Modification factors

Depth factor (Clause 3.2) \( kh = (150/hc)^{0.2} = 1.2612 \)
Width modification factor \( kw = (150/hc')^{0.2} = 1.0911 \)
Load-sharing modification factor \( ksys = 1.0 \)

The bending design of the beam is based on the modulus of the flanges and ignores the contribution of the web plate(s).
Design strength to be based on the bending strength of the timber flanges.

Web service class \( servc = 2 \)
Web to flange connection is to be by nailing using ordinary round wire nails. Plywood or OSB to timber connection with nails driven into side grain.
Number of nail shear surfaces \( ngl = 2 \)

Nailed joint position:

Design SF at nail position \( sfs = 6.525 \text{ kN} \)

Headside member - plywood group 1 comprising:
(a) American construction and industrial plywood
(b) Canadian Douglas fir plywood
(c) Canadian softwood plywood
(d) Swedish softwood plywood
(e) Finnish conifer plywood

Ply characteristic density \( pk1 = 390 \text{ kg/m}^3 \)
Timber characteristic density \( pk2 = 350 \text{ kg/m}^3 \)
Diameter of nails \( dn = 4 \text{ mm} \)
Length of nails \( l = 75 \text{ mm} \)

Basic loads in N will be derived from expressions given in Section 8 assuming round nails.

Min value of equations (a)-(f) \( G_{min} = 968.09 \)

Char. nail shear capacity \( basic = G_{min} = 968.09 \text{ N} \)
Material factor for connections \( gamM = 1.3 \)
Design resistance per nail \( pnl = kmod*basic/gamM = 595.75 \text{ N} \)
Required centres of nails \( ncrs = ngl*pnl*Iy/(sfs*1000*fmarea) \)
\( = 128.26 \text{ mm} \)

Minimum nail spacings - EC5 Table 8.2 and Fig 8.7

Minimum spacing of nails within one row // to grain \( a1 = (5+7\cdot\cos(\alpha))\cdot dn = 48 \text{ mm} \)
Actual spacing of nails within one row // to grain \( a1 = 48 \text{ mm} \)
Spacing of rows of nails perpendicular to grain \( a2 = 5\cdot dn = 20 \text{ mm} \)
End distance (loaded) \( a3t = (10+5\cdot\cos(\alpha))\cdot dn = 60 \text{ mm} \)
End distance (unloaded) \( a3c = 10\cdot dn = 40 \text{ mm} \)
Edge distance (loaded) \( a4t = (5+2\cdot\sin(\alpha))\cdot dn = 20 \text{ mm} \)
Edge distance (unloaded) \( a4c = (5+5\cdot\sin(\alpha))\cdot dn = 20 \text{ mm} \)

Fix ply webs to the top and bottom chords of the beam. Adopt 1 row(s) of 4 mm nails 75 mm long at 128 mm centres in each chord.
Length of bearing \( l_b = 100 \text{ mm} \)

Bearing modification factor \( k_{c90} = 1.5 \)

Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur. Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 356 mm to 712 mm centres.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Depth of ply section</th>
<th>450 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Overall beam width</td>
<td>133 mm</td>
</tr>
<tr>
<td>Design bending moment</td>
<td>16.313 kNm</td>
<td></td>
</tr>
<tr>
<td>Design moment capacity</td>
<td>30.788 kNm</td>
<td></td>
</tr>
<tr>
<td>Deflection</td>
<td>31.821 mm</td>
<td></td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>40 mm</td>
<td></td>
</tr>
<tr>
<td>Design bearing reaction</td>
<td>6.525 kN</td>
<td></td>
</tr>
<tr>
<td>Design bearing capacity</td>
<td>45.905 kN</td>
<td></td>
</tr>
</tbody>
</table>
Location: Default case 5

Laterally restrained glued ply or
OSB box beam design to EC5 with
National Annex Amendment No.2

Depth of ply or OSB section \( h = 1200 \text{ mm} \)
Nominal width of each ply web \( bp' = 18 \text{ mm} \)
Depth of each timber section \( hc = 147 \text{ mm} \)
Width of timber section \( b = 147 \text{ mm} \)
Face grain is perpendicular to span of beam.
Timber strength class C24.
Plywood is Finnish Birch faced (sanded).
Since \( \text{Iratio} > 40 \) (47.823 > 40) the beam should be fully restrained along its compression edge and web stiffeners should be used.

Simply supported beam subject to vertical loads.

Beam span \( L = 16 \text{ m} \)
Dist. from left support to start \( Lau(1) = 0 \text{ m} \)
Distance from left support to end \( Lbu(1) = 16 \text{ m} \)
Permanent load (unfactored) \( Gku(1) = 1.2 \text{ kN/m} \)
Variable load 1 (unfactored) \( Qua(1) = 1 \text{ kN/m} \)

Characteristic values of actions

Permanent:
Reaction at left hand end \( 9.6 \text{ kN} \)
Reaction at right hand end \( 9.6 \text{ kN} \)
Maximum span moment \( Mkp = 38.4 \text{ kNm} \)
Maximum shear force \( Fkp = 9.6 \text{ kN} \)
Variable 1:
Reaction at left hand end \( 8 \text{ kN} \)
Reaction at right hand end \( 8 \text{ kN} \)
Maximum span moment \( Mkvl = 32 \text{ kNm} \)
Maximum shear force \( Fkvl = 8 \text{ kN} \)
Modification factors

Depth factor (Clause 3.2) \( k_h = (150/hc)^{0.2} = 1.004 \)
Width modification factor \( k_w = (150/hc')^{0.2} = 1.004 \)
Load-sharing modification factor \( k_{sys} = 1.0 \)

Bending design is to be based on the full composite section and includes the contribution of the web(s).

Design strength to be based on the tension strength of the timber flanges \( p_{bs} = f_{td} = 7.2794 \text{ N/mm}^2 \)
Web service class \( s_{ervc} = 2 \)
Web to flange connection is to be by glueing.
No. of glue lines \( n_{gl} = 2 \)
Plywood or OSB is to be adhered to timber using clamps as the bonding pressure.
Length of bearing \( l_b = 150 \text{ mm} \)
Bearing modification factor \( k_c90 = 1.5 \)

Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur. Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 906 mm to 1812 mm centres.

**DESIGN**

- Depth of ply section: 1200 mm
- Overall beam width: 181.2 mm
- Design bending moment: 99.84 kNm
- Design moment capacity: 284.89 kNm
- Deflection: 40.662 mm
- Limiting deflection: 64 mm
- Flange/web shear stress: 0.056875 N/mm²
- Rolling shear strength: 0.78421 N/mm²
- Design bearing reaction: 24.96 kN
- Design bearing capacity: 116.93 kN
Location: Ply box beams with udl & point loads

Laterally restrained glued ply or
OSB box beam design to EC5 with
National Annex Amendment No.2

Depth of ply or OSB section $h=350$ mm
Actual width of each OSB web $bp'=9$ mm
Depth of each timber section $hc=66$ mm
Width of timber section $b=100$ mm
Face grain is parallel to span of beam.
Timber strength class C24.
Oriented Strand Board:
Oriented strand board of Grade 3 (OSB/3) to BS EN12369-1:2001 and
BS EN300:1997.
Oriented strand board service class 1
Actual thickness of OSB $bp=9$ mm
Since $I_{ratio} > 10$ and $\sigma < 20$ ($11.268 > 10$ and $\sigma < 20$) the beam should be held in line at the ends and web stiffeners should be used.

Simply supported beam subject to vertical loads.

Beam span $L=7.5$ m
Dist. from left support to start $L_{au(1)}=0$ m
Distance from left support to end $L_{bu(1)}=7.5$ m
Permanent load (unfactored) $G_{ku(1)}=0.425$ kN/m
Variable load 1 (unfactored) $Q_{ua(1)}=0.6$ kN/m

Characteristic values of actions

Permanent:
Reaction at left hand end $1.5938$ kN
Reaction at right hand end $1.5938$ kN
Maximum span moment $M_{kp}=2.9883$ kNm
Maximum shear force $F_{kp}=1.5938$ kN
Variable 1:
Reaction at left hand end $2.25$ kN
Reaction at right hand end $2.25$ kN
Maximum span moment $M_{kv1}=4.2188$ kNm
Maximum shear force $F_{kv1}=2.25$ kN
Modification factors

Depth factor (Clause 3.2) \[ kh = (150/h_c)^{0.2} = 1.1784 \]
Width modification factor \[ kw = (150/h_c')^{0.2} = 1.0845 \]
Load-sharing modification factor \[ k_{sys} = 1.0 \]

The bending design of the beam is based on the modulus of the flanges and ignores the contribution of the web plate(s).

Design strength to be based on the tension strength of the timber flanges.

Web to flange connection is to be by glueing.

No. of glue lines \[ n_{gl} = 2 \]

Plywood or OSB is to be adhered to timber using nails or staples as the bonding pressure.

Length of bearing \[ l_b = 150 \text{ mm} \]

Bearing modification factor \[ k_{c90} = 1.5 \]

Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur. Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 218 mm to 436 mm centres.

Web splice case 1

Splice plate half length \[ b_s = 450 \text{ mm} \]
Height of each splice plate \[ h_s = 200 \text{ mm} \]
Number of glue lines at this splice section \[ n_{sp} = 1 \]
Design shear force at splice \[ s_{fs} = 1.5 \text{ kN} \]

Since \( pfv_{90d} > s_{igr} \ (0.4105 \text{ N/mm}^2 > 0.061296 \text{ N/mm}^2) \) glue line design stress is within design strength. Hence splice is OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of OSB/3 section</td>
<td>350 mm</td>
</tr>
<tr>
<td>Overall beam width</td>
<td>118 mm</td>
</tr>
<tr>
<td>Design bending moment</td>
<td>10.362 kNm</td>
</tr>
<tr>
<td>Design moment capacity</td>
<td>14.983 kNm</td>
</tr>
<tr>
<td>Deflection</td>
<td>23.617 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>30 mm</td>
</tr>
<tr>
<td>Flange/web shear stress</td>
<td>0.11704 N/mm²</td>
</tr>
<tr>
<td>Rolling shear strength</td>
<td>0.4105 N/mm²</td>
</tr>
<tr>
<td>Design bearing reaction</td>
<td>5.5266 kN</td>
</tr>
<tr>
<td>Design bearing capacity</td>
<td>57.835 kN</td>
</tr>
</tbody>
</table>
Laterally restrained glued ply or
OSB web I beam design to EC5 with
National Annex Amendment No.2

Depth of ply or OSB section       h=250 mm
Nominal width of ply web          bp'=12.5 mm
Depth of each timber section      hc=50 mm
Width of timber section           b=45 mm
Face grain is perpendicular to span of beam.

Bending parallel to the grain     fmk=18 N/mm²
Compress parallel to the grain    fc0k=18 N/mm²
Compress perp to the grain       fc90k=2.2 N/mm²
Tension parallel to the grain    ft0k=11 N/mm²
Shear parallel to the grain       fvk=3.4 N/mm²
Mean modulus of elasticity       E0mean=9000 N/mm²
5th percentile MOE               E005=6000 N/mm²
User defined plywood characteristic strength properties.
Minimum thickness of ply          bp=12 mm
Bending face grain // to span     pfmk=20.2 N/mm²
Tension // to surface grain      pft0k=11.1 N/mm²
Bending face grain ⊥ to span     pfm90k=10.4 N/mm²
Tension ⊥ to surface grain      pft90k=7.4 N/mm²
Compress // to surface grain     pfc0k=14.5 N/mm²
Compress ⊥ to surface grain      pfc90k=9.7 N/mm²
Rolling shear                    pfvrk=0.64 N/mm²
Panel shear                      pfv0k=3.5 N/mm²
Surface grain ⊥ to the span      Emper=3960 N/mm²
Panel shear modulus of rigidity  Gw=430 N/mm²
Since Iratio > 10 and ≤ 20 (12.126 > 10 and ≤ 20) the beam should
be held in line at the ends and web stiffeners should be used.

Simply supported beam subject to
vertical loads.

Beam span                          L=4 m
Dist. from left support to start   Lau(1)=0 m
Distance from left support to end  Lbu(1)=4 m
Permanent load (unfactored)       Gku(1)=0.405 kN/m
Variable load 1 (unfactored)      Qua(1)=0.9 kN/m
Characteristic values of actions

Permanent:
Reaction at left hand end 0.81 kN
Reaction at right hand end 0.81 kN
Maximum span moment Mkp=0.81 kNm
Maximum shear force Fkp=0.81 kN

Variable 1:
Reaction at left hand end 1.8 kN
Reaction at right hand end 1.8 kN
Maximum span moment Mkvl=1.8 kNm
Maximum shear force Fkvl=1.8 kN

Modification factors

Depth factor (Clause 3.2) kh=(150/hc)^0.2=1.2457
Width modification factor kw=(150/hc')^0.2=1.2457
Load-sharing modification factor ksys=1.0
Bending design is to be based on the full composite section and includes the contribution of the web(s).
Design strength to be based on the bending strength of the timber flanges pbs=fmd=13.799 N/mm²
Web service class servc=2
Web to flange connection is to be by glueing.
No. of glue lines ngl=2
Plywood or OSB is to be adhered to timber using clamps as the bonding pressure.
Length of bearing lb=100 mm
Bearing modification factor kc90=1.5
Web stiffeners should be placed under any concentrated loads and at bearing positions. Internal stiffeners should also be placed at intervals to ensure buckling of the web members cannot occur.
Typically these stiffeners are at centres of between one to two times the clear depth of the web. In this case this would be at 150 mm to 300 mm centres.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of ply section 250 mm</td>
<td></td>
</tr>
<tr>
<td>Overall beam width 102 mm</td>
<td></td>
</tr>
<tr>
<td>Design bending moment 3.7935 kNm</td>
<td></td>
</tr>
<tr>
<td>Design moment capacity 11.611 kNm</td>
<td></td>
</tr>
<tr>
<td>Deflection 11.978 mm</td>
<td></td>
</tr>
<tr>
<td>Limiting deflection 16 mm</td>
<td></td>
</tr>
<tr>
<td>Flange/web shear stress 0.1588 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Rolling shear strength 0.23718 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Design bearing reaction 3.7935 kN</td>
<td></td>
</tr>
<tr>
<td>Design bearing capacity 29.877 kN</td>
<td></td>
</tr>
</tbody>
</table>
Stressed skin timber floor joist

Glued stressed skin floor joist
with top and bottom ply sheathing

to BS5268-2:2002

Location: Stressed skin floor joists

Depth of timber joist section \( d = 200 \text{ mm} \)
Width of timber joist section \( b = 63 \text{ mm} \)
Nominal depth of top ply \( \text{dtp}' = 22.5 \text{ mm} \)
Nominal depth of bottom ply \( \text{dbp}' = 12.5 \text{ mm} \)
Plywood face grain is perpendicular to span of floor joist.

Joist span \( L = 5 \text{ m} \)
Centres of joists \( c_{rs} = 400 \text{ mm} \)
Effective width of top flange \( e_{\text{width}} = 400 \text{ mm} \)

Grade stresses

Strength class C24 to Table 8.
Plywood is Canadian Douglas Fir (unsanded) to Table 45 of BS5268
Top plywood sheath comprises 7 plies.
Bottom plywood sheath comprises 4 plies.
Timber service class adopted \( \text{tmclass} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16
Member is in a load-sharing system as defined by Clause 13.
Load-sharing modification factor \( K_8 = 1.1 \)
The mean modulus of elasticity should be used to calculate deflections and displacements under both dead and imposed loads.

Loading - UDL

Other dead UDL \( G_{kux}' = 0.25 \text{ kN/m}^2 \)
Imposed load \( Q_{kux} = 1.5 \text{ kN/m}^2 \)
Distance from left support \( L_{c(1)} = 2.5 \text{ m} \)
Dead load (unfactored) \( G_{kc(1)} = 0.5 \text{ kN} \)
Imposed load (unfactored) \( Q_{kc(1)} = 0.5 \text{ kN} \)
Maximum span moment \( M' = 1.25 \text{ kNm} \)

Analysis results

Unfactored dead shear \( R_{udlhe} = G_k L/2 + Pl_ds = 0.79378 \text{ kN} \)
Unfactored imposed shear \( R_{uilhe} = Q_k L/2 + Pl_is = 1.75 \text{ kN} \)
Shear force \( F_{ve} = (G_k + Q_k) L/2 + Pl_ds + Pl_is = 2.5438 \text{ kN} \)
Maximum span bending moment \( M = (G_k + Q_k) L^2/8 + M' = 3.8047 \text{ kNm} \)
Duration of loading \( K_3 = 1.25 \)

Depth factor \( K_7 = (300/d)^{0.11} = 1.0456 \)

Width modification factor \( K_{14} = (300/h)^{0.11} = 1.0456 \)

No of spans which ply is continuous over plycon=3

Transverse stresses and deflection to be based on the clear span of ply.

**Rolling shear stress at ply/ joist interface (position B)**

Web to flange connection is to be by glueing.

Since \( E_{ayfa} \geq E_{ayfb} \) the 1st moment of area of the top flange is greater than the 1st moment of area of the bottom flange. Rolling shear will be more critical at the top ply.

1st moment of area of critical flange \( E_{aroll} = E_{ayfa} = 1.6542E9 \) Nmm

Rolling shear stress @ position B \( \tau_{0B} = F_{ve} * 1E3 * E_{aroll} / E_{Ix} / \text{conwid} \)

\( = 0.086688 \) N/mm²

Plywood is to be adhered to timber using nails or staples as the bonding pressure. Mod factor \( K_{70} = 0.9 \)

Since \( \tau_{rollp} \geq \tau_{0B} \) the applied rolling shear is less than the permissible shear. Hence OK.

Length of bearing \( l_b = 100 \) mm

From BS5268-2 Table 18, bearing is < 75 mm from joist end.

Bearing Modification factor \( K_4 = 1.0 \)

**Stresses due to bending action**

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
<th>N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compress @ Top face of top ply</td>
<td>1.0228</td>
<td></td>
</tr>
<tr>
<td>Perm Compress stress in top ply</td>
<td>5.2375</td>
<td></td>
</tr>
<tr>
<td>Bending in joist @ joist/top ply</td>
<td>5.1748</td>
<td></td>
</tr>
<tr>
<td>Bending in joist @ joist/btm ply</td>
<td>5.4916</td>
<td></td>
</tr>
<tr>
<td>Perm bending stress in joist</td>
<td>10.783</td>
<td></td>
</tr>
<tr>
<td>Tension @ btm face of btm ply</td>
<td>1.3569</td>
<td></td>
</tr>
<tr>
<td>Perm tension stress in btm ply</td>
<td>3.9375</td>
<td></td>
</tr>
</tbody>
</table>

Bending stress in top ply

spanning between timber joist \( 0.7593 \)

Perm ply bending stress \( 15.5 \) N/mm²

Transverse deflection of top ply \( 0.11205 \) mm

Permissible transverse deflection \( 1.011 \) mm

Deflection of composite section \( 12.552 \) mm

Limiting deflection of comp sect \( 15 \) mm

Lateral shear @ neutral axis \( 0.25454 \) N/mm²

Perm. lateral shear in joist \( 0.97625 \) N/mm²

Ply flange to joist shear stress \( 0.086688 \) N/mm²

Permissible rolling shear in ply \( 0.31556 \) N/mm²

Bearing stress on ply \( 0.063595 \) N/mm²

Perm. ply bearing stress \( 2.7 \) N/mm²

Bearing stress on joist \( 0.40378 \) N/mm²

Permissible joist bearing stress \( 3.3 \) N/mm²
Location: Stressed skin floor joists

Glued stressed skin floor joist
with top and bottom ply sheathing
to BS5268-2:2002

Depth of timber joist section \( d = 200 \text{ mm} \)
Width of timber joist section \( b = 47 \text{ mm} \)
Nominal depth of top ply \( \text{dtp}' = 24 \text{ mm} \)
Nominal depth of bottom ply \( \text{dbp}' = 12 \text{ mm} \)
Plywood face grain is perpendicular to span of floor joist.

\[ \begin{aligned}
\text{Joist span} & \quad \text{L} = 6 \text{ m} \\
\text{Centres of joists} & \quad \text{crs} = 600 \text{ mm} \\
\text{Effective width of top flange} & \quad \text{ewidth} = 600 \text{ mm}
\end{aligned} \]

**Grade stresses**

Strength class C24 to Table 8.
Plywood is Finnish Birch-faced (sanded) to Table 49 of BS5268
Top plywood sheath comprises 17 plies.
Bottom plywood sheath comprises 9 plies.
Timber service class adopted \( \text{tmclass} = 1 \)
Timber service class modification factor \( \text{K2} = 1 \) as Table 16
Member is in a load-sharing system as defined by Clause 13.
Load-sharing modification factor \( \text{K8} = 1.1 \)
The mean modulus of elasticity should be used to calculate deflections and displacements under both dead and imposed loads.

**Loading - UDL**

Other dead UDL \( Gkux' = 0.183 \text{ kN/m}^2 \)
Imposed load \( Qkux = 1.5 \text{ kN/m} \)

**Analysis results**

Unfactored dead shear \( \text{Rudlhe} = Gk \cdot L / 2 = 0.91568 \text{ kN} \)
Unfactored imposed shear \( \text{Ruilhe} = Qk \cdot L / 2 = 2.7 \text{ kN} \)
Shear force \( Fve = (Gk+Qk) \cdot L / 2 = 3.6157 \text{ kN} \)
Maximum span bending moment \( M = (Gk+Qk) \cdot L^2 / 8 = 5.4235 \text{ kNm} \)
Duration of loading \( K3 = 1.25 \)
Depth factor \( K7 = (300/d)^{0.11} = 1.0456 \)
Width modification factor \( K14 = (300/h)^{0.11} = 1.0456 \)
No of spans which ply is continuous over plycon=3
Transverse stresses and deflection to be based on the clear span of ply.

**Rolling shear stress at ply/joist interface (position B)**

Web to flange connection is to be by glueing. Since $E_{ayfa} \geq E_{ayfb}$ (4.9686E9 $\geq$ 3.1897E9) the 1st moment of area of the top flange is greater than the 1st moment of area of the bottom flange. Rolling shear will be more critical at the top ply

1st moment of area of critical flange $E_{aroll}=E_{ayfa}=4.9686E9$ Nmm
Rolling shear stress @ position B $\tau_{rB}=Fve*1E3*E_{aroll}/E_{Ix}/conwid=0.31024$ N/mm²

Plywood is to be adhered to timber using nails or staples as the bonding pressure. Mod factor $K_70=0.9$
Since $rollp \geq \tau_{rB}$ (0.76106 $\geq$ 0.31024) the applied rolling shear is less than the permissible shear. Hence OK.
Length of bearing $lb=100$ mm
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor $K_4=1.0$

<table>
<thead>
<tr>
<th>Stresses due to bending action</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DESIGN</strong></td>
</tr>
<tr>
<td>Compress @ Top face of top ply</td>
</tr>
<tr>
<td>Perm Compress stress in top ply</td>
</tr>
<tr>
<td>Bending in joist @ joist/top ply</td>
</tr>
<tr>
<td>Bending in joist @ joist/btm ply</td>
</tr>
<tr>
<td>Perm bending stress in joist</td>
</tr>
<tr>
<td>Tension @ btm face of btm ply</td>
</tr>
<tr>
<td>Perm tension stress in btm ply</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bending stress in top ply</th>
</tr>
</thead>
<tbody>
<tr>
<td>spanning between timber joist</td>
</tr>
<tr>
<td>Perm ply bending stress</td>
</tr>
<tr>
<td>Transverse deflection of top ply</td>
</tr>
<tr>
<td>Permissible transverse deflection</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Deflection of composite section</th>
</tr>
</thead>
<tbody>
<tr>
<td>17.533 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Limiting deflection of comp sect</th>
</tr>
</thead>
<tbody>
<tr>
<td>18 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Lateral shear @ neutral axis</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.41804 N/mm²</td>
</tr>
<tr>
<td>Permiss. lateral shear in joist</td>
</tr>
<tr>
<td>Ply flange to joist shear stress</td>
</tr>
<tr>
<td>Permissible rolling shear in ply</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bearing stress on ply</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.060261 N/mm²</td>
</tr>
<tr>
<td>Permissible ply bearing stress</td>
</tr>
<tr>
<td>Bearing stress on joist</td>
</tr>
<tr>
<td>Permissible joist bearing stress</td>
</tr>
</tbody>
</table>
Location: Stressed skin floor joists

Glued stressed skin floor joist
with top and bottom ply sheathing
to BS5268-2:2002

Depth of timber joist section  d=150 mm
Width of timber joist section  b=50 mm
Nominal depth of top ply      dtp'=15.5 mm
Nominal depth of bottom ply   dbp'=9.5 mm
Plywood face grain is parallel to span of floor joist.

Joist span                  L=3.6 m
Centres of joists           crs=360 mm
Effective width of top flange  ewidth=360 mm

Grade stresses

Strength class C24 to Table 8.
User defined plywood grade stresses.
Minimum thickness of top ply  dtp=15 mm
Minimum thickness of bottom ply dbp=9 mm
Bending: grain \perp to ply span  pbperd=4.73 N/mm²
Compress \perp to face grain  pcperd=3.84 N/mm²
Compress \parallel to face grain  pcpar=8.13 N/mm²
Rolling shear  prolls=0.51 N/mm²
Face grain \perp to span  Emper=3150 N/mm²
Face grain \perp to ply span  Ebper=1055 N/mm²
Panel shear modulus of rigidity  Gw=255 N/mm²
Tension \perp to face grain  ptbperd=4.97 N/mm²
Face grain \parallel to the span  Emperpar=2920 N/mm²
Bearing stress on ply face  beargs=2.16 N/mm²
Timber service class adopted  tmclass=1
Timber service class modification factor K2=1 as Table 16
Member is not in a load-sharing system as defined by Clause 13.
Load-sharing modification factor  K8=1.0
Loading - UDL

- Density of top ply: \( \text{gamtply} = 8.91 \, \text{kg/m}^2 \)
- Density of bottom ply: \( \text{gambply} = 4.3 \, \text{kg/m}^2 \)
- Other dead UDL: \( G_{kux'} = 0.1 \, \text{kN/m}^2 \)
- Imposed load: \( Q_{kux} = 1.75 \, \text{kN/m}^2 \)

Analysis results

- Unfactored dead shear: \( R_{udlhe} = \frac{G_k \times L}{2} = 0.20816 \, \text{kN} \)
- Unfactored imposed shear: \( R_{uilhe} = \frac{Q_k \times L}{2} = 1.134 \, \text{kN} \)
- Shear force: \( F_{ve} = (G_k + Q_k) \times \frac{L}{2} = 1.3422 \, \text{kN} \)
- Maximum span bending moment: \( M = (G_k + Q_k) \times \frac{L^2}{8} = 1.2079 \, \text{kNm} \)
- Duration of loading: \( K_3 = 1.5 \)
- Depth factor: \( K_7 = (300/d)^{0.11} = 1.0792 \)
- Width modification factor: \( K_{14} = (300/h)^{0.11} = 1.0792 \)

No of spans which ply is continuous over \( \text{plycon} = 2 \)
Transverse stresses and deflection to be based on the effective

Rolling shear stress at ply/joist interface (position B)

Web to flange connection is to be by glueing.
Since \( E_{ayfa} \geq E_{ayfb} \) (1.2657E9 ≥ 828.69E6) the 1st moment of area of the top flange is greater than the 1st moment of area of the bottom flange. Rolling shear will be more critical at the top ply

1st moment of area of critical flange \( E_{aroll} = E_{ayfa} = 1.2657E9 \, \text{Nm} \)
Rolling shear stress @ position B \( \tau_{orB} = \frac{F_{ve} \times 1E3 \times E_{aroll}}{E_{Ix} \times \text{conwid}} = 0.12494 \, \text{N/mm}^2 \)

Plywood is to be adhered to timber using clamps under pressure.
Bonding pressure Mod factor: \( K_{70} = 1.0 \)
Since \( rollp \geq \tau_{orB} \) (0.3825 ≥ 0.12494) the applied rolling shear is less than the permissible shear. Hence OK.
Length of bearing: \( l_b = 75 \, \text{mm} \)
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor: \( K_4 = 1.0 \)
### Stresses due to bending action

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Compress @ Top face of top ply</td>
<td>1.1461 N/mm²</td>
</tr>
<tr>
<td>Perm Compress stress in top ply</td>
<td>12.195 N/mm²</td>
</tr>
<tr>
<td>Bending in joist @ joist/top ply</td>
<td>2.1399 N/mm²</td>
</tr>
<tr>
<td>Bending in joist @ joist/btm ply</td>
<td>2.6575 N/mm²</td>
</tr>
<tr>
<td>Perm bending stress in joist</td>
<td>12.141 N/mm²</td>
</tr>
<tr>
<td>Tension @ btm face of btm ply</td>
<td>1.1945 N/mm²</td>
</tr>
<tr>
<td>Perm tension stress in btm ply</td>
<td>7.455 N/mm²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress in top ply spanning between timber joist</td>
<td>0.89477 N/mm²</td>
</tr>
<tr>
<td>Perm ply bending stress</td>
<td>7.095 N/mm²</td>
</tr>
<tr>
<td>Transverse deflection of top ply</td>
<td>0.63376 mm</td>
</tr>
<tr>
<td>Permissible transverse deflection</td>
<td>1.08 mm</td>
</tr>
<tr>
<td>Deflection of composite section</td>
<td>6.4263 mm</td>
</tr>
<tr>
<td>Limiting deflection of comp sect</td>
<td>10.8 mm</td>
</tr>
<tr>
<td>Lateral shear @ neutral axis</td>
<td>0.20448 N/mm²</td>
</tr>
<tr>
<td>Permiss. lateral shear in joist</td>
<td>1.065 N/mm²</td>
</tr>
<tr>
<td>Ply flange to joist shear stress</td>
<td>0.12494 N/mm²</td>
</tr>
<tr>
<td>Permissible rolling shear in ply</td>
<td>0.3825 N/mm²</td>
</tr>
</tbody>
</table>

| Bearing stress on ply                     | 0.04971 N/mm² |
| Permissible ply bearing stress            | 3.24 N/mm² |
| Bearing stress on joist                   | 0.35791 N/mm² |
| Permissible joist bearing stress          | 3.6 N/mm² |
Location: Stressed skin floor joists

![Glued stressed skin floor joist with top ply sheathing](image)

- Depth of timber joist section \( d = 225 \text{ mm} \)
- Width of timber joist section \( b = 75 \text{ mm} \)
- Nominal depth of top ply \( \text{d}tp' = 20.5 \text{ mm} \)
- Plywood face grain is parallel to span of floor joist.

Depth of timber joist section \( d = 225 \text{ mm} \)
Width of timber joist section \( b = 75 \text{ mm} \)
Nominal depth of top ply \( \text{d}tp' = 20.5 \text{ mm} \)

Plywood face grain is parallel to span of floor joist.

- Joist span \( L = 5.5 \text{ m} \)
- Centres of joists \( \text{crs} = 330 \text{ mm} \)
- Effective width of top flange \( \text{ewidth} = 330 \text{ mm} \)

### Grade stresses

Strength class C24 to Table 8.
Plywood is Canadian Douglas Fir (unsanded) to Table 45 of BS5268
Top plywood sheath comprises 7 plies.
Timber service class adopted \( \text{tmclass} = 1 \)
Timber service class modification factor \( K_2 = 1 \) as Table 16
Member is in a load-sharing system as defined by Clause 13.
Load-sharing modification factor \( K_8 = 1.1 \)
The mean modulus of elasticity should be used to calculate deflections and displacements under both dead and imposed loads.

### Loading - UDL

Other dead UDL \( Gkux' = 0.10091 \text{ kN/m}^2 \)
Imposed load \( Qkux = 2.5 \text{ kN/m}^2 \)

### Analysis results

- Unfactored dead shear \( \text{Rudlhe} = Gk \times \frac{L}{2} = 0.3589 \text{ kN} \)
- Unfactored imposed shear \( \text{Ruilhe} = Qk \times \frac{L}{2} = 2.2687 \text{ kN} \)
- Shear force \( Fv = (Gk + Qk) \times \frac{L}{2} = 2.6276 \text{ kN} \)
- Maximum span bending moment \( M = (Gk + Qk) \times \frac{L^2}{8} = 3.613 \text{ kNm} \)
- Duration of loading \( K_3 = 1 \)
- Depth factor \( K_7 = (300/d)^{0.11} = 1.0322 \)
- Width modification factor \( K_{14} = (300/h)^{0.11} = 1.0322 \)

No of spans which ply is continuous over \( \text{plycon} = 4 \)
Transverse stresses and deflection to be based on the clear span of ply.
Rolling shear stress at ply/Joist interface (position B)

Web to flange connection is to be by glueing.

1st moment of area of top flange  \( \text{Earoll} = EAtf \times y5 = 2.1779 \times 10^9 \text{ Nmm} \)

Rolling shear stress @ position B

\[
\tau_B = \frac{Fve \times 10^3 \times \text{Earoll}}{EIx \times \text{conwid}} = 0.07363 \text{ N/mm}^2
\]

Plywood is to be adhered to timber using nails or staples as the bonding pressure. Mod factor \( K70 = 0.9 \)

Since \( rollp \geq \tau_B \ (0.25245 \geq 0.07363) \) the applied rolling shear is less than the permissible shear. Hence OK.

Length of bearing \( lb = 100 \text{ mm} \)

From BS5268-2 Table 18, bearing is < 75 mm from joist end.

Bearing Modification factor \( K4 = 1.0 \)

<table>
<thead>
<tr>
<th>Stresses due to bending action</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESIGN SUMMARY</td>
</tr>
<tr>
<td>Compress @ Top face of top ply</td>
</tr>
<tr>
<td>Perm Compress stress in top ply</td>
</tr>
<tr>
<td>Bending in joist @ joist/top ply</td>
</tr>
<tr>
<td>Bending in joist @ joist/btm ply</td>
</tr>
<tr>
<td>Perm bending stress in joist</td>
</tr>
<tr>
<td>Bending stress in top ply spanning between timber joist</td>
</tr>
<tr>
<td>Perm ply bending stress</td>
</tr>
<tr>
<td>Transverse deflection of top ply</td>
</tr>
<tr>
<td>Permissible transverse deflection</td>
</tr>
<tr>
<td>Deflection of composite section</td>
</tr>
<tr>
<td>Limiting deflection of comp sect</td>
</tr>
<tr>
<td>Lateral shear @ neutral axis</td>
</tr>
<tr>
<td>Permiss. lateral shear in joist</td>
</tr>
<tr>
<td>Ply flange to joist shear stress</td>
</tr>
<tr>
<td>Permissible rolling shear in ply</td>
</tr>
<tr>
<td>Bearing stress on joist</td>
</tr>
<tr>
<td>Permissible joist bearing stress</td>
</tr>
</tbody>
</table>
Location: Stressed skin floor joists

Glued stressed skin floor joist with top and bottom ply sheathing to EC5 with NA Amendment No.2

Depth of timber joist section \( h=200 \text{ mm} \)
Width of timber joist section \( b=63 \text{ mm} \)
Nominal depth of top ply \( h_{tp}'=22.5 \text{ mm} \)
Nominal depth of bottom ply \( h_{bp}'=12.5 \text{ mm} \)
Plywood surface grain is perpendicular to span of floor joist.

Joist span \( L=5 \text{ m} \)
Bearing length \( lb=100 \text{ mm} \)
Centres of joists \( crs=400 \text{ mm} \)
Effective width of top flange \( ewidth=400 \text{ mm} \)

Characteristic strength properties

Timber: Based on Table 1 & 3 in BS EN 338:2016(E).
Timber strength class C24.
Plywood:
Plywood is Canadian Douglas Fir (unsanded).
Top plywood sheath comprises 7 plies.
Bottom plywood sheath comprises 4 plies.

Member is in a load-sharing system as defined in EC5 Clause 6.6.
Load-sharing modification factor \( k_{sys}=1.1 \)
The mean modulus of elasticity should be used to calculate deflections and displacements under both permanent and imposed loads.
Bending moments and shear forces

Simply supported beam subject to vertical loads.

Beam span \( L = 5 \text{ m} \)
Distance from left support \( L_c(1) = 2.5 \text{ m} \)
Permanent load (unfactored) \( G_{kc}(1) = 0.5 \text{ kN} \)
Variable load 1 (unfactored) \( Q_{ca}(1) = 0.5 \text{ kN} \)
Dist. from left support to start \( L_a(1) = 0 \text{ m} \)
Distance from left support to end \( L_{bu}(1) = 5 \text{ m} \)
Permanent load (unfactored) \( G_{ku}(1) = 0.18 \text{ kN/m} \)
Variable load 1 (unfactored) \( Q_{ua}(1) = 0.6 \text{ kN/m} \)

Characteristic values of actions

Permanent:
Reaction at left hand end \( 0.7 \text{ kN} \)
Reaction at right hand end \( 0.7 \text{ kN} \)
Maximum span moment \( M_{kp} = 1.1875 \text{ kNm} \)
Maximum shear force \( F_{kp} = 0.7 \text{ kN} \)
Variable 1:
Reaction at left hand end \( 1.75 \text{ kN} \)
Reaction at right hand end \( 1.75 \text{ kN} \)
Maximum span moment \( M_{kv1} = 2.5 \text{ kNm} \)
Maximum shear force \( F_{kv1} = 1.75 \text{ kN} \)
Depth factor (Clause 3.2) \( k_h = 1.0 \)
Width modification factor \( k_w = (150/lardim)^{0.2} = 0.94409 \)
Width modification factor \( k_w = 1.0 \)
Bearing modification factor \( k_{c90} = 1.5 \)
Shear reduction factor \( k_{sh} = 0.67 \)
No of spans which ply is continuous over \( plycon = 3 \)
Transverse stresses and deflection to be based on the clear span of ply.
Permanent udl load on floor \( G_{udl} = 0.45 \text{ kN/m}^2 \)
Variable udl load on floor \( Q_{udl} = 1.5 \text{ kN/m}^2 \)
Permanent max point load on floor \( G_{pl} = 0.5 \text{ kN} \)
Variable max point load on floor \( Q_{pl} = 0.5 \text{ kN} \)
Rolling shear stress at sheathing/joist interface (position B)

Web to flange connection is to be by glueing. Since Eayfa ≥ Eayfb (3.2451E9 ≥ 2.5475E9) the 1st moment of area of the top flange is greater than the 1st moment of area of the bottom flange. Rolling shear will be more critical at the top sheathing.

1st moment of area of critical flange Earoll = Eayfa = 3.2451E9 Nmm

Design rolling shear stress at B:

Design Stresses due to bending

| DESIGN          | Compress @ top face of top ply | 1.9922 N/mm² |
| SUMMARY         | Design compr.strength in top ply | 5.6667 N/mm² |
|                 | Bending in joist @ joist/top ply | 5.1127 N/mm² |
|                 | Bending in joist @ joist/btm ply | 5.6546 N/mm² |
|                 | Design bending strength in joist | 16.246 N/mm² |
|                 | Tension @ btm face of btm ply   | 2.7379 N/mm² |
|                 | Design tension strength (btm ply) | 4.2667 N/mm² |
|                 | Design bending stress in top ply spanning between timber joist | 1.0728 N/mm² |
|                 | Design ply bending strength     | 16.867 N/mm² |
|                 | Transverse deflection of top ply | 0.054869 mm |
|                 | Allowable transverse deflection | 1.011 mm    |
|                 | Deflection of composite section | 14.032 mm   |
|                 | Limiting deflection of comp.sect | 20 mm      |
|                 | Lateral shear @ neutral axis    | 0.33003 N/mm² |
|                 | Lateral shear strength in joist | 1.8142 N/mm² |
|                 | Ply flange to joist shear stress | 0.16813 N/mm² |
|                 | Rolling shear strength of ply   | 0.85067 N/mm² |
|                 | Bearing stress on ply           | 0.08925 N/mm² |
|                 | Design ply bearing strength     | 1.44 N/mm²   |
|                 | Bearing stress on joist         | 0.56667 N/mm² |
|                 | Design bearing strength of joist| 2.5385 N/mm² |
Location: Stressed skin floor joists

Glued stressed skin floor joist with top and bottom ply sheathing to EC5 with NA Amendment No.2

Depth of timber joist section \( h = 200 \text{ mm} \)
Width of timber joist section \( b = 47 \text{ mm} \)
Nominal depth of top ply \( h_{tp}' = 24 \text{ mm} \)
Nominal depth of bottom ply \( h_{bp}' = 12 \text{ mm} \)
Plywood surface grain is perpendicular to span of floor joist.

Joist span \( L = 6 \text{ m} \)
Bearing length \( l_{b} = 100 \text{ mm} \)
Centres of joists \( c_{rs} = 600 \text{ mm} \)
Effective width of top flange \( e_{\text{width}} = 600 \text{ mm} \)

Characteristic strength properties

Timber: Based on Table 1 & 3 in BS EN 338:2016(E).
Timber strength class C24.
Plywood:
- Plywood is Finnish Birch-faced (sanded).
- Top plywood sheath comprises 17 plies.
- Bottom plywood sheath comprises 9 plies.

Member is in a load-sharing system as defined in EC5 Clause 6.6.
Load-sharing modification factor \( k_{sys} = 1.1 \)
The mean modulus of elasticity should be used to calculate deflections and displacements under both permanent and imposed loads.
Bending moments and shear forces

Simply supported beam subject to vertical loads.

Beam span \( L = 6 \text{ m} \)
Distance from left support to start \( L_{\text{au}}(1) = 0 \text{ m} \)
Distance from left support to end \( L_{\text{bu}}(1) = 6 \text{ m} \)
Permanent load (unfactored) \( G_{k\text{u}}(1) = 0.24 \text{ kN/m} \)
Variable load 1 (unfactored) \( Q_{a\text{u}}(1) = 0.9 \text{ kN/m} \)

Characteristic values of actions

 Permanent:
 Reaction at left hand end \( 0.72 \text{ kN} \)
 Reaction at right hand end \( 0.72 \text{ kN} \)
 Maximum span moment \( M_{k\text{p}} = 1.08 \text{ kNm} \)
 Maximum shear force \( F_{k\text{p}} = 0.72 \text{ kN} \)
 Variable 1:
 Reaction at left hand end \( 2.7 \text{ kN} \)
 Reaction at right hand end \( 2.7 \text{ kN} \)
 Maximum span moment \( M_{k\text{v}1} = 4.05 \text{ kNm} \)
 Maximum shear force \( F_{k\text{v}1} = 2.7 \text{ kN} \)
 Depth factor (Clause 3.2) \( k_{\text{h}} = 1.0 \)
 Width modification factor \( k_{w} = (150/l_{\text{ardim}})^{0.2} = 0.94409 \)
 Bearing modification factor \( k_{c90} = 1.5 \)
 Shear reduction factor \( k_{\text{sh}} = 0.67 \)
 No of spans which ply is continuous over \( \text{plycon} = 3 \)
 Transverse stresses and deflection to be based on the clear span of ply.
 Permanent udl load on floor \( G_{\text{udl}} = 0.45 \text{ kN/m}^2 \)
 Variable udl load on floor \( Q_{\text{udl}} = 1.5 \text{ kN/m}^2 \)

Rolling shear stress at sheathing/joist interface (position B)

Web to flange connection is to be by glueing.
Since \( E_{\text{ayfa}} \geq E_{\text{ayfb}} \) (10E9 \( \geq 7.3847E9 \)) the 1st moment of area of the top flange is greater than the 1st moment of area of the bottom flange. Rolling shear will be more critical at the top sheathing.
1st moment of area of critical flange \( E_{\text{aroll}} = E_{\text{ayfa}} = 10E9 \text{ Nmm} \)
Design rolling shear stress at B:
Since \( p_{\text{fvr}} \geq p_{\text{svr}} \) (1.9213 \( \geq 0.47579 \)) the design rolling shear stress is less than the design shear strength. Hence OK.
### Design Stresses due to bending

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARIES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compress @ top face of top ply</td>
<td>2.7657 N/mm²</td>
</tr>
<tr>
<td>Design compr.strength in top ply</td>
<td>16.673 N/mm²</td>
</tr>
<tr>
<td>Bending in joist @ joist/top ply</td>
<td>2.7563 N/mm²</td>
</tr>
<tr>
<td>Bending in joist @ joist/btm ply</td>
<td>4.623 N/mm²</td>
</tr>
<tr>
<td>Design bending strength in joist</td>
<td>16.246 N/mm²</td>
</tr>
<tr>
<td>Tension @ btm face of btm ply</td>
<td>3.7474 N/mm²</td>
</tr>
<tr>
<td>Design tension strength (btm ply)</td>
<td>23.333 N/mm²</td>
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<tr>
<td>Design bending stress in top ply spanning between timber joist</td>
<td>1.0848 N/mm²</td>
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<tr>
<td>Design ply bending strength</td>
<td>25.933 N/mm²</td>
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<td>Transverse deflection of top ply</td>
<td>0.14033 mm</td>
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<tr>
<td>Allowable transverse deflection</td>
<td>1.659 mm</td>
</tr>
<tr>
<td>Deflection of composite section</td>
<td>15.114 mm</td>
</tr>
<tr>
<td>Limiting deflection of comp sect</td>
<td>24 mm</td>
</tr>
<tr>
<td>Lateral shear @ neutral axis</td>
<td>0.54443 N/mm²</td>
</tr>
<tr>
<td>Lateral shear strength in joist</td>
<td>1.8142 N/mm²</td>
</tr>
<tr>
<td>Ply flange to joist shear stress</td>
<td>0.47579 N/mm²</td>
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<tr>
<td>Rolling shear strength of ply</td>
<td>1.9213 N/mm²</td>
</tr>
<tr>
<td>Bearing stress on ply</td>
<td>0.0837 N/mm²</td>
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<tr>
<td>Design ply bearing strength</td>
<td>2.62 N/mm²</td>
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<tr>
<td>Bearing stress on joist</td>
<td>1.0685 N/mm²</td>
</tr>
<tr>
<td>Design bearing strength of joist</td>
<td>2.5385 N/mm²</td>
</tr>
</tbody>
</table>
Location: Stressed skin floor joists

Glued stressed skin floor joist with top and bottom ply sheathing to EC5 with NA Amendment No.2

Depth of timber joist section \( h=150 \) mm
Width of timber joist section \( b=50 \) mm
Nominal depth of top ply \( h_{tp}'=15.5 \) mm
Nominal depth of bottom ply \( h_{bp}'=9.5 \) mm
Plywood surface grain is parallel to span of floor joist.

Joist span \( L=3.6 \) m
Bearing length \( l_{b}=75 \) mm
Centres of joists \( c_{rs}=360 \) mm
Effective width of top flange \( e_{\text{width}}=360 \) mm

Characteristic strength properties

Timber: Based on Table 1 & 3 in BS EN 338:2016(E).
Timber strength class C24.

Plywood:
User defined sheathing strength properties.
Minimum thickness of top ply \( h_{tp}=15 \) mm
Minimum thickness of bottom ply \( h_{bp}=9 \) mm
Bending: grain \( \perp \) to ply span \( p_{f90k}=15 \) N/mm²
Compression // to face grain \( p_{fc0k}=25 \) N/mm²
Compression \( \perp \) to face grain \( p_{fc90k}=24 \) N/mm²
Rolling shear \( p_{fvrk}=3.3 \) N/mm²
Surface grain // to span \( E_{mp}=3150 \) N/mm²
Surface grain \( \perp \) to ply span \( E_{bper}=1055 \) N/mm²
Panel shear modulus of rigidity \( G_{w}=255 \) N/mm²
Tension // to face grain \( p_{bf0k}=10 \) N/mm²
Surface grain \( \perp \) to the span \( E_{bpar}=2920 \) N/mm²
Bearing stress on sheathing face \( b_{args}=2.16 \) N/mm²

The mean modulus of elasticity should be used to calculate deflections and displacements under both permanent and imposed loads.
Timber modulus of elasticity \( E_{t}=E_{0}\text{mean}=11000 \) N/mm²
Member is not in a load-sharing system as defined in EC5 Clause 6.6.
Load-sharing modification factor \( k_{sys}=1.0 \)
Bending moments and shear forces

Simply supported beam subject to vertical loads.

Beam span $L = 3.6 \text{ m}$
Dist. from left support to start $L_{au(1)} = 0 \text{ m}$
Distance from left support to end $L_{bu(1)} = 3.6 \text{ m}$
Permanent load (unfactored) $G_{ku(1)} = 0.108 \text{ kN/m}$
Variable load 1 (unfactored) $Q_{u(1)} = 0.63 \text{ kN/m}$

Characteristic values of actions

Permanent:
Reaction at left hand end $0.1944 \text{ kN}$
Reaction at right hand end $0.1944 \text{ kN}$
Maximum span moment $M_{kp} = 0.17496 \text{ kNm}$
Maximum shear force $F_{kp} = 0.1944 \text{ kN}$
Variable 1:
Reaction at left hand end $1.134 \text{ kN}$
Reaction at right hand end $1.134 \text{ kN}$
Maximum span moment $M_{kv1} = 1.0206 \text{ kNm}$
Maximum shear force $F_{kv1} = 1.134 \text{ kN}$
Depth factor (Clause 3.2) $k_{h} = 1.0$
Width modification factor $k_{w} = (150/lardim)^{0.2} = 1$
Bearing modification factor $k_{c90} = 1.5$
Shear reduction factor $k_{sh} = 0.67$
No of spans which ply is continuous over $\text{plycon} = 2$
Transverse stresses and deflection to be based on the effective
Permanent udl load on floor $G_{udl} = 0.3 \text{ kN/m}^2$
Variable udl load on floor $Q_{udl} = 1.73 \text{ kN/m}^2$

Rolling shear stress at sheathing/joist interface (position B)

Web to flange connection is to be by glueing.
Since $E_{ayfa} \geq E_{ayfb}$ (1.3017E9 $\geq$ 808.67E6) the 1st moment of area of the top flange is greater than the 1st moment of area of the bottom flange. Rolling shear will be more critical at the top sheathing.

1st moment of area of critical flange $E_{aroll} = E_{ayfa} = 1.3017E9 \text{ Nm}$

Design rolling shear stress at B:
Since $p_{fvr} \geq p_{svr}$ (2.475 $\geq$ 0.15644) the design rolling shear stress is less than the design shear strength. Hence OK.
### Design Stresses due to Bending

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compress @ top face of top ply</td>
<td>1.4314 N/mm²</td>
</tr>
<tr>
<td>Design compr. strength in top ply</td>
<td>18.75 N/mm²</td>
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<tr>
<td>Bending in joist @ joist/top ply</td>
<td>4.1062 N/mm²</td>
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<tr>
<td>Bending in joist @ joist/btm ply</td>
<td>4.8172 N/mm²</td>
</tr>
<tr>
<td>Design bending strength in joist</td>
<td>16.615 N/mm²</td>
</tr>
<tr>
<td>Tension @ btm face of btm ply</td>
<td>1.4209 N/mm²</td>
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<tr>
<td>Design tension strength (btm ply)</td>
<td>7.5 N/mm²</td>
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<tr>
<td>Design bending stress in top ply spanning between timber joist</td>
<td>1.296 N/mm²</td>
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<tr>
<td>Design ply bending strength</td>
<td>11.25 N/mm²</td>
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<tr>
<td>Transverse deflection of top ply</td>
<td>0.62114 mm</td>
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<tr>
<td>Allowable transverse deflection</td>
<td>1.08 mm</td>
</tr>
<tr>
<td>Deflection of composite section</td>
<td>8.9037 mm</td>
</tr>
<tr>
<td>Limiting deflection of comp.sect</td>
<td>14.4 mm</td>
</tr>
<tr>
<td>Lateral shear @ neutral axis</td>
<td>0.3139 N/mm²</td>
</tr>
<tr>
<td>Lateral shear strength in joist</td>
<td>1.8554 N/mm²</td>
</tr>
<tr>
<td>Ply flange to joist shear stress</td>
<td>0.15644 N/mm²</td>
</tr>
<tr>
<td>Rolling shear strength of ply</td>
<td>2.475 N/mm²</td>
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<tr>
<td>Bearing stress on ply</td>
<td>0.07272 N/mm²</td>
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<tr>
<td>Design ply bearing strength</td>
<td>1.62 N/mm²</td>
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<tr>
<td>Bearing stress on joist</td>
<td>0.52358 N/mm²</td>
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<tr>
<td>Design bearing strength of joist</td>
<td>2.5962 N/mm²</td>
</tr>
</tbody>
</table>
Location: Stressed skin floor joists

Depth of timber joist section \( h = 225 \text{ mm} \)
Width of timber joist section \( b = 75 \text{ mm} \)
Nominal depth of top ply \( \text{htp}' = 20.5 \text{ mm} \)
Plywood surface grain is parallel to span of floor joist.

Joist span \( L = 5.5 \text{ m} \)
Bearing length \( l_b = 100 \text{ mm} \)
Centres of joists \( c_{rs} = 330 \text{ mm} \)
Effective width of top flange \( e_{\text{width}} = 330 \text{ mm} \)

Characteristic strength properties

Timber: Based on Table 1 & 3 in BS EN 338:2016(E).
Timber strength class C24.
Plywood:
Plywood is Canadian Douglas Fir (unsanded).
Top plywood sheath comprises 7 plies.

Member is in a load-sharing system as defined in EC5 Clause 6.6.
Load-sharing modification factor \( k_{\text{sys}} = 1.1 \)
The mean modulus of elasticity should be used to calculate
deflections and displacements under both permanent and imposed loads.
Bending moments and shear forces

Simply supported beam subject to vertical loads.

Beam span \( L = 5.5 \) m
Distance from left support to start \( \text{Lau}(1) = 0 \) m
Distance from left support to end \( \text{Lbu}(1) = 5.5 \) m
Permanent load (unfactored) \( \text{Gku}(1) = 0.099 \) kN/m
Variable load 1 (unfactored) \( \text{Qua}(1) = 0.8415 \) kN/m

Characteristic values of actions

Permanent:
Reaction at left hand end \( 0.27225 \) kN
Reaction at right hand end \( 0.27225 \) kN
Maximum span moment \( M_{kP} = 0.37434 \) kNm
Maximum shear force \( F_{kP} = 0.27225 \) kN
Variable 1:
Reaction at left hand end \( 2.3141 \) kN
Reaction at right hand end \( 2.3141 \) kN
Maximum span moment \( M_{kv1} = 3.1819 \) kNm
Maximum shear force \( F_{kv1} = 2.3141 \) kN
Depth factor (Clause 3.2) \( k_h = 1.0 \)
Width modification factor \( k_w = (150/\text{lardim})^{0.2} = 0.92211 \)
Bearing modification factor \( k_c90 = 1.5 \)
Shear reduction factor \( k_{sh} = 0.67 \)

No of spans which ply is continuous over \( \text{plycon} = 4 \)
Transverse stresses and deflection to be based on the clear span of ply.
Permanent udl load on floor \( \text{Gudl} = 0.3 \) kN/m²
Variable udl load on floor \( \text{Qudl} = 2.5 \) kN/m²

Rolling shear stress at sheathing/joist interface (position B)

Web to flange connection is to be by gluing.
1st moment of area of top flange \( \text{Earoll} = \text{EAtf} \cdot \text{y5} = 3.9816 \) E9 Nmm
Design rolling shear stress at B:
Since \( p_fvrd \geq p_svrd \ (0.7315 \geq 0.16019) \) the design rolling shear stress is less than the design shear strength. Hence OK.

Design Stresses due to bending

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th></th>
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</thead>
<tbody>
<tr>
<td>Compress @ top face of top ply</td>
<td>2.7507 N/mm²</td>
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<tr>
<td>Design compr. strength in top ply</td>
<td>9.1 N/mm²</td>
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<tr>
<td>Bending in joist @ joist/top ply</td>
<td>4.1554 N/mm²</td>
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<tr>
<td>Bending in joist @ joist/btm ply</td>
<td>6.1133 N/mm²</td>
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<tr>
<td>Design bending strength in joist</td>
<td>14.215 N/mm²</td>
</tr>
<tr>
<td>Description</td>
<td>Value</td>
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<tr>
<td>-------------------------------------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>Design bending stress in top ply spanning between timber joist</td>
<td>0.55207 N/mm²</td>
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<tr>
<td>Design ply bending strength</td>
<td>6.3583 N/mm²</td>
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<tr>
<td>Transverse deflection of top ply</td>
<td>0.052547 mm</td>
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<tr>
<td>Allowable transverse deflection</td>
<td>0.765 mm</td>
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<td>Deflection of composite section</td>
<td>14.768 mm</td>
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<td>Limiting deflection of comp. sect</td>
<td>22 mm</td>
</tr>
<tr>
<td>Lateral shear @ neutral axis</td>
<td>0.29777 N/mm²</td>
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<tr>
<td>Lateral shear strength in joist</td>
<td>1.5874 N/mm²</td>
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<tr>
<td>Ply flange to joist shear stress</td>
<td>0.16019 N/mm²</td>
</tr>
<tr>
<td>Rolling shear strength of ply</td>
<td>0.7315 N/mm²</td>
</tr>
<tr>
<td>Bearing stress on joist</td>
<td>0.51183 N/mm²</td>
</tr>
<tr>
<td>Design bearing strength of joist</td>
<td>2.2212 N/mm²</td>
</tr>
</tbody>
</table>
Location: Stressed skin floor joists

Glued stressed skin floor joist with top and bottom OSB sheathing to EC5 with NA Amendment No.2

Depth of timber joist section $h=160$ mm
Width of timber joist section $b=47$ mm
Actual depth of top OSB $htp'=16$ mm
Actual depth of bottom OSB $hbp'=11$ mm
The OSB surface grain is parallel to span of floor joist.

Joist span $L=3.6$ m
Bearing length $lb=75$ mm
Centres of joists $crs=360$ mm
Effective width of top flange $ewidth=360$ mm

Characteristic strength properties

Timber: Based on Table 1 & 3 in BS EN 338:2016(E).
Timber strength class C24.
Oriented Strand Board (OSB):
Oriented strand board of Grade 3 (OSB/3) to BS EN12369-1:2001 and BS EN300:1997.
Oriented strand board will be of service class 1.
User defined sheathing strength properties.
Bending: grain $\perp$ to OSB span $pfm90k=8.2$ N/mm$^2$
Compression $\parallel$ to face grain $pfc0k=15.4$ N/mm$^2$
Compression $\perp$ to face grain $pfc90k=12.7$ N/mm$^2$
Rolling shear $pfvrk=1$ N/mm$^2$
Surface grain $\parallel$ to span $Empar=3800$ N/mm$^2$
Surface grain $\perp$ to OSB span $Ebper=4930$ N/mm$^2$
Panel shear modulus of rigidity $Gw=50$ N/mm$^2$
Tension $\parallel$ to face grain $pbft0k=9.4$ N/mm$^2$
Surface grain $\parallel$ to the span $Empar=3800$ N/mm$^2$
Bearing stress on sheathing face $beargs=2.16$ N/mm$^2$

Member is in a load-sharing system as defined in EC5 Clause 6.6.
Load-sharing modification factor $ksys=1.1$
The mean modulus of elasticity should be used to calculate deflections and displacements under both permanent and imposed loads.
Bending moments and shear forces

Simply supported beam subject to vertical loads.

Beam span \( L = 3.6 \) m
Distance from left support to start \( L_{au(1)} = 0 \) m
Distance from left support to end \( L_{bu(1)} = 3.6 \) m
Permanent load (unfactored) \( G_{ku(1)} = 0.108 \) kN/m
Variable load 1 (unfactored) \( Q_{ua(1)} = 0.63 \) kN/m

Characteristic values of actions

Permanent:
- Reaction at left hand end \( R_{ku} = 0.1944 \) kN
- Reaction at right hand end \( R_{ku} = 0.1944 \) kN
- Maximum span moment \( M_{kp} = 0.17496 \) kNm
- Maximum shear force \( F_{kp} = 0.1944 \) kN

Variable 1:
- Reaction at left hand end \( R_{ku} = 1.134 \) kN
- Reaction at right hand end \( R_{ku} = 1.134 \) kN
- Maximum span moment \( M_{kv1} = 1.0206 \) kNm
- Maximum shear force \( F_{kv1} = 1.134 \) kN

Depth factor (Clause 3.2) \( kh = 1.0 \)
Width modification factor \( kw = \left( \frac{150}{l_{ardim}} \right)^{0.2} = 0.98718 \)
Width modification factor \( kw = 1.0 \)
Bearing modification factor \( kc90 = 1.5 \)
Shear reduction factor \( k_{sh} = 0.67 \)

No of spans which OSB is continuous over \( plycon = 4 \)
Transverse stresses and deflection to be based on the clear span of OSB.

Permanent udl load on floor \( G_{udl} = 0.3 \) kN/m²
Variable udl load on floor \( Q_{udl} = 1.73 \) kN/m²

Rolling shear stress at sheathing/joist interface (position B)

Web to flange connection is to be by glueing.
Since \( E_{ayfa} \geq E_{ayfb} \) (1.8092E9 \( \geq 1.367E9 \)) the 1st moment of area of the top flange is greater than the 1st moment of area of the bottom flange. Rolling shear will be more critical at the top sheathing.

1st moment of area of critical flange \( E_{aroll} = E_{ayfa} = 1.8092E9 \) Nmm
Design rolling shear stress at B:
Since \( pfvd \geq psvd \) (0.825 \( \geq 0.16677 \)) the design rolling shear stress is less than the design shear strength. Hence OK.
### Design Stresses due to bending

<table>
<thead>
<tr>
<th>Category</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compress @ top face of top OSB</td>
<td>1.3433 N/mm²</td>
</tr>
<tr>
<td>Design compr. strength in top OSB</td>
<td>11.55 N/mm²</td>
</tr>
<tr>
<td>Bending in joist @ joist/top OSB</td>
<td>3.2022 N/mm²</td>
</tr>
<tr>
<td>Bending in joist @ joist/btm OSB</td>
<td>3.6607 N/mm²</td>
</tr>
<tr>
<td>Design bending strength in joist</td>
<td>18.277 N/mm²</td>
</tr>
<tr>
<td>Tension @ btm face of btm OSB</td>
<td>1.4276 N/mm²</td>
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<tr>
<td>Design tension strength (btm OSB)</td>
<td>7.05 N/mm²</td>
</tr>
<tr>
<td>Design bending stress in top OSB spanning between timber joist</td>
<td>0.83398 N/mm²</td>
</tr>
<tr>
<td>Design OSB bending strength</td>
<td>6.15 N/mm²</td>
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<tr>
<td>Transverse deflection of top OSB</td>
<td>0.086474 mm</td>
</tr>
<tr>
<td>Allowable transverse deflection</td>
<td>0.939 mm</td>
</tr>
<tr>
<td>Deflection of composite section</td>
<td>12.134 mm</td>
</tr>
<tr>
<td>Limiting deflection of comp sect</td>
<td>14.4 mm</td>
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<tr>
<td>Lateral shear @ neutral axis</td>
<td>0.29959 N/mm²</td>
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<tr>
<td>Lateral shear strength in joist</td>
<td>2.0409 N/mm²</td>
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<tr>
<td>Ply flange to joist shear stress</td>
<td>0.16677 N/mm²</td>
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<tr>
<td>Rolling shear strength of OSB</td>
<td>0.825 N/mm²</td>
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<tr>
<td>Bearing stress on OSB</td>
<td>0.07272 N/mm²</td>
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<tr>
<td>Design OSB bearing strength</td>
<td>1.62 N/mm²</td>
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<tr>
<td>Bearing stress on joist</td>
<td>0.557 N/mm²</td>
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<tr>
<td>Design bearing strength of joist</td>
<td>2.8558 N/mm²</td>
</tr>
</tbody>
</table>
**Location:** Howe truss on grid line D

Span of truss \( L = 8 \text{ m} \)
Effective height of truss \( H = 2 \text{ m} \)
Top chord node point loading \( W = 9 \text{ kN} \)
Bottom chord node point loading \( W_c = 2 \text{ kN} \)

\[
\begin{align*}
0 & \quad 16.5 \\
9 & \quad 9 \text{ kN}
\end{align*}
\]

\[
\begin{align*}
23.335 & \quad -7.5 & \quad 7.7782 & \quad -2 & \quad \text{-ve tension}
\end{align*}
\]

\[
\begin{align*}
-16.5 & \quad -22 \\
2 & \quad 2 \text{ kN}
\end{align*}
\]

Deflection of frame by strain energy unit load method \( 0.0063373 \text{ m} \)

Empirical value

\[
defl = \frac{L^2}{(4290*H)(L/26.6+1)}
\]

\[
= 0.0097026 \text{ m}
\]
Location: Pratt truss on grid line D

Axial loading coefficients for truss

The coefficients of axial loading are based on tables given in the 'Timber Designers' Manual' by Ozelton and Baird.

Four-panel Pratt truss

Span of truss \( L = 8 \text{ m} \)
Effective height of truss \( H = 2 \text{ m} \)
Top chord node point loading \( W = 12 \text{ kN} \)
Bottom chord node point loading \( W_c = 4 \text{ kN} \)

\[
\begin{align*}
\text{Top chord:} & \quad W = 12 \text{ kN} \\
\text{Bottom chord:} & \quad W_c = 4 \text{ kN}
\end{align*}
\]
Central deflection of frame

Applying a unit load at the central node of the frame and using the strain energy formula, deflection = \[ \sum \left( \frac{FUL}{AE} \right) \]

<table>
<thead>
<tr>
<th>Member Load</th>
<th>Unit</th>
<th>Length</th>
<th>Area</th>
<th>Modulus</th>
<th>Symmetry</th>
<th>FUL/AE</th>
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<tr>
<td>1</td>
<td>24</td>
<td>0.5</td>
<td>2</td>
<td>11280</td>
<td>5.98</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>32</td>
<td>1</td>
<td>2</td>
<td>11280</td>
<td>5.98</td>
<td>2</td>
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<tr>
<td>3</td>
<td>-33.9</td>
<td>-0.7</td>
<td>2.828</td>
<td>11280</td>
<td>5.98</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>-11.3</td>
<td>-0.7</td>
<td>2.828</td>
<td>11280</td>
<td>5.98</td>
<td>2</td>
</tr>
<tr>
<td>5</td>
<td>-24</td>
<td>-0.5</td>
<td>2</td>
<td>11280</td>
<td>5.98</td>
<td>2</td>
</tr>
<tr>
<td>6</td>
<td>20</td>
<td>0.5</td>
<td>2</td>
<td>11280</td>
<td>5.98</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>12</td>
<td>1</td>
<td>2</td>
<td>11280</td>
<td>5.98</td>
<td>2</td>
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<tr>
<td>8</td>
<td>24</td>
<td>0.5</td>
<td>2</td>
<td>11280</td>
<td>5.98</td>
<td>2</td>
</tr>
</tbody>
</table>

Deflection of frame under applied loading: 0.0080205 m
Empirical value: \[ \text{def1} = \frac{L^2}{(4290 \times H)} \times (\frac{L}{26.6} + 1) \]
= 0.0097026 m
Location: Fink truss on grid line D

Span of truss \( L = 6 \, \text{m} \)
Rafter point load \( W = 7.6 \, \text{kN} \)
Slope of roof \( \theta' = 21.8^\circ \)

\[ \begin{align*}
-\text{ve tension} & \quad 7.6 \, \text{kN} \\
7.6 & \quad 68.805 \\
6.627 & \quad -9.5007 \\
7.0565 & \quad 14.113 \\
-9.5007 & \quad -19.001 \\
71.627 & \quad 7.0565 \\
-66.505 \, \text{kN} & \quad -57.004 \, \text{kN} & \quad -38.003 \, \text{kN}
\end{align*} \]

Deflection of frame under applied loading \( 0.01444 \, \text{m} \)
Location: Wall Panel Grid Line A/2

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel: h=2500 mm
Total width of panel: w=3000 mm
Applied load at top of panel: RF=4 kN
Applied vertical load: P=5.01 kN
Wall panel comprises C16 stress grade studs with Canadian Douglas Fir plywood sheathing.
Ply sheathing thickness: pst=9.5 mm

Racking resistance

Nail diameter: Dn=3 mm
Nail spacing: Sp=150 mm

Permissible racking shear load taking into account all modification factors of BS5268:Section 6.1:1996 for ply and OSB sheathing is given by the following equation:

PRS=1680*K101*K102*K103*K104*K105*K106*K107*K108=2198.9 N/m

Permissible racking shear load per metre for 9.5 mm sheathing with 3 mm diameter, 50 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 2198.9 N/m.

Since applied shear load (1333.3 N/m) is less than permissible shear load (2198.9 N/m) then panel is adequate.

Over-turning checks

Density of sheathing: mass=5.5 kg/m²
Stud width: b=75 mm
Stud depth: d=100 mm
Stud centres: scrs=400 mm
FOS against overturning: FOS=1.33
Density of stud timber: dstud'=370 kg/m³
Overall weight of panel: panow=stud+ow=1.0798 kN
Combined vertical load: cvl=P+panow=6.0898 kN
Restoring moment due to vert. load $RM = \frac{c\cdot l \cdot w}{2000} = 9.1347$ kNm

Applied over-turning moment 10 kNm

Factor of safety to overturning $OTFOS = \frac{RM}{aotm} = 0.91347$

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Racking shear load 1.3333 kN/m</td>
<td>Permiss shear load 2.1989 kN/m</td>
</tr>
<tr>
<td>Applied overturning moment 10 kNm</td>
<td>Restoring moment 9.1347 kNm</td>
</tr>
<tr>
<td>Req'd uplift strap moment 4.1653 kNm</td>
<td>Sliding load at base 1.3333 kN/m</td>
</tr>
<tr>
<td>Nail diameter 3 mm</td>
<td>Nail Length 50 mm</td>
</tr>
<tr>
<td>Perimeter nail crs 150 mm</td>
<td>Intermediate nail crs 300 mm</td>
</tr>
<tr>
<td>Ply thickness 9.5 mm</td>
<td></td>
</tr>
</tbody>
</table>
Location:

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel \( h = 2800 \text{ mm} \)
Total width of panel \( w = 4000 \text{ mm} \)
Applied load along side of panel \( R_F = 6.01 \text{ kN} \)
Applied vertical load \( P = 3.25 \text{ kN} \)
Wall panel comprises C16 stress grade studs with British Hardwood plywood sheathing.
Ply sheathing thickness \( \text{pst} = 12 \text{ mm} \)

Racking resistance

Nail diameter \( D_n = 3 \text{ mm} \)
Nail spacing \( S_p = 150 \text{ mm} \)

Permissible racking shear load taking into account all modification factors of BS5268:Section 6.1:1996 for ply and OSB sheathing is given by the following equation:

\[
PRS = 1680 \times K_{101} \times K_{102} \times K_{103} \times K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108} = 2427.2 \text{ N/m}
\]

Permissible racking shear load per metre for 12 mm sheathing with 3 mm diameter, 65 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 2427.2 N/m.

Since applied shear load (1502.5 N/m) is less than permissible shear load (2427.2 N/m) then panel is adequate.

Over-turning checks

Density of sheathing \( \text{mass} = 7.32 \text{ kg/m}^2 \)
Stud width \( b = 50 \text{ mm} \)
Stud depth \( d = 125 \text{ mm} \)
Stud centres \( S_{crs} = 450 \text{ mm} \)
FOS against overturning \( \text{FOS} = 1.4 \)
Density of stud timber \( d_{\text{stud}}' = 370 \text{ kg/m}^3 \)
Overall weight of panel \( \text{cvl} = P + \text{panow} = 4.787 \text{ kN} \)
Restoring moment due to vert. load RM = cvl * w / 2000 = 9.574 kNm
Applied over-turning moment 8.414 kNm
Factor of safety to overturning OTFOS = RM / aotm = 1.1379

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Racking shear load</td>
<td>1.5025 kN/m</td>
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<tr>
<td>Permiss shear load</td>
<td>2.4272 kN/m</td>
</tr>
<tr>
<td>Applied overturning moment</td>
<td>8.414 kNm</td>
</tr>
<tr>
<td>Restoring moment</td>
<td>9.574 kNm</td>
</tr>
<tr>
<td>Req'd uplift strap moment</td>
<td>2.2056 kNm</td>
</tr>
<tr>
<td>Sliding load at base</td>
<td>1.5025 kN/m</td>
</tr>
<tr>
<td>Nail diameter</td>
<td>3 mm</td>
</tr>
<tr>
<td>Nail Length</td>
<td>65 mm</td>
</tr>
<tr>
<td>Perimeter nail crs</td>
<td>150 mm</td>
</tr>
<tr>
<td>Intermediate nail crs</td>
<td>300 mm</td>
</tr>
<tr>
<td>Ply thickness</td>
<td>12 mm</td>
</tr>
</tbody>
</table>

SUMMARY

Applied overturning moment 8.414 kNm
Restoring moment           9.574 kNm
Reqd uplift strap moment   2.2056 kNm
Sliding load at base       1.5025 kN/m
Location:

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel \( h = 4035 \text{ mm} \)
Total width of panel \( w = 5050 \text{ mm} \)
Applied load at top of panel \( RF = 7.12 \text{ kN} \)
Applied vertical load \( P = 9.15 \text{ kN} \)
Wall panel comprises C16 stress grade studs with American construction plywood (grade C-D or grade C-C) sheathing.
Ply sheathing thickness \( pst = 9 \text{ mm} \)

Racking resistance

Nail diameter \( Dn = 3 \text{ mm} \)
Nail spacing \( Sp = 150 \text{ mm} \)

Permissible racking shear load taking into account all modification factors of BS5268:Section 6.1:1996 for ply and OSB sheathing is given by the following equation:

\[
PRS = 1680 \times K_{101} \times K_{102} \times K_{103} \times K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108} = 2314.5 \text{ N/m}
\]

Permissible racking shear load per metre for 9 mm sheathing with 3 mm diameter, 50 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 2314.5 N/m.

Since applied shear load (1409.9 N/m) is less than permissible shear load (2314.5 N/m) then panel is adequate.

Over-turning checks

Density of sheathing \( \text{mass} = 4.9 \text{ kg/m}^2 \)
Stud width \( b = 97 \text{ mm} \)
Stud depth \( d = 147 \text{ mm} \)
Stud centres \( scrs = 600 \text{ mm} \)
FOS against overturning \( \text{FOS} = 1.4 \)
Density of stud timber \( dstud' = 370 \text{ kg/m}^3 \)
Overall weight of panel \( \text{panow} = \text{stud} + \text{ow} = 3.2914 \text{ kN} \)
Combined vertical load \( cvl = P + \text{panow} = 12.441 \text{ kN} \)
Restoring moment due to vert. load \( RM = \frac{cvl \cdot w}{2000} = 31.414 \) kNm

Applied over-turning moment \( 28.729 \) kNm

Factor of safety to overturning \( OTFOS = \frac{RM}{aotm} = 1.0935 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Racking shear load</th>
<th>1.4099 kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Permiss shear load</td>
<td>2.3145 kN/m</td>
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<tr>
<td></td>
<td>Applied overturning moment</td>
<td>28.729 kNm</td>
</tr>
<tr>
<td></td>
<td>Restoring moment</td>
<td>31.414 kNm</td>
</tr>
<tr>
<td></td>
<td>Req'd uplift strap moment</td>
<td>8.8064 kNm</td>
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<td></td>
<td>Sliding load at base</td>
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<td></td>
</tr>
<tr>
<td>Nail Length</td>
<td>50 mm</td>
<td></td>
</tr>
<tr>
<td>Perimeter nail crs</td>
<td>150 mm</td>
<td></td>
</tr>
<tr>
<td>Intermediate nail crs</td>
<td>300 mm</td>
<td></td>
</tr>
<tr>
<td>Ply thickness</td>
<td>9 mm</td>
<td></td>
</tr>
</tbody>
</table>
Location:

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel \( h = 2150 \text{ mm} \)
Total width of panel \( w = 1800 \text{ mm} \)
Applied load along side of panel \( RF = 1.57 \text{ kN} \)
Applied vertical load \( P = 1.55 \text{ kN} \)
Wall panel comprises C16 stress grade studs with Finnish Birch plywood sheathing.
Ply sheathing thickness \( pst = 9 \text{ mm} \)

Racking resistance

Nail diameter \( \text{Dn}=3 \text{ mm} \)
Nail spacing \( \text{Sp}=150 \text{ mm} \)

Permissible racking shear load taking into account all modification factors of BS5268:Section 6.1:1996 for ply and OSB sheathing is given by the following equation:

\[
\text{PRS}=1680\text{ }\times\text{ }K101\text{ }\times\text{ }K102\text{ }\times\text{ }K103\text{ }\times\text{ }K104\text{ }\times\text{ }K105\text{ }\times\text{ }K106\text{ }\times\text{ }K107\text{ }\times\text{ }K108=1604.4 \text{ N/m}
\]

Permissible racking shear load per metre for 9 mm sheathing with 3 mm diameter, 50 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 1604.4 N/m.

Since applied shear load (872.22 N/m) is less than permissible shear load (1604.4 N/m) then panel is adequate.

Over-turning checks

Density of sheathing \( \text{mass}=6.2 \text{ kg/m}^2 \)
Stud width \( \text{b}=47 \text{ mm} \)
Stud depth \( \text{d}=97 \text{ mm} \)
Stud centres \( \text{scrs}=450 \text{ mm} \)
FOS against overturning \( \text{FOS}=1.4 \)
Density of stud timber \( \text{dstud}'=370 \text{ kg/m}^3 \)
Overall weight of panel \( \text{panow}=\text{stud}+\text{ow}=0.46506 \text{ kN} \)
Combined vertical load \( \text{cvl}=P+\text{panow}=2.0151 \text{ kN} \)
Restoring moment due to vert. load RM=csvl*w/2000=1.8136 kNm

Applied over-turning moment 1.6878 kNm

Factor of safety to overturning OTFOS=RM/aotm=1.0745

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
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<td>Racking shear load</td>
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<tr>
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<td>1.6044 kN/m</td>
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<tr>
<td>Applied overturning moment</td>
<td>1.6878 kNm</td>
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<td>Restoring moment</td>
<td>1.8136 kNm</td>
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<td>Req'd uplift strap moment</td>
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<tr>
<td>Nail Length</td>
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<tr>
<td>Perimeter nail crs</td>
<td>150 mm</td>
</tr>
<tr>
<td>Intermediate nail crs</td>
<td>300 mm</td>
</tr>
<tr>
<td>Ply thickness</td>
<td>9 mm</td>
</tr>
</tbody>
</table>
Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

- Height of panel, $h = 6000$ mm
- Total width of panel, $w = 6750$ mm
- Applied load at top of panel, $RF = 18.5$ kN
- Applied vertical load, $P = 15.5$ kN
- Wall panel comprises C16 stress grade studs with Swedish Softwood Construction plywood sheathing.
- Ply sheathing thickness, $p_{st} = 12.5$ mm

### Racking resistance

Nail diameter, $D_n = 3$ mm
Nail spacing, $S_p = 150$ mm

Permissible racking shear load taking into account all modification factors of BS5268:Section 6.1:1996 for ply and OSB sheathing is given by the following equation:

$$ PRS = 1680 \times K_{101} \times K_{102} \times K_{103} \times K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108} = 2791 \text{ N/m} $$

Permissible racking shear load per metre for 12.5 mm sheathing with 3 mm diameter, 75 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 2791 N/m.

Since applied shear load (2740.7 N/m) is less than permissible shear load (2791 N/m) then panel is adequate.

### Over-turning checks

- Density of sheathing, $\text{mass} = 5.3$ kg/m²
- Stud width, $b = 63$ mm
- Stud depth, $d = 197$ mm
- Stud centres, $s_{crs} = 400$ mm
- FOS against overturning, $FOS = 1.4$
- Density of stud timber, $d_{stud} = 370$ kg/m³
- Overall weight of panel, $p_{anow} = \text{stud+ow} = 7.2123$ kN
- Combined vertical load, $c_{vl} = P + p_{anow} = 22.712$ kN
Restoring moment due to vert.load RM=cvl*w/2000=76.654 kNm
Applied over-turning moment 111 kNm
Factor of safety to overturning OTFOS=RM/aotm=0.69058

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Racking shear load</td>
<td>Applied overturing moment</td>
</tr>
<tr>
<td>2.7407 kN/m</td>
<td>111 kNm</td>
</tr>
<tr>
<td>Permiss shear load</td>
<td>Resting moment</td>
</tr>
<tr>
<td>2.791 kN/m</td>
<td>76.654 kNm</td>
</tr>
<tr>
<td>Applied overturing moment</td>
<td>Reqd uplift strap moment</td>
</tr>
<tr>
<td>111 kNm</td>
<td>78.746 kNm</td>
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<tr>
<td>Reqd uplift strap moment</td>
<td>Nail diameter</td>
</tr>
<tr>
<td>78.746 kNm</td>
<td>3 mm</td>
</tr>
<tr>
<td>Sliding load at base</td>
<td>Nail Length</td>
</tr>
<tr>
<td>2.7407 kN/m</td>
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<tr>
<td>Nail diameter</td>
<td>Perimeter nail crs</td>
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<tr>
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<tr>
<td>Nail Length</td>
<td>Intermediate nail crs</td>
</tr>
<tr>
<td>75 mm</td>
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<tr>
<td>Perimeter nail crs</td>
<td>Ply thickness</td>
</tr>
<tr>
<td>150 mm</td>
<td>12.5 mm</td>
</tr>
</tbody>
</table>
Location:

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel \( h = 5200 \text{ mm} \)
Total width of panel \( w = 14600 \text{ mm} \)
Applied load along side of panel \( RF = 18.528 \text{ kN} \)
Applied vertical load \( P = 3.872 \text{ kN} \)
Wall panel comprises C16 stress grade studs with Finnish Birch faced plywood sheathing.
Ply sheathing thickness \( pst = 8 \text{ mm} \)

Racking resistance

Nail diameter \( Dn = 3 \text{ mm} \)
Nail spacing \( Sp = 150 \text{ mm} \)

Permissible racking shear load taking into account all modification factors of BS5268:Section 6.1:1996 for ply and OSB sheathing is given by the following equation:

\[
PRS = 1680 \times K_{101} \times K_{102} \times K_{103} \times K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108} = 1861.8 \text{ N/m}
\]

Permissible racking shear load per metre for 8 mm sheathing with 3 mm diameter, 50 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 1861.8 N/m.

Since applied shear load (1269 N/m) is less than permissible shear load (1861.8 N/m) then panel is adequate.

Over-turning checks

Density of sheathing \( \text{mass} = 4.9 \text{ kg/m}^2 \)
Stud width \( b = 47 \text{ mm} \)
Stud depth \( d = 175 \text{ mm} \)
Stud centres \( scrs = 400 \text{ mm} \)
FOS against overturning \( \text{FOS} = 1.25 \)
Density of stud timber \( dstud' = 370 \text{ kg/m}^3 \)
Overall weight of panel \( \text{panow} = \text{stud+ow} = 10.161 \text{ kN} \)
Combined vertical load \( cvl = P + \text{panow} = 14.033 \text{ kN} \)
Restoring moment due to vert. load $RM = \frac{cvl \times w}{2000} = 102.44$ kNm

Applied over-turning moment 48.173 kNm

Factor of safety to overturning $OTFOS = \frac{RM}{aotm} = 2.1266$

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Racking shear load 1.269 kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Permiss shear load 1.8618 kN/m</td>
</tr>
<tr>
<td></td>
<td>Applied overturning moment 48.173 kNm</td>
</tr>
<tr>
<td></td>
<td>Restoring moment 102.44 kNm</td>
</tr>
<tr>
<td></td>
<td>FOS against overturning 2.1266</td>
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<tr>
<td></td>
<td>Min reqd overturning FOS 1.25</td>
</tr>
<tr>
<td></td>
<td>Sliding load at base 1.269 kN/m</td>
</tr>
<tr>
<td></td>
<td>Nail diameter 3 mm</td>
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<tr>
<td></td>
<td>Nail Length 50 mm</td>
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<tr>
<td></td>
<td>Perimeter nail crs 150 mm</td>
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<tr>
<td></td>
<td>Intermediate nail crs 300 mm</td>
</tr>
<tr>
<td></td>
<td>Ply thickness 8 mm</td>
</tr>
</tbody>
</table>
Location:

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel $h = 3000$ mm
Total width of panel $w = 4100$ mm
Applied load along side of panel $RF = 4.912$ kN
Applied vertical load $P = 4.1$ kN

WARNING:
Plasterboard alone should not be relied upon to provide the racking resistance of a dwelling; however plasterboard will contribute to the racking resistance if used with ply or OSB sheathing. This is outside the scope of this proforma, the user should refer to the relevant clauses of BS5268:Section 6.1:1996 (Cl.4.7.4.1 to 4.7.5).
Location:

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel \( h = 3000 \text{ mm} \)
Total width of panel \( w = 4100 \text{ mm} \)
Applied load along side of panel \( RF = 4.912 \text{ kN} \)
Applied vertical load \( P = 4.1 \text{ kN} \)
Wall panel comprises C16 stress grade studs with Swedish Softwood Construction OSB sheathing.
OSB sheathing thickness \( pst = 8 \text{ mm} \)

Racking resistance

Nail diameter \( Dn = 3 \text{ mm} \)
Nail spacing \( Sp = 150 \text{ mm} \)
Width of opening in wall panel \( wopen = 800 \text{ mm} \)
Height of opening in wall panel \( hopen = 800 \text{ mm} \)

Permissible racking shear load taking into account all modification factors of BS5268:Section 6.1:1996 for ply and OSB sheathing is given by the following equation:

\[
PRS = 1680 \times K101 \times K102 \times K103 \times K104 \times K105 \times K106 \times K107 \times K108 = 1703.8 \text{ N/m}
\]

Permissible racking shear load per metre for 8 mm sheathing with 3 mm diameter, 50 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 1703.8 N/m.

Since applied shear load (1198 N/m) is less than permissible shear load (1703.8 N/m) then panel is adequate.

Over-turning checks

Density of sheathing \( \text{mass} = 5 \text{ kg/m}^2 \)
Stud width \( b = 47 \text{ mm} \)
Stud depth \( d = 97 \text{ mm} \)
Stud centres \( scrs = 600 \text{ mm} \)
FOS against overturning \( FOS = 1.33 \)
Density of stud timber \( dstud' = 370 \text{ kg/m}^3 \)
Overall weight of panel  \( \text{panow}=\text{stud}+\text{ow}=1.0756 \text{ kN} \)

Combined vertical load  \( \text{cvl}=P+\text{panow}=5.1756 \text{ kN} \)

Restoring moment due to vert.load  \( \text{RM}=\text{cvl}*w/2000=10.61 \text{ kNm} \)

Applied over-turning moment  \( 7.368 \text{ kNm} \)

Factor of safety to overturning  \( \text{OTFOS}=\text{RM}/aotm=1.44 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Racking shear load</td>
<td>1.198 kN/m</td>
</tr>
<tr>
<td>Permiss shear load</td>
<td>1.7038 kN/m</td>
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<tr>
<td>Applied overturning moment</td>
<td>7.368 kNm</td>
</tr>
<tr>
<td>Restoring moment</td>
<td>10.61 kNm</td>
</tr>
<tr>
<td>FOS against overturning</td>
<td>1.44</td>
</tr>
<tr>
<td>Min reqd overturning FOS</td>
<td>1.33</td>
</tr>
<tr>
<td>Sliding load at base</td>
<td>1.198 kN/m</td>
</tr>
<tr>
<td>Nail diameter</td>
<td>3 mm</td>
</tr>
<tr>
<td>Nail Length</td>
<td>50 mm</td>
</tr>
<tr>
<td>Perimeter nail crs</td>
<td>150 mm</td>
</tr>
<tr>
<td>Intermediate nail crs</td>
<td>300 mm</td>
</tr>
<tr>
<td>OSB (type F2) thickness</td>
<td>8 mm</td>
</tr>
</tbody>
</table>
Location: Wall Panel Grid Line A/2

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel \( h = 2500 \text{ mm} \)
Total width of panel \( w = 3000 \text{ mm} \)
Design side PL on panel \( F_{vEd} = 5.4 \text{ kN} \)
Design vertical load (permanent) \( gd = 2.2545 \text{ kN/m} \)
Wall panel comprises C16 stress grade studs with Canadian Douglas Fir plywood sheathing.
Ply sheathing thickness \( pst = 9.5 \text{ mm} \)

Design racking resistance

The design (racking) shear resistance \( F_{vRd} \) will be evaluated in accordance with EC5, Section 9.2.4.3.2 for ply and OSB sheathing.

Nail diameter \( dn = 3 \text{ mm} \)
Nail spacing along perimeter \( sf = 150 \text{ mm} \)
Lateral design capac of fastener \( F_{fRd} = 370 \text{ N} \)
Density of stud timber \( pk1 = 370 \text{ kg/m}^3 \)
Basic fastener spacing \( so = 9.7*dn/pk1 = 0.078649 \text{ m} \)
Panel dimension factor \( kd = (w/h)^0.4 = 1.0757 \)
Uniformly distributed load factor \( kq = 1 + (0.083*gd - 0.0008*gd^2)*(2400/w)^0.4 = 1.1674 \)
Fastener spacing factor \( ks = 1/(0.86*sf/(so*1000) + 0.57) = 0.45245 \)
Sheathing is assumed to be on one side of the panel only.
Sheathing material factor \( kn = 1.0 \)

Design racking strength of wall \( F_{vRd} = F_{fRd} * kd * kq * ks * kn / so = 2672.9 \text{ N/m} \)

Design racking shear resistance per metre for 9.5 mm sheathing with 3 mm diameter, 50 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 2672.9 N/m.

Since the base design shear load (1800 N/m) is less than the design shear resistance (2672.9 N/m) the panel is adequate.
Over-turning checks

Density of sheathing                   \( mass = 5.5 \, \text{kg/m}^2 \)  
Stud width                        \( b = 75 \, \text{mm} \)  
Stud depth                        \( d = 100 \, \text{mm} \)  
Stud centres                      \( scrs = 400 \, \text{mm} \)  
FOS against overturning           \( FOS = 1.33 \)  
Overall weight of panel            \( panow = \text{stud+ow} = 1.0798 \, \text{kN} \)  
Permanent actions factor          \( \text{gamG} = 1.35 \)  
Combined design vertical load      \( cvl = gd' + panow\cdot\text{gamG} = 8.2212 \, \text{kN} \)  
Restoring moment due to vert. load \( RM = cvl\cdot w / 2000 = 12.332 \, \text{kNm} \)  
Design over-turning moment        \( Mdot = 13.5 \, \text{kNm} \)  
Factor of safety to overturning    \( OTFOS = RM / Mdot = 0.91347 \)  
Strap design uplift moment        \( Mdstr = Mdot \cdot FOS - RM = 5.6232 \, \text{kNm} \)

Buckling strength of the sheathing

The buckling of the sheets under the action of shear force \( FvEd \) may be disregarded provided EC5 expression 9.33 is satisfied.

Expression 9.33
\[ \text{ratio} = \frac{scrs - b}{pst} = 34.211 \]

The ratio of clear distance between studs and thickness of sheathing is less than the upper limit ratio of 100. Hence OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design racking shear</td>
<td>1.8 , \text{kN/m}</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>2.6729 , \text{kN/m}</td>
</tr>
<tr>
<td>Design overturning moment</td>
<td>13.5 , \text{kNm}</td>
</tr>
<tr>
<td>Design restoring moment</td>
<td>12.332 , \text{kNm}</td>
</tr>
<tr>
<td>Strap design uplift moment</td>
<td>5.6232 , \text{kNm}</td>
</tr>
<tr>
<td>Sliding load at base</td>
<td>1.8 , \text{kN/m}</td>
</tr>
<tr>
<td>Nail diameter</td>
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<tr>
<td>Nail Length</td>
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<tr>
<td>Perimeter nail crs</td>
<td>150 mm</td>
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<tr>
<td>Intermediate nail crs</td>
<td>300 mm</td>
</tr>
<tr>
<td>Fly thickness</td>
<td>9.5 mm</td>
</tr>
</tbody>
</table>
Timber design to BS5268 and Eurocode 5
Made by: IFB
Date: 02/12/19

Location:

---

Location:

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

- **Height of panel**: h=2800 mm
- **Total width of panel**: w=4000 mm
- **Design side UDL on panel**: FvEd=8.414 kN
- **Design vertical load (permanent)**: gd=1.0969 kN/m
- **Wall panel comprises C16 stress grade studs with British Hardwood plywood sheathing.**
- **Ply sheathing thickness**: pst=12 mm

### Design racking resistance

The design (racking) shear resistance FvRd will be evaluated in accordance with EC5, Section 9.2.4.3.2 for ply and OSB sheathing.

- **Nail diameter**: dn=3 mm
- **Nail spacing along perimeter**: sf=150 mm
- **Lateral design capac of fastener**: FfRd=370 N
- **Density of stud timber**: pk1=370 kg/m³
- **Basic fastener spacing**: so=9.7*dn/pk1=0.078649 m
- **Panel dimension factor**: kd=(w/h)^0.4=1.1533
- **Uniformly distributed load factor**: kg=1+(0.083*gd-0.0008*gd^2)*(2400/w)^0.4=1.0734
- **Fastener spacing factor**: ks=1/(0.86*sf/(so*1000)+0.57)=0.45245
- **Sheathing is assumed to be on one side of the panel only.**
- **Sheathing material factor**: kn=1.0

- **Design racking strength of wall**: FvRd=FfRd*kd*kq*ks*kn/so=2635.2 N/m

- **Design racking shear resistance per metre for 12 mm sheathing with 3 mm diameter, 65 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 2635.2 N/m.**

Since the base design shear load (2103.5 N/m) is less than the design shear resistance (2635.2 N/m) the panel is adequate.
Over-turning checks

Density of sheathing      mass = 7.32 kg/m²
Stud width               \( b = 50 \text{ mm} \)
Stud depth               \( d = 125 \text{ mm} \)
Stud centres             \( \text{scrs} = 450 \text{ mm} \)
FOS against overturning  \( \text{FOS} = 1.4 \)
Overall weight of panel  \( \text{panow} = \text{stud+ow} = 1.537 \text{ kN} \)
Permanent actions factor \( \text{gamG} = 1.35 \)
Combined design vertical load \( \text{cvl} = gd' + \text{panow} \times \text{gamG} = 6.4626 \text{ kN} \)
Restoring moment due to vert.load \( \text{RM} = \frac{\text{cvl} \times w}{2000} = 12.925 \text{ kNm} \)
Design over-turning moment \( \text{Mdot} = 11.78 \text{ kNm} \)
Factor of safety to overturning \( \text{OTFOS} = \frac{\text{RM}}{\text{Mdot}} = 1.0972 \)
Strap design uplift moment \( \text{Mdstr} = \text{Mdot} \times \text{FOS} - \text{RM} = 3.5669 \text{ kNm} \)

Buckling strength of the sheathing

The buckling of the sheets under the action of shear force \( FvEd \) may be disregarded provided EC5 expression 9.33 is satisfied.

Expression 9.33  \( \text{ratio} = \frac{\text{scrs} - b}{p} = 33.333 \)

The ratio of clear distance between studs and thickness of sheathing is less than the upper limit ratio of 100. Hence OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design racking shear</td>
<td>2.1035 kN/m</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>2.6352 kN/m</td>
</tr>
<tr>
<td>Design overturning moment</td>
<td>11.78 kNm</td>
</tr>
<tr>
<td>Design restoring moment</td>
<td>12.925 kNm</td>
</tr>
<tr>
<td>Strap design uplift moment</td>
<td>3.5669 kNm</td>
</tr>
<tr>
<td>Sliding load at base</td>
<td>2.1035 kN/m</td>
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<tr>
<td>Nail diameter</td>
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<td>Nail Length</td>
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<td>Intermediate nail crs</td>
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</tr>
<tr>
<td>Fly thickness</td>
<td>12 mm</td>
</tr>
</tbody>
</table>
Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel: h=4035 mm
Total width of panel: w=5050 mm
Design side PL on panel: FvEd=9.968 kN
Design vertical load (permanent): gd=2.446 kN/m
Wall panel comprises C16 stress grade studs with American construction plywood (grade C-D or grade C-C) sheathing.
Ply sheathing thickness: pst=9 mm

Design racking resistance

The design (racking) shear resistance FvRd will be evaluated in accordance with EC5, Section 9.2.4.3.2 for ply and OSB sheathing.

Nail diameter: dn=3 mm
Nail spacing along perimeter: sf=150 mm
Lateral design capacity of fastener: FfRd=370 N
Density of stud timber: pk1=370 kg/m³
Basic fastener spacing: so=9.7*dn/pk1=0.078649 m
Panel dimension factor: kd=(4800/h)^0.4=1.0719
Uniformly distributed load factor: kq=1+(0.083*gd-0.0008*gd^2)*(2400/w)^0.4=1.1472
Fastener spacing factor: ks=1/(0.86*sf/(so*1000)+0.57)=0.45245
Sheathing is assumed to be on one side of the panel only.
Sheathing material factor: kn=1.0

Design racking strength of wall: FvRd=FfRd*kd*kq*ks*kn/so=2617.5 N/m

Design racking shear resistance per metre for 9 mm sheathing with 3 mm diameter, 50 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 2617.5 N/m.

Since the base design shear load (1973.9 N/m) is less than the design shear resistance (2617.5 N/m) the panel is adequate.
Over-turning checks

Density of sheathing  mass = 4.9 kg/m²
Stud width  b = 97 mm
Stud depth  d = 147 mm
Stud centres  scrs = 600 mm
FOS against overturning  FOS = 1.4
Overall weight of panel  panow = stud + ow = 3.2914 kN
Permanent actions factor  gamG = 1.35
Combined design vertical load  cvl = gd' + panow * gamG = 16.796 kN
Restoring moment due to vert.load  RM = cvl * w / 2000 = 42.409 kNm
Design over-turning moment  Mdot = 40.221 kNm
Factor of safety to overturning  OTFOS = RM / Mdot = 1.0544
Strap design uplift moment  Mdstr = Mdot * FOS - RM = 13.9 kNm

Buckling strength of the sheathing

The buckling of the sheets under the action of shear force FvEd may be disregarded provided EC5 expression 9.33 is satisfied.

Expression 9.33  ratio = (scrs - b) / pst = 55.889
The ratio of clear distance between studs and thickness of sheathing is less than the upper limit ratio of 100. Hence OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
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</thead>
<tbody>
<tr>
<td>Design racking shear</td>
<td>1.9739 kN/m</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>2.6175 kN/m</td>
</tr>
<tr>
<td>Design overturning moment</td>
<td>40.221 kNm</td>
</tr>
<tr>
<td>Design restoring moment</td>
<td>42.409 kNm</td>
</tr>
<tr>
<td>Strap design uplift moment</td>
<td>13.9 kNm</td>
</tr>
<tr>
<td>Sliding load at base</td>
<td>1.9739 kN/m</td>
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<tr>
<td>Nail diameter</td>
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</tr>
<tr>
<td>Nail Length</td>
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</tr>
<tr>
<td>Perimeter nail crs</td>
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<td>Intermediate nail crs</td>
<td>300 mm</td>
</tr>
<tr>
<td>Fly thickness</td>
<td>9 mm</td>
</tr>
</tbody>
</table>

SUMMARY

Design shear strength  2.6175 kN/m
Design overturning moment  40.221 kNm
Design restoring moment  42.409 kNm
Strap design uplift moment  13.9 kNm
Sliding load at base  1.9739 kN/m
Nail diameter  3 mm
Nail Length  50 mm
Perimeter nail crs  150 mm
Intermediate nail crs  300 mm
Fly thickness  9 mm
Location:

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel  \( h = 2150 \) mm  
Total width of panel  \( w = 1800 \) mm  
Design side UDL on panel  \( F_{vEd} \) = 2.198 kN  
Design vertical load (permanent)  \( g_d \) = 1.1625 kN/m  
Wall panel comprises C16 stress grade studs with Finnish Birch plywood sheathing.  
Ply sheathing thickness  \( p_{st} \) = 9 mm

Design racking resistance

The design (racking) shear resistance \( F_{vRd} \) will be evaluated in accordance with EC5, Section 9.2.4.3.2 for ply and OSB sheathing.

Nail diameter  \( d_n = 3 \) mm  
Nail spacing along perimeter  \( s_f = 150 \) mm  
Lateral design capac of fastener  \( F_{fRd} = 370 \) N  
Density of stud timber  \( p_{kl} = 370 \) kg/m³  
Basic fastener spacing  \( s_o = 9.7 \times d_n / p_{kl} = 0.078649 \) m  
Panel dimension factor  \( k_d = w / h = 0.83721 \)  
Uniformly distributed load factor  \( k_q = 1 + (0.083 \times g_d - 0.0008 \times g_d^2) \times (2400 / w)^{0.4} = 1.107 \)  
Fastener spacing factor  \( k_s = 1 / (0.86 \times s_f / (s_o \times 1000) + 0.57) = 0.45245 \)  
Sheathing is assumed to be on one side of the panel only.  
Sheathing material factor  \( k_n = 1.0 \)  
Design racking strength of wall  \( F_{vRd} = F_{fRd} \times k_d \times k_q \times k_s \times k_n / s_o = 1972.8 \) N/m

Design racking shear resistance per metre for 9 mm sheathing with 3 mm diameter, 50 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 1972.8 N/m.

Since the base design shear load (1221.1 N/m) is less than the design shear resistance (1972.8 N/m) the panel is adequate.
Over-turning checks

Density of sheathing \( \text{mass}=6.2 \text{ kg/m}^2 \)
Stud width \( b=47 \text{ mm} \)
Stud depth \( d=97 \text{ mm} \)
Stud centres \( \text{scrs}=450 \text{ mm} \)
FOS against overturning \( \text{FOS}=1.4 \)
Overall weight of panel \( \text{panow}=\text{stud+ow}=0.46506 \text{ kN} \)
Permanent actions factor \( \text{gamG}=1.35 \)
Combined design vertical load \( \text{cvl}=gd'+\text{panow}\cdot\text{gamG}=2.7203 \text{ kN} \)
Restoring moment due to vert.load \( \text{RM}=\text{cvl}\cdot w/2000=2.4483 \text{ kNm} \)
Design over-turning moment \( \text{Mdot}=2.3629 \text{ kNm} \)
Factor of safety to overturning \( \text{OTFOS}=\text{RM}/\text{Mdot}=1.0361 \)
Strap design uplift moment \( \text{Mdstr}=\text{Mdot}\cdot\text{FOS}-\text{RM}=0.85976 \text{ kNm} \)

Buckling strength of the sheathing

The buckling of the sheets under the action of shear force \( FvEd \) may be disregarded provided EC5 expression 9.33 is satisfied.

Expression 9.33 \( \text{ratio}=(\text{scrs}-b)/\text{pst}=44.778 \)
The ratio of clear distance between studs and thickness of sheathing is less than the upper limit ratio of 100. Hence OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design racking shear</td>
<td>1.2211 \text{ kN/m}</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>1.9728 \text{ kN/m}</td>
</tr>
<tr>
<td>Design overturning moment</td>
<td>2.3629 \text{ kNm}</td>
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<tr>
<td>Design restoring moment</td>
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<td>300 \text{ mm}</td>
</tr>
<tr>
<td>Fly thickness</td>
<td>9 \text{ mm}</td>
</tr>
</tbody>
</table>
Location:

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel \( h = 6000 \text{ mm} \)
Total width of panel \( w = 6750 \text{ mm} \)
Design side PL on panel \( FvEd = 22.2 \text{ kN} \)
Design vertical load (permanent) \( gd = 3.1 \text{ kN/m} \)
Wall panel comprises C16 stress grade studs with Swedish Softwood Construction plywood sheathing.
Ply sheathing thickness \( pst = 12.5 \text{ mm} \)

Design racking resistance

The design (racking) shear resistance \( FvRd \) will be evaluated in accordance with EC5, Section 9.2.4.3.2 for ply and OSB sheathing.

Nail diameter \( dn = 3 \text{ mm} \)
Nail spacing along perimeter \( sf = 150 \text{ mm} \)
Lateral design capac of fastener \( FfRd = 570 \text{ N} \)
Density of stud timber \( pk1 = 370 \text{ kg/m}^3 \)
Basic fastener spacing \( so = 9.7 \times \frac{dn}{pk1} = 0.078649 \text{ m} \)
Panel dimension factor \( kd = (4800/h)^{0.4} = 0.91461 \)
Uniformly distributed load factor \( kq = 1 + (0.083 \times gd - 0.0008 \times gd^2) \times (2400/w)^{0.4} = 1.1651 \)
Fastener spacing factor \( ks = 1 / (0.86 \times sf / (so \times 1000) + 0.57) = 0.45245 \)
Sheathing is assumed to be on one side of the panel only.
Sheathing material factor \( kn = 1.0 \)

Design racking strength of wall \( FvRd = FfRd \times kd \times kq \times ks \times kn / so = 3494.1 \text{ N/m} \)

Design racking shear resistance per metre for 12.5 mm sheathing with 3 mm diameter, 75 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 3494.1 N/m.

Since the base design shear load (3288.9 N/m) is less than the design shear resistance (3494.1 N/m) the panel is adequate.
Over-turning checks

Density of sheathing mass=5.3 kg/m²
Stud width b=63 mm
Stud depth d=197 mm
Stud centres scrs=400 mm
FOS against overturning FOS=1.4
Overall weight of panel panow=stud+ow=7.2123 kN
Permanent actions factor gamG=1.35
Combined design vertical load cvl=gd'+panow*gamG=30.662 kN
Restoring moment due to vert.load RM=cvl*w/2000=103.48 kNm
Design over-turning moment Mdot=133.2 kNm
Factor of safety to overturning OTFOS=RM/Mdot=0.7769
Strap design uplift moment Mdstr=Mdot*FOS-RM=82.997 kNm

Buckling strength of the sheathing

The buckling of the sheets under the action of shear force FvEd may be disregarded provided EC5 expression 9.33 is satisfied.

Expression 9.33 ratio=(scrs-b)/pst=26.96
The ratio of clear distance between studs and thickness of sheathing is less than the upper limit ratio of 100. Hence OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design racking shear</td>
<td>3.2889 kN/m</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>3.4941 kN/m</td>
</tr>
<tr>
<td>Design overturning moment</td>
<td>133.2 kNm</td>
</tr>
<tr>
<td>Design restoring moment</td>
<td>103.48 kNm</td>
</tr>
<tr>
<td>Strap design uplift moment</td>
<td>82.997 kNm</td>
</tr>
<tr>
<td>Sliding load at base</td>
<td>3.2889 kN/m</td>
</tr>
<tr>
<td>Nail diameter</td>
<td>3 mm</td>
</tr>
<tr>
<td>Nail Length</td>
<td>75 mm</td>
</tr>
<tr>
<td>Perimeter nail crs</td>
<td>150 mm</td>
</tr>
<tr>
<td>Intermediate nail crs</td>
<td>300 mm</td>
</tr>
<tr>
<td>Fly thickness</td>
<td>12.5 mm</td>
</tr>
</tbody>
</table>
Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel $h=5200$ mm
Total width of panel $w=14600$ mm
Design side UDL on panel $F_{vEd}=25.939$ kN
Design vertical load (permanent) $g_d=0.35803$ kN/m
Wall panel comprises C16 stress grade studs with Finnish Birch faced plywood sheathing.
Ply sheathing thickness $p_s=8$ mm

**Design racking resistance**

The design (racking) shear resistance $F_{vRd}$ will be evaluated in accordance with EC5, Section 9.2.4.3.2 for ply and OSB sheathing.

Nail diameter $d_n=3$ mm
Nail spacing along perimeter $s_f=150$ mm
Lateral design capac of fastener $F_{fRd}=370$ N
Density of stud timber $p_{k1}=370$ kg/m$^3$
Basic fastener spacing $s_o=9.7\cdot d_n/p_{k1}=0.078649$ m
Panel dimension factor $k_d=(4800/h)^{0.4}=0.96849$
Uniformly distributed load factor $k_q=1+(0.083\cdot g_d-0.0008\cdot g_d^2)\cdot (2400/w)^{0.4}=1.0144$
Fastener spacing factor $k_s=1/(0.86\cdot s_f/(s_o\cdot 1000)+0.57)=0.45245$
Sheathing is assumed to be on one side of the panel only.
Sheathing material factor $k_n=1.0$
Design racking strength of wall $F_{vRd}=F_{fRd}\cdot k_d\cdot k_q\cdot k_s\cdot k_n/s_o=2091.1$ N/m

Design racking shear resistance per metre for 8 mm sheathing with 3 mm diameter, 50 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 2091.1 N/m.

Since the base design shear load (1776.6 N/m) is less than the design shear resistance (2091.1 N/m) the panel is adequate.
Over-turning checks

Density of sheathing \( \text{mass}=4.9 \, \text{kg/m}^2 \)
Stud width \( b=47 \, \text{mm} \)
Stud depth \( d=175 \, \text{mm} \)
Stud centres \( \text{scrs}=400 \, \text{mm} \)
FOS against overturning \( \text{FOS}=1.25 \)
Overall weight of panel \( \text{panow}=\text{stud}+\text{ow}=10.161 \, \text{kN} \)
Permanent actions factor \( \text{gamG}=1.35 \)
Combined design vertical load \( \text{cvl}=\text{gd}'+\text{panow}^*\text{gamG}=18.945 \, \text{kN} \)
Restoring moment due to vert.load \( \text{RM}=\text{cvl}^*\text{w}/2000=138.3 \, \text{kNm} \)
Design over-turning moment \( \text{Mdot}=67.441 \, \text{kNm} \)
Factor of safety to overturning \( \text{OTFOS}=\text{RM}/\text{Mdot}=2.0507 \)

Buckling strength of the sheathing

The buckling of the sheets under the action of shear force \( FvE_d \) may be disregarded provided EC5 expression 9.33 is satisfied.

Expression 9.33 \( \text{ratio}=(\text{scrs}-b)/\text{pst}=44.125 \)
The ratio of clear distance between studs and thickness of sheathing is less than the upper limit ratio of 100. Hence OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design racking shear</td>
<td>1.7766 , \text{kN/m}</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>2.0911 , \text{kN/m}</td>
</tr>
<tr>
<td>Design overturning moment</td>
<td>67.441 , \text{kNm}</td>
</tr>
<tr>
<td>Design restoring moment</td>
<td>138.3 , \text{kNm}</td>
</tr>
<tr>
<td>FOS against overturning</td>
<td>2.0507</td>
</tr>
<tr>
<td>Min reqd. overturning FOS</td>
<td>1.25</td>
</tr>
<tr>
<td>Sliding load at base</td>
<td>1.7766 , \text{kN/m}</td>
</tr>
<tr>
<td>Nail diameter</td>
<td>3 , \text{mm}</td>
</tr>
<tr>
<td>Nail Length</td>
<td>50 , \text{mm}</td>
</tr>
<tr>
<td>Perimeter nail crs</td>
<td>150 , \text{mm}</td>
</tr>
<tr>
<td>Intermediate nail crs</td>
<td>300 , \text{mm}</td>
</tr>
<tr>
<td>Fly thickness</td>
<td>8 , \text{mm}</td>
</tr>
</tbody>
</table>
Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel \( h = 3000 \text{ mm} \)
Total width of panel \( w = 4100 \text{ mm} \)
Design side UDL on panel \( FvEd = 6.8768 \text{ kN} \)
Design vertical load (permanent) \( gd = 1.1268 \text{ kN/m} \)

**WARNING:**
Plasterboard alone should not be relied upon to provide the racking resistance of a dwelling; however plasterboard will contribute to the racking resistance if used with ply or OSB sheathing. This is outside the scope of this proforma.
Location:

Panel assumed to be supported along full length of base sole plate and laterally restrained perpendicular to plane of racking.

Height of panel \( h = 3000 \text{ mm} \)

Total width of panel \( w = 4100 \text{ mm} \)

Design side UDL on panel \( F_v Ed = 6.8768 \text{ kN} \)

Design vertical load (permanent) \( g_d = 1.35 \text{ kN/m} \)

Wall panel comprises C16 stress grade studs with Swedish Softwood Construction OSB sheathing.

OSB sheathing thickness \( p_{st} = 8 \text{ mm} \)

**Design racking resistance**

The design (racking) shear resistance \( F_v Rd \) will be evaluated in accordance with EC5, Section 9.2.4.3.2 for ply and OSB sheathing.

Nail diameter \( d_n = 3 \text{ mm} \)

Nail spacing along perimeter \( s_f = 150 \text{ mm} \)

Lateral design capacity of fastener \( F_{f Rd} = 370 \text{ N} \)

Density of stud timber \( p_{kl} = 370 \text{ kg/m}^3 \)

Basic fastener spacing \( s_o = 9.7 \times d_n / p_{kl} = 0.078649 \text{ m} \)

Panel dimension factor \( k_d = (w / h)^{0.4} = 1.1331 \)

Uniformly distributed load factor \( k_q = 1 + (0.083 \times g_d - 0.0008 \times g_d^2) \times (2400 / w)^{0.4} = 1.0893 \)

Fastener spacing factor \( k_s = 1 / (0.86 \times s_f / (s_o \times 1000) + 0.57) = 0.45245 \)

Sheathing material factor \( k_n = 1.0 \)

Design racking strength of wall \( F_v Rd = F_{f Rd} \times k_d \times k_q \times k_s \times k_n / s_o = 2627.1 \text{ N/m} \)

Design racking shear resistance per metre for 8 mm sheathing with 3 mm diameter, 50 mm long round wire nails at 150 mm centres along the full perimeter of the panel is 2627.1 N/m.

Since the base design shear load (1677.3 N/m) is less than the design shear resistance (2627.1 N/m) the panel is adequate.
Over-turning checks

Density of sheathing mass=5 kg/m²
Stud width b=47 mm
Stud depth d=97 mm
Stud centres scrs=600 mm
FOS against overturning FOS=1.33
Overall weight of panel panow=stud+ow=1.0756 kN
Permanent actions factor gamG=1.35
Combined design vertical load cvl=gd’+panow*gamG=6.9871 kN
Restoring moment due to vert.load RM=cvl*w/2000=14.324 kNm
Design over-turning moment Mdot=10.315 kNm
Factor of safety to overturning OTFOS=RM/Mdot=1.3886

Buckling strength of the sheathing

The buckling of the sheets under the action of shear force FvEd may be disregarded provided EC5 expression 9.33 is satisfied.

Expression 9.33 ratio=(scrs-b)/pst=69.125
The ratio of clear distance between studs and thickness of sheathing is less than the upper limit ratio of 100. Hence OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design racking shear</td>
<td>1.6773 kN/m</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>2.6271 kN/m</td>
</tr>
<tr>
<td>Design overturning moment</td>
<td>10.315 kNm</td>
</tr>
<tr>
<td>Design restoring moment</td>
<td>14.324 kNm</td>
</tr>
<tr>
<td>FOS against overturning</td>
<td>1.3886</td>
</tr>
<tr>
<td>Min reqd. overturning FOS</td>
<td>1.33</td>
</tr>
<tr>
<td>Sliding load at base</td>
<td>1.6773 kN/m</td>
</tr>
<tr>
<td>Nail diameter</td>
<td>3 mm</td>
</tr>
<tr>
<td>Nail Length</td>
<td>50 mm</td>
</tr>
<tr>
<td>Perimeter nail crs</td>
<td>150 mm</td>
</tr>
<tr>
<td>Intermediate nail crs</td>
<td>300 mm</td>
</tr>
<tr>
<td>OSB thickness</td>
<td>8 mm</td>
</tr>
</tbody>
</table>
Location: Timber beam with udl & point loads

Structural design of a timber member for fire resistance

BS5268 Part 4 Section 4.1: 1978 sets out a method for calculating the fire resistance of timber members. Fire resistance of a member is the period of time which the member is required to support the design load without failure whilst subjected to the fire. The method assumes that a depth of charring occurs around the exposed face of the section, and the corners of the beam become rounded. The residual section is checked for stability strength and the beam deflection is limited to Span/20 (1990 revision).

Basic fire exposure period period=30 min
Timber is Western red cedar

Charring depth char=25 mm
Original depth of section od=297 mm
Original width of section ob=147 mm

Residual depth of beam d=od-char=272 mm
Residual width of beam b=ob-2*char=97 mm

Simply supported beam subjected to vertical loads.

Beam span \(L=7.2\ m\)
Dist. from left support to start \(L_{au}(1)=0\ m\)
Distance from left support to end \(L_{bu}(1)=7.2\ m\)
Dead load (unfactored) \(G_{k\mu}(1)=0.25\ kN/m\)
Imposed load (unfactored) \(Q_{k\mu}(1)=1.2\ kN/m\)
Maximum span bending moment 9.396 kNm
Design shear force \(F_{ve}'=5.22\)
Eff. length for bending about xx \( \text{Lex} = 7200 \text{ mm} \)
Eff. length for bending about yy \( \text{Ley} = 0 \text{ mm} \)
Length of bearing \( \text{lb} = 147 \text{ mm} \)

From BS5268 Part 2 Table 18 and bearing is < 75 mm from joist end

Bearing Modification factor \( K_4 = 1.0 \)
Strength class C16 to Table 8.

Depth factor \( K_7 = (300/d)^{0.11} = 1.0108 \)
Load-sharing modification factor \( K_8 = 1.1 \)
No notches exist at the support. \( K_5 = 1.0 \)

**Permissible stresses**

BS 5268 Part 4 Section 4.1 does not give any guidance on the enhancement to the shear stress or for compression perpendicular to the grain using the residual section method.

Shear enhancement factor adopted \( \text{Shefac} = 2 \)
Shear parallel to grain \( \text{torad} = \text{Shefac} \times K_2 \times \text{shr} \times K_3 \times K_5 \times K_8 \times \text{torad} = 1.474 \text{ N/mm}^2 \)
Compress perp to grain enhancement factor adopted \( \text{Cpgfac} = 2 \)
Compress perp to grain (no wane) \( \text{sigbad} = \text{Cpgfac} \times K_2 \times \text{per} \times K_3 \times K_4 \times K_8 \times \text{sigbad} = 4.84 \text{ N/mm}^2 \)

### DESIGN

- Bending stress \( 7.8587 \text{ N/mm}^2 \)
- Permissible bending \( 13.26 \text{ N/mm}^2 \)
- Deflection \( 36.236 \text{ mm} \)
- Limiting deflection \( 360 \text{ mm} \)
- Shear stress \( 0.29677 \text{ N/mm}^2 \)
- Permissible shear \( 1.474 \text{ N/mm}^2 \)
- Bearing stress \( 0.36608 \text{ N/mm}^2 \)
- Permissible bearing \( 4.84 \text{ N/mm}^2 \)
Location: Timber beam with udl & point loads (2)

Structural design of a timber member for fire resistance

BS5268 Part 4 Section 4.1: 1978 sets out a method for calculating the fire resistance of timber members. Fire resistance of a member is the period of time which the member is required to support the design load without failure whilst subjected to the fire. The method assumes that a depth of charring occurs around the exposed face of the section, and the corners of the beam become rounded. The residual section is checked for stability strength and the beam deflection is limited to Span/20 (1990 revision).

Basic fire exposure period period=60 min
Timber is a structural species included in BS 5268-2:2002 except Western red cedar or a hardwood having a nominal density of 650 kg/m³ at 18% moisture content.

Charring depth char=40 mm
Original depth of section od=305 mm
Original width of section ob=197 mm

Timber beam partly exposed with protective structure part way down beam's depth.

Depth of beam below fire proof structure d1=125 mm

Simply supported beam subjected to vertical loads.

Beam span L=8.5 m
Dist. from left support to start Lau(1)=0 m
Distance from left support to end Lbu(1)=8.5 m
Dead load (unfactored) Gku(1)=0.45 kN/m
Imposed load (unfactored) Qku(1)=0.9 kN/m
Maximum span bending moment 12.192 kNm
Design shear force Fve'=5.7375
Design axial load (+ve compress) \( F_a = 9.2 \text{ kN} \)
Eff. length for bending about \( xx \) \( L_{ex} = 8500 \text{ mm} \)
Eff. length for bending about \( yy \) \( L_{ey} = 0 \text{ mm} \)
Length of bearing \( l_b = 100 \text{ mm} \)
> From BS5268 Part 2 Table 18 and bearing is > 75 mm from joist end
Bearing modification factor \( K_4 = 1.1 \)
Strength class C18 to Table 8.

Depth factor \( K_7 = (300/d)^{0.11} = 1.018 \)
Load-sharing modification factor \( K_8 = 1.1 \)
Modulus of elasticity mod factor \( K_9 = 1 \)
Modification factor \( K_{12} = (0.5 + (1 + \eta)*C/2)^2 - ((0.5 + (1 + \eta)*C/2)^2 - C)^{0.5} \)
\( = 0.25615 \)
Depth of notch at support \( \text{notch} = 20 \text{ mm} \)
Extent of notch \( a_l = 100 \text{ mm} \)
\( K_5 = (d*(he-al)+(al*he))/he/he = 1.0489 \)

**Permissible stresses**

BS 5268 Part 4 Section 4.1 does not give any guidance on the enhancement to the shear stress or for compression perpendicular to the grain using the residual section method.
Shear enhancement factor adopted \( \text{Shefac} = 2 \)
Shear parallel to grain \( \text{torad} = \text{Shefac}*K_{2shr}*K_3*K_5*K_8*\text{torad} \)
\( = 1.5461 \text{ N/mm}^2 \)
Compress perp to grain enhancement factor adopted \( \text{Cpgfac} = 2 \)
Compress perp to grain (no wane) \( \text{sigbad} = \text{Cpgfac}*K_{2per}*K_3*K_4*K_8*\text{sigbad} \)
\( = 5.324 \text{ N/mm}^2 \)

**DESIGN**

- **Bending stress** 8.401 N/mm²
- **Permissible bending** 14.614 N/mm²
- **Comp Stress @ Mids.** 0.21528 N/mm²
- **Permissible compression** 4.001 N/mm²
- **Interaction factor** 0.64239
- **Comp.stress @ Supp.** 0.26632 N/mm²
- **Deflection** 55.181 mm
- **Limiting deflection** 425 mm

**SUMMARY**

- **Shear stress** 0.24913 N/mm²
- **Permissible shear** 1.5461 N/mm²
- **Bearing stress** 0.39031 N/mm²
- **Permissible bearing** 5.324 N/mm²
**Location:** Timber beam with udl & point loads (3)

**Structural design of a timber member for fire resistance**

BS5268 Part 4 Section 4.1: 1978 sets out a method for calculating the fire resistance of timber members. Fire resistance of a member is the period of time which the member is required to support the design load without failure whilst subjected to the fire. The method assumes that a depth of charring occurs around the exposed face of the section, and the corners of the beam become rounded. The residual section is checked for stability strength and the beam deflection is limited to Span/20 (1990 revision).

Basic fire exposure period: period=15 min
Timber is a hardwood having a nominal density of 650 kg/m³ at 18% moisture content.

**Charring depth**  \( \text{char} = 7.5 \text{ mm} \)
**Original depth of section**  \( \text{od} = 272 \text{ mm} \)
**Original width of section**  \( \text{ob} = 63 \text{ mm} \)

\[
\begin{align*}
\text{d} & = \text{od} - 2 \times \text{char} = 253.25 \text{ mm} \\
\text{b} & = \text{ob} - 2 \times \text{char} = 44.25 \text{ mm}
\end{align*}
\]

Timber beam exposed on all four sides.

**Simply supported beam subjected to vertical loads.**

**Beam span**  \( L = 4.5 \text{ m} \)
**Dist. from left support to start**  \( \text{Lau}(1) = 0 \text{ m} \)
**Distance from left support to end**  \( \text{Lbu}(1) = 4.5 \text{ m} \)
**Dead load (unfactored)**  \( G\text{ku}(1) = 0.35 \text{ kN/m} \)
**Imposed load (unfactored)**  \( Q\text{ku}(1) = 1.75 \text{ kN/m} \)
**Maximum span bending moment**  \( 5.3156 \text{ kNm} \)
**Design shear force**  \( F\text{ve}' = 4.725 \)
Design axial load (+ve compress) $F_a = -3 \text{kN}$
Eff.length for bending about $xx$ $L_{ex} = 0 \text{ mm}$
Eff.length for bending about $yy$ $L_{ey} = 0 \text{ mm}$
Length of bearing $l_b = 100 \text{ mm}$
>From BS5268 Part 2 Table 18 and bearing is $> 75 \text{ mm}$ from joist end
Bearing modification factor $K_4 = 1.1$
Strength class D40 to Table 8.

Tensile area of cross section $A_t = 5500 \text{ mm}^2$
Depth factor $K_7 = (300/d)^{0.11} = 1.0188$
Load-sharing modification factor $K_8 = 1.0$
Modulus of elasticity mod factor $K_9 = 1.06$
No notches exist at the support. $K_5 = 1.0$

**Permissible stresses**

Width modification factor $K_{14} = (300/h)^{0.11} = 1.0188$
BS 5268 Part 4 Section 4.1 does not give any guidance on the enhancement to the shear stress or for compression perpendicular to the grain using the residual section method.
Shear enhancement factor adopted $Shefac = 2$
Shear parallel to grain $torad = Shefac \times K_{2shr} \times K_3 \times K_5 \times K_8 \times torad$
$= 4 \text{ N/mm}^2$
Compress perp to grain enhancement factor adopted $C_{pgfac} = 2$
Compress perp to grain (no wane) $\sigma_{bad} = C_{pgfac} \times K_{2per} \times K_3 \times K_4 \times K_8 \times \sigma_{bad}$
$= 8.58 \text{ N/mm}^2$

<table>
<thead>
<tr>
<th>Stress Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress</td>
<td>11.462 N/mm²</td>
</tr>
<tr>
<td>Permissible bending</td>
<td>25.47 N/mm²</td>
</tr>
<tr>
<td>Tension stress</td>
<td>0.54545 N/mm²</td>
</tr>
<tr>
<td>Permiss. tension</td>
<td>15.282 N/mm²</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.48572</td>
</tr>
</tbody>
</table>

**DESIGN**

| Deflection                  | 25.164 mm        |

**SUMMARY**

| Limiting deflection         | 225 mm           |
| Shear stress                | 0.63246 N/mm²    |
| Permissible shear           | 4 N/mm²          |
| Bearing stress              | 1.0678 N/mm²     |
| Permissible bearing         | 8.58 N/mm²       |
Location: Timber beam with udl & point loads (4)

Structural design of a timber member for fire resistance

BS5268 Part 4 Section 4.1: 1978 sets out a method for calculating the fire resistance of timber members. Fire resistance of a member is the period of time which the member is required to support the design load without failure whilst subjected to the fire. The method assumes that a depth of charring occurs around the exposed face of the section, and the corners of the beam become rounded. The residual section is checked for stability strength and the beam deflection is limited to Span/20 (1990 revision).

Basic fire exposure period period=60 min
Timber is a structural species included in BS 5268-2:2002 except Western red cedar or a hardwood having a nominal density of 650 kg/m³ at 18% moisture content.

Charring depth char=40 mm
Original depth of section od=247 mm
Original width of section ob=97 mm

Timber beam exposed on all four sides.

Residual depth of beam d=od-2*char=167 mm
Residual width of beam b=ob-2*char=17 mm
Residual width is less than 2 x char depth so depth of charred corners overlap. Neglect contribution from remaining section below char rounding. Reduced depth of section d=d-2*char=87 mm

Simply supported beam subjected to vertical loads.

Beam span L=2.4 m
Distance from left support Lc(1)=1.2 m
Dead load (unfactored) Gkc(1)=0.3 kN
Imposed load (unfactored) Qkc(1)=0.35 kN
Maximum span bending moment: 0.39 kNm
Design shear force: $F_{ve}' = 0.325$
Eff. length for bending about $xx$: $L_{ex} = 0$ mm
Eff. length for bending about $yy$: $L_{ey} = 0$ mm
Length of bearing: $l_b = 75$ mm
From BS5268 Part 2 Table 18 and bearing is < 75 mm from joist end
Bearing Modification factor: $K_4 = 1.0$
Strength class C24 to Table 8.

Depth factor: $K_7 = (300/d)^{0.11} = 1.1459$
Load-sharing modification factor: $K_8 = 1.0$
No notches exist at the support. $K_5 = 1.0$

**Permissible stresses**

BS 5268 Part 4 Section 4.1 does not give any guidance on the enhancement to the shear stress or for compression perpendicular to the grain using the residual section method.
Shear enhancement factor adopted: $Shefac = 1$
Shear parallel to grain: $\tau_{rad} = Shefac \times K_{2shr} \times K_3 \times K_5 \times K_8 \times \tau_{rad}$

$= 1.42 \text{ N/mm}^2$

Compress perp to grain enhancement factor adopted: $C_{pgfac} = 1$
Compress perp to grain (no wane): $\sigma_{bad} = C_{pgfac} \times K_{2per} \times K_3 \times K_4 \times K_8 \times \sigma_{bad}$

$= 4.8 \text{ N/mm}^2$

**DESIGN**

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress</td>
<td>18.186 N/mm$^2$</td>
</tr>
<tr>
<td>Permissible bending</td>
<td>19.337 N/mm$^2$</td>
</tr>
<tr>
<td>Deflection</td>
<td>28.574 mm</td>
</tr>
</tbody>
</table>

**SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limiting deflection</td>
<td>120 mm</td>
</tr>
<tr>
<td>Shear stress</td>
<td>0.32961 N/mm$^2$</td>
</tr>
<tr>
<td>Permissible shear</td>
<td>1.42 N/mm$^2$</td>
</tr>
<tr>
<td>Bearing stress</td>
<td>0.2549 N/mm$^2$</td>
</tr>
<tr>
<td>Permissible bearing</td>
<td>4.8 N/mm$^2$</td>
</tr>
</tbody>
</table>
Location: Timber beam with udl & point loads

Structural design of a timber member for fire resistance

BS EN 1995-1-2:2004 sets out a method for calculating the fire resistance of timber members. Fire resistance of a member is the period of time which the member is required to support the design load without failure whilst subjected to the fire. The method assumes that a depth of charring occurs around the exposed face of the section, and the corners of the beam become rounded. The residual section is checked for strength and the beam deflection is limited to Span/20 which is the recommended vertical deflection limit given in the EC5 Manual.

Basic fire exposure period  period=30 min
Enter the charring rate $\beta n$  betan=0.6

Effective charring depth  char=betan*period+k0*d0=25 mm
Original depth of section  oh=297 mm
Original width of section  ob=147 mm

Timber beam is exposed on three sides and protected by structure on top face.

Residual depth of beam  h=oh-char=272 mm
Residual width of beam  b=ob-2*char=97 mm

Timber beam to EC5

Simply supported beam subject to vertical loads.

Beam span  L=7.2 m
Dist. from left support to start  Lau(1)=0 m
Distance from left support to end  Lbu(1)=7.2 m
Permanent load (unfactored)  Gku(1)=0.25 kN/m
Variable load 1 (unfactored)  Qua(1)=1.2 kN/m
### Characteristic values of actions

<table>
<thead>
<tr>
<th>Category</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent: Reaction at left hand end</td>
<td>0.9 kN</td>
</tr>
<tr>
<td>Permanent: Reaction at right hand end</td>
<td>0.9 kN</td>
</tr>
<tr>
<td>Maximum span moment</td>
<td>Mkp=1.62 kNm</td>
</tr>
<tr>
<td>Maximum shear force</td>
<td>Fkp=0.9 kN</td>
</tr>
<tr>
<td>Variable 1: Reaction at left hand end</td>
<td>4.32 kN</td>
</tr>
<tr>
<td>Variable 1: Reaction at right hand end</td>
<td>4.32 kN</td>
</tr>
<tr>
<td>Variable 1: Maximum span moment</td>
<td>Mkv1=7.776 kNm</td>
</tr>
<tr>
<td>Variable 1: Maximum shear force</td>
<td>Fkv1=4.32 kN</td>
</tr>
<tr>
<td>Design axial load (+ve compress)</td>
<td>Nd=0 kN</td>
</tr>
</tbody>
</table>

### Section design parameters

![Diagram of section design parameters]

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eff. length for bending about yy</td>
<td>Ley'=7200 mm</td>
</tr>
<tr>
<td>Eff. length for bending about zz</td>
<td>Lez=0 mm</td>
</tr>
<tr>
<td>Length of bearing</td>
<td>lb=147 mm</td>
</tr>
<tr>
<td>Bearing modification factor</td>
<td>kc90=1.5</td>
</tr>
<tr>
<td>Timber strength class C16</td>
<td></td>
</tr>
<tr>
<td>Depth factor (Clause 3.2)</td>
<td>kh=1.0</td>
</tr>
<tr>
<td>Load-sharing modification factor</td>
<td>ksys=1.1</td>
</tr>
<tr>
<td>No notches exist at the support</td>
<td>kv=1.0</td>
</tr>
<tr>
<td>Effective length about zz</td>
<td>Le=5.76 m</td>
</tr>
</tbody>
</table>

### Design bending stress

Design bending stress: 7.8587 N/mm²

### Summary

<table>
<thead>
<tr>
<th>Category</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design bending strength</td>
<td>22 N/mm²</td>
</tr>
<tr>
<td>Deflection</td>
<td>39.859 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>360 mm</td>
</tr>
<tr>
<td>Design shear stress</td>
<td>0.29677 N/mm²</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>4.4 N/mm²</td>
</tr>
<tr>
<td>Design bearing stress</td>
<td>0.36608 N/mm²</td>
</tr>
<tr>
<td>Design bearing strength</td>
<td>4.5375 N/mm²</td>
</tr>
</tbody>
</table>
Location: Timber beam with udl & point loads (2)

Structural design of a timber member for fire resistance

BS EN 1995-1-2:2004 sets out a method for calculating the fire resistance of timber members. Fire resistance of a member is the period of time which the member is required to support the design load without failure whilst subjected to the fire. The method assumes that a depth of charring occurs around the exposed face of the section, and the corners of the beam become rounded. The residual section is checked for strength and the beam deflection is limited to Span/20 which is the recommended vertical deflection limit given in the EC5 Manual.

Basic fire exposure period \( \text{period}=60 \text{ min} \)
Enter the charring rate \( \beta_n \) \( \beta_n=0.7 \)

Effective charring depth \( \text{char} = \beta_n \times \text{period} + k_0 \times d_0 = 49 \text{ mm} \)

Original depth of section \( \text{oh}=305 \text{ mm} \)
Original width of section \( \text{ob}=197 \text{ mm} \)

Timber beam partly exposed part way down beam's depth.

Depth of beam below fire proof structure \( h_1=125 \text{ mm} \)

Timber beam to EC5

Simply supported beam subject to vertical loads.

Beam span \( L=8.5 \text{ m} \)
Dist. from left support to start \( L_{au(1)}=0 \text{ m} \)
Distance from left support to end \( L_{bu(1)}=8.5 \text{ m} \)
Permanent load (unfactored) \( G_{ku(1)}=0.45 \text{ kN/m} \)
Variable load 1 (unfactored) \( Q_{ua(1)}=0.9 \text{ kN/m} \)
Characteristic values of actions

Permanent:
Reaction at left hand end 1.9125 kN  
Reaction at right hand end 1.9125 kN  
Maximum span moment Mkp=4.0641 kNm  
Maximum shear force Fkp=1.9125 kN

Variable 1:
Reaction at left hand end 3.825 kN  
Reaction at right hand end 3.825 kN  
Maximum span moment Mkvl=8.1281 kNm  
Maximum shear force Fkvl=3.825 kN

Design axial load (+ve compress) Nd=9.2 kN

Section design parameters

Eff.length for bending about yy Ley'=8500 mm
Eff.length for bending about zz Lez=0 mm
Length of bearing lb=100 mm
Bearing modification factor kc90=1.5
Timber strength class C18.

Depth factor (Clause 3.2) kh=1.0
Load-sharing modification factor ksys=1.1
Instability factor kcy=1/(ky+(ky^2-rsry^2)^0.5)=0.18288
Instability factor kcz=1/(kz+(kz^2-rsrz^2)^0.5)=1.0638
Max instability factor kcz=1
Depth of notch at support notch=20 mm
Notch is on top edge of beam kv=1
Effective length about zz Le=6.8 m

Design bending stress 8.2567 N/mm²  
Design bending strength 24.75 N/mm²  
Design comp stress @ Mids. 0.21403 N/mm²  
Design comp strength 24.75 N/mm²  
Interaction factor 0.38089
Design comp stress @ Supp. 0.2634 N/mm²  
Design Deflection 54.627 mm
Limiting deflection 425 mm
Design shear stress 0.2464 N/mm²  
Design shear strength 4.675 N/mm²  
Design bearing stress 0.38767 N/mm²  
Design bearing strength 4.5375 N/mm²
Location: Timber beam with udl & point loads (3)

Structural design of a timber member for fire resistance

BS EN 1995-1-2:2004 sets out a method for calculating the fire resistance of timber members. Fire resistance of a member is the period of time which the member is required to support the design load without failure whilst subjected to the fire. The method assumes that a depth of charring occurs around the exposed face of the section, and the corners of the beam become rounded. The residual section is checked for strength and the beam deflection is limited to Span/20 which is the recommended vertical deflection limit given in the EC5 Manual.

Basic fire exposure period
Enter the charring rate $\beta_n$
Effective charring depth
Original depth of section
Original width of section

Timber beam exposed on all four sides.

Residual depth of beam
Residual width of beam

Timber beam to EC5

Simply supported beam subject to vertical loads.

Beam span
Dist. from left support to start
Distance from left support to end
Permanent load (unfactored)
Variable load 1 (unfactored)
Characteristic values of actions

Permanent:
Reaction at left hand end 0.7875 kN
Reaction at right hand end 0.7875 kN
Maximum span moment Mkp=0.88594 kNm
Maximum shear force Fkp=0.7875 kN

Variable 1:
Reaction at left hand end 3.9375 kN
Reaction at right hand end 3.9375 kN
Maximum span moment Mkv1=4.4297 kNm
Maximum shear force Fkv1=3.9375 kN
Design axial load (+ve compress) Nd=-3 kN

Section design parameters

Eff. length for bending about yy Ley'=0 mm
Eff. length for bending about zz Lez=0 mm
Length of bearing lb=100 mm
Bearing modification factor kc90=1.5
Timber strength class C40.

Tensile area of cross section At=5500 mm²
Depth factor (Clause 3.2) kh=1.0
Load-sharing modification factor ksys=1.0
No notches exist at the support kv=1.0
Width modification factor kw=(150/lardim)^0.2=0.9099

Design bending stress 19.019 N/mm²
Design bending strength 50 N/mm²
Design tension stress 0.54545 N/mm²
Design tension strength 32.5 N/mm²
Interaction factor 0.39717

Design shear stress 0.93555 N/mm²
Design shear strength 5 N/mm²
Design bearing stress 1.5 N/mm²
Design bearing strength 5.25 N/mm²

DESIGN
Deflection 24.793 mm
Limiting deflection 225 mm

SUMMARY

SCALE 5.48 Office 1007 Proforma 268
**Location: Timber beam with udl & point loads (4)**

**Structural design of a timber member for fire resistance**

BS EN 1995-1-2:2004 sets out a method for calculating the fire resistance of timber members. Fire resistance of a member is the period of time which the member is required to support the design load without failure whilst subjected to the fire. The method assumes that a depth of charring occurs around the exposed face of the section, and the corners of the beam become rounded. The residual section is checked for strength and the beam deflection is limited to Span/20 which is the recommended vertical deflection limit given in the EC5 Manual.

Basic fire exposure period \( \text{period} = 60 \text{ min} \)
Enter the charring rate \( \beta_n \) \( \beta_n = 0.55 \)

Effective charring depth \( \text{char} = \beta_n \times \text{period} + k_0 \times d_0 = 40 \text{ mm} \)
Original depth of section \( \text{oh} = 247 \text{ mm} \)
Original width of section \( \text{ob} = 97 \text{ mm} \)

Timber beam exposed on all four sides.

Residual depth of beam \( h = \text{oh} - 2 \times \text{char} = 167 \text{ mm} \)
Residual width of beam \( b = \text{ob} - 2 \times \text{char} = 17 \text{ mm} \)
Residual width is less than 2 x char depth so depth of charred corners overlap. Neglect contribution from remaining section below char rounding. Reduced depth of section \( h = h - 2 \times \text{char} = 87 \text{ mm} \).

**Timber beam to EC5**

Simply supported beam subject to vertical loads.

Beam span \( L = 2.4 \text{ m} \)
Distance from left support \( L_c(1) = 1.2 \text{ m} \)
Permanent load (unfactored) \( G_{kc}(1) = 0.3 \text{ kN} \)
Variable load 1 (unfactored) \( Q_{ca}(1) = 0.35 \text{ kN} \)
Characteristic values of actions

Permanent:
- Reaction at left hand end: 0.15 kN
- Reaction at right hand end: 0.15 kN
- Maximum span moment: Mkp=0.18 kNm
- Maximum shear force: Fkp=0.15 kN

Variable 1:
- Reaction at left hand end: 0.175 kN
- Reaction at right hand end: 0.175 kN
- Maximum span moment: Mkv1=0.21 kNm
- Maximum shear force: Fkv1=0.175 kN

Design axial load (+ve compress): Nd=0 kN

Section design parameters

Eff. length for bending about yy: Ley'=0 mm
Eff. length for bending about zz: Lez=0 mm
Length of bearing: lb=75 mm
Bearing modification factor: kc90=1.5
Timber strength class: C24

Depth factor (Clause 3.2): kh=(150/h)^0.2=1.1151
Load-sharing modification factor: ksys=1.0
No notches exist at the support: kv=1.0

Design bending stress: 18.186 N/mm²
Design bending strength: 33.453 N/mm²
Design deflection: 18.703 mm
Limiting deflection: 120 mm
Design shear stress: 0.32961 N/mm²
Design shear strength: 5 N/mm²
Design bearing stress: 0.2549 N/mm²
Design bearing strength: 4.6875 N/mm²
Location: Ex1 - Solid timber joists & OSB/3 boarding

The design requirements in EC5 relate solely to residential floors and the following vibration verifications will need to be considered:

- fundamental frequency check
- static deflection under a point load check
- unit impulse velocity response check.

Effective span of joist \( L = 3.75 \text{ m} \)
Width of floor structure \( B = 4 \text{ m} \)
Joist spacing \( s = 400 \text{ mm} \)
Mass of the floor \( m_f = 35 \text{ kg/m}^2 \)
Depth of section \( h = 200 \text{ mm} \)
Width of section \( b = 47 \text{ mm} \)
Flooring thickness \( t = 18 \text{ mm} \)

Floor joist properties

Floor joist selected are solid timber joists.
Timber strength class C18.
Mean modulus of elasticity \( E_0\text{mean}=9000 \text{ N/mm}^2 \)

Flooring properties

Oriented strand board (OSB/3), service class 1 or 2 to BS EN 12369-1.
Mean modulus of elasticity \( E_f\text{mean}=4930 \text{ N/mm}^2 \)
Function value \( k\text{strut}=1 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of floor structure</td>
<td>4 m</td>
</tr>
<tr>
<td>Flooring thickness</td>
<td>18 mm</td>
</tr>
<tr>
<td>Floor fundamental frequency</td>
<td>15.853 Hz</td>
</tr>
<tr>
<td>Min allowable frequency</td>
<td>8 Hz</td>
</tr>
<tr>
<td>Floor static deflection</td>
<td>1.466 mm</td>
</tr>
<tr>
<td>Max allowable static deflection</td>
<td>1.8 mm</td>
</tr>
<tr>
<td>Actual unit impulse velocity</td>
<td>0.024465 m/(Ns²)</td>
</tr>
<tr>
<td>Max allowable impulse velocity</td>
<td>0.04267 m/(Ns²)</td>
</tr>
</tbody>
</table>

Therefore floor is OK.
Location: Ex2 - Solid timber joists & ply flooring

The design requirements in EC5 relate solely to residential floors and the following vibration verifications will need to be considered:

- fundamental frequency check
- static deflection under a point load check
- unit impulse velocity response check.

Effective span of joist          \( L = 3.6 \text{ m} \)
Width of floor structure        \( B = 7.2 \text{ m} \)
Joist spacing                   \( s = 600 \text{ mm} \)
Mass of the floor               \( mf = 47.204 \text{ kg/m}^2 \)
Depth of section                \( h = 195 \text{ mm} \)
Width of section                \( b = 47 \text{ mm} \)
Flooring thickness              \( t = 12 \text{ mm} \)

**Floor joist properties**

Floor joist selected are solid timber joists.
Timber strength class C24.
Mean modulus of elasticity       \( E0\text{mean}=11000 \text{ N/mm}^2 \)

**Flooring properties**

Mean modulus of elasticity       \( Ef0\text{mean}=11000 \text{ N/mm}^2 \)
Function value                   \( kstrut=1 \)

**DESIGN**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span of floor structure</td>
<td>3.6 m</td>
</tr>
<tr>
<td>Width of floor structure</td>
<td>7.2 m</td>
</tr>
<tr>
<td>Flooring thickness</td>
<td>12 mm</td>
</tr>
<tr>
<td>Floor fundamental frequency</td>
<td>12.872 Hz</td>
</tr>
<tr>
<td>Min allowable frequency</td>
<td>8 Hz</td>
</tr>
<tr>
<td>Floor static deflection</td>
<td>1.6652 mm</td>
</tr>
<tr>
<td>Max allowable static deflection</td>
<td>1.8 mm</td>
</tr>
<tr>
<td>Actual unit impulse velocity</td>
<td>0.025889 m/(Ns²)</td>
</tr>
<tr>
<td>Max allowable impulse velocity</td>
<td>0.034431 m/(Ns²)</td>
</tr>
</tbody>
</table>

Therefore floor is OK.
Location: Ex3 - Domestic floor (thin webbed joists & ply flooring)

The design requirements in EC5 relate solely to residential floors and the following vibration verifications will need to be considered:

- fundamental frequency check
- static deflection under a point load check
- unit impulse velocity response check.

Effective span of joist \( L = 3.75 \) m
Width of floor structure \( B = 4 \) m
Joist spacing \( s = 400 \) mm
Mass of the floor \( mf = 35 \) kg/m²
Flooring thickness \( t = 18.5 \) mm

**Floor joist properties**

Floor joist selected are glued thin webbed joists.
Timber strength class C18.
Mean modulus of elasticity \( E_0\text{\text{mean}} = 9000 \) N/mm²

**Flooring properties**

Plywood to be of service class 1 or 2 to BS EN 12369-1.
Plywood is Canadian Douglas Fir (unsanded).
Plywood comprises 7 plies.
Mean modulus of elasticity \( E_f\text{\text{mean}} = 9620 \) N/mm²
Second moment of area of a joist \( I_{\text{joist}} = 31E6 \) mm⁴
Function value \( k_{\text{strut}} = 1 \)

### DESIGN

- Span of floor structure: 3.75 m
- Width of floor structure: 4 m
- Flooring thickness: 18.5 mm
- Floor fundamental frequency: 15.769 Hz
- Min allowable frequency: 8 Hz
- Floor static deflection: 1.2404 mm
- Max allowable static deflection: 1.8 mm

### SUMMARY

- Actual unit impulse velocity: 0.020666 m/(Ns²)
- Max allowable impulse velocity: 0.039937 m/(Ns²)
- Therefore floor is OK.
Location: Ex4 - Domestic floor (joists are floor trusses)

The design requirements in EC5 relate solely to residential floors and the following vibration verifications will need to be considered:

- fundamental frequency check
- static deflection under a point load check
- unit impulse velocity response check.

Effective span of joist: L=3.75 m
Width of floor structure: B=4 m
Joist spacing: s=400 mm
Mass of the floor: mf=35 kg/m²
Flooring thickness: t=18 mm

Floor joist properties

Floor joist selected are mechanically jointed floor trusses.
Mean modulus of elasticity: E₀mean=11000 N/mm²

Flooring properties

Mean modulus of elasticity: Eᵢmean=4930 N/mm²
Second moment of area of a joist: Ijoist=31E6 mm⁴
Function value: kstrut=1

Design

<table>
<thead>
<tr>
<th>Component</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span of floor structure</td>
<td>3.75 m</td>
</tr>
<tr>
<td>Width of floor structure</td>
<td>4 m</td>
</tr>
<tr>
<td>Flooring thickness</td>
<td>18 mm</td>
</tr>
<tr>
<td>Floor fundamental frequency</td>
<td>17.433 Hz</td>
</tr>
<tr>
<td>Min allowable frequency</td>
<td>8 Hz</td>
</tr>
<tr>
<td>Floor static deflection</td>
<td>1.501 mm</td>
</tr>
<tr>
<td>Max allowable static deflection</td>
<td>1.8 mm</td>
</tr>
<tr>
<td>Actual unit impulse velocity</td>
<td>0.024245 m/(Ns²)</td>
</tr>
<tr>
<td>Max allowable impulse velocity</td>
<td>0.049823 m/(Ns²)</td>
</tr>
</tbody>
</table>

Therefore floor is OK.
Location: Ex5 - Domestic floor (glulam beams as floor joists)

The design requirements in EC5 relate solely to residential floors and the following vibration verifications will need to be considered:

- fundamental frequency check
- static deflection under a point load check
- unit impulse velocity response check.

Effective span of joist \( L = 5 \, \text{m} \)
Width of floor structure \( B = 4 \, \text{m} \)
Joist spacing \( s = 600 \, \text{mm} \)
Mass of the floor \( m_f = 100 \, \text{kg/m}^2 \)
Depth of section \( h = 300 \, \text{mm} \)
Width of section \( b = 100 \, \text{mm} \)
Flooring thickness \( t = 18 \, \text{mm} \)

Floor joist properties

Floor joist selected are glulam beams.
Glulam strength class GL24h
Mean modulus of elasticity \( E_{0\text{mean}} = 11500 \, \text{N/mm}^2 \)

Flooring properties

Mean modulus of elasticity \( E_{f0\text{mean}} = 4930 \, \text{N/mm}^2 \)
Function value \( k_{\text{strut}} = 1 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Span of floor structure</td>
<td>5 m</td>
</tr>
<tr>
<td>Width of floor structure</td>
<td>4 m</td>
</tr>
<tr>
<td>Flooring thickness</td>
<td>18 mm</td>
</tr>
<tr>
<td>Floor fundamental frequency</td>
<td>13.048 Hz</td>
</tr>
<tr>
<td>Min allowable frequency</td>
<td>8 Hz</td>
</tr>
<tr>
<td>Floor static deflection</td>
<td>0.51584 mm</td>
</tr>
<tr>
<td>Max allowable static deflection</td>
<td>1.408 mm</td>
</tr>
<tr>
<td>Actual unit impulse velocity</td>
<td>0.010404 m/(Ns²)</td>
</tr>
<tr>
<td>Max allowable impulse velocity</td>
<td>0.024764 m/(Ns²)</td>
</tr>
</tbody>
</table>

Therefore floor is OK.
Location: Ex6 - Domestic floor (stressed skin timber floor joists)

The design requirements in EC5 relate solely to residential floors and the following vibration verifications will need to be considered:

- fundamental frequency check
- static deflection under a point load check
- unit impulse velocity response check.

Effective span of joist \( L = 5.5 \text{ m} \)
Width of floor structure \( B = 4 \text{ m} \)
Joist spacing \( s = 330 \text{ mm} \)
Mass of the floor \( m_f = 35 \text{ kg/m}^2 \)
Depth of section \( h = 225 \text{ mm} \)
Width of section \( b = 75 \text{ mm} \)
Flooring thickness \( t = 20.5 \text{ mm} \)

Floor joist properties

Floor joist selected is a stressed skin timber floor joist.
Mean modulus of elasticity \( E_{0\text{mean}} = 11600 \text{ N/mm}^2 \)

Flooring properties

Plywood to be of service class 1 or 2 to BS EN 12369-1.
Plywood is Canadian Douglas Fir (unsanded).
Plywood comprises 7 plies.
Mean modulus of elasticity \( E_{f0\text{mean}} = 8930 \text{ N/mm}^2 \)
Function value \( k_{\text{strut}} = 1 \)

Span of floor structure 5.5 m
Width of floor structure 4 m
Flooring thickness 20.5 mm
Floor fundamental frequency 14.89 Hz
Min allowable frequency 8 Hz
Floor static deflection 1.1497 mm
Max allowable static deflection 1.2679 mm
Actual unit impulse velocity 0.014728 m/(Ns^2)
Max allowable impulse velocity 0.035942 m/(Ns^2)
Therefore floor is OK.
The design requirements in EC5 relate solely to residential floors and the following vibration verifications will need to be considered:

- fundamental frequency check
- static deflection under a point load check
- unit impulse velocity response check.

**Effective span of joist**
L = 3.75 m

**Width of floor structure**
B = 4 m

**Joist spacing**
s = 400 mm

**Mass of the floor**
mf = 35 kg/m²

**Flooring thickness**
t = 18 mm

### Floor joist properties

- Mean modulus of elasticity
  E₀mean = 11000 N/mm²

### Flooring properties

- Mean modulus of elasticity
  Ef₀mean = 4930 N/mm²

- Second moment of area of a joist
  I₀joist = 31E6 mm⁴

- Function value
  kstrut = 1

**DESIGN**

- Width of floor structure
  4 m

**SUMMARY**

- Flooring thickness
  18 mm

- Floor fundamental frequency
  17.433 Hz

- Min allowable frequency
  8 Hz

- Floor static deflection
  1.2124 mm

- Max allowable static deflection
  1.8 mm

- Actual unit impulse velocity
  0.024245 m/(Ns²)

- Max allowable impulse velocity
  0.046399 m/(Ns²)

Therefore floor is OK.
Location: Ax8 - Floor vibrations to EC5

The design requirements in EC5 relate solely to residential floors and the following vibration verifications will need to be considered:

- fundamental frequency check
- static deflection under a point load check
- unit impulse velocity response check.

Effective span of joist \( L = 1 \) m
Width of floor structure \( B = 10 \) m
Joist spacing \( s = 100 \) mm
Mass of the floor \( mf = 0.05 \) kg/m²
Flooring thickness \( t = 10 \) mm

Floor joist properties

Mean modulus of elasticity \( E_0 \text{mean} = 13000 \) N/mm²

Flooring properties

Oriented strand board (OSB/3), service class 1 or 2 to BS EN 12369-1.
Mean modulus of elasticity \( E_f \text{mean} = 1980 \) N/mm²
Second moment of area of a joist \( I_{\text{joist}} = 5E9 \) mm⁴
Function value \( k_{\text{strut}} = 1 \)

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Span of floor structure \( 1 \) m
Width of floor structure \( 10 \) m
Flooring thickness \( 10 \) mm
Floor fundamental frequency \( 179098 \) Hz
Min allowable frequency \( 8 \) Hz
Floor static deflection \( 0.10096E-3 \) mm
Max allowable static deflection \( 1.8 \) mm
Actual unit impulse velocity \( 0.00798 \) m/(Ns²)
Max allowable impulse velocity \( 179.99 \) m/(Ns²)
Therefore floor is OK.
Location: Timber connection default case 1

Timber to timber joint without pre-drilled holes to BS5268-2:2002

Axial load to be carried by joint F=6 kN
Thickness of headside member th=50 mm
Strength class C24 to Table 8.
Timber characteristic density pk1=350 kg/m³
Strength class C24 to Table 8.
Thickness of pointside timber tp=63 mm
Timber characteristic density pk2=350 kg/m³
End grain modification factor K43=1
Diameter of nails d=4.2 mm
Length of nails l=85 mm
Improved nail modification factor K44=1
Basic loads in N will be derived from formula given in Annex G.2.
Equations G1 to G6 in annex G in BS5268-2:2002.

Basic nail shear capacity basic=1*Gmin/(Fd*Kd)=518 N
Mod factor for duration of load K48=1.12
Mod factor for number of nails K50=1

Minimum nail spacings

Edge distance ⊥ to timber grain 21 mm
Timber is a softwood except Douglas fir therefore the spacings given in Table 60 for timber-to-timber joints are multiplied by 0.8.
Edge distance ⊥ to timber grain 21 mm
End distance parallel to grain 68 mm

Joint slip check

Slip per shear plane U=F*1000/Kser=7.2676 mm
Slip per fastener per shear plane Uf=U/n=0.66069 mm
Location: Timber connection default case 2

Timber to timber joint with pre-drilled holes to BS5268-2:2002

Holes not greater than 0.8 of the nail diameter

Axial load to be carried by joint $F=4$ kN
Thickness of headside member $th=38$ mm
Strength class C18 to Table 8.
Timber characteristic density $pk_1=320$ kg/m$^3$
Strength class C22 to Table 8.
Thickness of pointside timber $tp=38$ mm
Timber characteristic density $pk_2=340$ kg/m$^3$
End grain modification factor $K_{43}=1$
Diameter of nails $d=4.6$ mm
Length of nails $l=75$ mm
Improved nail modification factor $K_{44}=1.2$
Basic nail shear capacity $basich=620$ N
Basic nail shear capacity $basicp=620$ N
Pointside basic load is critical $basic=basicp=500.51$ N
Mod factor for duration of load $K_{48}=1$
Mod factor for number of nails $K_{50}=0.9$

Minimum nail spacings

Edge distance $\perp$ to timber grain 23 mm
Timber is a softwood except Douglas fir therefore the spacings given in Table 60 for timber-to-timber joints are multiplied by 0.8.
Edge distance $\perp$ to timber grain 23 mm
End distance parallel to grain 52 mm

Joint slip check

Slip per shear plane $U=F \times 1000/K_{ser}=3.0381$ mm
Slip per fastener per shear plane $Uf=U/n=0.37977$ mm
Location: Timber connection default case 3

Steel plate-to-timber joint to BS5268-2:2002

Axial load to be carried by joint $F=8$ kN  
Thickness of headside steel plate $th=10$ mm  
Strength class C24 to Table 8.

Thickness of pointside timber $tp=72$ mm  
Timber characteristic density $p_{k2}=350$ kg/m$^3$  
Diameter of nails $d=6$ mm  
Length of nails $l=58$ mm  
Improved nail modification factor $K_{44}=1.2$

Basic loads in N will be derived from formula given in Annex G.2.  
Equations G1 to G6 in annex G in BS5268-2:2002.

Basic nail shear capacity $basic=1 \cdot G_{min}/(F_d \cdot K_d)=1022$ N

Mod factor for duration of load $K_{48}=1.25$  
Mod factor for number of nails $K_{50}=1$

Minimum nail spacings

Nails are driven at right angles to the glued surface of pre-glued laminated members, therefore the spacings are multiplied by 0.9.  
Edge distance $\perp$ to timber grain 30 mm  
End distance parallel to grain 76 mm
Location: Timber connection default case 4

Plywood to timber joint to BS5268-2:2002 Nails driven into side grain

Axial load to be carried by joint $F=15$ kN

Plywood group 1 (see Clause 6.1) comprising:
(a) American construction and industrial plywood
(b) Canadian Douglas fir plywood
(c) Canadian softwood plywood
(d) Swedish softwood plywood
(e) Finnish conifer plywood

Thicknass of headside member $th=19$ mm
Timber characteristic density $pk_1=505$ kg/m$^3$
Strength class C18 to Table 8.
Thickness of pointside timber $tp=63$ mm
Timber characteristic density $pk_2=320$ kg/m$^3$
Diameter of nails $d=3.4$ mm
Length of nails $l=75$ mm

Basic loads in N will be derived from formula given in Annex G.2 Equations G1 to G6 in annex G in BS5268-2:2002.

Basic nail shear capacity $basic=1*Gmin/(Fd*Kd)=280.69$ N

Mod factor for duration of load $K48=1.12$
Mod factor for number of nails $K50=1$

Minimum nail spacings

End distance parallel to grain $48$ mm
Edge distance in the plywood $11$ mm
Loaded edge distance in timber $17$ mm
Distance between lines of nails perpendicular to grain $24$ mm
Distance between adjacent nails in any one line, parallel to grain $48$ mm

Joint slip check

Slip per shear plane $U=F*1000/Kser=24.611$ mm
Slip per fastener per shear plane $Uf=U/n=0.51272$ mm
Location: Timber connection default case 5

Particleboard to timber joint in accordance with BS5268-2:2002

Headside is particleboard without pre-drilled holes and nails driven into side grain of timber.
Axial load to be carried by joint $F=2.5$ kN
Thickness of headside member $th=18$ mm
Strength class C30 to Table 8.
Thickness of pointside timber $tp=47$ mm
Timber characteristic density $pk_2=380$ kg/m$^3$
Diameter of nails $d=3.8$ mm
Length of nails $l=50$ mm
Basic single shear lateral load $basic=140$ N
Mod factor for duration of load $K_{48}=1.4$
Mod factor for number of nails $K_{50}=0.9$

Minimum nail spacings

End distance parallel to grain $54$ mm
Loaded edge distance in particleboard $23$ mm
Unloaded edge distance in timber & part'board $12$ mm
Loaded edge distance in timber $19$ mm
Distance between lines of nails perpendicular to grain $27$ mm
Distance between adjacent nails in any one line, parallel to grain $54$ mm

Joint slip check

Slip per shear plane $U=F*1000/K_{ser}=2.8999$ mm
Slip per fastener per shear plane $U_f=U/n=0.19332$ mm
Location: Timber connection default case 6

Timber joint with nails in withdrawal - design to BS5268-2:2002

Axial load to be carried by joint \( F = 0.5 \text{ kN} \)
Strength class C22 to Table 8.
Thickness of pointside timber \( t_p = 75 \text{ mm} \)
Timber characteristic density \( p_k^2 = 340 \text{ kg/m}^3 \)
Diameter of nails \( d = 3.4 \text{ mm} \)
Pointside embedment of nail \( p_{en} = 65 \text{ mm} \)
Improved nail modification factor \( K_{45} = 1 \)
Basic withdrawal load \( W_{\text{basic}} = 1.93 \text{ N/mm penetration} \)
Mod factor for duration of load \( K_{48} = 1.25 \)
Mod factor for number of nails \( K_{50} = 1 \)
Permissible nail load \( p_{nl} = W_{\text{basic}} \times p_{en} \times K_{48} \times K_{49} \times K_{50} = 156.81 \text{ N} \)
Number of nails required \( n = \text{INT}(F \times 1000/p_{nl}) + 1 = 4 \)

Minimum nail spacings

Edge dist perpend to timber grain 17 mm
Timber is a softwood except Douglas fir therefore the spacings given in Table 60 for timber-to-timber joints are multiplied by 0.8.
Edge dist perpend to timber grain 17 mm
End distance parallel to grain 55 mm
Loaded edge distance in timber 17 mm
Distance between lines of nails perpendicular to grain 28 mm
Distance between adjacent nails in any one line, parallel to grain 55 mm
Location: Timber connection default case 4

Plywood to timber joint to BS5268-2:2002 Nails driven into side grain

Axial load to be carried by joint F=1.5 kN

Plywood group 2 (see Clause 6.1) comprising:
(a) Finnish birch-faced plywood
(b) Finnish birch plywood

Thickness of headside member       th=9 mm
Timber characteristic density      pk1=641 kg/m³
Strength class C14 to Table 8.
Thickness of pointside timber      tp=50 mm
Timber characteristic density      pk2=290 kg/m³
Diameter of nails                  d=2.7 mm
Length of nails                    l=50 mm

Basic loads in N will be derived from formula given in Annex G.2 Equations G1 to G6 in annex G in BS5268-2:2002.

Basic nail shear capacity          basic=1*Gmin/(Fd*Kd)=195.99 N

Mod factor for duration of load    K48=1.12
Mod factor for number of nails     K50=1

Minimum nail spacings

End distance parallel to grain     38 mm
Edge distance in the plywood      9 mm
Loaded edge distance in timber    14 mm
Distance between lines of nails
perpendicular to grain            19 mm
Distance between adjacent nails in any one line, parallel to grain 38 mm
Location: Timber connection default case 2

Timber to timber joint with pre-drilled holes to BS5268-2:2002

Holes not greater than 0.8 of the nail diameter

Axial load to be carried by joint $F=4$ kN
Thickness of headside member $th=60$ mm
Strength class C30 to Table 8.
Timber characteristic density $pk1=380$ kg/m³
Strength class C30 to Table 8.
Thickness of pointside timber $tp=75$ mm
Timber characteristic density $pk2=380$ kg/m³
End grain modification factor $K43=1$
Diameter of nails $d=5$ mm
Length of nails $l=120$ mm
Improved nail modification factor $K44=1.2$
Basic nail shear capacity $basich=778$ N
Basic nail shear capacity $basicp=778$ N
Pointside basic load is critical $basic=basicp=933.6$ N
Mod factor for duration of load $K48=1$
Mod factor for number of nails $K50=0.9$

Minimum nail spacings

Edge distance $\perp$ to timber grain 25 mm
Timber is a softwood except Douglas fir therefore the spacings given in Table 60 for timber-to-timber joints are multiplied by 0.8.
Edge distance $\perp$ to timber grain 25 mm
End distance parallel to grain 56 mm

Joint slip check

Slip per shear plane $U=F*1000/Kser=2.16$ mm
Slip per fastener per shear plane $Uf=U/n=0.43199$ mm
Location: Timber connection default case 3

Steel plate-to-timber joint to BS5268-2:2002

Axial load to be carried by joint $F=8$ kN
Thickness of headside steel plate $t_h=10$ mm
Strength class C24 to Table 8.
Thickness of pointside timber $t_p=125$ mm
Timber characteristic density $p_k^2=350$ kg/m$^3$
Diameter of nails $d=8$ mm
Length of nails $l=106$ mm
Improved nail modification factor $K_{44}=1.2$
Basic loads in N will be derived from formula given in Annex G.2.
Equations G1 to G6 in annex G in BS5268-2:2002.

Basic nail shear capacity $\text{basic}=1*G_{\min}/(F_d*K_d)=1642.9$ N

Mod factor for duration of load $K_{48}=1.25$
Mod factor for number of nails $K_{50}=1$

Minimum nail spacings

Nails are driven at right angles to the glued surface of pre-glued laminated members, therefore the spacings are multiplied by 0.9.
Edge distance $\perp$ to timber grain 40 mm
End distance parallel to grain 101 mm
Location: Timber connection default case 4

Plywood to timber joint to BS5268-2:2002 Nails driven into side grain

Axial load to be carried by joint \( F = 5 \text{ kN} \)

Plywood group 1 (see Clause 6.1) comprising:
(a) American construction and industrial plywood
(b) Canadian Douglas fir plywood
(c) Canadian softwood plywood
(d) Swedish softwood plywood
(e) Finnish conifer plywood

Thickness of headside member \( t_h = 15 \text{ mm} \)
Timber characteristic density \( p_{k1} = 505 \text{ kg/m}^3 \)
Strength class C14 to Table 8.

Thickness of pointside timber \( t_p = 63 \text{ mm} \)
Timber characteristic density \( p_{k2} = 290 \text{ kg/m}^3 \)

Diameter of nails \( d = 3.4 \text{ mm} \)
Length of nails \( l = 65 \text{ mm} \)

Basic loads in N will be derived from formula given in Annex G.2 Equations G1 to G6 in annex G in BS5268-2:2002.

Basic nail shear capacity \( \text{basic} = 1 \times G_{\text{min}} / (F_d \times K_d) = 215.24 \text{ N} \)

Mod factor for duration of load \( K_{48} = 1.12 \)
Mod factor for number of nails \( K_{50} = 1 \)

Minimum nail spacings

End distance parallel to grain \( 48 \text{ mm} \)
Edge distance in the plywood \( 11 \text{ mm} \)
Loaded edge distance in timber \( 17 \text{ mm} \)
Distance between lines of nails perpendicular to grain \( 24 \text{ mm} \)
Distance between adjacent nails in any one line, parallel to grain \( 48 \text{ mm} \)

Joint slip check

\( U = F \times 1000 / K_{\text{ser}} = 9.5089 \text{ mm} \)
\( U_f = U / n = 0.4528 \text{ mm} \)
Location: Timber connection default case 1

Timber to timber joint without pre-drilled holes to BS5268-2:2002

Axial load to be carried by joint $F=6\text{ kN}$
Thickness of headside member $th=50\text{ mm}$
Strength class C22 to Table 8.
Timber characteristic density $p_{k1}=340\text{ kg/m}^3$
Strength class C24 to Table 8.
Thickness of pointside timber $tp=50\text{ mm}$
Timber characteristic density $p_{k2}=350\text{ kg/m}^3$
End grain modification factor $K_{43}=1$
Diameter of nails $d=4.2\text{ mm}$
Length of nails $l=90\text{ mm}$
Improved nail modification factor $K_{44}=1$
Basic loads in N will be derived from formula given in Annex G.2.
Equations G1 to G6 in annex G in BS5268-2:2002.

Basic nail shear capacity $basic=1\times\frac{G_{\text{min}}}{(Fd\times K_{4d})}=563.24\text{ N}$
Mod factor for duration of load $K_{48}=1.12$
Mod factor for number of nails $K_{50}=1$

Minimum nail spacings

Edge distance $\perp$ to timber grain 21 mm
Timber is a softwood except Douglas fir therefore the spacings given in Table 60 for timber-to-timber joints are multiplied by 0.8.
Edge distance $\perp$ to timber grain 21 mm
End distance parallel to grain 68 mm

Joint slip check

Slip per shear plane $U=F\times1000/K_{\text{ser}}=7.5906\text{ mm}$
Slip per fastener per shear plane $Uf=U/n=0.75906\text{ mm}$
**Location:** Ex 10.13.1 Jack Porteous book p.417

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**Timber-to-timber joint without pre-drilled holes to EC5**

Characteristic permanent load \( G_k = 2.5 \) kN  
Characteristic variable load \( Q_k = 3.5 \) kN

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**PLAN**

Thickness of headside member \( t_h = 36 \) mm  
Timber strength class C22.  
Timber characteristic density \( p_{k1} = 340 \) kg/m³  
Timber strength class C22.  
Thickness of pointside timber \( t_p = 50 \) mm  
Timber characteristic density \( p_{k2} = 340 \) kg/m³  
Diameter of nails \( d = 3.35 \) mm  
Length of nails \( l = 65 \) mm  
Basic loads will be derived from expressions given in Section 8 assuming round nails.  
Min value of equations (a)-(f) \( G_{min} = 802.9 \)  
Char nail shear capacity basic = \( G_{min} = 802.9 \) N  
Material factor for connections \( \gamma_{M} = 1.3 \)  
Design resistance per nail \( p_{n} = k_{mod} \cdot \text{basic} / \gamma_{M} = 494.09 \) N  
Number of nails one side (A-B) \( n = \text{INT}(F_d/2 \cdot 1000/p_{n}) + 1 = 9 \)  
Number of nails both sides (A-B) \( N_{tot} = 18 \)  

**Minimum nail spacings - EC5 Table 8.2 & Figure 8.7**

Minimum spacing of nails within  
one row parallel to grain \( a_1 = (5+5 \cdot \cos(\alpha)) \cdot d = 33.5 \) mm  
Actual spacing of nails within  
one row parallel to grain \( a_1 = 33.5 \) mm  
Spacing of rows of nails  
perpendicular to grain \( a_2 = 5 \cdot d = 16.75 \) mm  
End distance (loaded) \( a_3l = (10+5 \cdot \cos(\alpha)) \cdot d = 50.25 \) mm  
End distance (unloaded) \( a_3c = 10 \cdot d = 33.5 \) mm  
Edge distance (loaded) \( a_4l = (5+2 \cdot \sin(\alpha)) \cdot d = 16.75 \) mm  
Edge distance (unloaded) \( a_4c = (5+5 \cdot \sin(\alpha)) \cdot d = 16.75 \) mm  

**Joint slip check - EC5 Section 7**

Total instantaneous slip SLS \( u_{s} = 2 \cdot u_{i, s} = 2.4255 \) mm  
Total instantaneous slip ULS \( u_{t} = 2 \cdot u_{i, t} = 3.4867 \) mm  
Psi 2 factor (variable action) \( \psi_{2} = 0.3 \)  
\( u_{s} = 2 \cdot (u_{i, s} \cdot G_{k} / F_{sd} + u_{i, s} \cdot Q_{k} / F_{ds}) \cdot (1 + \psi_{2} \cdot k_{def}) = 3.5736 \) mm  
\( u_{t} = 2 \cdot (u_{i, t} \cdot G_{k} / F_{sd} + u_{i, t} \cdot Q_{k} / F_{ds}) \cdot (1 + \psi_{2} \cdot k_{def}) = 5.1371 \) mm
DESIGN SUMMARY

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<td>Design axial load</td>
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<td>Headside member thickness</td>
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<td>Points side member thickness</td>
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<td>Nail details between A-B:</td>
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<td>Number of nails per side</td>
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<td>Total number of nails</td>
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<tr>
<td>Nail details between B-C:</td>
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<tr>
<td>Number of nails per side</td>
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<td>Total number of nails</td>
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<tr>
<td>Diameter of nails</td>
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<tr>
<td>Length of smooth nails</td>
<td>65 mm</td>
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</tbody>
</table>

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Pre-drilled holes are not required.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 46.9 mm.
Location: Timber connection default case 1

Timber-to-timber joint without pre-drilled holes to EC5

Characteristic permanent load $G_k=3$ kN
Characteristic variable load $Q_k=3$ kN

PLAN

- Thickness of headside member $th=50$ mm
- Timber characteristic density $p_{k1}=350$ kg/m³
- Thickness of pointside timber $tp=63$ mm
- Timber characteristic density $p_{k2}=350$ kg/m³
- Diameter of nails $d=4.2$ mm
- Length of nails $l=85$ mm

Basic loads will be derived from expressions given in Section 8 assuming round nails.
Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.

Min value of equations (a)-(f) $G_{min}=1173.5$
Char nail shear capacity basic=$G_{min}=1173.5$ N
Material factor for connections $\gamma_{M}=1.3$
Design resistance per nail $p_{nl}=k_{mod} \times G_{min}/\gamma_{M}=722.17$ N
Number of nails required $n=\text{INT}(F_d \times 1000/p_{nl})+1=12$

Minimum nail spacings - EC5 Table 8.2 & Figure 8.7

Minimum spacing of nails within one row parallel to grain $a_1=(5+5 \times \text{COS}(\alpha)) \times d=42$ mm
Actual spacing of nails within one row parallel to grain $a_1=42$ mm
Spacing of rows of nails perpendicular to grain $a_2=5 \times d=21$ mm
End distance (loaded) $a_3t=(10+5 \times \text{COS}(\alpha)) \times d=63$ mm
End distance (unloaded) $a_3c=10 \times d=42$ mm
Edge distance (loaded) $a_4t=(5+2 \times \text{SIN}(\alpha)) \times d=21$ mm
Edge distance (unloaded) $a_4c=(5+5 \times \text{SIN}(\alpha)) \times d=21$ mm

Joint slip check - EC5 Section 7

Total instantaneous slip SLS $u_{ser}=2 \times u_{iser}=1.4535$ mm
Total instantaneous slip ULS $u_{ult}=2 \times u_{iult}=2.0713$ mm
Psi 2 factor (variable action) $\psi_2=0.3$

$u_{ser}=2 \times (u_{iser} \times G_k/F_{dser}+(1+\psi_2p \times k_{def})+u_{iser} \times Q_k/F_{dser}+(1+\psi_2p \times k_{def}))=2.0204$ mm

$u_{ult}=2 \times (u_{iult} \times G_k/F_{dser}+(1+\psi_2p \times k_{def})+u_{iult} \times Q_k/F_{dser}+(1+\psi_2p \times k_{def}))=2.8791$ mm
DESIGN SUMMARY

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<td>Pointside member thickness</td>
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<td>Number of nails required</td>
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<td>Diameter of nails</td>
<td>4.2 mm</td>
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<tr>
<td>Length of smooth nails</td>
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</table>

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Pre-drilled holes are not required.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 58.8 mm.
Location: Timber connection default case 2

Timber-to-timber joint with pre-drilled holes to EC5

Holes not greater than 0.8 of the nail diameter

**Characteristic permanent load** \( G_k = 2 \) kN

**Characteristic variable load** \( Q_k = 2 \) kN

**Plan**

- Thickness of headside member \( th = 38 \) mm
- Timber strength class C22.
- Timber characteristic density \( p_k1 = 340 \) kg/m³
- Timber strength class C22.
- Thickness of pointside timber \( tp = 38 \) mm
- Timber characteristic density \( p_k2 = 340 \) kg/m³
- Diameter of nails \( d = 4.6 \) mm
- Length of nails \( l = 75 \) mm

Basic loads in N will be derived from expressions given in Section 8 assuming round nails.

Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.

Min value of equations (a)-(f) \( G_{min} = 1754.8 \) N

Char nail shear capacity basic \( = G_{min} = 1754.8 \) N

Material factor for connections \( \gamma_m = 1.3 \)

Design resistance per nail \( p_{nl} = k_{mod} \times \text{basic} / \gamma_m = 1079.9 \) N

Number of nails required \( n = \text{INT} (F_d \times 1000 / p_{nl}) + 1 = 6 \)

**Minimum nail spacings - EC5 Table 8.2 & Figure 8.7**

- Spacing of nails within one row parallel to grain \( a_1 = (4 + \cos(\alpha)) \times d = 23 \) mm
- Actual spacing of nails within one row parallel to grain \( a_1 = 23 \) mm
- Spacing of rows of nails perpendicular to grain \( a_2 = (3 + \sin(\alpha)) \times d = 13.8 \) mm
- End distance (loaded) \( a_3t = (7 + 5 \cos(\alpha)) \times d = 55.2 \) mm
- End distance (unloaded) \( a_3c = 7 \times d = 32.2 \) mm
- Edge distance (loaded) \( a_4t = (3 + 2 \sin(\alpha)) \times d = 13.8 \) mm
- Edge distance (unloaded) \( a_4c = 3 \times d = 13.8 \) mm

**Joint slip check - EC5 Section 7**

- Total instantaneous slip SLS \( u_{ser} = 2 \times u_{ser} = 1.0634 \) mm
- Total instantaneous slip ULS \( u_{ult} = 2 \times u_{ult} = 1.5153 \) mm
- Psi 2 factor (variable action) \( \psi_2v = 0.3 \)
- \( u_{ser} = 2 \times (u_{ser} \times G_k' / F_{dser} \times (1 + \psi_2p \times k_{def}) + u_{ser} \times Q_k' / F_{dser} \times (1 + \psi_2v \times k_{def})) = 1.6163 \) mm
ufult=2*(uiult*Gk'/Fdser*(1+psi2p*kdef)+uiult*Qk'/Fdser*(1+psi2v *kdef))=2.3033 mm

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load</td>
<td>5.7 kN</td>
</tr>
<tr>
<td>Headside member thickness</td>
<td>38 mm</td>
</tr>
<tr>
<td>Pointside member thickness</td>
<td>38 mm</td>
</tr>
<tr>
<td>Number of nails required</td>
<td>6</td>
</tr>
<tr>
<td>Diameter of nails</td>
<td>4.6 mm</td>
</tr>
<tr>
<td>Length of smooth nails</td>
<td>75 mm</td>
</tr>
</tbody>
</table>

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Provide pre-drilled holes.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 64.4 mm.
Location: Timber connection default case 3

Steel plate-to-timber joint to EC5

Characteristic permanent load \( G_k = 4 \, \text{kN} \)
Characteristic variable load \( Q_k = 4 \, \text{kN} \)

\[
\begin{align*}
\text{Thickess of headside steel plate } & \text{th} = 10 \, \text{mm} \\
\text{Timber strength class } & \text{C24.} \\
\text{Thickness of pointside timber } & \text{tp} = 72 \, \text{mm} \\
\text{Timber characteristic density } & \text{pk2} = 350 \, \text{kg/m}^3 \\
\text{Diameter of nails } & \text{d} = 6 \, \text{mm} \\
\text{Length of nails } & \text{l} = 58 \, \text{mm} \\
\text{Basic loads in N will be derived from expressions given in Section 8} \\
\text{assuming round nails.} \\
\text{Expressions (a) to (e) for fasteners in single shear refer to section} \\
8.2.3 \text{ in EC5. Zero contribution from rope effect will be considered.} \\
\text{Min value of equations (a)-(e) } & \text{Gmin} = 2538.3 \\
\text{Char nail shear capacity } & \text{basic} = \text{Gmin} = 2538.3 \, \text{N} \\
\text{Material factor for connections } & \text{gmM} = 1.3 \\
\text{Design resistance per nail } & \text{pn1} = \text{kmod} \times \text{basic} / \text{gmM} = 1269.2 \, \text{N} \\
\text{Number of nails required } & \text{n} = \text{INT}(\text{Fd} \times 1000 / \text{pn1}) + 1 = 9 \\
\end{align*}
\]

Minimum nail spacings - EC5 Table 8.2 & Figure 8.7

\[
\begin{align*}
\text{Minimum spacing of nails within} \\
\text{one row parallel to grain} & \text{a1} = (5+7\times\text{COS(alpha)}) \times \text{d} = 72 \, \text{mm} \\
\text{Actual spacing of nails within} \\
\text{one row parallel to grain} & \text{a1} = 72 \, \text{mm} \\
\text{Spacing of rows of nails} \\
\text{perpendicular to grain} & \text{a2} = 5 \times \text{d} = 30 \, \text{mm} \\
\text{End distance (loaded)} & \text{a3t} = (10+5\times\text{COS(alpha)}) \times \text{d} = 90 \, \text{mm} \\
\text{End distance (unloaded)} & \text{a3c} = 10 \times \text{d} = 60 \, \text{mm} \\
\text{Edge distance (loaded)} & \text{a4t} = (5+2\times\text{SIN(alpha)}) \times \text{d} = 30 \, \text{mm} \\
\text{Edge distance (unloaded)} & \text{a4c} = (5+5\times\text{SIN(alpha)}) \times \text{d} = 30 \, \text{mm} \\
\end{align*}
\]

Joint slip check - EC5 Section 7

\[
\begin{align*}
\text{Total instantaneous slip SLS } & \text{utser} = 2 \times \text{uiser} = 0.97128 \, \text{mm} \\
\text{Total instantaneous slip ULS } & \text{utuls} = 2 \times \text{uiult} = 1.3841 \, \text{mm} \\
\text{Psi 2 factor (variable action) } & \text{psi2v} = 0.3 \\
\text{ufser} = 2 \times (\text{uiser} \times \text{Gk} / \text{Fdser} \times (1+\text{psi2p} \times \text{kdef}) + \text{uiser} \times \text{Qk} / \text{Fdser} \times (1+\text{psi2v} \times \text{kdef})) = 2.234 \, \text{mm} \\
\text{ufult} = 2 \times (\text{uiult} \times \text{Gk} / \text{Fdser} \times (1+\text{psi2p} \times \text{kdef}) + \text{uiult} \times \text{Qk} / \text{Fdser} \times (1+\text{psi2v} \times \text{kdef})) = 3.1834 \, \text{mm} \\
\end{align*}
\]
DESIGN SUMMARY

- Design axial load: 11.4 kN
- Headside member thickness: 10 mm
- Pointside member thickness: 72 mm
- Number of nails required: 9
- Diameter of nails: 6 mm
- Length of smooth nails: 58 mm

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Pre-drilled holes are not required.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 84 mm.
Location: Timber connection default case 4

Plywood-to-timber joint to EC5 nails driven into side grain

Characteristic permanent load \( G_k = 7.5 \) kN
Characteristic variable load \( Q_k = 7.5 \) kN

\[
\begin{array}{c}
F_d \\
\text{th} \\
\text{tp}
\end{array}
\]

PLAN

Plywood group 1 comprising:
(a) American construction and industrial plywood
(b) Canadian Douglas fir plywood
(c) Canadian softwood plywood
(d) Swedish softwood plywood
(e) Finnish conifer plywood

Thickness of headside member \( \text{th} = 19 \) mm
Timber characteristic density \( p_k1 = 505 \) kg/m\(^3\)
Timber strength class C18.
Thickness of pointside timber \( \text{tp} = 63 \) mm
Timber characteristic density \( p_k2 = 320 \) kg/m\(^3\)
Diameter of nails \( d = 3.4 \) mm
Length of nails \( l = 75 \) mm

Basic loads in N will be derived from expressions given in Section 8 assuming round nails.
Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.

Min value of equations (a)-(f) \( G_{\text{min}} = 925.51 \) N

Char nail shear capacity basic = \( G_{\text{min}} = 925.51 \) N
Material factor for connections \( \gamma_M = 1.3 \)
Design resistance per nail \( p_n1 = k_{\text{mod}} \times \text{basic} / \gamma_M = 569.55 \) N
Number of nails required \( n = \text{INT}(F_d \times 1000 / p_n1) + 1 = 38 \)

Minimum nail spacings - EC5 Table 8.2 & Figure 8.7

Minimum spacing of nails within one row parallel to grain \( a_1 = (5 + 5 \times \text{COS}(\alpha)) \times d = 34 \) mm
Actual spacing of nails within one row parallel to grain \( a_1 = 34 \) mm
Spacing of rows of nails perpendicular to grain \( a_2 = 5 \times d = 17 \) mm
End distance (loaded) \( a_{3t} = (10 + 5 \times \text{COS}(\alpha)) \times d = 51 \) mm
End distance (unloaded) \( a_{3c} = 10 \times d = 34 \) mm
Edge distance (loaded) \( a_{4t} = (5 + 2 \times \text{SIN}(\alpha)) \times d = 17 \) mm
Edge distance (unloaded) \( a_{4c} = (5 + 2 \times \text{SIN}(\alpha)) \times d = 17 \) mm
Joint slip check - EC5 Section 7

Total instantaneous slip SLS \( u_{SLS} = 2 * u_{user} = 1.5544 \text{ mm} \)

Total instantaneous slip ULS \( u_{ULS} = 2 * u_{ult} = 2.215 \text{ mm} \)

Psi 2 factor (variable action) \( \psi_2v = 0.3 \)

\[
u_{SLS} = 2 \left( u_{user} \frac{G_k'}{F_{dser}} (1 + \psi_2p \times k_{def}) + u_{user} \frac{Q_k'}{F_{dser}} (1 + \psi_2v \times k_{def}) \right) = 2.1606 \text{ mm}
\]

\[
u_{ULS} = 2 \left( u_{ult} \frac{G_k'}{F_{dser}} (1 + \psi_2p \times k_{def}) + u_{ult} \frac{Q_k'}{F_{dser}} (1 + \psi_2v \times k_{def}) \right) = 3.0788 \text{ mm}
\]

**DESIGN SUMMARY**

Design axial load 21.375 kN

Headside member thickness 19 mm

Pointside member thickness 63 mm

Number of nails required 38

Diameter of nails 3.4 mm

Length of smooth nails 75 mm

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Pre-drilled holes are not required.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 47.6 mm.
Particleboard to timber joint in accordance with EC5

Headside is particleboard without pre-drilled holes and nails are driven into side grain of timber.
Characteristic permanent load $G_k=1.25$ kN
Characteristic variable load $Q_k=1.25$ kN

Plan

Thickness of headside member $th=18$ mm
Particleboard char density $p_{k1}=400$ kg/m³
Timber strength class C30.
Thickness of pointside timber $tp=47$ mm
Timber characteristic density $p_{k2}=380$ kg/m³
Diameter of nails $d=3.8$ mm
Length of nails $l=50$ mm

Basic loads in N will be derived from expressions given in Section 8 assuming round nails.
Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.

Min value of equations (a)-(f) $G_{\text{min}}=1010.3$ N

Char nail shear capacity $G_{\text{basic}}=G_{\text{min}}=1010.3$ N
Material factor for connections $\Gamma_{\text{M}}=1.3$

Design resistance per nail $p_{\text{n1}}=k_{\text{mod}}*G_{\text{basic}}/\Gamma_{\text{M}}=621.75$ N
Number of nails required $n=\text{INT}(F_d*1000/p_{\text{n1}})+1=6$

Minimum nail spacings - EC5 Table 8.2 & Figure 8.7

Minimum spacing of nails within one row parallel to grain $a_1=(5+5*\text{COS}(\alpha))*d=38$ mm
Actual spacing of nails within one row parallel to grain $a_1=38$ mm
Spacing of rows of nails perpendicular to grain $a_2=5*d=19$ mm
End distance (loaded) $a_3=(10+5*\text{COS}(\alpha))*d=57$ mm
End distance (unloaded) $a_3c=10*d=38$ mm
Edge distance (loaded) $a_4=(5+2*\text{SIN}(\alpha))*d=19$ mm
Edge distance (unloaded) $a_4c=(5+5*\text{SIN}(\alpha))*d=19$ mm

Joint slip check - EC5 Section 7

Total instantaneous slip SLS $u_{\text{user}}=2*u_{\text{isuer}}=1.1599$ mm
Total instantaneous slip ULS $u_{\text{ult}}=2*u_{\text{isuer}}=1.6529$ mm
Psi 2 factor (variable action) $\psi_{2v}=0.3$

$u_{\text{user}}=2*(u_{\text{isuer}}*G_k/F_d*1+p_{\text{isuer}}*k_{\text{def}})+u_{\text{isuer}}*Q_k/F_d*1+p_{\text{isuer}}*k_{\text{def}})=1.7631$ mm
ufult=2*(uiult*Gk'/Fdser*(1+psi2p*kdef)+uiult*Qk'/Fdser*(1+psi2v *kdef))=2.5124 mm

### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load</td>
<td>3.5625 kN</td>
</tr>
<tr>
<td>Headside member thickness</td>
<td>18 mm</td>
</tr>
<tr>
<td>Pointside member thickness</td>
<td>47 mm</td>
</tr>
<tr>
<td>Number of nails required</td>
<td>6</td>
</tr>
<tr>
<td>Diameter of nails</td>
<td>3.8 mm</td>
</tr>
<tr>
<td>Length of smooth nails</td>
<td>50 mm</td>
</tr>
</tbody>
</table>

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Pre-drilled holes are not required.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 53.2 mm.
Location: Timber connection default case 6

Timber joint with nails in withdrawal to EC5

Characteristic permanent load \( G_k = 0.25 \text{ kN} \)
Characteristic variable load \( Q_k = 0.25 \text{ kN} \)
Thickness of headside member \( t_h = 50 \text{ mm} \)
Timber strength class C22.
Thickness of pointside timber \( t_p = 75 \text{ mm} \)
Timber characteristic density \( p_{k2} = 340 \text{ kg/m}^3 \)
Diameter of nails \( d = 3.4 \text{ mm} \)
Pointside embedment of nail \( p_{en} = 65 \text{ mm} \)
Nail head diameter \( d_h = 7.65 \text{ mm} \)

Material factor for connections \( \gamma_M = 1.3 \)
Nail design withdrawal capacity \( F_{ax,Rd} = k_{mod} \cdot F_{ax,Rk} \cdot w_{crf} / \gamma_M = 314.43 \text{ N} \)
Number of nails required \( n = \text{INT} (F_d / F_{ax,Rd}) + 1 = 3 \)

Minimum nail spacings - EC5 Table 8.2 & Figure 8.7

Minimum spacing of nails within one row parallel to grain \( a_1 = (5 + 5 \cdot \cos(\alpha)) \cdot d = 34 \text{ mm} \)
Actual spacing of nails within one row parallel to grain \( a_1 = 34 \text{ mm} \)
Spacing of rows of nails perpendicular to grain \( a_2 = 5 \cdot d = 17 \text{ mm} \)
End distance (loaded) \( a_{3t} = (10 + 5 \cdot \cos(\alpha)) \cdot d = 51 \text{ mm} \)
End distance (unloaded) \( a_{3c} = 10 \cdot d = 34 \text{ mm} \)
Edge distance (loaded) \( a_{4t} = (5 + 2 \cdot \sin(\alpha)) \cdot d = 17 \text{ mm} \)
Edge distance (unloaded) \( a_{4c} = (5 + 5 \cdot \sin(\alpha)) \cdot d = 17 \text{ mm} \)

DESIGN SUMMARY

- Design axial load: 0.7125 kN
- Headside member thickness: 50 mm
- Pointsie member thickness: 75 mm
- Number of nails required: 3
- Diameter of nails: 3.4 mm
- Length of smooth nails: 115 mm

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Pre-drilled holes are not required.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 47.6 mm.
Location: Timber connection default case 4

Plywood-to-timber joint to EC5 nails driven into side grain

Characteristic permanent load \( G_k = 0.75 \text{ kN} \)
Characteristic variable load \( Q_k = 0.75 \text{ kN} \)

PLAN

Plywood group 2 comprising:
  (a) Finnish birch-faced plywood
  (b) Finnish birch plywood

Thickness of headside member \( \text{th} = 9 \text{ mm} \)
Timber characteristic density \( \rho_{k1} = 641 \text{ kg/m}^3 \)
Timber strength class C14.
Thickness of pointside timber \( \text{tp} = 50 \text{ mm} \)
Timber characteristic density \( \rho_{k2} = 290 \text{ kg/m}^3 \)
Diameter of nails \( d = 2.7 \text{ mm} \)
Length of nails \( l = 50 \text{ mm} \)

Basic loads in N will be derived from expressions given in Section 8 assuming round nails.

Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.

Min value of equations (a)-(f) \( G_{\text{min}} = 564.44 \text{ N} \)

Char nail shear capacity basic \( = G_{\text{min}} = 564.44 \text{ N} \)
Material factor for connections \( \gamma_M = 1.3 \)
Design resistance per nail \( p_{\text{n1}} = k_{\text{mod}} \times \text{basic} / \gamma_M = 282.22 \text{ N} \)
Number of nails required \( n = \text{INT} \left( F_d \times 1000 / p_{\text{n1}} \right) + 1 = 8 \)

Minimum nail spacings – EC5 Table 8.2 & Figure 8.7

Spacing of nails within one row parallel to grain \( a_1 = (4 + \cos(\alpha)) \times d = 13.5 \text{ mm} \)
Actual spacing of nails within one row parallel to grain \( a_1 = 13.5 \text{ mm} \)
Spacing of rows of nails perpendicular to grain \( a_2 = (3 + \sin(\alpha)) \times d = 8.1 \text{ mm} \)
End distance (loaded) \( a_3t = (7 + 5 \times \cos(\alpha)) \times d = 32.4 \text{ mm} \)
End distance (unloaded) \( a_3c = 7 \times d = 18.9 \text{ mm} \)
Edge distance (loaded) \( a_4t = (3 + 2 \times \sin(\alpha)) \times d = 8.1 \text{ mm} \)
Edge distance (unloaded) \( a_4c = 3 \times d = 8.1 \text{ mm} \)

Joint slip check – EC5 Section 7

Total instantaneous slip SLS \( u_{\text{suser}} = 2 \times u_{\text{user}} = 0.64684 \text{ mm} \)
Total instantaneous slip ULS \( u_{\text{ult}} = 2 \times u_{\text{ult}} = 0.92175 \text{ mm} \)
Psi 2 factor (variable action) \( \psi_{2v} = 0.3 \)
ufser=2*(uiser*Gk'/Fdser*(1+psi2p*kdef)+uiser*Qk'/Fdser*(1+psi2v
 *kdef))=1.4877 mm
ufult=2*(uiult*Gk'/Fdser*(1+psi2p*kdef)+uiult*Qk'/Fdser*(1+psi2v
 *kdef))=2.12 mm

DESIGN SUMMARY
Design axial load 2.1375 kN
Headside member thickness 9 mm
Pointside member thickness 50 mm
Number of nails required 8
Diameter of nails 2.7 mm
Length of smooth nails 50 mm

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Provide pre-drilled holes.
In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 37.8 mm.
Location: Timber connection default case 2

Timber-to-timber joint with pre-drilled holes to EC5

Holes not greater than 0.8 of the nail diameter

Characteristic permanent load \( G_k = 2 \text{kN} \)

Characteristic variable load \( Q_k = 2 \text{kN} \)

\[
\begin{align*}
F_d & \rightarrow \\
\text{th} & \\
\text{tp} & \\
\end{align*}
\]

PLAN

Thickness of headside member \( \text{th} = 60 \text{ mm} \)

Timber strength class C30.

Timber characteristic density \( \rho_{k1} = 380 \text{ kg/m}^3 \)

Timber strength class C30.

Thickness of pointside timber \( \text{tp} = 75 \text{ mm} \)

Timber characteristic density \( \rho_{k2} = 380 \text{ kg/m}^3 \)

Diameter of nails \( d = 5 \text{ mm} \)

Length of nails \( l = 120 \text{ mm} \)

Basic loads in N will be derived from expressions given in Section 8 assuming round nails.

Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.

Min value of equations (a)-(f) \( G_{\text{min}} = 2151.1 \text{N} \)

Char nail shear capacity \( \text{basic} = G_{\text{min}} = 2151.1 \text{N} \)

Material factor for connections \( \gamma_M = 1.3 \)

Design resistance per nail \( p_{nl} = k_{mod} \times \text{basic} / \gamma_M = 1323.7 \text{N} \)

Number of nails required \( n = \text{INT} (F_d \times 1000 / p_{nl}) + 1 = 5 \)

Minimum nail spacings - EC5 Table 8.2 & Figure 8.7

Spacing of nails within one
row parallel to grain \( a_1 = (4 + \cos(\alpha)) \times d = 25 \text{ mm} \)

Actual spacing of nails within
one row parallel to grain \( a_1 = 25 \text{ mm} \)

Spacing of rows of nails
perpendicular to grain \( a_2 = (3 + \sin(\alpha)) \times d = 15 \text{ mm} \)

End distance (loaded) \( a_3t = (7 + 5 \times \cos(\alpha)) \times d = 60 \text{ mm} \)

End distance (unloaded) \( a_3c = 7 \times d = 35 \text{ mm} \)

Edge distance (loaded) \( a_4t = (3 + 4 \times \sin(\alpha)) \times d = 15 \text{ mm} \)

Edge distance (unloaded) \( a_4c = 3 \times d = 15 \text{ mm} \)

Joint slip check - EC5 Section 7

Total instantaneous slip SLS \( u_{\text{user}} = 2 \times u_{\text{user}} = 0.99358 \text{ mm} \)

Total instantaneous slip ULS \( u_{\text{ult}} = 2 \times u_{\text{ult}} = 1.4159 \text{ mm} \)

Psi 2 factor (variable action) \( \psi_{2v} = 0.3 \)

\[
\begin{align*}
\text{ufser} &= 2 \times (u_{\text{user}} \times G_k / F_d \times (1 + \psi_{2p} \times \kappa_{\text{def}}) + u_{\text{user}} \times Q_k / F_d \times (1 + \psi_{2v} \times \kappa_{\text{def}})) = 1.5102 \text{ mm}
\end{align*}
\]
ufult=2*(uiult*Gk'/Fdser*(1+ψ2p*kdef)+uiult*Qk'/Fdser*(1+ψ2v *kdef))=2.1521 mm

DESIGN SUMMARY

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
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</thead>
<tbody>
<tr>
<td>Design axial load</td>
<td>5.7 kN</td>
</tr>
<tr>
<td>Headside member thickness</td>
<td>60 mm</td>
</tr>
<tr>
<td>Pointside member thickness</td>
<td>75 mm</td>
</tr>
<tr>
<td>Number of nails required</td>
<td>5</td>
</tr>
<tr>
<td>Diameter of nails</td>
<td>5 mm</td>
</tr>
<tr>
<td>Length of smooth nails</td>
<td>120 mm</td>
</tr>
</tbody>
</table>

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Provide pre-drilled holes.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 70 mm.
Location: Timber connection default case 3

Steel plate-to-timber joint to EC5

Characteristic permanent load \( G_k = 4 \) kN
Characteristic variable load \( Q_k = 4 \) kN

Thickness of headside steel plate \( \text{th} = 10 \) mm
Timber strength class C24.
Thickness of pointside timber \( \text{tp} = 125 \) mm
Timber characteristic density \( \rho_k = 350 \) kg/m\(^3\)
Diameter of nails \( d = 8 \) mm
Length of nails \( l = 106 \) mm

Basic loads in N will be derived from expressions given in Section 8 assuming round nails.

Expressions (a) to (e) for fasteners in single shear refer to section 8.2.3 in EC5. Zero contribution from rope effect will be considered.

Min value of equations (a)-(e) \( G_{\text{min}} = 5473.5 \) N

Char nail shear capacity \( \text{basic} = G_{\text{min}} = 5473.5 \) N
Material factor for connections \( \gamma_M = 1.3 \)
Design resistance per nail \( \text{pnl} = k_{\text{mod}} \times \text{basic} / \gamma_M = 2736.7 \) N
Number of nails required \( n = \text{INT}(F_d \times 1000 / \text{pnl}) + 1 = 5 \)

Minimum nail spacings - EC5 Table 8.2 & Figure 8.7

Minimum spacing of nails within one row parallel to grain \( a_1 = (5 + 7 \times \text{COS(alpha)}) \times d = 96 \) mm
Actual spacing of nails within one row parallel to grain \( a_1 = 96 \) mm
Spacing of rows of nails perpendicular to grain \( a_2 = 5 \times d = 40 \) mm
End distance (loaded) \( a_3t = (10 + 5 \times \text{COS(alpha)}) \times d = 120 \) mm
End distance (unloaded) \( a_3c = 10 \times d = 80 \) mm
Edge distance (loaded) \( a_4t = (5 + 2 \times \text{SIN(alpha)}) \times d = 40 \) mm
Edge distance (unloaded) \( a_4c = (5 + 5 \times \text{SIN(alpha)}) \times d = 40 \) mm

Joint slip check - EC5 Section 7

Total instantaneous slip SLS \( u_{\text{user}} = 2 \times u_{\text{user}} = 1.3889 \) mm
Total instantaneous slip ULS \( u_{\text{ult}} = 2 \times u_{\text{ult}} = 1.9792 \) mm
Psi 2 factor (variable action) \( \psi_2 = 0.3 \)
\( u_{\text{ser}} = 2 \times (u_{\text{ser}} \times G_k / F_d \times (1 + \psi_2 \times \kappa_{\text{def}}) + u_{\text{ser}} \times Q_k / F_d \times (1 + \psi_2 \times \kappa_{\text{def}})) = 3.1944 \) mm
\( u_{\text{ult}} = 2 \times (u_{\text{ult}} \times G_k / F_d \times (1 + \psi_2 \times \kappa_{\text{def}}) + u_{\text{ult}} \times Q_k / F_d \times (1 + \psi_2 \times \kappa_{\text{def}})) = 4.5521 \) mm
### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load</td>
<td>11.4 kN</td>
</tr>
<tr>
<td>Headsisde member thickness</td>
<td>10 mm</td>
</tr>
<tr>
<td>Pointside member thickness</td>
<td>125 mm</td>
</tr>
<tr>
<td>Number of nails required</td>
<td>5</td>
</tr>
<tr>
<td>Diameter of nails</td>
<td>8 mm</td>
</tr>
<tr>
<td>Length of smooth nails</td>
<td>106 mm</td>
</tr>
</tbody>
</table>

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Pre-drilled holes are not required.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 112 mm.
**Location:** Timber connection default case 4

**Plywood-to-timber joint to EC5 nails driven into side grain**

Characteristic permanent load  \( G_k = 2.5 \, \text{kN} \)

Characteristic variable load  \( Q_k = 2.5 \, \text{kN} \)

---

Plywood group 1 comprising:
(a) American construction and industrial plywood
(b) Canadian Douglas fir plywood
(c) Canadian softwood plywood
(d) Swedish softwood plywood
(e) Finnish conifer plywood

Thickness of headside member  \( t_h = 15 \, \text{mm} \)

Timber characteristic density  \( p_k1 = 505 \, \text{kg/m}^3 \)

Timber strength class C14.

Thickness of pointside timber  \( t_p = 63 \, \text{mm} \)

Timber characteristic density  \( p_k2 = 290 \, \text{kg/m}^3 \)

Diameter of nails  \( d = 3.4 \, \text{mm} \)

Length of nails  \( l = 65 \, \text{mm} \)

Basic loads in N will be derived from expressions given in Section 8 assuming round nails.

Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.

Min value of equations (a)-(f)  \( G_{\text{min}} = 786.36 \) N

Char nail shear capacity basic  \( = G_{\text{min}} = 786.36 \, \text{N} \)

Material factor for connections  \( \gamma_{\text{M}} = 1.3 \)

Design resistance per nail  \( p_n = k_{\text{mod}} \times \text{basic} / \gamma_{\text{M}} = 483.91 \, \text{N} \)

Number of nails required  \( n = \text{INT}(F_d \times 1000 / p_n) + 1 = 15 \)

**Minimum nail spacings - EC5 Table 8.2 & Figure 8.7**

Minimum spacing of nails within one row parallel to grain  \( a_1 = (5 + 5 \times \cos(\alpha)) \times d = 34 \, \text{mm} \)

Actual spacing of nails within one row parallel to grain  \( a_1 = 34 \, \text{mm} \)

Spacing of rows of nails perpendicular to grain  \( a_2 = 5 \times d = 17 \, \text{mm} \)

End distance (loaded)  \( a_3t = (10 + 5 \times \cos(\alpha)) \times d = 51 \, \text{mm} \)

End distance (unloaded)  \( a_3c = 10 \times d = 34 \, \text{mm} \)

Edge distance (loaded)  \( a_4t = (5 + 2 \times \sin(\alpha)) \times d = 17 \, \text{mm} \)

Edge distance (unloaded)  \( a_4c = (5 + 2 \times \sin(\alpha)) \times d = 17 \, \text{mm} \)
Joint slip check - EC5 Section 7

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total instantaneous slip SLS</td>
<td>utser=2*uiser=1.5214 mm</td>
</tr>
<tr>
<td>Total instantaneous slip ULS</td>
<td>utuls=2*uiult=2.168 mm</td>
</tr>
<tr>
<td>Psi 2 factor (variable action)</td>
<td>psi2v=0.3</td>
</tr>
<tr>
<td>ufser=2*(uiser<em>Gk'/Fdser</em>(1+psi2p<em>kdef)+uiser</em>Qk'/Fdser*(1+psi2v*kdef))=2.1148 mm</td>
<td></td>
</tr>
<tr>
<td>ufult=2*(uiult<em>Gk'/Fdser</em>(1+psi2p<em>kdef)+uiult</em>Qk'/Fdser*(1+psi2v*kdef))=3.0136 mm</td>
<td></td>
</tr>
</tbody>
</table>

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load</td>
<td>7.125 kN</td>
</tr>
<tr>
<td>Headside member thickness</td>
<td>15 mm</td>
</tr>
<tr>
<td>Pointside member thickness</td>
<td>63 mm</td>
</tr>
<tr>
<td>Number of nails required</td>
<td>15</td>
</tr>
<tr>
<td>Diameter of nails</td>
<td>3.4 mm</td>
</tr>
<tr>
<td>Length of smooth nails</td>
<td>65 mm</td>
</tr>
</tbody>
</table>

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Pre-drilled holes are not required.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 47.6 mm.
Location: Timber connection default case 1

Timber-to-timber joint without pre-drilled holes to EC5

Characteristic permanent load \( G_k = 3 \text{ kN} \)
Characteristic variable load \( Q_k = 3 \text{ kN} \)

Thickness of headside member \( t_h = 50 \text{ mm} \)
Timber strength class C22.
Timber characteristic density \( p_{k1} = 340 \text{ kg/m}^3 \)
Timber strength class C22.
 Thickness of pointside timber \( t_p = 50 \text{ mm} \)
Timber characteristic density \( p_{k2} = 340 \text{ kg/m}^3 \)
 Diameter of nails \( d = 4.2 \text{ mm} \)
Length of nails \( l = 90 \text{ mm} \)

Basic loads will be derived from expressions given in Section 8 assuming round nails.
Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.

Min value of equations (a)-(f) \( G_{\text{min}} = 1229.9 \text{ N} \)
Char nail shear capacity \( \text{basic} = \frac{G_{\text{min}}}{\gamma_M} = 935.28 \text{ N} \)
Material factor for connections \( \gamma_M = 1.3 \)
Design resistance per nail \( p_{nl} = \text{mod} \times \text{basic} / \gamma_M = 756.84 \text{ N} \)
Number of nails required \( n = \left\lfloor \frac{F_d}{p_{nl}} \right\rfloor + 1 = 12 \)

Minimum nail spacings - EC5 Table 8.2 & Figure 8.7

Minimum spacing of nails within one row parallel to grain \( a_1 = (5 + 5 \times \cos(\alpha)) \times d = 42 \text{ mm} \)
Actual spacing of nails within one row parallel to grain \( a_1 = 42 \text{ mm} \)
Spacing of rows of nails perpendicular to grain \( a_2 = 5 \times d = 21 \text{ mm} \)
End distance (loaded) \( a_3t = (10 + 5 \times \cos(\alpha)) \times d = 63 \text{ mm} \)
End distance (unloaded) \( a_3c = 10 \times d = 42 \text{ mm} \)
Edge distance (loaded) \( a_4t = (5 + 2 \times \sin(\alpha)) \times d = 21 \text{ mm} \)
Edge distance (unloaded) \( a_4c = (5 + 5 \times \sin(\alpha)) \times d = 21 \text{ mm} \)

Joint slip check - EC5 Section 7

Total instantaneous slip SLS \( u_{\text{ser}} = 2 \times u_{\text{ser}} = 1.5181 \text{ mm} \)
Total instantaneous slip ULS \( u_{\text{ult}} = 2 \times u_{\text{ult}} = 2.1633 \text{ mm} \)
Psi 2 factor (variable action) \( \psi_{2v} = 0.3 \)
\[ u_{\text{ser}} = 2 \times (u_{\text{ser}} \times G_k / F_d \times (1 + \psi_{2p} \times k_{def}) + u_{\text{ser}} \times Q_k / F_d \times (1 + \psi_{2v} \times k_{def})) = 2.1102 \text{ mm} \]
\[ u_{\text{ult}} = 2 \times (u_{\text{ult}} \times G_k / F_d \times (1 + \psi_{2p} \times k_{def}) + u_{\text{ult}} \times Q_k / F_d \times (1 + \psi_{2v} \times k_{def})) = 3.007 \text{ mm} \]
**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load</td>
<td>8.55 kN</td>
</tr>
<tr>
<td>Headside member thickness</td>
<td>50 mm</td>
</tr>
<tr>
<td>Pointside member thickness</td>
<td>50 mm</td>
</tr>
<tr>
<td>Number of nails required</td>
<td>12</td>
</tr>
<tr>
<td>Diameter of nails</td>
<td>4.2 mm</td>
</tr>
<tr>
<td>Length of smooth nails</td>
<td>90 mm</td>
</tr>
</tbody>
</table>

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Pre-drilled holes are not required.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 58.8 mm.
**Location:** Ex 10.13.1 Jack Porteous book p.417

**Timber-to-timber joint without pre-drilled holes to EC5**

Characteristic permanent load \( G_k = 2.5 \text{ kN} \)
Characteristic variable load \( Q_k = 3.5 \text{ kN} \)

**PLAN**

Thickness of headside member \( th = 36 \text{ mm} \)
Timber strength class C22.
Timber characteristic density \( p_{k1} = 340 \text{ kg/m}^3 \)
Timber strength class C22.
Thickness of pointside timber \( tp = 50 \text{ mm} \)
Timber characteristic density \( p_{k2} = 340 \text{ kg/m}^3 \)
Diameter of nails \( d = 3.35 \text{ mm} \)
Length of nails \( l = 65 \text{ mm} \)

Basic loads will be derived from expressions given in Section 8 assuming round nails.

**Minimum nail spacings - EC5 Table 8.2 & Figure 8.7**

Minimum spacing of nails within
one row parallel to grain \( a_1 = (5+5*\cos(\alpha))*d = 33.5 \text{ mm} \)
Actual spacing of nails within
one row parallel to grain \( a_1 = 33.5 \text{ mm} \)
Spacing of rows of nails perpendicular to grain \( a_2 = 5*d = 16.75 \text{ mm} \)
End distance (loaded) \( a_3t = (10+5*\cos(\alpha))*d = 50.25 \text{ mm} \)
End distance (unloaded) \( a_3c = 10*d = 33.5 \text{ mm} \)
Edge distance (loaded) \( a_4t = (5+2*\sin(\alpha))*d = 16.75 \text{ mm} \)
Edge distance (unloaded) \( a_4c = (5+5*\sin(\alpha))*d = 16.75 \text{ mm} \)

**Joint slip check - EC5 Section 7**

Total instantaneous slip SLS \( u_{ser} = 2*u_{iser} = 2.4255 \text{ mm} \)
Total instantaneous slip ULS \( u_{ult} = 2*u_{iult} = 3.4867 \text{ mm} \)
Psi 2 factor (variable action) \( \psi_{2v} = 0.3 \)

\[
uf_{ser} = 2*(u_{iser} * G_k/Fd_{ser}*(1+\psi_{2p} k_{def})+u_{iser} * Q_k/Fd_{ser}*(1+\psi_{2v} k_{def})) = 3.5736 \text{ mm}
\]

\[
uf_{ult} = 2*(u_{iult} * G_k/Fd_{ser}*(1+\psi_{2p} k_{def})+u_{iult} * Q_k/Fd_{ser}*(1+\psi_{2v} k_{def})) = 5.1371 \text{ mm}
\]
DESIGN SUMMARY

Design axial load 8.625 kN
Headside member thickness 36 mm
Pointside member thickness 50 mm
Nail details between A-B:
Number of nails per side 9
Total number of nails 18
Nail details between B-C:
Number of nails per side 9
Total number of nails 18
Diameter of nails 3.35 mm
Length of smooth nails 65 mm

Nails should comply with BS EN 14592 and should have a minimum ultimate tensile strength of 600 N/mm². Pre-drilled holes are not required.

In order to eliminate the effect of number of nails in a row the minimum spacing of nails in a row parallel to grain needs to be 46.9 mm.
**Location:** Screwed timber-to-timber joint connection

**Timber to timber screwed joint - BS5268-2:2002**

Axial load to be carried by joint $F=5.5$ kN  
Thickness of headside member $th=38$ mm  
Thickness of pointside timber $tp=76$ mm  
End grain modification factor $K_43=0.7$  
Diameter of screw shank $d=4$ mm  
Length of screws $l=90$ mm  
Basic screw shear headside capacity $basich=361$ N  
Basic screw shear pointside capacity $basicp=361$ N  
Mod factor for duration of load $K_52=1.12$  
Mod factor for number of screws $K_54=0.9$  
Permissible screw load $psl=basic*K_52*K_53*K_54=363.89$ N  
Number of screws required $n=\text{INT}(F*1000/psl)+1=16$

**Minimum screw spacings**

End distance parallel to grain $40$ mm  
Edge dist perpendicular to grain $20$ mm  
Distance between lines of screws perpendicular to grain $12$ mm  
Distance between adjacent screws on a line, parallel to grain $40$ mm

**Joint slip check**

Slip per shear plane $U=F*1000/Kser=4.8041$ mm  
Slip per fastener per shear plane $Uf=U/n=0.30025$ mm
Location: Screwed metal plate-to-timber joint connection

Metal plate-to-timber screwed joint - BS5268-2:2002

Axial load to be carried by joint \( F = 4.8 \text{ kN} \)
Thickness of headside steel plate \( t_h = 4 \text{ mm} \)
Thickness of pointside timber \( t_p = 38 \text{ mm} \)
End grain modification factor \( K_{43} = 1 \)
Diameter of screw shank \( d = 4 \text{ mm} \)
Length of screws \( l = 38 \text{ mm} \)

Basic loads in kN will be derived from formula given in Annex G.2

Basic screw shear capacity \( \text{basic} = \frac{1 \cdot G_{\text{min}}}{(F_d \cdot K_d)} = 388.12 \text{ N} \)

Basic screw shear capacity \( \text{basic} = K_{46} \cdot \text{basic} = 485.14 \text{ N} \)
Mod factor for duration of load \( K_{52} = 1.12 \)
Mod factor for number of screws \( K_{54} = 1 \)
Permissible screw load \( \text{psl} = \text{basic} \cdot K_{52} \cdot K_{53} \cdot K_{54} = 380.35 \text{ N} \)
Number of screws required \( n = \text{INT}(F \cdot 1000/\text{psl}) + 1 = 13 \)

Minimum screw spacings

End distance parallel to grain \( 40 \text{ mm} \)
Edge dist perpendicular to grain \( 20 \text{ mm} \)
Distance between lines of screws perpendicular to grain \( 12 \text{ mm} \)
Distance between adjacent screws on a line, parallel to grain \( 40 \text{ mm} \)
Location: Screwed ply-to-timber joint

Ply plate to timber screwed joint - BS5268-2:2002

Axial load to be carried by joint $F=4.8$ kN
Thickness of headside ply $\text{th}=9$ mm
Plywood group 1 (Clause 38) comprising:
(a) American construction and industrial plywood
(b) Canadian Douglas fir plywood
(c) Canadian softwood plywood
(d) Swedish softwood plywood
(e) Finnish conifer plywood
Thickness of pointside timber $\text{tp}=63$ mm
Diameter of screw shank $d=4$ mm
Length of screws $l=63$ mm
Basic screw shear capacity $\text{basic}=238$ N
Mod factor for duration of load $K52=1.12$
Mod factor for number of screws $K54=1$
Permissible screw load $\text{psl}=\text{basic}*K52*K53*K54=266.56$ N
Number of screws required $n=\text{INT}(F*1000/\text{psl})+1=19$

Minimum screw spacings

End distance parallel to grain $40$ mm
Edge dist perpendicular to grain $20$ mm
Distance between lines of screws perpendicular to grain $12$ mm
Distance between adjacent screws on a line, parallel to grain $40$ mm

Joint slip check

Slip per shear plane $U=F*1000/\text{Kser}=4.3971$ mm
Slip per fastener per shear plane $U_f=U/n=0.23143$ mm
Location: Screwed timber joint connection in withdrawal

Timber joint with screws in withdrawal - BS5268-2:2002

Axial load to be carried by joint  \( F=1.6 \text{ kN} \)
Thickness of pointside timber  \( t_p=48 \text{ mm} \)
Diameter of screw shank  \( d=4 \text{ mm} \)
Pointside embedment of screw  \( p_{en}=48 \text{ mm} \)
Basic withdrawal load (Table 67)  \( W_{basic}=11.53 \text{ N/mm penetration} \)
Mod factor for duration of load  \( K_52=1.12 \)
Mod factor for number of screws  \( K_54=1 \)
Permissible screw load  \( p_{nl}=W_{basic}p_{en}K_52K_53K_54=619.85 \text{ N} \)
Number of screws required  \( n=\text{INT}(F*1000/p_{nl})+1=3 \)

Minimum screw spacings

End distance parallel to grain  \( 40 \text{ mm} \)
Edge dist perpendicular to grain  \( 20 \text{ mm} \)
Distance between lines of screws perpendicular to grain  \( 12 \text{ mm} \)
Distance between adjacent screws on a line, parallel to grain  \( 40 \text{ mm} \)
Location: Screwed timber to timber joint

**Metal plate-to-timber screwed joint - BS5268-2:2002**

Axial load to be carried by joint $F = 4.8$ kN  
Thickness of headside steel plate $th = 4$ mm  
Thickness of pointside timber $tp = 38$ mm  
End grain modification factor $K43 = 1$  
Diameter of screw shank $d = 4$ mm  
Length of screws $l = 36$ mm

Basic loads in kN will be derived from formula given in Annex G.2 Equations G1 to G6 in annex G in BS5268-2:2002.

Basic screw shear capacity $\text{basic} = \frac{1 \cdot G_{\text{min}}}{(F_d \cdot K_d)} = 388.12$ N

Basic screw shear capacity $\text{basic} = K_{46} \cdot \text{basic} = 485.14$ N

Mod factor for duration of load $K_{52} = 1.12$

Mod factor for number of screws $K_{54} = 1$

Permissible screw load $psl = \text{basic} \cdot K_{52} \cdot K_{53} \cdot K_{54} = 543.36$ N

Number of screws required $n = \text{INT}(F \cdot 1000/psl) + 1 = 9$

**Minimum screw spacings**

End distance parallel to grain $40$ mm  
Edge dist perpendicular to grain $20$ mm  
Distance between lines of screws perpendicular to grain $12$ mm  
Distance between adjacent screws on a line, parallel to grain $40$ mm

**Joint slip check**

Slip per shear plane $U = \frac{F \cdot 1000}{K_{\text{ser}}} = 2.2969$ mm  
Slip per fastener per shear plane $U_f = U/n = 0.25521$ mm
Location: Screwed metal plate-to-timber joint connection

Metal plate-to-timber screwed joint – BS5268-2:2002

Axial load to be carried by joint $F=4.8$ kN
Thickness of headside steel plate $th=4$ mm
Thickness of pointside timber $tp=38$ mm
End grain modification factor $K43=1$
Diameter of screw shank $d=4$ mm
Length of screws $l=38$ mm

Basic loads in kN will be derived from formula given in Annex G.2

Basic screw shear capacity $basic=1\times G_{min}/(F_d\times K_d)=388.12$ N

Basic screw shear capacity $basic=K46\times basic=485.14$ N
Mod factor for duration of load $K52=1.12$
Mod factor for number of screws $K54=1$
Permissible screw load $psl=basic\times K52\times K53\times K54=543.36$ N
Number of screws required $n=\text{INT}(F\times 1000/psl)+1=9$

Minimum screw spacings

End distance parallel to grain $40$ mm
Edge dist perpendicular to grain $20$ mm
Distance between lines of screws perpendicular to grain $12$ mm
Distance between adjacent screws on a line, parallel to grain $40$ mm

Joint slip check

Slip per shear plane $U=F\times 1000/K_{ser}=2.2969$ mm
Slip per fastener per shear plane $U_f=U/n=0.25521$ mm
Location: Screwed timber-to-timber joint connection

Timber-to-timber screwed joint to EC5

Plan

Characteristic permanent load $G_k = 2.75 \text{ kN}$
Characteristic variable load $Q_k = 2.75 \text{ kN}$
Design axial load at joint $F_d = 1.35 \times G_k + 1.5 \times Q_k = 7.8375 \text{ kN}$
Timber strength class C22.
Thickness of headside member $t_h = 38 \text{ mm}$
Timber strength class C22.
Thickness of pointside timber $t_p = 76 \text{ mm}$
Diameter of screw shank $d = 4 \text{ mm}$
Length of screws $l = 90 \text{ mm}$

Basic loads in N will be derived from expressions given in Section 8 assuming smooth shank screws.

Min value of equations (a)-(f) $G_{\text{min}} = 1134.7$
Character shear capacity basic $G_{\text{basic}} = G_{\text{min}} = 1134.7 \text{ N}$
Material factor for connections $\gamma_M = 1.3$
Design resistance per screw $p_{nl} = k_{mod} \times G_{\text{basic}} / \gamma_M = 698.3 \text{ N}$
Number of screws required $n = \text{INT}(F_d \times 1000 / p_{nl}) + 1 = 12$

Minimum screw spacings – EC5 Table 8.2

Minimum spacing of screws within one row parallel to grain $a_1 = (5 + 5 \times \cos(\alpha)) \times d = 40 \text{ mm}$
Actual spacing of screws within one row parallel to grain $a_1 = 40 \text{ mm}$
Spacing of rows of screws perpendicular to grain $a_2 = 5 \times d = 20 \text{ mm}$
End distance (loaded) $a_3 t = (10 + 5 \times \cos(\alpha)) \times d = 60 \text{ mm}$
End distance (unloaded) $a_3 c = 10 \times d = 40 \text{ mm}$
Edge distance (loaded) $a_4 t = (5 + 2 \times \sin(\alpha)) \times d = 20 \text{ mm}$
Edge distance (unloaded) $a_4 c = (5 + 5 \times \sin(\alpha)) \times d = 20 \text{ mm}$

Joint slip check – EC5 Section 7

Total instantaneous slip SLS $u_{\text{SLS}} = 2 \times u_{\text{IS}} = 1.447 \text{ mm}$
Total instantaneous slip ULS $u_{\text{ULS}} = 2 \times u_{\text{IUL}} = 2.062 \text{ mm}$
Psi 2 factor (variable action) $\psi_{2v} = 0.3$
$u_{\text{SLS}} = 2 \times (u_{\text{IS}} \times G_k' / F_d \times (1 + \psi_2 p \times k_{\text{def}}) + u_{\text{IS}} \times Q_k' / F_d \times (1 + \psi_2 v \times k_{\text{def}})) = 2.0113 \text{ mm}$
$u_{\text{ULS}} = 2 \times (u_{\text{IUL}} \times G_k' / F_d \times (1 + \psi_2 p \times k_{\text{def}}) + u_{\text{IUL}} \times Q_k' / F_d \times (1 + \psi_2 v \times k_{\text{def}})) = 2.8661 \text{ mm}$
DESIGN SUMMARY

Design axial load: 7.8375 kN
Headsite member thickness: 38 mm
Pointsite member thickness: 76 mm
Number of screws required: 12
Diameter of screws: 4 mm
Length of screws: 90 mm

Provide smooth shank screws to EN 14592 of min ultimate tensile strength of 540 N/mm². Pre-drilled holes are not required. In order to eliminate the effect of number of screws in a row the min spacing of screws in a row parallel to grain needs to be 56 mm.
Location: Screwed metal plate-to-timber joint connection

Metal plate-to-timber screwed joint to EC5

Characteristic permanent load $G_k=2.4$ kN
Characteristic variable load $Q_k=2.4$ kN
Design axial load at joint $F_d=1.35*G_k+1.5*Q_k=6.84$ kN
Thickness of headside steel plate $t_h=4$ mm
Timber strength class C22.
Thickness of pointside timber $t_p=38$ mm
Diameter of screw shank $d=4$ mm
Length of screws $l=38$ mm

Basic loads in N will be derived from expressions given in Section 8 assuming smooth shank screws.

Min value of equations (a)-(e) $G_{min}=1301.5$ N
Char screw shear capacity basic=$G_{min}=1301.5$ N
Material factor for connections $\gamma_{M}=1.3$
Design resistance per screw $p_n=k_{mod}*basic/\gamma_{M}=650.73$ N
Number of screws required $n=\text{INT}(F_d*1000/p_n)+1=11$

Minimum screw spacings - EC5 Table 8.2

Minimum spacing of screws within one row parallel to grain $a_1=(5+5*COS(\alpha))*d=40$ mm
Actual spacing of screws within one row parallel to grain $a_1=40$ mm
Spacing of rows of screws perpendicular to grain $a_2=5*d=20$ mm
End distance (loaded) $a_3t=(10+5*COS(\alpha))*d=60$ mm
End distance (unloaded) $a_3c=10*d=40$ mm
Edge distance (loaded) $a_4t=(5+2*SIN(\alpha))*d=20$ mm
Edge distance (unloaded) $a_4c=(5+5*SIN(\alpha))*d=20$ mm

Joint slip check - EC5 Section 7

Total instantaneous slip SLS $u_{user}=2*u_{isuer}=0.68882$ mm
Total instantaneous slip ULS $u_{ult}=2*u_{iult}=0.98156$ mm
Psi 2 factor (variable action) $psi_{2v}=0.3$

$u_{ser}=2*(u_{isuer}*G_k/Fdser*(1+psi_{2p}*kdef)+u_{isuer}*Q_k/Fdser*(1+psi_{2v} *kdef))=1.5843$ mm
$u_{ult}=2*(u_{iult}*G_k/Fdser*(1+psi_{2p}*kdef)+u_{iult}*Q_k/Fdser*(1+psi_{2v} *kdef))=2.2576$ mm
<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Design axial load</th>
<th>6.84 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hheads member thickness</td>
<td>4 mm</td>
<td></td>
</tr>
<tr>
<td>Pointside member thickness</td>
<td>38 mm</td>
<td></td>
</tr>
<tr>
<td>Number of screws required</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td>Diameter of screws</td>
<td>4 mm</td>
<td></td>
</tr>
<tr>
<td>Length of screws</td>
<td>38 mm</td>
<td></td>
</tr>
</tbody>
</table>

Provide smooth shank screws to EN 14592 of min ultimate tensile strength of 540 N/mm². Pre-drilled holes are not required.
In order to eliminate the effect of number of screws in a row the min spacing of screws in a row parallel to grain needs to be 56 mm.
Location: Screwed ply-to-timber joint

Ply plate-to-timber screwed joint to EC5

Characteristic permanent load \( G_k = 2.4 \text{ kN} \)
Characteristic variable load \( Q_k = 2.4 \text{ kN} \)
Design axial load at joint \( F_d = 1.35 \times G_k + 1.5 \times Q_k = 6.84 \text{ kN} \)
Plywood group 1 comprising:
(a) American construction and industrial plywood
(b) Canadian Douglas fir plywood
(c) Canadian softwood plywood
(d) Swedish softwood plywood
(e) Finnish conifer plywood
Thickness of headside member \( \text{th} = 9 \text{ mm} \)
Timber strength class C16.
Thickness of pointside timber \( \text{tp} = 63 \text{ mm} \)
Diameter of screw shank \( d = 4 \text{ mm} \)
Length of screws \( l = 63 \text{ mm} \)

Min value of equations (a)-(f) \( G_{\text{min}} = 846.75 \text{ N} \)
Char screw shear capacity \( \text{basic} = G_{\text{min}} = 846.75 \text{ N} \)
Material factor for connections \( \text{gamM} = 1.3 \)
Design resistance per screw \( \text{pnl} = \text{kmod} \times \text{basic} / \text{gamM} = 521.08 \text{ N} \)
Number of screws required \( n = \text{INT} (F_d \times 1000 / \text{pnl}) + 1 = 14 \)

Minimum screw spacings - EC5 Table 8.2

Minimum spacing of screws within one row parallel to grain \( a_1 = (5 + 5 \times \text{COS} (\alpha)) \times d = 40 \text{ mm} \)
Actual spacing of screws within one row parallel to grain \( a_1 = 40 \text{ mm} \)
Spacing of rows of screws perpendicular to grain \( a_2 = 5 \times d = 20 \text{ mm} \)
End distance (loaded) \( a_3t = (10 + 5 \times \text{COS} (\alpha)) \times d = 60 \text{ mm} \)
End distance (unloaded) \( a_3c = 10 \times d = 40 \text{ mm} \)
Edge distance (loaded) \( a_4t = (5 + 2 \times \text{SIN} (\alpha)) \times d = 20 \text{ mm} \)
Edge distance (unloaded) \( a_4c = (5 + 5 \times \text{SIN} (\alpha)) \times d = 20 \text{ mm} \)

Joint slip check - EC5 Section 7

Total instantaneous slip SLS \( u_{\text{iser}} = 2 \times u_{\text{iser}} = 1.2433 \text{ mm} \)
Total instantaneous slip ULS \( u_{\text{ult}} = 2 \times u_{\text{ult}} = 1.7717 \text{ mm} \)
Psi 2 factor (variable action) \( \text{psi2v} = 0.3 \)
ufser=2*(uiuser*Gk'/Fdser*(1+psi2p*kdef)+uiuser*Qk'/Fdser*(1+psi2v *kdef))=1.8898 mm
ufult=2*(uiult*Gk'/Fdser*(1+psi2p*kdef)+uiult*Qk'/Fdser*(1+psi2v *kdef))=2.693 mm

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Design axial load</td>
<td>6.84 kN</td>
</tr>
<tr>
<td>Headside member thickness</td>
<td>9 mm</td>
</tr>
<tr>
<td>Pointside member thickness</td>
<td>63 mm</td>
</tr>
<tr>
<td>Number of screws required</td>
<td>14</td>
</tr>
<tr>
<td>Diameter of screws</td>
<td>4 mm</td>
</tr>
<tr>
<td>Length of screws</td>
<td>63 mm</td>
</tr>
</tbody>
</table>

Provide smooth shank screws to EN 14592 of min ultimate tensile strength of 540 N/mm². Pre-drilled holes are not required.
In order to eliminate the effect of number of screws in a row the min spacing of screws in a row parallel to grain needs to be 56 mm.
Location: Screwed timber joint connection in withdrawal

Timber joint with screws in withdrawal to EC5

Characteristic permanent load \( G_k = 0.8 \text{ kN} \)
Characteristic variable load \( Q_k = 0.8 \text{ kN} \)
Design axial load at joint \( F_d = 1.35 \times G_k + 1.5 \times Q_k = 2.28 \text{ kN} \)
Thickness of headside member \( t_h = 25 \text{ mm} \)

Bending parallel to the grain \( f_{mk} = 22 \text{ N/mm}^2 \)
Compress parallel to the grain \( f_{ck} = 20 \text{ N/mm}^2 \)
Compress perp to the grain \( f_{c90k} = 2.4 \text{ N/mm}^2 \)
Tension parallel to the grain \( f_{tk} = 13 \text{ N/mm}^2 \)
Shear parallel to the grain \( f_{vk} = 3.8 \text{ N/mm}^2 \)
Mean modulus of elasticity \( E_{0\text{mean}} = 10000 \text{ N/mm}^2 \)
5th percentile MOE \( E_{0.05} = 6700 \text{ N/mm}^2 \)
Characteristic density timber \( 2 \) \( p_k2 = 340 \text{ kg/m}^3 \)
Thickness of pointside timber \( t_p = 48 \text{ mm} \)
Diameter of screw shank \( d = 4 \text{ mm} \)
Pointside embedment of screw \( p_{en} = 48 \text{ mm} \)
Screw head diameter \( d_h = 8 \text{ mm} \)

Material factor for connections \( \gamma_{M} = 1.3 \)
Screw design withdrawal capacity \( F_{axRd} = 4 \times k_{mod} \times F_{axRk} / \gamma_{M} = 1092.7 \text{ N} \)
Number of screws required \( n = \text{INT} (F_d \times 1000 / F_{axRd}) + 1 = 3 \)

The design withdrawal capacity of the connection taking into account the effective number of screws as per EC5, Clause 8.7.2(8) is given by the following expression:

Spacing in a plane \( // \) grain \( a_1 = 28 \text{ mm} \)
Perpendicular to the grain \( a_2 = 20 \text{ mm} \)
End distance as per Fig 8.11a \( a_{1cg} = 40 \text{ mm} \)
Edge distance as per Fig 8.11a \( a_{2cg} = 16 \text{ mm} \)

DESIGN SUMMARY

- Design axial load \( 2.28 \text{ kN} \)
- Headside member thickness \( 25 \text{ mm} \)
- Pointside member thickness \( 48 \text{ mm} \)
- Number of screws required \( 3 \)
- Diameter of screws \( 4 \text{ mm} \)
- Length of screws \( 73 \text{ mm} \)

Provide smooth shank screws to EN 14592 of min ultimate tensile strength of 540 N/mm². Pre-drilled holes are not required.
Location: Screwed timber to timber joint

Metal plate-to-timber screwed joint to EC5

Characteristic permanent load \( G_k = 2.4 \text{ kN} \)
Characteristic variable load \( Q_k = 2.4 \text{ kN} \)
Design axial load at joint \( F_d = 1.35 \times G_k + 1.5 \times Q_k = 6.84 \text{ kN} \)
Thickness of headside steel plate \( t_h = 4 \text{ mm} \)
Timber strength class C22.
Thickness of pointside timber \( t_p = 38 \text{ mm} \)
Diameter of screw shank \( d = 4 \text{ mm} \)
Length of screws \( l = 36 \text{ mm} \)
Basic loads in N will be derived from expressions given in Section 8 assuming smooth shank screws.
Expressions (a) to (e) for fasteners in single shear refer to section 8.2.3 in EC5. Zero contribution from rope effect will be considered.
Min value of equations (a)-(e) \( G_{\text{min}} = 1255.8 \text{ N} \)
Char screw shear capacity \( k_{\text{mod}} \times G_{\text{min}} = 1255.8 \text{ N} \)
Material factor for connections \( \gamma_M = 1.3 \)
Design resistance per screw \( p_{nl} = k_{\text{mod}} \times G_{\text{min}} / \gamma_M = 772.81 \text{ N} \)
Number of screws required \( n = \text{INT}(F_d \times 1000 / p_{nl}) + 1 = 9 \)

Minimum screw spacings - EC5 Table 8.2

Minimum spacing of screws within
one row parallel to grain \( a_1 = (5 + 5 \times \cos(\alpha)) \times d = 40 \text{ mm} \)
Actual spacing of screws within
one row parallel to grain \( a_1 = 40 \text{ mm} \)
Spacing of rows of screws perpendicular to grain \( a_2 = 5 \times d = 20 \text{ mm} \)
End distance (loaded) \( a_3t = (10 + 5 \times \cos(\alpha)) \times d = 60 \text{ mm} \)
End distance (unloaded) \( a_3c = 10 \times d = 40 \text{ mm} \)
Edge distance (loaded) \( a_4t = (5 + 2 \times \sin(\alpha)) \times d = 20 \text{ mm} \)
Edge distance (unloaded) \( a_4c = (5 + 5 \times \sin(\alpha)) \times d = 20 \text{ mm} \)

Joint slip check - EC5 Section 7

Total instantaneous slip SLS \( u_{\text{ser}} = 2 \times u_{\text{ser}} = 0.84189 \text{ mm} \)
Total instantaneous slip ULS \( u_{\text{ult}} = 2 \times u_{\text{ult}} = 1.1997 \text{ mm} \)
Psi 2 factor (variable action) \( \psi_{2v} = 0.3 \)
\( u_{\text{ser}} = 2 \times (u_{\text{ser}} \times G_k / F_d \times (1 + \psi_{2v} \times k_{\text{def}}) + u_{\text{ser}} \times Q_k / F_d \times (1 + \psi_{2v} \times k_{\text{def}})) = 1.1702 \text{ mm} \)
\( u_{\text{ult}} = 2 \times (u_{\text{ult}} \times G_k / F_d \times (1 + \psi_{2v} \times k_{\text{def}}) + u_{\text{ult}} \times Q_k / F_d \times (1 + \psi_{2v} \times k_{\text{def}})) = 1.6676 \text{ mm} \)
**Design Summary**

- **Design axial load**: 6.84 kN
- **Headside member thickness**: 4 mm
- **Pointside member thickness**: 38 mm
- **Number of screws required**: 9
- **Diameter of screws**: 4 mm
- **Length of screws**: 36 mm

Provide smooth shank screws to EN 14592 of min ultimate tensile strength of 540 N/mm². Pre-drilled holes are not required. In order to eliminate the effect of number of screws in a row the min spacing of screws in a row parallel to grain needs to be 56 mm.
Location: Screwed metal plate-to-timber joint connection

Metal plate-to-timber screwed joint to EC5

![Diagram](Plan)

Characteristic permanent load \( G_k = 2.4 \text{ kN} \)
Characteristic variable load \( Q_k = 2.4 \text{ kN} \)
Design axial load at joint \( F_d = 1.35 \times G_k + 1.5 \times Q_k = 6.84 \text{ kN} \)
Thickness of headside steel plate \( \text{th} = 4 \text{ mm} \)
Timber strength class C22.
Thickness of pointside timber \( \text{tp} = 38 \text{ mm} \)
Diameter of screw shank \( d = 4 \text{ mm} \)
Length of screws \( l = 38 \text{ mm} \)
Basic loads in N will be derived from expressions given in Section 8 assuming smooth shank screws.
Min value of equations (a)-(e) \( G_{\text{min}} = 1301.5 \text{ N} \)
Char screw shear capacity \( \text{basic} = G_{\text{min}} = 1301.5 \text{ N} \)
Material factor for connections \( \gamma_M = 1.3 \)
Design resistance per screw \( \text{pnl} = k_{\text{mod}} \times \text{basic} / \gamma_M = 800.89 \text{ N} \)
Number of screws required \( n = \text{INT}(F_d \times 1000 / \text{pnl}) + 1 = 9 \)

Minimum screw spacings - EC5 Table 8.2

Minimum spacing of screws within one row parallel to grain \( a_1 = (5 + 5 \times \cos(\alpha)) \times d = 40 \text{ mm} \)
Actual spacing of screws within one row parallel to grain \( a_1 = 40 \text{ mm} \)
Spacing of rows of screws perpendicular to grain \( a_2 = 5 \times d = 20 \text{ mm} \)
End distance (loaded) \( a_{3t} = (10 + 5 \times \cos(\alpha)) \times d = 60 \text{ mm} \)
End distance (unloaded) \( a_{3c} = 10 \times d = 40 \text{ mm} \)
Edge distance (loaded) \( a_{4t} = (5 + 2 \times \sin(\alpha)) \times d = 20 \text{ mm} \)
Edge distance (unloaded) \( a_{4c} = (5 + 5 \times \sin(\alpha)) \times d = 20 \text{ mm} \)

Joint slip check - EC5 Section 7

Total instantaneous slip SLS \( u_{\text{SLS}} = 2 \times u_{\text{user}} = 0.84189 \text{ mm} \)
Total instantaneous slip ULS \( u_{\text{ULS}} = 2 \times u_{\text{ult}} = 1.1997 \text{ mm} \)
Psi 2 factor (variable action) \( \psi_{2\text{v}} = 0.3 \)
\( u_{\text{SLS}} = 2 \times (u_{\text{user}} \times G_k / F_d + u_{\text{user}} \times Q_k / F_d + 1.1702 \text{ mm} \)
\( u_{\text{ULS}} = 2 \times (u_{\text{ult}} \times G_k / F_d + u_{\text{ult}} \times Q_k / F_d + 1.6676 \text{ mm} \)
DESIGN SUMMARY

- Design axial load: 6.84 kN
- Headside member thickness: 4 mm
- Pointside member thickness: 38 mm
- Number of screws required: 9
- Diameter of screws: 4 mm
- Length of screws: 38 mm

Provide smooth shank screws to EN 14592 of min ultimate tensile strength of 540 N/mm². Pre-drilled holes are not required.
In order to eliminate the effect of number of screws in a row the min spacing of screws in a row parallel to grain needs to be 56 mm.
Location: Bolt design

Axial load to be carried by joint $F=4.55$ kN

| $F$ (2) | : | (1) | $F$ |

**ELEVATION**

In line connection

$\alpha_1'=0^\circ$ & $\alpha_2'=0^\circ$

(1) = member 1
(2) = member 2

**PLAN**

Thickness of thinner member $t_1=47$ mm
Strength class C24 to Table 8.

Thickness of other member $t_2=47$ mm
Strength class C24 to Table 8.

Joint to be connected by bolts $K_{2b}=1.33$

Fixing diameter (8 to 36 mm) $b_d=16$ mm

Basic loads in kN will be derived from formula given in Annex G.3

Load duration for bolt/dowel connection = Medium term.

Fixing spacing parallel to the grain $\alpha_{par}=64$ mm

Basic bolt shear capacity $\text{basic}=1\times\min(G_{min}/(F_d\times K_d))=3.2882$ kN

Mod factor for moisture content $K_{56}=1$

Mod factor for number of bolts $K_{57}=1$

Permissible bolt load $p_{bl}=\text{basic}\times K_{56}\times K_{57}=3.2882$ kN

Number of bolts required $n=\lfloor F/p_{bl}\rfloor+1=2$

**Minimum bolt/dowel spacing**

The fixing spacing in the direction of load should be not less than 64 mm. For loading parallel to the grain, the spacing across the grain between fixings or rows of fixings should be not less than 64 mm.

For loading perpendicular to the grain, the spacing parallel to the grain between fixings or rows of fixings should be not less than 79 mm.

When two timber members are fixed together at right-angles to each other, the minimum bolt/dowel spacing should be 80 mm.

The distance from the end of the timber to the centre of the nearest bolt/dowel hole for parallel to the grain loading should be not less than 64 mm where the bolts/dowels are bearing away from the end of the member, or 112 mm where bolts/dowels are bearing towards the end of the member.
The distance from the edge of the timber to the centre of the nearest bolt/dowel hole should be not less than 24 mm except when loading is perpendicular to the grain and then the distance from the edge of the timber towards which the fixings are bearing and the centre of the nearest fixing hole, should be not less than 64 mm.

**DESIGN**
- Axial load to be carried by joint: 4.55 kN

**SUMMARY**
- Permissible load on each bolt: 3.2882 kN
- Number of bolts required per joint: 2
- Load capacity of the joint: 6.5765 kN
Location: Default case 2

Axial load to be carried by joint $F=6.5\text{ kN}$

**ELEVATION**

- Members with grain at $90^\circ$ to one another
  - $\alpha_1'=90^\circ$ (member 1)
  - $\alpha_2'=0^\circ$ (member 2)

**PLAN**

- $(1) = \text{member 1}$
- $(2) = \text{member 2}$

**Thickness of member 1** $t_1=35\text{ mm}$
- Default case with $\text{scr}=0$

**Characteristic density timber 1** $p_{k1}=310\text{ kg/m}^3$

**Angle between grain and load** $\alpha_1'=90$ degrees

**Thickness of member 2** $t_2=35\text{ mm}$
- Default case with $\text{scr}=0$

**Characteristic density timber 2** $p_{k2}=310\text{ kg/m}^3$

**Angle between grain and load** $\alpha_2'=0$ degrees

**Joint to be connected by bolts** $K_{2b}=1.33$

**Fixing diameter (8 to 36 mm)** $b_d=12\text{ mm}$

**Basic loads in kN will be derived from formula given in Annex G.3**

**Load duration for bolt/dowel connection** = Medium term.

**Fixing spacing parallel to the grain** $\alpha_{\text{par}}=48\text{ mm}$

**Basic bolt shear capacity** $\text{basic}=1\times G_{\text{min}}/(F_d\times K_d)=1.5875\text{ kN}$

**Mod factor for moisture content** $K_{56}=1$

**Mod factor for number of bolts** $K_{57}=1$

**Permissible bolt load** $p_{bl}=\text{basic}\times K_{56}\times K_{57}=1.5875\text{ kN}$

**Number of bolts required** $n=\text{INT}(F/p_{bl})+1=5$

### DESIGN
- **Axial load to be carried by joint** 6.5 kN

### SUMMARY
- **Permissible load on each bolt** 1.5875 kN
- **Number of bolts required per joint** 5
- **Load capacity of the joint** 7.9376 kN
Location: Default case 3

Axial load to be carried by joint \( F = 8.5 \) kN

---

**ELEVATION**

Member inclined at an angle alpha' degrees

(1) = member 1
(2) = member 2

---

**PLAN**

- Thickness of member 1 \( t_1 = 72 \) mm
- Strength class C16 to Table 8.
- Timber characteristic density \( p_{k1} = 310 \) kg/m³
- Angle between grain and load \( \alpha_{l1} = 0^\circ \)
- Thickness of member 2 \( t_2 = 72 \) mm
- Strength class C16 to Table 8.
- Timber characteristic density \( p_{k2} = 310 \) kg/m³
- Angle between grain and load \( \alpha_{l2} = 45^\circ \)
- Joint to be connected by bolts \( K_2b = 1.33 \)
- Fixing diameter (8 to 36 mm) \( b_d = 16 \) mm
- Basic loads in kN will be derived from formula given in Annex G.3
- Load duration for bolt/dowel connection = Short term.
- Fixing spacing parallel to the grain \( \alpha_{par} = 64 \) mm

---

- Basic bolt shear capacity \( \text{basic} = 1 \times G_{\text{min}}/(F_d \times K_d) = 3.9774 \) kN
- Mod factor for moisture content \( K_{56} = 1 \)
- Mod factor for number of bolts \( K_{57} = 1 \)
- Permissible bolt load \( p_{bl} = \text{basic} \times K_{56} \times K_{57} = 3.9774 \) kN
- Number of bolts required \( n = \text{INT}(F/p_{bl}) + 1 = 3 \)

---

**DESIGN**

- Axial load to be carried by joint \( 8.5 \) kN

**SUMMARY**

- Permissible load on each bolt \( 3.9774 \) kN
- Number of bolts required per joint \( 3 \)
- Load capacity of the joint \( 11.932 \) kN
Location: Default case 4

Axial load to be carried by joint $F = 8.5$ kN

Members inclined at an angle $\alpha'$ degrees

Thickness of member 1 $t_1 = 36$ mm
Strength class C30 to Table 8.
Timber characteristic density $p_{k1} = 380$ kg/m³
Angle between grain and load $\alpha_{p1}' = 0°$
Thickness of member 2 $t_2 = 72$ mm
Strength class C30 to Table 8.
Timber characteristic density $p_{k2} = 380$ kg/m³
Angle between grain and load $\alpha_{p2}' = 45°$
Joint to be connected by bolts $K_2b = 1.33$
Fixing diameter (8 to 36 mm) $b_d = 16$ mm
Basic loads in kN will be derived from formula given in Annex G.3
Constant $K_d = 1$
Load duration for bolt/dowel connection = Short term.
Fixing spacing parallel to the grain $\alpha_{par} = 64$ mm
Equations $G7$ $G8$ $G9$ $G10$
$7922.1$ $6541.8$ $6933.4$ $9731.9$
Min value of equations $G7$ to $G10$ $G_{min} = 6541.8$
Basic bolt shear capacity $basic = 1 \times G_{min} / (F_d \times K_d) = 4.6727$ kN
Mod factor for moisture content $K_{56} = 1$
Mod factor for number of bolts $K_{57} = 1$
Permissible bolt load $p_{bl} = basic \times K_{56} \times K_{57} = 4.6727$ kN
Number of bolts required $n = \text{INT}(F / p_{bl}) + 1 = 2$

DESIGN
- Axial load to be carried by joint $8.5$ kN

SUMMARY
- Permissible load on each bolt $4.6727$ kN
- Number of bolts required per joint $2$
- Load capacity of the joint $9.3454$ kN
Location: Default case 5

Axial load to be carried by joint F=8.5 kN

---

**ELEVATION**

(1)

In line connection  
\( \alpha_1' = 0° \) & \( \alpha_2' = 0° \)

(1) = member 1  
(2) = member 2

---

**PLAN**

(1)

Thickness of outer members \( t_1 = 48.5 \) mm  
Strength class C16 to Table 8.  
Timber characteristic density \( p_{k1} = 310 \) kg/m\(^3\)  
Angle between grain and load \( \alpha_1' = 0 \) degrees  
Thickness of other member \( t_2 = 97 \) mm  
Strength class C16 to Table 8.  
Timber characteristic density \( p_{k2} = 310 \) kg/m\(^3\)  
Angle between grain and load \( \alpha_2' = 0 \) degrees  
Joint to be connected by bolts \( K_{2b} = 1.33 \)  
Fixing diameter (8 to 36 mm) \( b_d = 20 \) mm  
Basic loads in kN will be derived from formula given in Annex G.3  
Constant \( K_d = 1 \)  
Load duration for bolt/dowel connection = Long term.  
Fixing spacing parallel to the grain \( \alpha_{par} = 80 \) mm  
Equations G7 G8 G9 G10  
6910.2 6910.2 8140.9 11507  
Min value of equations G7 to G10 \( G_{min} = 6910.2 \)

Basic bolt shear capacity \( \text{basic} = 1 \times G_{min} / (F_d \times K_d) = 5.1186 \) kN  
Mod factor for moisture content \( K_{36} = 1 \)  
Mod factor for number of bolts \( K_{57} = 1 \)  
Permissible bolt load \( p_{bl} = \text{basic} \times K_{36} \times K_{57} = 5.1186 \) kN  
Number of bolts required \( n = \text{INT}(F/p_{bl}) + 1 = 2 \)

---

**DESIGN**  
Axial load to be carried by joint 8.5 kN

**SUMMARY**  
Permissible load on each bolt 5.1186 kN  
Number of bolts required per joint 2  
Load capacity of the joint 10.237 kN
Location: Default case 6

Axial load to be carried by joint $F = 4.55$ kN

![Diagram](location_diagram.png)

Thickness of timber member $t_1 = 47$ mm
Strength class C24 to Table 8.
Timber characteristic density $p_{k1} = 350$ kg/m³
Angle between grain and load $\alpha'_{L1} = 45°$
Thickness of steel plate $t_3 = 15$ mm
Load is inclined at an angle to the grain of the timber thus:
Modification factor $K_{46} = 1.0$
Joint to be connected by bolts $K_{2b} = 1.33$
Fixing diameter (8 to 36 mm) $b_d = 12$ mm

**Bolt shear capacities**

Basic loads in kN will be derived from formula given in Annex G.3
Load duration for bolt/dowel connection = Medium term.
Fixing spacing parallel to the grain $\alpha_{par} = 48$ mm
Embedded strength for timber 1
Embedding strength $f_{ hod } = 0.050 \times (1 - 0.01 \times b_d) \times p_{k1} \times K_a = 12.449$
Embedding strength (Timb 1) $f_{hd1} = f_{hod} / ((K_{90} \times \sin(\alpha'_{L1})^2) + \cos(\alpha'_{L1})^2) = 9.8409$
$\beta = 1$
Basic bolt shear capacity $\text{basic} = 1 \times G_{\min} / (F_d \times K_d) = 2.184$ kN
Mod factor for moisture content $K_{56} = 1$
Mod factor for number of bolts $K_{57} = 1$
Permissible bolt load $p_{bl} = \text{basic} \times K_{46} \times K_{56} \times K_{57} = 2.184$ kN
Number of bolts required $n = \text{INT}(F/p_{bl}) + 1 = 3$

**Minimum bolt/dowel spacing**

The fixing spacing in the direction of load should be not less than 48 mm. For loading parallel to the grain, the spacing across the grain between fixings or rows of fixings should be not less than 48 mm.

For loading perpendicular to the grain, the spacing parallel to the grain between fixings or rows of fixings should be not less than 60 mm.
When two timber members are fixed together at right-angles to each other, the minimum bolt/dowel spacing should be 60 mm.

The distance from the end of the timber to the centre of the nearest bolt/dowel hole for parallel to the grain loading should be not less than 48 mm where the bolts/dowels are bearing away from the end of the member, or 84 mm where bolts/dowels are bearing towards the end of the member.

The distance from the edge of the timber to the centre of the nearest bolt/dowel hole should be not less than 18 mm except when loading is perpendicular to the grain and then the distance from the edge of the timber towards which the fixings are bearing and the centre of the nearest fixing hole, should be not less than 48 mm.

**DESIGN**  
Axial load to be carried by joint 4.55 kN

**SUMMARY**  
Permissible load on each bolt 2.184 kN  
Number of bolts required per joint 3  
Load capacity of the joint 6.5521 kN
**Location: Default case 7**

Axial load to be carried by joint $F=9.7$ kN

---

**ELEVATION**

- Member inclined at an angle $\alpha'$ degrees

- (1) = member 1
- (2) = member 2

---

**PLAN**

<table>
<thead>
<tr>
<th>Thickness of member 1</th>
<th>$t_1=97$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength class C24 to Table 8.</td>
<td></td>
</tr>
<tr>
<td>Timber characteristic density</td>
<td>$p_{k1}=350$ kg/m$^3$</td>
</tr>
<tr>
<td>Angle between grain and load</td>
<td>$\alpha_{l1}'=0^\circ$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Thickness of member 2</th>
<th>$t_2=72$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength class C24 to Table 8.</td>
<td></td>
</tr>
<tr>
<td>Timber characteristic density</td>
<td>$p_{k2}=350$ kg/m$^3$</td>
</tr>
<tr>
<td>Angle between grain and load</td>
<td>$\alpha_{l2}'=42^\circ$</td>
</tr>
<tr>
<td>Joint to be connected by bolts</td>
<td>$K_{2b}=1.33$</td>
</tr>
<tr>
<td>Fixing diameter (8 to 36 mm)</td>
<td>$b_d=20$ mm</td>
</tr>
</tbody>
</table>

Basic loads in kN will be derived from formula given in Annex G.3

- Load duration for bolt/dowel connection = Medium term.
- Fixing spacing parallel to the grain $\alpha_{par}=140$ mm

<table>
<thead>
<tr>
<th>Basic bolt shear capacity</th>
<th>$basic=1.0\times G_{min}/(F_d\times K_d)=6.6875$ kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mod factor for moisture content</td>
<td>$K_{56}=1$</td>
</tr>
<tr>
<td>Mod factor for number of bolts</td>
<td>$K_{57}=1$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Permissible bolt load</th>
<th>$p_{bl}=basic\times K_{56}\times K_{57}=6.6875$ kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bolts required</td>
<td>$n=INT(F/pbl)+1=2$</td>
</tr>
</tbody>
</table>

**DESIGN**

Axial load to be carried by joint $9.7$ kN

**SUMMARY**

- Permissible load on each bolt $6.6875$ kN
- Number of bolts required per joint $2$
- Load capacity of the joint $13.375$ kN
Location: Bolt design

Joint comprising 2 members (fasteners in single shear)

Design axial load at joint \( F_d' = 5.8 \text{ kN} \)

---

### ELEVATION

- \( t_1 = 47 \text{ mm} \) (1)
- \( t_2 = 47 \text{ mm} \) (2)

### PLAN

- Member 1 (1)
- Member 2 (2)

Thickness of thinner member \( t_1 = 47 \text{ mm} \)
Thickness of other member \( t_2 = 47 \text{ mm} \)
Bolt/dowel diameter (8 to 30 mm) \( b_d = 16 \text{ mm} \)
Timber strength class C24.

Basic loads are in kN and will be derived from EC5 Section 8.

Charact fastener yield moment \( M_y R_k = 0.3 \times 400 \times b_d^2.6 = 162141 \text{ Nmm} \)

Charact.embedment strength \( f_{h1k} = 0.082 \times (1 - 0.01 \times b_d) \times p_k \)
\[ = 24.108 \frac{\text{N}}{\text{mm}^2} \]

Denominator (expression 8.31) \( \text{denom}_1 = ((K_90 \times \text{SIN}(\alpha_1)^2) + \text{COS}(\alpha_1)^2) = 1 \)

Embedment strength (Timber 1) \( f_{h1k} = f_{h1k} / \text{denom}_1 = 24.108 \frac{\text{N}}{\text{mm}^2} \)

Charact.embedment strength \( f_{h2k} = 0.082 \times (1 - 0.01 \times b_d) \times p_k \)
\[ = 24.108 \frac{\text{N}}{\text{mm}^2} \]

Denominator (expression 8.31) \( \text{denom}_2 = ((K_90 \times \text{SIN}(\alpha_2)^2) + \text{COS}(\alpha_2)^2) = 1 \)

Embedment strength (Timber 2) \( f_{h2k} = f_{h2k} / \text{denom}_2 = 24.108 \frac{\text{N}}{\text{mm}^2} \)

Factor \( \beta = f_{h2k} / f_{h1k} = 1 \)

Min value of equations (a)-(f) \( G_{min} = 7509.4 \)

Char fastener shear resistance basic \( G_{min} / 1000 = 7.5094 \text{ kN} \)

Design resistance per fastener \( p_{bl} = k_{mod} \times \text{basic} / \gamma_{M} = 4.0435 \text{ kN} \)

Number of fasteners required \( n = \text{INT}(F_d' / p_{bl}) + 1 = 2 \)

Total No of fasteners at joint \( n = 2 \)

Number of fasteners per row \( n_{fpr} = 2 \)
Lines of fasteners to be adopted \( r_{pl} = 1 \)
Spacing between fasteners \( a_1 = 80 \text{ mm} \)
Effect No of fasteners per row \( n_{ef} = n_{fpr} \times 0.9 \times (a_1 / (13 \times b_d))^{0.25} = 1.4695 \)
Effective joint design capacity \( F_{vef} = r_{pl} \times n_{ef} \times p_{bl} = 5.9421 \text{ kN} \)

Minimum bolt spacings

- Angle alpha \( \alpha = 0 \text{ radians} \)
- Spacing parallel to grain \( a_1 = (4 + \text{COS}(\alpha)) \times b_d = 80 \text{ mm} \)
- Spacing perpendicular to grain \( a_2 = 4 \times b_d = 64 \text{ mm} \)
- End distance (loaded end) \( a_{3t} = 7 \times b_d = 112 \text{ mm} \)
- End distance (unloaded end) \( a_{3c} = 4 \times b_d = 64 \text{ mm} \)
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load at joint</td>
<td>Number of fasteners per joint</td>
<td>5.8 kN</td>
</tr>
<tr>
<td>Effective joint design capacity</td>
<td>2</td>
<td>5.9421 kN</td>
</tr>
</tbody>
</table>
Location: Default case 2

Joint comprising 2 members (fasteners in single shear)

Design axial load at joint  \( F_{d}' = 9.1 \text{ kN} \)

![Elevation and Plan Diagram]

\( t_1 = 35 \text{ mm} \)

Default case with \( \text{scr}=0 \)

- Bending parallel to the grain:  \( f_{mk} = 22 \text{ N/mm}^2 \)
- Compress parallel to the grain:  \( f_{c0k} = 20 \text{ N/mm}^2 \)
- Compress perp to the grain:  \( f_{c90k} = 2.4 \text{ N/mm}^2 \)
- Tension parallel to the grain:  \( f_{t0k} = 13 \text{ N/mm}^2 \)
- Shear parallel to the grain:  \( f_{vk} = 2.4 \text{ N/mm}^2 \)
- Mean modulus of elasticity:  \( E_{0\text{mean}} = 10000 \text{ N/mm}^2 \)
- 5th percentile MOE:  \( E_{005} = 6700 \text{ N/mm}^2 \)
- Characteristic density timber 1:  \( p_{k1} = 310 \text{ kg/m}^3 \)
- Angle between grain and load:  \( \alpha_{l1}' = 90 \text{ degrees} \)
- Thickness of member 2:  \( t_2 = 35 \text{ mm} \)

Default case with \( \text{scr}=0 \)

- Bending parallel to the grain:  \( f_{mk} = 22 \text{ N/mm}^2 \)
- Compress parallel to the grain:  \( f_{c0k} = 20 \text{ N/mm}^2 \)
- Compress perp to the grain:  \( f_{c90k} = 2.4 \text{ N/mm}^2 \)
- Tension parallel to the grain:  \( f_{t0k} = 13 \text{ N/mm}^2 \)
- Shear parallel to the grain:  \( f_{vk} = 2.4 \text{ N/mm}^2 \)
- Mean modulus of elasticity:  \( E_{0\text{mean}} = 10000 \text{ N/mm}^2 \)
- 5th percentile MOE:  \( E_{005} = 6700 \text{ N/mm}^2 \)
- Characteristic density timber 2:  \( p_{k2} = 310 \text{ kg/m}^3 \)
- Angle between grain and load:  \( \alpha_{l2}' = 0 \text{ degrees} \)
- Bolt/dowel diameter (8 to 30 mm):  \( b_{d} = 12 \text{ mm} \)

Basic loads are in kN and will be derived from EC5 Section 8.

Charact fastener yield moment:  \( M_{yRk} = 0.3 \times 400 \times b_{d} \times 2.6 = 76745 \text{ Nmm} \)

Charact.embedment strength:

\[
\begin{align*}
\text{fhok1} &= 0.082 \times (1 - 0.01 \times b_{d}) \times p_{k1} \\
&= 22.37 \text{ N/mm}^2 \\
\text{denom1} &= ((K_{90} \times \sin(\alpha_{l1})^2) + \cos(\alpha_{l1})^2) = 1.53
\end{align*}
\]

Embedment strength (Timber 1):

\[
\begin{align*}
\text{fh1k} &= \frac{\text{fhok1}}{\text{denom1}} = 14.621 \text{ N/mm}^2 \\
\text{fhok2} &= 0.082 \times (1 - 0.01 \times b_{d}) \times p_{k2} \\
&= 22.37 \text{ N/mm}^2 \\
\text{denom2} &= ((K_{90} \times \sin(\alpha_{l2})^2) + \cos(\alpha_{l2})^2) = 1
\end{align*}
\]
Embedment strength (Timber 2) \( fh_2k = \frac{fh_0k2}{denom2} = 22.37 \text{ N/mm}^2 \)

Factor \( \beta = \frac{fh_2k}{fh_1k} = 1.53 \)

Min value of equations (a)-(f) \( G_{\text{min}} = 3196.2 \)

Char fastener shear resistance \( \text{basic} = \frac{G_{\text{min}}}{1000} = 3.1962 \text{ kN} \)

Design resistance per fastener \( p_{bl} = k_{\text{mod}} \times \text{basic} / \gamma_M = 1.4752 \text{ kN} \)

Number of fasteners required \( n = \text{INT}(F_d' / p_{bl}) + 1) = 7 \)

Total No of fasteners at joint \( n = 7 \)

Shear force on LHS of joint \( F_{vEd1} = 4.55 \text{ kN} \)

Shear force on RHS of joint \( F_{vEd2} = 4.55 \text{ kN} \)

Height of member (2) \( h = 225 \text{ mm} \)

Loaded edge distance \( h_e = 175 \text{ mm} \)

**DESIGN**

- Design axial load at joint \( 9.1 \text{ kN} \)

**SUMMARY**

- Number of fasteners per joint \( 7 \)
- Design joint resistance \( 10.326 \text{ kN} \)
Location: Default case 3

Joint comprising 2 members (fasteners in single shear)

Design axial load at joint \( Fd' = 11.9 \text{ kN} \)

Member inclined at an angle \( \alpha' \) degrees

ELEVATION

(1) = member 1
(2) = member 2

Thickness of member 1 \( t1 = 72 \text{ mm} \)
Timber strength class C16.
Timber characteristic density \( pkl = 310 \text{ kg/m}^3 \)
Angle between grain and load \( \alpha1' = 0^\circ \)

Thickness of member 2 \( t2 = 72 \text{ mm} \)
Timber strength class C16.
Timber characteristic density \( pkl = 310 \text{ kg/m}^3 \)
Angle between grain and load \( \alpha2' = 45^\circ \)
Bolt/dowel diameter (8 to 30 mm) \( bd = 16 \text{ mm} \)

Basic loads are in kN and will be derived from EC5 Section 8.
Charact fastener yield moment \( MyRk = 0.3 \times 400 \times bd^2 \times 1.62141 = 162141 \text{ Nmm} \)
Charact.embedment strength \( fhok1 = 0.082 \times (1 - 0.01 \times bd) \times pkl \)
\( = 21.353 \text{ N/mm}^2 \)
Denominator (expression 8.31) \( denom1 = ((K90 \times \sin(\alpha1'))^2) + \cos(\alpha1')^2 = 1 \)
Embedment strength (Timber 1) \( fh1k = fhok1 / denom1 = 21.353 \text{ N/mm}^2 \)
Charact.embedment strength \( fhok2 = 0.082 \times (1 - 0.01 \times bd) \times pkl \)
\( = 21.353 \text{ N/mm}^2 \)
Denominator (expression 8.31) \( denom2 = ((K90 \times \sin(\alpha2'))^2) + \cos(\alpha2')^2 = 1.295 \)
Embedment strength (Timber 2) \( fh2k = fhok2 / denom2 = 16.489 \text{ N/mm}^2 \)
Factor \( \beta = fh2k / fh1k = 0.7722 \)
Min value of equations (a)-(f) \( Gmin = 9006.4 \)
Char fastener shear resistance \( basic = Gmin / 1000 = 9.0064 \text{ kN} \)

Design resistance per fastener \( pbl = kmod \times basic / \gammaM = 4.8496 \text{ kN} \)
Number of fasteners required \( n = \text{INT}(Fd' / pbl) + 1 = 3 \)
Total No of fasteners at joint \( n = 3 \)
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load at joint</td>
<td>Number of fasteners per joint</td>
</tr>
<tr>
<td>$FvEd1 = 4.2073$ kN</td>
<td>3</td>
</tr>
<tr>
<td>$FvEd2 = 4.2073$ kN</td>
<td>Design joint resistance</td>
</tr>
<tr>
<td>$h = 225$ mm</td>
<td>14.549 kN</td>
</tr>
<tr>
<td>$he = 175$ mm</td>
<td></td>
</tr>
</tbody>
</table>
Location: Default case 4

Joint comprising 3 members (fasteners in double shear)

Design axial load at joint $F_d' = 11.9$ kN

---

**ELEVATION**

(1) ______________—˜ _ _ _—˜

Member inclined at an angle $\alpha'$ degrees

(2) ______________—˜

---

**PLAN**

T1 (1) = member 1
T2 (2) = member 2

Thickness of member 1 $t_1 = 36$ mm
Timber strength class C30.
Timber characteristic density $p_{k1} = 380$ kg/m$^2$
Angle between grain and load $\alpha_{p1}' = 0^\circ$

Thickness of member 2 $t_2 = 72$ mm
Timber strength class C30.
Timber characteristic density $p_{k2} = 380$ kg/m$^2$
Angle between grain and load $\alpha_{p2}' = 45^\circ$

Bolt/dowel diameter (8 to 30 mm) $b_d = 16$ mm

Basic loads are in kN and will be derived from EC5 Section 8.
Charact fastener yield moment $M_{yRk} = 0.3 \times 400 \times b_d^2 \times 2.6 = 162141$ Nmm
Charact. embedment strength $f_{hok1} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k1}$

Denominator (expression 8.31) $\text{denom1} = ((K90 \times \sin(\alpha_{p1})^2) + \cos(\alpha_{p1})^2) = 1$
Embedment strength (Timber 1) $f_{h1k} = f_{hok1} / \text{denom1} = 26.174$ N/mm$^2$
Charact. embedment strength $f_{hok2} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k1}$

Denominator (expression 8.31) $\text{denom2} = ((K90 \times \sin(\alpha_{p2})^2) + \cos(\alpha_{p2})^2) = 1.295$
Embedment strength (Timber 2) $f_{h2k} = f_{hok2} / \text{denom2} = 20.212$ N/mm$^2$
Factor $\beta = f_{h2k} / f_{h1k} = 0.7722$

Equations $g$ $h$ $j$ $k$

15076 15076 8730.5 12511

Min value of equations $(g) - (k)$ $G_{\text{min}} = 8730.5$
Char fastener shear resistance $\text{basic} = G_{\text{min}} / 1000 = 8.7305$ kN

Design resistance per fastener $p_{bl} = k_{mod} \times 2 \times \text{basic} / \gamma_{M} = 9.4021$ kN
Number of fasteners required $n = \text{INT}(F_d' / p_{bl}) + 1 = 2$
Total No of fasteners at joint $n = 2$
Shear force on LHS of joint       FvEd1=4.2073 kN
Shear force on RHS of joint       FvEd2=4.2073 kN
Height of member (2)              h=200 mm
Loaded edge distance              he=100 mm

**DESIGN**    Design axial load at joint   11.9 kN
**SUMMARY**   Number of fasteners per joint 2
               Design joint resistance        18.804 kN
Location: Default case 5

Joint comprising 3 members (fasteners in double shear)

Design axial load at joint \( F_d' = 11.9 \text{ kN} \)

---

In line connection
\( \alpha_1' = 0^\circ \) & \( \alpha_2' = 0^\circ \)

(1) = member 1
(2) = member 2

---

### ELEVATION

- \( t_1 \)
- \( t_2 \)

### PLAN

- \( t_1 \)

---

Thickness of outer members \( t_1 = 48.5 \text{ mm} \)
Timber strength class D35.
Timber characteristic density \( p_{k1} = 540 \text{ kg/m}^3 \)
Angle between grain and load \( \alpha_1' = 0^\circ \) degrees
Thickness of other member \( t_2 = 97 \text{ mm} \)
Timber strength class D35.
Timber characteristic density \( p_{k2} = 540 \text{ kg/m}^3 \)
Angle between grain and load \( \alpha_2' = 0^\circ \) degrees
Bolt/dowel diameter (8 to 30 mm) \( b_d = 20 \text{ mm} \)
Basic loads are in kN and will be derived from EC5 Section 8.

Charact. fastener yield moment \( M_{yRk} = 0.3 \times 400 \times b_d^2 \times 2.6 = 289640 \text{ Nmm} \)
Charact. embedment strength \( f_{hok1} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k1} = 35.424 \text{ N/mm}^2 \)
Denominator (expression 8.31) \( \text{denom}_1 = (K_{90} \times \sin(\alpha_1')^2) + \cos(\alpha_1')^2 = 1 \)
Embedment strength (Timber 1) \( f_{h1k} = f_{hok1} / \text{denom}_1 = 35.424 \text{ N/mm}^2 \)
Charact. embedment strength \( f_{hok2} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k1} = 35.424 \text{ N/mm}^2 \)
Denominator (expression 8.31) \( \text{denom}_2 = (K_{90} \times \sin(\alpha_2')^2) + \cos(\alpha_2')^2 = 1 \)
Embedment strength (Timber 2) \( f_{h2k} = f_{hok2} / \text{denom}_2 = 35.424 \text{ N/mm}^2 \)
Factor \( \beta = f_{h2k} / f_{h1k} = 1 \)

Equations \( g \quad h \quad j \quad k \)
| 34361 | 34361 | 17642 | 23297 |

Min value of equations \( (g) - (k) \) \( G_{\text{min}} = 17642 \)
Char fastener shear resistance \( \text{basic} = G_{\text{min}} / 1000 = 17.642 \text{ kN} \)

Design resistance per fastener \( p_{bl} = \text{kmod} \times 2 \times \text{basic} / \text{gamM} = 18.999 \text{ kN} \)
Number of fasteners required \( n = \text{INT}(F_d' / p_{bl}) + 1 = 1 \)
Total No of fasteners at joint \( n = 1 \)
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load at joint</td>
<td>11.9 kN</td>
</tr>
<tr>
<td>Number of fasteners per joint</td>
<td>1</td>
</tr>
<tr>
<td>Design joint resistance</td>
<td>18.999 kN</td>
</tr>
</tbody>
</table>
Location: Default case 6

Joint comprising timber member fixed to metal plate

with fasteners in single shear

Design axial load at joint  \( F_d' = 6.37 \text{ kN} \)

---

(1) ____________—˜ _ _ _—˜

(2) —˜      —˜

(1) = member 1

(2) = member 2

---

**ELEVATION**

---

**PLAN**

Thickness of timber member  \( t_1 = 47 \text{ mm} \)

Timber strength class C24.

Timber characteristic density  \( p_{k1} = 350 \text{ kg/m}^3 \)

Angle between grain and load  \( \alpha_{l1}' = 45^\circ \)

Thickness of steel plate  \( t_3 = 15 \text{ mm} \)

Bolt diameter (10 to 12 mm)  \( b_d = 12 \text{ mm} \)

**Fastener shear capacities**

Basic loads are in kN and will be derived from EC5 Section 8.

Charact fastener yield moment  \( M_{yRk} = 0.3 \times 400 \times b_d^2 \times 2.6 = 76745 \text{ Nmm} \)

Charact. embedment strength  \( f_{hok1} = 0.082 \times (1-0.01 \times b_d) \times p_{k1} \)

\[= 25.256 \text{ N/mm}^2\]

Denominator (expression 8.31)  \[ \text{denom1} = ((K90 \times \sin(\alpha_{l1})^2) + \cos(\alpha_{l1})^2) = 1.265 \]

Embedment strength (Timber 1)  \( f_{h1k} = f_{hok1}/\text{denom1} = 19.965 \text{ N/mm}^2 \)

Factor  \( \beta = f_{h2k}/f_{h1k} = 1 \)

Min value of equations (a)-(e)  \( G_{\text{min}} = 6826.6 \)

Char fastener shear resistance  \( \text{basic} = G_{\text{min}}/1000 = 6.8266 \text{ kN} \)

Design resistance per fastener  \( p_{bl} = k_{\text{mod}} \times \text{basic}/\gamma_{M} = 3.6759 \text{ kN} \)

Total No of fasteners at joint  \( n = 2 \)

Number of fasteners per row  \( n_{fpr} = 2 \)

Lines of fasteners to be adopted  \( r_{pl} = 1 \)

Spacing between fasteners  \( a_1 = 80 \text{ mm} \)

Effect No of fasteners per row  \( n_{ef} = n_{fpr}^{0.9} (a_1/(13 \times b_d))^0.25 = 1.5791 \)

Effective joint design capacity  \( F_{vef} = r_{pl} \times n_{ef} \times p_{bl} = 5.8047 \text{ kN} \)

**Toothed-plate connector assembly**

Toothed-plate connectors should comply with BS EN 912 and BS EN 14545.

Connector nominal diameter  \( d_c = 50 \text{ mm} \)
**Design shear capacity for one toothed-plate connector unit**

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of metal in connector</td>
<td>$t_{met}=0.9 \text{ mm}$</td>
</tr>
<tr>
<td>Toothed-plate connector height</td>
<td>$h_c=15 \text{ mm}$</td>
</tr>
<tr>
<td>Material factor for connection</td>
<td>$\gamma_M=1.3$</td>
</tr>
<tr>
<td>Design capacity of one TPC unit</td>
<td>$F_{vRd}=\text{interf}*k_{mod}<em>F_{vRk}/(1000</em>\gamma_M)=3.4267 \text{ kN}$</td>
</tr>
</tbody>
</table>

**Combined effective joint design resistance**

Combined joint design resistance $p_{bl}=p_{bl1}+p_{bl2}=12.658 \text{ kN}$

**Minimum bolt spacings**

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle alpha</td>
<td>$\alpha=0.7854 \text{ radians}$</td>
</tr>
<tr>
<td>Spacing parallel to grain</td>
<td>$a_1=(4+\cos(\alpha))*b_d=56.485 \text{ mm}$</td>
</tr>
<tr>
<td>Spacing perpendicular to grain</td>
<td>$a_2=4*b_d=48 \text{ mm}$</td>
</tr>
<tr>
<td>End distance (loaded end)</td>
<td>$a_3t=7*b_d=84 \text{ mm}$</td>
</tr>
<tr>
<td>End distance (unloaded end)</td>
<td>$a_3c=4*b_d=48 \text{ mm}$</td>
</tr>
<tr>
<td>Edge distance (loaded edge)</td>
<td>$a_4t=(2+2*\sin(\alpha))*b_d=40.971 \text{ mm}$</td>
</tr>
<tr>
<td>Edge distance (unloaded edge)</td>
<td>$a_4c=3*b_d=36 \text{ mm}$</td>
</tr>
</tbody>
</table>

**Minimum spacings for toothed-plate connectors - to EC5 Table 8.8**

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing within one row parallel to grain</td>
<td>$a_1=(1.2+0.3*\cos(\alpha))*d_c=70.607 \text{ mm}$</td>
</tr>
<tr>
<td>Spacing between connector rows perpendicular to grain:</td>
<td>$a_2=1.2*d_c=60 \text{ mm}$</td>
</tr>
<tr>
<td>End distance (loaded)</td>
<td>$a_3t=1.5*d_c=75 \text{ mm}$</td>
</tr>
<tr>
<td>End distance (unloaded)</td>
<td>$a_3c=(0.9+0.6*\sin(\alpha))*d_c=66.213 \text{ mm}$</td>
</tr>
<tr>
<td>Edge distance (loaded)</td>
<td>$a_4t=(0.6+0.2*\sin(\alpha))*d_c=37.071 \text{ mm}$</td>
</tr>
<tr>
<td>Edge distance (unloaded)</td>
<td>$a_4c=0.6*d_c=30 \text{ mm}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear force on LHS of joint</td>
<td>$F_{vEd1}=2.2521 \text{ kN}$</td>
</tr>
<tr>
<td>Shear force on RHS of joint</td>
<td>$F_{vEd2}=2.2521 \text{ kN}$</td>
</tr>
<tr>
<td>Height of member (2)</td>
<td>$h=200 \text{ mm}$</td>
</tr>
<tr>
<td>Loaded edge distance</td>
<td>$h_e=100 \text{ mm}$</td>
</tr>
</tbody>
</table>

**Joint slip check - to EC5 Section 7**

Stiffness of joint final condition at the SLS: $K_{serfj}=K_{serj}/(1+2*k_{def})=2983 \text{ N}$

Final deflection of the joint at SLS: $u_{ser}=F_{d_{sls}}/K_{serfj}+2=3.4933 \text{ mm}$

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load at joint</td>
<td>6.37 \text{ kN}</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of fasteners per joint</td>
<td>2</td>
</tr>
<tr>
<td>Contribution from bolts</td>
<td>5.8047 \text{ kN}</td>
</tr>
<tr>
<td>Contribution from connectors</td>
<td>6.8535 \text{ kN}</td>
</tr>
</tbody>
</table>

**Combined design joint resistance** 12.658 \text{ kN}
Location: Default case 7

Joint comprising 2 members (fasteners in single shear)

Design axial load at joint \( F_d' = 13.58 \text{ kN} \)

```
(2) ______________—˜ _ _ _—˜
—˜      —˜
Member inclined at an angle \( \alpha' \) degrees
—˜      —˜
(1) = member 1
(2) = member 2

ELEVATION

PLAN
```

Thickness of member 1 \( t_1 = 97 \text{ mm} \)
Timber strength class C24.
Timber characteristic density \( p_{k1} = 350 \text{ kg/m}^3 \)
Angle between grain and load \( \alpha_{p1}' = 0^\circ \)
Thickness of member 2 \( t_2 = 72 \text{ mm} \)
Timber strength class C24.
Timber characteristic density \( p_{k2} = 350 \text{ kg/m}^3 \)
Angle between grain and load \( \alpha_{p2}' = 42^\circ \)
Bolt/dowel diameter (8 to 30 mm) \( b_d = 20 \text{ mm} \)

Basic loads are in kN and will be derived from EC5 Section 8.
Charact fastener yield moment \( M_{yRk} = 0.3 \times 400 \times b_d^2 \times 2.6 = 289640 \text{ Nmm} \)
Charact.embedment strength \( f_{hok1} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k1} \)
\[ = 22.96 \text{ N/mm}^2 \]

Denominator (expression 8.31) \( \text{denom}_1 = ((K_90 \times \sin(\alpha_{p1})^2 + \cos(\alpha_{p1})^2) = 1 \)
Embedment strength (Timber 1) \( f_{h1k} = f_{hok1} / \text{denom}_1 = 22.96 \text{ N/mm}^2 \)
Charact.embedment strength \( f_{hok2} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k2} \)
\[ = 22.96 \text{ N/mm}^2 \]

Denominator (expression 8.31) \( \text{denom}_2 = ((K_90 \times \sin(\alpha_{p2})^2 + \cos(\alpha_{p2})^2) = 1.291 \)
Embedment strength (Timber 2) \( f_{h2k} = f_{hok2} / \text{denom}_2 = 17.784 \text{ N/mm}^2 \)
Factor \( \beta = f_{h2k} / f_{h1k} = 0.77458 \)

Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.
Min value of equations (a)-(f) \( G_{\text{min}} = 13392 \text{ kN} \)
Char fastener shear resistance \( G_{\text{basic}} = G_{\text{min}} / 1000 = 13.392 \text{ kN} \)

Design resistance per fastener \( p_{bl} = k_{\text{mod}} \times G_{\text{basic}} / \gamma_M = 9.2713 \text{ kN} \)
Number of fasteners required \( n = \text{INT}(F_d' / p_{bl}) + 1 = 2 \)
Total No of fasteners at joint \( n = 2 \)
Number of fasteners per row \( n_{\text{fpr}} = 2 \)
Lines of fasteners to be adopted \( r_{\text{pl}} = 1 \)
Spacing between fasteners\nEffect No of fasteners per row\nEffective joint design capacity\nShear force on LHS of joint\nShear force on RHS of joint\nHeight of member (2)\nLoaded edge distance

DESIGN

SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load at joint</td>
<td>13.58 kN</td>
</tr>
<tr>
<td>Number of fasteners per joint</td>
<td>2</td>
</tr>
<tr>
<td>Effective joint design capacity</td>
<td>13.625 kN</td>
</tr>
</tbody>
</table>

Timber design to BS5268 and Eurocode 5
Bolted joint
Made by: IFB
Date: 02/12/19
Ref No: SC273 EC
Location: Default case 8

Joint comprising 3 members (fasteners in double shear)

Design axial load at joint \( F_d^\prime = 22 \text{ kN} \)

In line connection

\[ \text{alp}_1^\prime = 0^\circ \text{ & alp}_2^\prime = 0^\circ \]

(1) = member 1
(2) = member 2

Thickness of outer members \( t_1 = 48.5 \text{ mm} \)
Timber characteristic density \( p_k_1 = 540 \text{ kg/m}^3 \)
Angle between grain and load \( \text{alp}_1^\prime = 0 \text{ degrees} \)
Thickness of other member \( t_2 = 97 \text{ mm} \)
Timber characteristic density \( p_k_2 = 540 \text{ kg/m}^3 \)
Angle between grain and load \( \text{alp}_2^\prime = 0 \text{ degrees} \)
Bolt/dowel diameter (8 to 30 mm) \( b_d = 16 \text{ mm} \)

Basic loads are in kN and will be derived from EC5 Section 8.

Charact fastener yield moment \( M_{yRk} = 0.3 \times 400 \times b_d \times \sqrt{2.6} = 162141 \text{ Nmm} \)
Charact. embedment strength \( f_{hok1} = 0.082 \times (1 - 0.01 \times b_d) \times p_k_1 \)
\[ = 37.195 \text{ N/mm}^2 \]

Denominator (expression 8.31) \( \text{denom}_1 = ((K90 \times \sin(\text{alp}_1)^2) + \cos(\text{alp}_1)^2) = 1 \)
Embedment strength (Timber 1) \( f_{h1k} = f_{hok1} / \text{denom}_1 = 37.195 \text{ N/mm}^2 \)
Charact. embedment strength \( f_{hok2} = 0.082 \times (1 - 0.01 \times b_d) \times p_k_2 \)
\[ = 37.195 \text{ N/mm}^2 \]
Denominator (expression 8.31) \( \text{denom}_2 = ((K90 \times \sin(\text{alp}_2)^2) + \cos(\text{alp}_2)^2) = 1 \)
Embedment strength (Timber 2) \( f_{h2k} = f_{hok2} / \text{denom}_2 = 37.195 \text{ N/mm}^2 \)
Factor \( \beta = f_{h2k} / f_{h1k} = 1 \)

Expressions (g)-(k) for fasteners in double shear, refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.

Equations \( g \quad h \quad j \quad k \)
\[ 28863 \quad 28863 \quad 13351 \quad 15976 \]
Min value of equations (g)-(k) \( G_{\text{min}} = 13351 \)
Char fastener shear resistance \( \text{basic} = G_{\text{min}} / 1000 = 13.351 \text{ kN} \)

Design resistance per fastener \( p_{bl} = k_{\text{mod}} \times 2 \times \text{basic} / \gamma_{M} = 14.378 \text{ kN} \)
Number of fasteners required \( n = \text{INT}(F_d^\prime / p_{bl}) + 1 = 2 \)
Total No of fasteners at joint \( n = 2 \)
Number of fasteners per row \( n_{fpr} = 1 \)
Lines of fasteners to be adopted \( r_{pl} = 2 \)
Spacing between fasteners \( a_1 = 112 \text{ mm} \)
Effect No of fasteners per row  \( n_{ef} = n_f \times r_{pl} \times a_1 \times (13 \times b_d)^{0.25} = 0.85662 \)

Effective joint design capacity  \( F_{vef} = r_{pl} \times n_{ef} \times p_{bl} = 24.633 \, \text{kN} \)

### Minimum bolt spacings

- **Angle alpha**: \( \alpha = 0 \) radians
- **Spacing parallel to grain**: \( a_1 = (4 + \cos(\alpha)) \times b_d = 80 \, \text{mm} \)
- **Spacing perpendicular to grain**: \( a_2 = 4 \times b_d = 64 \, \text{mm} \)
- **End distance (loaded end)**: \( a_3t = 7 \times b_d = 112 \, \text{mm} \)
- **End distance (unloaded end)**: \( a_3c = 4 \times b_d = 64 \, \text{mm} \)
- **Edge distance (loaded edge)**: \( a_4t = 3 \times b_d = 48 \, \text{mm} \)
- **Edge distance (unloaded edge)**: \( a_4c = 3 \times b_d = 48 \, \text{mm} \)

### Design
- **Design axial load at joint**: 22 kN

### Summary
- **Number of fasteners per joint**: 2
- **Effective joint design capacity**: 24.633 kN
Location: Bolted timber connection with axial load and moment

Timber to timber joint with timber splice plate adopted.

<table>
<thead>
<tr>
<th>Timber joist</th>
<th>Timber joist</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber splice</td>
<td></td>
</tr>
</tbody>
</table>

Plan view on splice

Three bolt symmetrical joint with in line splice plate designed to BS5268-2:2002

Shear load to be carried by joint $F=1.15$ kN
Moment resisted by bolt group $BM=0.35$ kNm
Depth of timber section $dp=300$ mm
Thickness of member 1 $t1=47$ mm
Strength class C22 to Table 8.
Timber characteristic density $p1=340$ kg/m³
Thickness of timber splice plate $t2=63$ mm
Strength class C22 to Table 8.
Timber characteristic density $p2=340$ kg/m³
Gap distance between joists $gp=10$ mm
Joint to be connected by bolts $K2b=1.33$
Fixing diameter (8 mm to 36 mm) $d=16$ mm
Actual edge distance to bolt ctr $ed=65$ mm
Actual end distance $el=125$ mm
Basic loads in kN will be derived from formula given in Annex G.
Constant $Kd=1$
Load duration for bolted/dowelled connection = long term.
Embedding strength (Timb 1) $fhd1=fhod/den1=8.0742$ N/mm²
Embedding strength (Timb 2) $fhd2=fhod/den2=8.0742$ N/mm²
Factor $\beta=fhd2/fhd1=1$

Basic bolt shear capacity $basic=1*Gmin/(Fd*Kd)=2.9517$ kN
Mod factor for moisture content $K56=1$
Mod factor for number of bolts $K57=1$
Dry stresses are appropriate

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth factor</td>
<td>$K_7 = (300/dp)^{0.11} = 1$</td>
</tr>
</tbody>
</table>

**Bolt summary:**
- Max resultant force: 2.0942 kN
- Angle to grain: 10.547 degrees
- Permissible bolt load: 2.9517 kN

**Summary:**
- Combined bend moment: 0.4995 kNm
- Applied bend stress: 0.52857 N/mm²
- Permiss bend Stress: 7.48 N/mm²
- Applied shear stress: 0.10952 N/mm²
- Permiss shear Stress: 0.781 N/mm²
Location: Timber splice plate type 1

Timber to timber joint with timber splice plate adopted.

Plan view on splice

Two bolt symmetrical joint with in line splice plate designed to BS5268-2:2002

Shear load to be carried by joint F=0.35 kN
Moment resisted by bolt group BM=0.15 kNm
Depth of timber section dp=200 mm
Thickness of member 1 t1=47 mm
Strength class C22 to Table 8.
Timber characteristic density pk1=340 kg/m³
Thickness of timber splice plate t2=47 mm
Strength class C22 to Table 8.
Timber characteristic density pk2=340 kg/m³
Gap distance between joists gp=5 mm
Joint to be connected by bolts K2b=1.33
Fixing diameter (8 mm to 36 mm) d=16 mm
Actual end distance el=100 mm
Actual centres of bolts // grain cag=150 mm

Basic loads in kN will be derived from formula given in Annex G.
Constant Kd=1
Load duration for bolted/dowelled connection = medium term.
Fixing spacing parallel to the grain alphapar=112 mm
Embedding strength (Timb 1) fhd1=fhod/den1=11.881 N/mm²
Embedding strength (Timb 2) fhd2=fhod/den2=11.881 N/mm²
Factor beta=fhd2/fhd1=1
Equations G1 to G6 in annex G in BS5268-2:2002.

Basic bolt shear capacity basic=1*Gmin/(Fd*Kd)=3.5158 kN
Mod factor for moisture content K56=1
Mod factor for number of bolts K57=1
Dry stresses are appropriate

Depth factor \[ K_7 = (300/d_p)^{0.11} = 1.0456 \]

**Bolt summary:**
- Max resultant force: 1.175 kN
- Angle to grain: 90 degrees
- Permissible bolt load: 3.5158 kN

**Timber splice plate summary:**
- Combined bend moment: 0.21213 kNm
- Applied bend stress: 0.67699 N/mm²
- Permiss bend Stress: 9.7765 N/mm²
- Applied shear stress: 0.061375 N/mm²
- Permiss shear Stress: 0.97625 N/mm²
Timber to timber joint with steel splice plate adopted.

<table>
<thead>
<tr>
<th>Timber joist</th>
<th>Timber joist</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel plate</td>
<td>Plan view on splice</td>
</tr>
</tbody>
</table>

Location: Steel splice plate type 2

X
<table>
<thead>
<tr>
<th>el</th>
<th>gp</th>
</tr>
</thead>
</table>

X
<table>
<thead>
<tr>
<th>ed</th>
<th>cpg</th>
<th>+</th>
<th>+</th>
<th>dp</th>
</tr>
</thead>
</table>
X
| ed |

Two bolt symmetrical joint with in line splice plate designed to BS5268-2:2002

Shear load to be carried by joint \( F = 0.35 \text{ kN} \)

Moment resisted by bolt group \( BM = 0.15 \text{ kNm} \)

Depth of timber section \( dp = 200 \text{ mm} \)

Thickness of member 1 \( t_1 = 47 \text{ mm} \)

Strength class C22 to Table 8.

Timber characteristic density \( p_k1 = 340 \text{ kg/m}^3 \)

Thickness of steel splice plate \( st = 10 \text{ mm} \)

Gap distance between joists \( gp = 0 \text{ mm} \)

Joint to be connected by bolts \( K_{2b} = 1.33 \)

Fixing diameter (8 mm to 36 mm) \( d = 16 \text{ mm} \)

Actual edge distance to bolt ctr \( ed = 64 \text{ mm} \)

Actual end distance \( el = 112 \text{ mm} \)

Basic loads in kN will be derived from formula given in Annex G.

Constant \( K_d = 1 \)

Load duration for bolted/dowelled connection = short term.

Embedding strength (Timb 1) \( f_{hd1} = f_{hd}/den1 = 12.265 \text{ N/mm}^2 \)

Basic bolt shear capacity \( \text{basic} = 1*G_{\text{min}}/(F_d*K_d) = 3.6292 \text{ kN} \)

Mod factor for moisture content \( K_{56} = 0.7 \)

Mod factor for number of bolts \( K_{57} = 1 \)
Bolt summary:
Max resultant force 2.0907 kN
Angle to grain 4.8016 degrees
Permissible bolt load 2.5405 kN

Steel splice plate summary:
Combined bend moment 0.1892 kNm
Applied bend stress 2.838 N/mm²
Permiss bend Stress 180 N/mm²
Applied shear stress 0.21084 N/mm²
Permiss shear stress 100 N/mm²
Location: Timber overlap splice type 3

Timber to timber overlap joint to be adopted.

Timber joist

Timber joist

Plan view on splice

Three bolt overlapped timber joint designed to BS5268-2:2002

Shear load to be carried by joint \( F = 2 \text{ kN} \)

Moment resisted by bolt group \( BM = 0.25 \text{ kNm} \)

Depth of timber section \( dp = 200 \text{ mm} \)

Thickness of member 1 \( t_1 = 75 \text{ mm} \)

Strength class C22 to Table 8.

Timber characteristic density \( p_k1 = 340 \text{ kg/m}^3 \)

Thickness of member 2 \( t_2 = 75 \text{ mm} \)

Strength class C22 to Table 8.

Timber characteristic density \( p_k2 = 340 \text{ kg/m}^3 \)

Joint to be connected by bolts \( K_{2b} = 1.33 \)

Fixing diameter (8 mm to 36 mm) \( d = 16 \text{ mm} \)

Actual end distance \( e_1 = 125 \text{ mm} \)

Actual centres of bolts // grain \( c_1 = 150 \text{ mm} \)

Basic loads in kN will be derived from formula given in Annex G.

Constant \( K_d = 1 \)

Load duration for bolted/dowelled connection = medium term.

Fixing spacing parallel to the grain \( \alpha_{par} = 100 \text{ mm} \)

Embedding strength (Timb 1) \( f_{hd1} = f_{hd}/den1 = 11.226 \text{ N/mm}^2 \)

Embedding strength (Timb 2) \( f_{hd2} = f_{hd}/den2 = 11.226 \text{ N/mm}^2 \)

Factor \( \beta = f_{hd2}/f_{hd1} = 1 \)

Equations G1 to G6 in annex G in BS5268-2:2002.

Basic bolt shear capacity \( \text{basic} = 1 \times G_{\text{min}}/(F_d \times K_d) = 5.3012 \text{ kN} \)

Mod factor for moisture content \( K_{56} = 0.4 \)

Mod factor for number of bolts \( K_{57} = 1 \)

Bolt summary:

| DESIGN | Max resultant force 1.5 kN |
| SUMMARY | Angle to grain 90 degrees |
|         | Permissible bolt load 2.1205 kN |
Location: Timber splice plate type 4

Timber to timber joint with timber splice plate adopted.

Plan view on splice

Three bolt symmetrical joint with in line splice plate designed to BS5268-2:2002

Shear load to be carried by joint F=0.5 kN
Moment resisted by bolt group BM=0.25 kNm
Depth of timber section dp=225 mm
Thickness of member 1 t1=76 mm
Strength class C22 to Table 8.
Timber characteristic density pk1=340 kg/m³
Thickness of timber splice plate t2=76 mm
Strength class C22 to Table 8.
Timber characteristic density pk2=340 kg/m³
Gap distance between joists gp=50 mm
Joint to be connected by bolts K2b=1.33
Fixing diameter (8 mm to 36 mm) d=12 mm
Actual edge distance to bolt ctr ed=48 mm
Actual end distance el=90 mm
Basic loads in kN will be derived from formula given in Annex G.
Constant Kd=1
Load duration for bolted/dowelled connection = long term.
Embedding strength (Timb 1) fhd1=fhod/den1=8.5681 N/mm²
Embedding strength (Timb 2) fhd2=fhod/den2=8.5681 N/mm²
Factor beta=fhd2/fhd1=1

Basic bolt shear capacity basic=1*Gmin/(Fd*Kd)=2.934 kN
Mod factor for moisture content K56=1
Mod factor for number of bolts K57=1
Dry stresses are appropriate

Depth factor

\[ K_7 = (300/dp)^{0.11} = 1.0322 \]

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Max resultant force 1.9451 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Angle to grain 4.9154 degrees</td>
</tr>
<tr>
<td>Permissible bolt load 2.934 kN</td>
<td></td>
</tr>
</tbody>
</table>

Timber splice plate summary:

- Combined bend moment 0.3075 kNm
- Applied bend stress 0.47953 N/mm²
- Permiss bend Stress 7.7205 N/mm²
- Applied shear stress 0.052772 N/mm²
- Permiss shear Stress 0.781 N/mm²
Location: Timber splice plate type 5

Timber to timber joint with timber splice plate adopted.

<table>
<thead>
<tr>
<th>Timber joist</th>
<th>Timber joist</th>
<th>Timber splice</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plan view on splice</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Four bolt symmetrical joint with in line splice plate designed to BS5268-2:2002

Shear load to be carried by joint $F = 0.9$ kN
Moment resisted by bolt group $BM = 0.45$ kNm
Depth of timber section $dp = 225$ mm
Strength of member 1 $t_1 = 47$ mm
Timber characteristic density $p_{kl} = 340$ kg/m$^3$
Thickness of timber splice plate $t_2 = 47$ mm
Strength class C22 to Table 8.
Depth of timber section $dp = 225$ mm
Thickness of timber splice plate $t_2 = 47$ mm
Strength class C22 to Table 8.
Timber characteristic density $p_{kl} = 340$ kg/m$^3$
Gap distance between joists $gp = 10$ mm
Joint to be connected by bolts $K_{2b} = 1.33$
Fixing diameter (8 mm to 36 mm) $d = 12$ mm
Actual edge distance to bolt ctr $ed = 65$ mm
Actual end distance $el = 125$ mm
Actual centres of bolts // grain $cag = 100$ mm
Basic loads in kN will be derived from formula given in Annex G.
Constant $K_d = 1$
Load duration for bolted/dowelled connection = long term.
Fixing spacing parallel to the grain $\alpha_{par} = 84$ mm
Embedding strength (Timb 1) $f_{hd1} = f_{hod}/den1 = 9.3782$ N/mm$^2$
Embedding strength (Timb 2) $f_{hd2} = f_{hod}/den2 = 9.3782$ N/mm$^2$
Factor $\beta = f_{hd2}/f_{hd1} = 1$
Basic bolt shear capacity $\text{basic} = 1 \times G_{min}/(F_d \times K_d) = 2.1585$ kN
Mod factor for moisture content $K_{56} = 1$
Mod factor for number of bolts $K_{57} = 1$

Bolt summary:

**DESIGN**
Max resultant force 1.8011 kN

**SUMMARY**
Angle to grain 51.405 degrees
Permissible bolt load 2.1585 kN
Location: Timber splice plate type 6

Timber to timber joint with timber splice plate adopted.

Timber joist | Timber joist
-------------|-------------
Timber splice

Plan view on splice

X

| cag | el | gp |

| ed | cpg | + | + | + | + | dp |
| ed | cpg | + | + | + | + |

Five bolt symmetrical joint with in line splice plate designed to BS5268-2:2002

Shear load to be carried by joint F=2.25 kN
Moment resisted by bolt group BM=0.8 kNm
Depth of timber section dp=300 mm
Thickness of member 1 t1=50 mm
Strength class C22 to Table 8.
Timber characteristic density pk1=340 kg/m³
Thickness of timber splice plate t2=50 mm
Strength class C22 to Table 8.
Timber characteristic density pk2=340 kg/m³
Gap distance between joists gp=0 mm
Joint to be connected by bolts K2b=1.33
Fixing diameter (8 mm to 36 mm) d=16 mm
Actual edge distance to bolt ctr ed=65 mm
Actual end distance el=125 mm
Actual centres of bolts // grain cag=100 mm
Basic loads in kN will be derived from formula given in Annex G.
Constant Kd=1
Load duration for bolted/dowelled connection = long term.
Fixing spacing parallel to the grain alphapar=100 mm
Embedding strength (Timb 1) fhd1=fhod/den1=8.6819 N/mm²
Embedding strength (Timb 2) fhd2=fhod/den2=8.6819 N/mm²
Factor beta=fhd2/fhd1

Basic bolt shear capacity basic=1*Gmin/(Fd*Kd)=2.8343 kN
Mod factor for moisture content K56=1
Mod factor for number of bolts K57=1
Dry stresses are appropriate

Depth factor \[ K_7 = (300/dp)^{0.11} = 1 \]

**Bolt summary:**
- Max resultant force 2.2893 kN
- Angle to grain 40.22 degrees
- Permissible bolt load 2.8343 kN

**Timber splice plate summary:**
- Combined bend moment 1.1938 kNm
- Applied bend stress 1.5917 N/mm²
- Permiss bend Stress 7.48 N/mm²
- Applied shear stress 0.25376 N/mm²
- Permiss shear Stress 0.781 N/mm²
Location: Bolted timber connection with axial load and moment

Timber to timber joint with timber splice plate adopted

Timber joist | Timber joist
-------------|-------------
Timber splice

Plan view on splice

X
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el gp

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cpg
+ +
cpg
+ +
ed

Three bolt symmetrical joint with in line splice plate to EC5

Design shear to be resisted Fd' = 1.61 kN
Design moment to be resisted Md = 0.49 kNm
Depth of timber section h = 300 mm
Thickness of member 1 t1 = 47 mm
Timber strength class C22.
Timber characteristic density pk1 = 340 kg/m³
Thickness of timber splice plate t2 = 63 mm
Timber strength class C22.
Timber characteristic density pk2 = 340 kg/m³
Gap distance between joists gp = 10 mm
Fixing diameter (8 mm to 30 mm) bd = 16 mm
Depth factor (Clause 3.2) kh = 1.0
Actual edge distance to bolt ctr ed = 65 mm
Actual end distance el = 125 mm
Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.
Char fastener yield moment MyRk = 0.3*400*bd^2.6 = 162141 Nmm
Char embedment strength fhok1 = 0.082*(1-0.01*bd)*pk1 = 23.419
Embedding strength (Timb 1) fh1k = fhok1/denom1 = 22.965 N/mm²
Char embedment strength fhok2 = 0.082*(1-0.01*bd)*pk2 = 23.419
Embedding strength (Timb 2) fh2k = fhok2/denom2 = 22.965 N/mm²
Factor beta = fh2k/fh1k = 1
Min value of equations (a)-(f) Gmin = 8521.8
Char bolt shear resistance basic = Gmin/1000 = 8.5218 kN
Design resistance per bolt pbl = kmod/gamM*basic = 5.2442 kN
Load-sharing modification factor \( k_{sys} = 1.1 \)

Design bending strength \( f_{md} = k_h \cdot k_{sys} \cdot f_{mk} \cdot k_{mod}/\gamma_M = 14.892 \text{ N/mm}^2 \)

Design shear strength \( f_{vd} = k_{sys} \cdot f_{vk} \cdot k_{mod}/\gamma_M = 2.5723 \text{ N/mm}^2 \)

Check splitting capacity of the connection

Design resultant force \( RF = 2.9319 \text{ kN} \)

Tension force component \( \perp \) grain \( F_{v Ed} = RF \cdot \sin(\alpha) = 0.53666 \text{ kN} \)

Loaded edge distance \( h_e = 250 \text{ mm} \)

Design splitting capacity \( F_{90Rd} = F_{90Rk} \cdot k_{mod}/\gamma_M = 15.683 \text{ kN} \)

**Bolt summary:**

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<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
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<tbody>
<tr>
<td>Max resultant force</td>
<td>2.9319 \text{ kN}</td>
<td></td>
</tr>
<tr>
<td>Angle to grain</td>
<td>10.547 degrees</td>
<td></td>
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<tr>
<td>Effect bolt design capacity</td>
<td>5.2442 \text{ kN}</td>
<td></td>
</tr>
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**Timber splice plate summary:**

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<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td>Combined bend moment</td>
<td>0.6993 \text{ kNm}</td>
</tr>
<tr>
<td>Design bend stress</td>
<td>0.74 \text{ N/mm}^2</td>
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<tr>
<td>Design bend strength</td>
<td>14.892 \text{ N/mm}^2</td>
</tr>
<tr>
<td>Design shear stress</td>
<td>0.15333 \text{ N/mm}^2</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>2.5723 \text{ N/mm}^2</td>
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</table>
Location: Timber splice plate type 1

Timber to timber joint
with timber splice plate adopted

<table>
<thead>
<tr>
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<tr>
<td>Timber splice</td>
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</table>

Plan view on splice

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</table>

Two bolt symmetrical joint with in line splice plate to EC5

Design shear to be resisted $F_d' = 0.49 \text{kN}$
Design moment to be resisted $M_d = 0.21 \text{kNm}$
Depth of timber section $h = 200 \text{ mm}$
Thickness of member 1 $t_1 = 47 \text{ mm}$
Timber strength class C22.
Timber characteristic density $p_{k1} = 340 \text{ kg/m}^3$
Thickness of timber splice plate $t_2 = 47 \text{ mm}$
Timber strength class C22.
Timber characteristic density $p_{k2} = 340 \text{ kg/m}^3$
Gap distance between joists $g_p = 5 \text{ mm}$
Fixing diameter (8 mm to 30 mm) $b_d = 16 \text{ mm}$
Depth factor (Clause 3.2) $k_h = 1.0$
Actual end distance $e_l = 100 \text{ mm}$
Actual centres of bolts // grain $c_{ag} = 150 \text{ mm}$

Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.
Char fastener yield moment $M_{yrk} = 0.3 \times 400 \times b_d^2.6 = 162141 \text{ Nmm}$
Fixing spacing // to the grain $\alpha_{p\text{par}} = 112 \text{ mm}$
Char embedment strength $f_{hok1} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k1} = 23.419$
Embedding strength (Timb 1) $f_{hlk1} = f_{hok1}/\text{denom1} = 14.729 \text{ N/mm}^2$
Char embedment strength $f_{hok2} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k2} = 23.419$
Embedding strength (Timb 2) $f_{hlk2} = f_{hok2}/\text{denom2} = 14.729 \text{ N/mm}^2$
Factor $\beta = f_{hlk2}/f_{hlk1}$
Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.
Min value of equations (a)-(f) $G_{\text{min}} = 4587.9$
Char bolt shear resistance $\text{basic} = G_{\text{min}}/1000 = 4.5879 \text{ kN}$
Design resistance per bolt $p_{bl} = k_{mod}/\gamma_{M} \times \text{basic} = 2.8233 \text{ kN}$
Load-sharing modification factor $k_{sys} = 1.1$

Design bending strength $f_{md} = k_{h} \cdot k_{sys} \cdot f_{mk} \cdot k_{mod} / \gamma_{M} = 14.892 \text{ N/mm}^2$

Design shear strength $f_{vd} = k_{sys} \cdot f_{vk} \cdot k_{mod} / \gamma_{M} = 2.5723 \text{ N/mm}^2$

Check splitting capacity of the connection

Design resultant force $RF = 1.645 \text{ kN}$

Tension force component perpendicular to grain $F_{vEd} = RF \cdot \sin(\alpha) = 1.645 \text{ kN}$

Loaded edge distance $he = 150 \text{ mm}$

Design splitting capacity $F_{90Rd} = F_{90Rk} \cdot k_{mod} / \gamma_{M} = 9.9185 \text{ kN}$

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<th>Bolt summary:</th>
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<tbody>
<tr>
<td>Max resultant force</td>
<td>1.645 kN</td>
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<tr>
<td>Angle to grain</td>
<td>90 degrees</td>
</tr>
<tr>
<td>Design capacity per bolt</td>
<td>2.8233 kN</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Timber splice plate summary:</th>
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</thead>
<tbody>
<tr>
<td>Combined bend moment</td>
<td>0.29698 kNm</td>
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<tr>
<td>Design bend stress</td>
<td>0.94779 N/mm$^2$</td>
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<tr>
<td>Design bend strength</td>
<td>14.892 N/mm$^2$</td>
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<tr>
<td>Design shear stress</td>
<td>0.085925 N/mm$^2$</td>
</tr>
<tr>
<td>Design shear strength</td>
<td>2.5723 N/mm$^2$</td>
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</tbody>
</table>
Location: Steel splice plate type 2

Timber to timber joint with steel splice plate

Timber joist | Timber joist
---|---
Steel plate |

Plan view on splice

X

el gp

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Two bolt symmetrical joint with in line splice plate to EC5

Design shear to be resisted: $F_d' = 0.49 \text{ kN}$
Design moment to be resisted: $M_d = 0.21 \text{ kNm}$
Depth of timber section: $h = 200 \text{ mm}$
Thickness of member 1: $t_1 = 47 \text{ mm}$
Timber strength class C22.
Timber characteristic density: $p_k = 340 \text{ kg/m}^3$
Thickness of steel splice plate: $s_t = 10 \text{ mm}$
Gap distance between joists: $g_p = 0 \text{ mm}$
Fixing diameter (8 mm to 30 mm): $b_d = 16 \text{ mm}$
Depth factor (Clause 3.2): $k_h = 1.0$
Actual edge distance to bolt ctr: $e_d = 64 \text{ mm}$
Actual end distance: $e_l = 112 \text{ mm}$

Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.

Char fastener yield moment: $M_{yRk} = 0.3 \times 400 \times b_d^2 \times 2.6 = 162141 \text{ Nmm}$
Char embedment strength: $f_{ho1} = 0.082 \times (1 - 0.01 \times b_d) \times p_k = 23.419$
Embedding strength (Timb 1): $f_{hl1} = f_{ho1} / \text{denom1} = 23.323 \text{ N/mm}^2$
Min value of equations (a)-(e): $G_{\text{min}} = 8196.6$
Char bolt shear resistance: $\text{basic} = G_{\text{min}} / 1000 = 8.1966 \text{ kN}$
Design resistance per bolt: $p_{bl} = k_{mod} / g_m * \text{basic} = 5.0441 \text{ kN}$
Check splitting capacity of the connection

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<table>
<thead>
<tr>
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<tbody>
<tr>
<td>Design resultant force</td>
<td>RF=2.9269 kN</td>
</tr>
<tr>
<td>Tension force component</td>
<td>FvEd=RF*SIN(alpha)=0.245 kN</td>
</tr>
<tr>
<td>Loaded edge distance</td>
<td>he=150 mm</td>
</tr>
<tr>
<td>Design splitting capacity</td>
<td>F90Rd=F90Rk*kmod/gamM=9.9185 kN</td>
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Bolt summary:

<table>
<thead>
<tr>
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</thead>
<tbody>
<tr>
<td>Max resultant force</td>
<td>2.9269 kN</td>
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<tr>
<td>Angle to grain</td>
<td>4.8016 degrees</td>
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<tr>
<td>Effect bolt design capacity</td>
<td>5.0441 kN</td>
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</tbody>
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Steel splice plate summary:

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<tbody>
<tr>
<td>Combined bend moment</td>
<td>0.26488 kNm</td>
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<tr>
<td>Design bend stress</td>
<td>3.9732 N/mm²</td>
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<td>Design shear stress</td>
<td>0.29518 N/mm²</td>
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<td>Design shear strength</td>
<td>255 N/mm²</td>
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**Location:** Timber overlap splice type 3

Timber to timber overlap joint

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</table>

Three bolt overlapped timber joint designed to Eurocode 5

Design shear to be resisted: \( F_d' = 2.8 \text{ kN} \)
Design moment to be resisted: \( M_d = 0.35 \text{ kNm} \)
Depth of timber section: \( h = 200 \text{ mm} \)
Thickness of member 1: \( t_1 = 75 \text{ mm} \)
Timber strength class C22.
Timber characteristic density: \( p_{k1} = 340 \text{ kg/m}^3 \)
Timber strength class C22.
Thickness of member 2: \( t_2 = 75 \text{ mm} \)
Timber characteristic density: \( p_{k2} = 340 \text{ kg/m}^3 \)
Fixing diameter (8 mm to 30 mm): \( b_d = 16 \text{ mm} \)
Depth factor (Clause 3.2): \( k_h = 1.0 \)
Actual end distance: \( e_l = 125 \text{ mm} \)
Actual centres of bolts // grain: \( c_a g = 150 \text{ mm} \)

Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.

Char fastener yield moment: \( M_yR_k = 0.3 \times 400 \times b_d^2 \times 2.6 = 162141 \text{ Nmm} \)
Fixing spacing // to the grain: \( \alpha_{par} = 100 \text{ mm} \)
Char embedment strength (Timb 1): \( f_{h1k} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k1} = 23.419 \text{ N/mm}^2 \)
Embedding strength (Timb 1): \( f_{h1k} = f_{h1k} / \text{denom1} = 14.729 \text{ N/mm}^2 \)
Char embedment strength (Timb 2): \( f_{h2k} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k2} = 23.419 \text{ N/mm}^2 \)
Embedding strength (Timb 2): \( f_{h2k} = f_{h2k} / \text{denom2} = 14.729 \text{ N/mm}^2 \)
Factor: \( \beta = f_{h2k} / f_{h1k} = 1 \)

Expressions (a)-(f) for fasteners in single shear refer to section 8.2.2 in EC5. Zero contribution from rope effect will be considered.

Min value of equations (a)-(f): \( G_{min} = 7321.2 \text{ kN} \)
Char bolt shear resistance: \( \text{basic} = G_{min} / 1000 = 7.3212 \text{ kN} \)
Design resistance per bolt: \( p_{bl} = k_{mod} / \gamma_{M} \times \text{basic} = 4.5053 \text{ kN} \)

Check splitting capacity of the connection

Design resultant force: \( R_F = 2.1 \text{ kN} \)
Tension force component \( \perp \) grain: \( F_{vEd} = R_F \times \text{SIN}(\alpha) = 2.1 \text{ kN} \)
Loaded edge distance: \( h_e = 150 \text{ mm} \)
Design splitting capacity: \( F_{90Rd} = F_{90Rk} \times k_{mod} / \gamma_{M} = 15.827 \text{ kN} \)
<table>
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<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max resultant force</td>
<td>Angle to grain</td>
</tr>
<tr>
<td>2.1 kN</td>
<td>90 degrees</td>
</tr>
<tr>
<td>Design capacity per bolt</td>
<td>4.5053 kN</td>
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</tbody>
</table>
Location: Timber splice plate type 4

Timber to timber joint with timber splice plate adopted

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<th>Timber joist</th>
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<tbody>
<tr>
<td>Timber splice</td>
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Plan view on splice

<table>
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<tr>
<th>ed</th>
<th>el</th>
<th>gp</th>
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Three bolt symmetrical joint with in line splice plate to EC5

Design shear to be resisted $F_d' = 0.7$ kN
Design moment to be resisted $M_d = 0.35$ kNm
Depth of timber section $h = 225$ mm
Thickness of member 1 $t_1 = 76$ mm
Timber strength class C22.
Timber characteristic density $p_{k1} = 340$ kg/m$^3$
Thickness of timber splice plate $t_2 = 76$ mm
Timber strength class C22.
Timber characteristic density $p_{k2} = 340$ kg/m$^3$
Gap distance between joists $g_{p} = 50$ mm
Fixing diameter (8 mm to 30 mm) $b_d = 12$ mm
Depth factor (Clause 3.2) $k_h = 1.0$
Actual edge distance to bolt ctr $e_d = 48$ mm
Actual end distance $e_l = 90$ mm
Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.
Char fastener yield moment $M_{yRk} = 0.3 \times 400 \times b_d^2 \times 2.6 = 76745$ Nmm
Char embedment strength $f_{hok1} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k1} = 24.534$
Embedding strength (Timb 1) $f_{h1k} = f_{hok1} / \text{denom1} = 24.439$ N/mm$^2$
Char embedment strength $f_{hok2} = 0.082 \times (1 - 0.01 \times b_d) \times p_{k2} = 24.534$
Embedding strength (Timb 2) $f_{h2k} = f_{hok2} / \text{denom2} = 24.439$ N/mm$^2$
Factor $\beta = f_{h2k} / f_{h1k} = 1$
Min value of equations (a)-(f) $G_{\text{min}} = 7715.7$
Char bolt shear resistance $\text{basic} = G_{\text{min}} / 1000 = 7.7157$ kN
Design resistance per bolt $p_{bl} = k_{mod} / \gamma M \times \text{basic} = 4.7481$ kN
Load-sharing modification factor \( k_{sys} = 1.1 \)
Design bending strength \( f_{md} = k_{h} \times k_{sys} \times f_{mk} \times k_{mod} / \gamma_{M} = 14.892 \text{ N/mm}^2 \)
Design shear strength \( f_{vd} = k_{sys} \times f_{vk} \times k_{mod} / \gamma_{M} = 2.5723 \text{ N/mm}^2 \)

Check splitting capacity of the connection

Design resultant force \( RF = 2.7232 \text{ kN} \)
Tension force component \( \perp \) grain \( F_{vEd} = RF \times \sin(\alpha) = 0.23333 \text{ kN} \)
Loaded edge distance \( h_e = 175 \text{ mm} \)
Design splitting capacity \( F_{90Rd} = F_{90Rk} \times k_{mod} / \gamma_{M} = 18.374 \text{ kN} \)

Bolt summary:

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Max resultant force</th>
<th>2.7232 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Angle to grain</td>
<td>4.9154 degrees</td>
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<tr>
<td>Effect bolt design capacity</td>
<td>4.7481 kN</td>
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Timber splice plate summary:

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<tr>
<th>Combined bend moment</th>
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<tbody>
<tr>
<td>Design bend stress</td>
<td>0.67135 N/mm²</td>
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<tr>
<td>Design bend strength</td>
<td>14.892 N/mm²</td>
</tr>
<tr>
<td>Design shear stress</td>
<td>0.073881 N/mm²</td>
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<tr>
<td>Design shear strength</td>
<td>2.5723 N/mm²</td>
</tr>
</tbody>
</table>
Location: Timber splice plate type 5

Timber to timber joint
with timber splice plate adopted

<table>
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<th>Timber joist</th>
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<tbody>
<tr>
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Plan view on splice

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<tr>
<th>cag</th>
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<th>gp</th>
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<tbody>
<tr>
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</table>

Four bolt symmetrical joint with in line splice plate to EC5

Design shear to be resisted $F_d' = 1.26 \text{ kN}$
Design moment to be resisted $M_d = 0.63 \text{ kNm}$
Depth of timber section $h = 225 \text{ mm}$
Thickness of member 1 $t_1 = 47 \text{ mm}$
Timber strength class C22.
Timber characteristic density $p_k_1 = 340 \text{ kg/m}^3$
Thickness of timber splice plate $t_2 = 47 \text{ mm}$
Timber strength class C22.
Timber characteristic density $p_k_2 = 340 \text{ kg/m}^3$
Gap distance between joists $g_p = 10 \text{ mm}$
Fixing diameter (10 mm or 12 mm) $b_d = 12 \text{ mm}$
Depth factor (Clause 3.2) $k_h = 1.0$
Actual edge distance to bolt ctr $e_d = 65 \text{ mm}$
Actual end distance $e_l = 125 \text{ mm}$
Actual centres of bolts // grain $c_a_g = 100 \text{ mm}$

Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.
Char fastener yield moment $M_{y_R k} = 0.3*400*b_d^2.6 = 76745 \text{ Nmm}$
Fixing spacing // to the grain $\alpha_{p a r} = 84 \text{ mm}$
Char embedment strength $f_{h_k 1} = 0.082*(1-0.01*b_d)*p_k_1 = 24.534$
Embedding strength (Timb 1) $f_{h_k 1} = f_{h_k 1}/denom_1 = 18.534 \text{ N/mm}^2$
Char embedment strength $f_{h_k 2} = 0.082*(1-0.01*b_d)*p_k_2 = 24.534$
Embedding strength (Timb 2) $f_{h_k 2} = f_{h_k 2}/denom_2 = 18.534 \text{ N/mm}^2$
Factor $\beta = f_{h_k 2}/f_{h_k 1} = 1$
Min value of equations (a)-(f) $G_{m i n} = 4329.8$
Char bolt shear resistance $basic = G_{m i n}/1000 = 4.3298 \text{ kN}$
Design resistance per bolt $p_b l = k_{mod}/\gamma_{M}basic = 2.6645 \text{ kN}$

Total No of bolts at joint $n = 4$
Number of bolts per row $n_f p r = 2$
Spacing between bolts // grain $a_l = 100 \text{ mm}$
EC5 effect No fasteners per row $n_e f = n_f p r^0.9*(a_l/(13*b_d))^0.25 = 1.6697$
TRADA effect No of bolts per row $n_e f = (n_f p r)^0.8 = 1.7411$
Effective No of bolts to be adopted \( \text{nef}=1.7411 \)
Effective bolt design capacity \( \text{Fvef}=pbl*\text{nef}/\text{nfpr}=2.3196 \text{ kN} \)

**Toothed-plate connector assembly**

Toothed-plate connectors should comply with BS EN 912 and BS EN 14545.
Connector nominal diameter \( \text{dc}=50 \text{ mm} \)

**Design shear capacity for one toothed-plate connector unit**

- Thickness of metal in connector \( \text{tmet}=0.9 \text{ mm} \)
- Toothed-plate connector height \( \text{hc}=15 \text{ mm} \)
- Material factor for connection \( \text{gamM}=1.3 \)
- Design capacity of one TPC unit
  \[ \text{FvRd}=\text{interf}*\text{kmod}*[\text{FvRk}/(1000*\text{gamM})]=3.8044 \text{ kN} \]

**Combined design resistance per fixing**

Combined design resistance \( \text{pbl}=pbl1+pbl2=6.124 \text{ kN} \)

**Check splitting capacity of the connection**

- Design resultant force \( \text{RF}=2.5215 \text{ kN} \)
- Tension force component \( \text{FvEd}=\text{RF}*[\text{SIN}(\text{alpha})]=1.9707 \text{ kN} \)
- Loaded edge distance \( \text{he}=175 \text{ mm} \)
- Design splitting capacity \( \text{F90Rd}=\text{F90Rk}*\text{kmod}/\text{gamM}=11.363 \text{ kN} \)

**Minimum spacings for toothed-plate connectors - to EC5 Table 8.8**

- Spacing within one row parallel to grain:
  \[ a1=(1.2+0.3*\text{COS}(\text{alpha}))*\text{dc}=69.357 \text{ mm} \]
- Spacing between connector rows perpendicular to grain:
  \[ a2=1.2*\text{dc}=60 \text{ mm} \]
- End distance (loaded) \( a3t=1.5*\text{dc}=75 \text{ mm} \)
- End distance (unloaded) \( a3c=(0.9+0.6*\text{SIN}(\text{alpha}))*\text{dc}=68.447 \text{ mm} \)
- Edge distance (loaded) \( a4t=(0.6+0.2*\text{SIN}(\text{alpha}))*\text{dc}=37.816 \text{ mm} \)
- Edge distance (unloaded) \( a4c=0.6*\text{dc}=30 \text{ mm} \)

**Joint slip check - to EC5 Section 7**

- Stiffness of joint final condition at the SLS:
  \( \text{Kserfj}=\text{Kserj}/(1+2*\text{kdef})=2897.7 \text{ N} \)
- Final deflection of the joint at SLS:
  \( \text{utser}=\text{Fdsls}/\text{Kserfj}+2=2.3041 \text{ mm} \)

Bolt/toothed-plate connector summary:

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max resultant force</td>
<td>2.5215 \text{ kN}</td>
</tr>
<tr>
<td>Angle to grain</td>
<td>51.405 degrees</td>
</tr>
<tr>
<td>Contribution from bolt</td>
<td>2.3196 \text{ kN}</td>
</tr>
<tr>
<td>Contribution from connector</td>
<td>3.8044 \text{ kN}</td>
</tr>
<tr>
<td>Combined design resistance</td>
<td>6.124 \text{ kN}</td>
</tr>
</tbody>
</table>

Toothed-plate connectors of nominal diameter 50 mm need to be provided with each 12 mm diameter bolt. Toothed-plate connectors should comply with BS EN 912 and BS EN 14545.
**Location:** Timber splice plate type 6

Timber to timber joint with timber splice plate adopted

<table>
<thead>
<tr>
<th>Timber joist</th>
<th>Timber joist</th>
<th>Timber splice</th>
</tr>
</thead>
</table>

Plan view on splice

X

<table>
<thead>
<tr>
<th>ed</th>
<th>cag</th>
<th>el</th>
<th>gp</th>
</tr>
</thead>
</table>

Five bolt symmetrical joint with in line splice plate to EC5

Design shear to be resisted $F_d'=3.15\, \text{kN}$

Design moment to be resisted $M_d=1.12\, \text{kNm}$

Depth of timber section $h=300\, \text{mm}$

Thickness of member 1 $t_1=50\, \text{mm}$

Timber strength class C22.

Timber characteristic density $p_{k1}=340\, \text{kg/m}^3$

Thickness of timber splice plate $t_2=50\, \text{mm}$

Timber strength class C22.

Timber characteristic density $p_{k2}=340\, \text{kg/m}^3$

Gap distance between joists $g_p=0\, \text{mm}$

Fixing diameter (8 mm to 30 mm) $b_d=16\, \text{mm}$

Depth factor (Clause 3.2) $k_h=1.0$

Actual edge distance to bolt ctr $e_d=65\, \text{mm}$

Actual end distance $e_l=125\, \text{mm}$

Actual centres of bolts // grain $c_a=g=100\, \text{mm}$

Basic loads in kN will be derived from the expressions in Section 8 of Eurocode 5.

Char fastener yield moment $M_{yRk}=0.3*400*b_d^2.6=162141\, \text{Nm}$

Fixing spacing // to the grain $\alpha_{\text{par}}=100\, \text{mm}$

Char embedment strength $f_{hok1}=0.082*(1-0.01*bd)*p_{k1}=23.419$

Embedding strength (Timb 1) $f_{h1k}=f_{hok1}/\text{denom}=18.795\, \text{N/mm}^2$

Char embedment strength $f_{hok2}=0.082*(1-0.01*bd)*p_{k2}=23.419$

Embedding strength (Timb 2) $f_{h2k}=f_{hok2}/\text{denom}=18.795\, \text{N/mm}^2$

Factor $\beta=f_{h2k}/f_{h1k}=1$

Min value of equations (a)-(f) $G_{\text{min}}=6228.3$

Char bolt shear resistance $\text{basic}=G_{\text{min}}/1000=6.2283\, \text{kN}$

Design resistance per bolt $p_{bl}=\text{kmod}/\text{gamM}^*\text{basic}=3.8328\, \text{kN}$

Total No of bolts at joint $n=5$

Number of bolts per row $n_{fpr}=2$

Spacing between bolts // grain $a_1=100\, \text{mm}$

EC5 effect No fasteners per row $n_{ef}=n_{fpr}^0.9*(a_1/(13*b_d))^{0.25}=1.5539$
TRADA effect No of bolts per row  nef=(nfpr)^0.8=1.7411
Effective No of bolts to be adopted  nef=1.7411
Effective bolt design capacity  Fvef=pbl*nef/nfpr=3.3366 kN

Load-sharing modification factor  ksys=1.1
Design bending strength  fmd=kh*ksys*fmk*kmod/gamM=14.892 N/mm²
Design shear strength  fvd=ksys*fvk*kmod/gamM=2.5723 N/mm²

Check splitting capacity of the connection

Design resultant force  RF=3.2051 kN
Tension force component  \( FvEd = RF \cdot \sin(\alpha) = 2.0696 \ \text{kN} \)
Loaded edge distance  he=250 mm
Design splitting capacity  F90Rd=F90Rk*kmod/gamM=16.684 kN

Bolt summary:
- **DESIGN**
  - Max resultant force  3.2051 kN
  - Angle to grain  40.22 degrees
  - Effect bolt design capacity  3.8328 kN

- **SUMMARY**
  - Combined bend moment  1.6713 kNm
  - Design bend stress  2.2283 N/mm²
  - Design bend strength  14.892 N/mm²
  - Design shear stress  0.35526 N/mm²
  - Design shear strength  2.5723 N/mm²
Location: Test case 1

Toothed-plate connector joint design to BS5268-2:2002

Toothed-plate connectors should comply with BS 1579. Bolts should comply with BS EN 20898-2 and washers with BS4320.

Axial load at joint \( F = 2.15 \) kN

Number of connectors in joint \( ncj = 2 \)

Angle between grain and load \( \alpha' = 45^\circ \)

Thickness of thinner member \( t = 22 \) mm

Strength class C14 to Table 8.

Mod factor for duration of load \( K58 = 1.25 \)

Angle of connector axis to grain \( acag = 30^\circ \)

Actual spacing of connectors \( aspace = 56 \) mm

End grain is loaded by connector.

Actual loaded end distance to Figure 7 BS5268-2:2002 \( alend = 60 \) mm

Actual edge distance to figure 7 BS5268-2:2002 \( aedge = 30 \) mm

Mod factor for No. of connectors \( K61 = 0.97 \)

Permissible connector load \( pcl = \text{basic} \times K58 \times K59 \times K60 \times K61 = 1.6643 \) kN

Actual load per connector \( Fa = F / (ncj \times interf) = 1.075 \) kN

Since \( pcl > Fa \ (1.6643 \) kN > 1.075 \) kN \) then connector capacity exceeds the applied load and the joint is satisfactory.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual load per connector</td>
<td>1.075 ) kN</td>
</tr>
<tr>
<td>Permissible connector load</td>
<td>1.6643 ) kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of connectors in joint</td>
<td>2</td>
</tr>
<tr>
<td>Number of bolts in joint</td>
<td>2</td>
</tr>
<tr>
<td>Provide toothed-plate connector on one side of the timber only</td>
<td></td>
</tr>
</tbody>
</table>
**Location:** Test case 2

**Toothed-plate connector joint design to BS5268-2:2002**

Toothed-plate connectors should comply with BS 1579. Bolts should comply with BS EN 20898-2 and washers with BS4320.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load at joint</td>
<td>F=1.55 kN</td>
</tr>
<tr>
<td>Number of connectors in joint</td>
<td>ncj=2</td>
</tr>
<tr>
<td>Angle between grain and load</td>
<td>alpha'=90°</td>
</tr>
<tr>
<td>Thickness of thinner member</td>
<td>t=60 mm</td>
</tr>
<tr>
<td>Strength class C16 to Table 8</td>
<td></td>
</tr>
<tr>
<td>Mod factor for duration of load</td>
<td>K58=1</td>
</tr>
<tr>
<td>Angle of connector axis to grain</td>
<td>acag=45°</td>
</tr>
<tr>
<td>Actual spacing of connectors</td>
<td>aspace=70 mm</td>
</tr>
<tr>
<td>End grain is loaded by connector</td>
<td></td>
</tr>
<tr>
<td>Actual loaded end distance to Figure 7 BS5268-2:2002</td>
<td>alend=50 mm</td>
</tr>
<tr>
<td>Actual edge distance to figure 7 BS5268-2:2002</td>
<td>aedge=35 mm</td>
</tr>
<tr>
<td>Mod factor for No. of connectors</td>
<td>K61=0.97</td>
</tr>
<tr>
<td>Permissible connector load</td>
<td>pcl=basic<em>K58</em>K59<em>K60</em>K61=1.1935 kN</td>
</tr>
<tr>
<td>Actual load per connector</td>
<td>Fa=F/(ncj*interf)=0.3875 kN</td>
</tr>
</tbody>
</table>

Since \( pcl > Fa \) (1.1935 kN > 0.3875 kN) then connector capacity exceeds the applied load and the joint is satisfactory.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual load per connector</td>
<td>0.3875 kN</td>
</tr>
<tr>
<td>Permissible connector load</td>
<td>1.1935 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of connectors in joint</td>
<td>2</td>
</tr>
<tr>
<td>Number of bolts in joint</td>
<td>2</td>
</tr>
<tr>
<td>Provide toothed-plate connectors on both sides of timber &amp; on same bolt</td>
<td></td>
</tr>
</tbody>
</table>
Location: Test case 3

Toothed-plate connector joint design to BS5268-2:2002

Toothed-plate connectors should comply with BS 1579. Bolts should comply with BS EN 20898-2 and washers with BS4320.

Axial load at joint \( F = 3.55 \text{ kN} \)
Number of connectors in joint \( n_{cj} = 1 \)
Angle between grain and load \( \alpha' = 0^\circ \)
Thickness of thinner member \( t = 29 \text{ mm} \)
Strength class C24 to Table 8.
Mod factor for duration of load \( K_5 = 1.25 \)
End grain is not loaded by connector.

Actual unloaded end distance to Figure 7 BS5268-2:2002 \( a_{uend} = 44 \text{ mm} \)
Actual edge distance to figure 7 BS5268-2:2002 \( a_{edge} = 38 \text{ mm} \)
Permissible connector load \( p_{cl} = \text{basic} \times K_5 \times K_9 \times K_60 \times K_61 = 5.5 \text{ kN} \)
Actual load per connector \( F_a = F / (n_{cj} \times \text{interf}) = 3.55 \text{ kN} \)

Since \( p_{cl} > F_a \) (5.5 kN > 3.55 kN) then connector capacity exceeds the applied load and the joint is satisfactory.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual load per connector</td>
<td>3.55 kN</td>
</tr>
<tr>
<td>Permissible connector load</td>
<td>5.5 kN</td>
</tr>
<tr>
<td>Number of connectors in joint</td>
<td>1</td>
</tr>
<tr>
<td>Number of bolts in joint</td>
<td>1</td>
</tr>
<tr>
<td>Provide toothed-plate connector on one side of the timber only</td>
<td></td>
</tr>
</tbody>
</table>
Toothed-plate connector joint design to BS5268-2:2002

Toothed-plate connectors should comply with BS 1579. Bolts should comply with BS EN 20898-2 and washers with BS4320.

Axial load at joint \( F=5.25 \text{ kN} \)

Number of connectors in joint \( ncj=2 \)

Angle between grain and load \( \alpha'=45^\circ \)

Thickness of thinner member \( t=63 \text{ mm} \)

Strength class C40 to Table 8.

Mod factor for duration of load \( K58=1 \)

Angle of connector axis to grain \( acag=0^\circ \)

Actual spacing of connectors \( a_{\text{space}}=95 \text{ mm} \)

End grain is loaded by connector.

Actual loaded end distance to Figure 7 BS5268-2:2002 \( a_{\text{end}}=80 \text{ mm} \)

Actual edge distance to figure 7 BS5268-2:2002 \( a_{\text{edge}}=45 \text{ mm} \)

Mod factor for No. of connectors \( K61=0.97 \)

Permissible connector load \( pcl=\text{basic}\times K58\times K59\times K60\times K61=2.8165 \text{ kN} \)

Actual load per connector \( F_a=F/(ncj*\text{interf})=1.3125 \text{ kN} \)

Since \( pcl > F_a \) (2.8165 kN > 1.3125 kN) then connector capacity exceeds the applied load and the joint is satisfactory.

| DESIGN | Permissible connector load | 2.8165 kN |
| SUMMARY | Number of connectors in joint | 2 |
| | Number of bolts in joint | 2 |
| | Provide toothed-plate connectors on both sides of timber & on same bolt | |
Location: Test case 5

Toothed-plate connector joint design to BS5268-2:2002

Toothed-plate connectors should comply with BS 1579. Bolts should comply with BS EN 20898-2 and washers with BS4320.

Axial load at joint \( F = 4.95 \) kN

Number of connectors in joint \( n_{cj} = 5 \)

Angle between grain and load \( \alpha' = 37.5^\circ \)

Thickness of thinner member \( t = 47 \) mm

Strength class C22 to Table 8.

Mod factor for duration of load \( K_{58} = 1 \)

Angle of connector axis to grain \( \alpha_{cag} = 12^\circ \)

Actual spacing of connectors \( a_{space} = 62.533 \) mm

End grain is loaded by connector.

Actual loaded end distance to Figure 7 BS5268-2:2002 \( a_{end} = 60 \) mm

Actual edge distance to figure 7 BS5268-2:2002 \( a_{edge} = 35 \) mm

Mod factor for No. of connectors \( K_{61} = 0.88 \)

Permissible connector load \( p_{cl} = \text{basic} \times K_{58} \times K_{59} \times K_{60} \times K_{61} = 0.99632 \) kN

Actual load per connector \( F_a = F / (n_{cj} \times \text{interf}) = 0.99 \) kN

Since \( p_{cl} > F_a \) (0.99632 kN > 0.99 kN) then connector capacity exceeds the applied load and the joint is satisfactory.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual load per connector</td>
<td>0.99 kN</td>
</tr>
<tr>
<td>Permissible connector load</td>
<td>0.99632 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of connectors in joint</td>
<td>5</td>
</tr>
<tr>
<td>Number of bolts in joint</td>
<td>5</td>
</tr>
</tbody>
</table>

Provide toothed-plate connector on one side of the timber only
Location: Test case 6

Toothed-plate connector joint design to BS5268-2:2002

Toothed-plate connectors should comply with BS 1579. Bolts should comply with BS EN 20898-2 and washers with BS4320.

Axial load at joint \( F = 3.2 \text{ kN} \)

Number of connectors in joint \( ncj = 1 \)

Angle between grain and load \( \alpha' = 90^\circ \)

Thickness of thinner member \( t = 50 \text{ mm} \)

Strength class C16 to Table 8.

Mod factor for duration of load \( K58 = 1 \)

End grain is not loaded by connector.

Actual unloaded end distance to Figure 7 BS5268-2:2002 \( auend = 102 \text{ mm} \)

Actual edge distance to figure 7 BS5268-2:2002 \( aedge = 44 \text{ mm} \)

Permissible connector load \( pcl = \text{basic} * K58 * K59 * K60 * K61 = 3.13 \text{ kN} \)

Actual load per connector \( Fa = F / (ncj*interf) = 1.6 \text{ kN} \)

Since \( pcl > Fa \ (3.13 \text{ kN} > 1.6 \text{ kN}) \) then connector capacity exceeds the applied load and the joint is satisfactory.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual load per connector</td>
<td>1.6 \text{ kN}</td>
</tr>
<tr>
<td>Permissible connector load</td>
<td>3.13 \text{ kN}</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of connectors in joint</td>
<td>1</td>
</tr>
<tr>
<td>Number of bolts in joint</td>
<td>1</td>
</tr>
</tbody>
</table>
| Provide toothed-plate connectors on both sides of timber & on same bolt | }
Location: Test case 1

Toothed-plate connector joint design to EC5

Toothed-plate connectors should comply with BS EN 912 & BS EN 14545 and be used in conjunction with connector bolts having a min tensile strength of 400 N/mm². Bolts should comply with BS EN 20898-2. Washers should have a minimum thickness of 0.3d and a minimum diameter or side length of 3d, where d is the bolt diameter. Washers need to be used under both the head and nut. Bolt holes in timber should have a diameter not more than 1.0 mm larger than the bolt diameter.

Characteristic permanent load \( G_k = 1.075 \) kN
Characteristic variable load \( Q_k = 1.075 \) kN
Design axial load at joint \( F_d = 1.35 \times G_k + 1.5 \times Q_k = 3.0638 \) kN
Number of bolts to be used \( n_bolt = 2 \)
Angle between grain and load \( \alpha' = 45^\circ \)
Connector nominal diameter \( d_c = 38 \) mm
Timber strength class C14.
Thickness of thinner member \( t_1 = 22 \) mm
Thickness of other member \( t_2 = 22 \) mm
Thickness of metal in connector \( t_{met} = 0.9 \) mm
Height of toothed-plate connector \( h_c = 12 \) mm
Material factor for connection \( \gamma_M = 1.3 \)
Design capacity of joint \( F_{vRd} = \frac{\text{interf} \times n_{cj} \times k_{mod} \times F_{vRk}}{1000 \times \gamma_M} = 3.835 \) kN
Design load at joint \( F_d = 3.0638 \) kN

Since \( F_{vRd} \geq F_d (3.835 \text{ kN} \geq 3.0638 \text{ kN}) \) design capacity of joint exceeds the design load at joint hence connection is satisfactory.

Minimum toothed-plate connector spacings - EC5 Table 8.8

Spacing within one row parallel to grain:
\( a_1 = (1.2 + 0.3 \times \cos(\alpha)) \times d_c = 53.661 \) mm

Spacing between connector rows perpendicular to grain:
\( a_2 = 1.2 \times d_c = 45.6 \) mm

End distance (loaded)
\( a_3t = 1.5 \times d_c = 57 \) mm
End distance (unloaded)
\( a_3c = (0.9 + 0.6 \times \sin(\alpha)) \times d_c = 50.322 \) mm
Edge distance (loaded)
\( a_4t = (0.6 + 0.2 \times \sin(\alpha)) \times d_c = 28.174 \) mm
Edge distance (unloaded)
\( a_4c = 0.6 \times d_c = 22.8 \) mm
Reduction factors for distance \( a_1 \) \( k_{a1} = 0.75 \)
Reduction factors for distance \( a_2 \) \( k_{a2} = 0.75 \)

Joint slip check - EC5 Section 7

Stiffness of joint final condition at the SLS:
\( K_{serfj} = K_{serj} / (1 + 2 \times k_{def}) = 3756.8 \) N
Final deflection of the joint at SLS:
\( u_{tser} = F_{dsls} / (K_{serfj} + 2) = 2.5723 \) mm
The connector bolts hold the members together and are assumed not to contribute to the load-carrying capacity of the connection. Use proformas 273 and 274 to evaluate the bolt contribution.
Location: Test case 2

Toothed-plate connector joint design to EC5

Toothed-plate connectors should comply with BS EN 912 & BS EN 14545 and be used in conjunction with connector bolts having a min tensile strength of 400 N/mm². Bolts should comply with BS EN 20898-2. Washers should have a minimum thickness of 0.3d and a minimum diameter or side length of 3d, where d is the bolt diameter. Washers need to be used under both the head and nut. Bolt holes in timber should have a diameter not more than 1.0 mm larger than the bolt diameter.

Characteristic permanent load \( G_k = 0.775 \) kN
Characteristic variable load \( Q_k = 0.775 \) kN
Design axial load at joint \( F_d = 1.35*G_k + 1.5*Q_k = 2.2088 \) kN
Number of bolts to be used \( n_{bolt} = 2 \)
Angle between grain and load \( \alpha' = 90^\circ \)
Connector nominal diameter \( d_c = 50 \) mm
Timber strength class C16.
Thickness of thinner member \( t_1 = 60 \) mm
Thickness of other member \( t_2 = 60 \) mm
Thickness of metal in connector \( t_{met} = 0.9 \) mm
Height of toothed-plate connector \( h_c = 15 \) mm
Material factor for connection \( \gamma_M = 1.3 \)
Design capacity of joint \( F_vR_d = \text{interf} * n_{cj} * \kappa_{mod} * F_vR_k / (1000 * \gamma_M) = 5.6367 \) kN
Design load at joint \( F_d = 2.2088 \) kN

Since \( F_vR_d \geq F_d \) \( (5.6367 \text{ kN} \geq 2.2088 \text{ kN}) \) design capacity of joint exceeds the design load at joint hence connection is satisfactory.

Minimum toothed-plate connector spacings - EC5 Table 8.8

Spacing within one row parallel to grain:
\[ a_1 = (1.2 + 0.3 \cos(\alpha')) \times d_c = 60 \text{ mm} \]

Spacing between connector rows perpendicular to grain:
\[ a_2 = 1.2 \times d_c = 60 \text{ mm} \]

End distance (loaded):
\[ a_{3t} = 1.5 \times d_c = 75 \text{ mm} \]

End distance (unloaded):
\[ a_{3c} = (0.9 + 0.6 \sin(\alpha')) \times d_c = 75 \text{ mm} \]

Edge distance (loaded):
\[ a_{4t} = (0.6 + 0.2 \sin(\alpha')) \times d_c = 40 \text{ mm} \]

Edge distance (unloaded):
\[ a_{4c} = 0.6 \times d_c = 30 \text{ mm} \]

Joint slip check - EC5 Section 7

Stiffness of joint final condition at the SLS:
\[ K_{serfj} = K_{serj} / (1 + 2 \kappa_{def}) = 2325 \text{ N} \]

Final deflection of the joint at SLS:
\[ u_{ser} = F_d \times s_l / K_{serfj} + 2 = 2.6667 \text{ mm} \]
The connector bolts hold the members together and are assumed not to contribute to the load-carrying capacity of the connection. Use proformas 273 and 274 to evaluate the bolt contribution.
Location: Test case 3

Toothed-plate connector joint design to EC5

Toothed-plate connectors should comply with BS EN 912 & BS EN 14545 and be used in conjunction with connector bolts having a min tensile strength of 400 N/mm². Bolts should comply with BS EN 20898-2. Washers should have a minimum thickness of 0.3d and a minimum diameter or side length of 3d, where d is the bolt diameter. Washers need to be used under both the head and nut. Bolt holes in timber should have a diameter not more than 1.0 mm larger than the bolt diameter.

Characteristic permanent load \( G_k = 3 \text{ kN} \)

Characteristic variable load \( Q_k = 0.55 \text{ kN} \)

Design axial load at joint \( F_d = 1.35 \times G_k + 1.5 \times Q_k = 4.875 \text{ kN} \)

Number of bolts to be used \( n_{bolt} = 1 \)

Angle between grain and load \( \alpha' = 0^\circ \)

Connector nominal diameter \( d_c = 63 \text{ mm} \)

Timber strength class C24.

Thickness of thinner member \( t_1 = 29 \text{ mm} \)

Thickness of other member \( t_2 = 29 \text{ mm} \)

Thickness of metal in connector \( t_{met} = 1.2 \text{ mm} \)

Height of toothed-plate connector \( h_c = 10 \text{ mm} \)

Material factor for connection \( \gamma_{M} = 1.3 \)

Design capacity of joint \( F_{V,Rd} = \text{interf} \times n_{cj} \times k_{mod} \times F_{V,Rk} / (1000 \times \gamma_{M}) = 5.539 \text{ kN} \)

Design load at joint \( F_d = 4.875 \text{ kN} \)

Since \( F_{V,Rd} \geq F_d \) \( (5.539 \text{ kN} \geq 4.875 \text{ kN}) \) design capacity of joint exceeds the design load at joint hence connection is satisfactory.

Minimum toothed-plate connector spacings - EC5 Table 8.8

Spacing within one row parallel to grain:
\[ a_1 = (1.2 + 0.3 \times \cos(\alpha)) \times d_c = 94.5 \text{ mm} \]

Spacing between connector rows perpendicular to grain:
\[ a_2 = 1.2 \times d_c = 75.6 \text{ mm} \]

End distance (loaded) \( a_{3t} = 1.5 \times d_c = 94.5 \text{ mm} \)

End distance (unloaded) \( a_{3c} = 1.2 \times d_c = 75.6 \text{ mm} \)

Edge distance (loaded) \( a_{4t} = (0.6 + 0.2 \times \sin(\alpha)) \times d_c = 37.8 \text{ mm} \)

Edge distance (unloaded) \( a_{4c} = 0.6 \times d_c = 37.8 \text{ mm} \)

Joint slip check - EC5 Section 7

Stiffness of joint final condition at the SLS:
\[ K_{serfj} = K_{serj} / (1 + 2 \times k_{def}) = 3180.3 \text{ N} \]

Final deflection of the joint at SLS:
\[ u_{ser} = F_{dsls} / K_{serfj} + 2 = 3.1163 \text{ mm} \]
### DESIGN
- Design load at joint: 4.875 kN
- Design capacity of joint: 5.539 kN

### SUMMARY
- Number of connectors in joint: 1
- Number of bolts in joint: 1
- Toothed-plate connector type: C6
- Provide a toothed-plate connector on one side of timber only

The connector bolts hold the members together and are assumed not to contribute to the load-carrying capacity of the connection. Use proformas 273 and 274 to evaluate the bolt contribution.
**Location: Test case 4**

### Toothed-plate connector joint design to EC5

Toothed-plate connectors should comply with BS EN 912 & BS EN 14545 and be used in conjunction with connector bolts having a min tensile strength of 400 N/mm². Bolts should comply with BS EN 20898-2. Washers should have a minimum thickness of 0.3d and a minimum diameter or side length of 3d, where d is the bolt diameter. Washers need to be used under both the head and nut. Bolt holes in timber should have a diameter not more than 1.0 mm larger than the bolt diameter.

- **Characteristic permanent load** $G_k = 2.625$ kN
- **Characteristic variable load** $Q_k = 2.625$ kN
- **Design axial load at joint** $F_d = 1.35 \times G_k + 1.5 \times Q_k = 7.4813$ kN
- **Number of bolts to be used** $n_{bolt} = 2$
- **Angle between grain and load** $\alpha' = 45^\circ$
- **Connector nominal diameter** $d_c = 63$ mm
- **Timber strength class** C40
- **Thickness of thinner member** $t_1 = 63$ mm
- **Thickness of other member** $t_2 = 63$ mm
- **Thickness of metal in connector** $t_{met} = 1.2$ mm
- **Height of toothed-plate connector** $h_c = 8.7$ mm
- **Material factor for connection** $\gamma_{M} = 1.3$
- **Design capacity of joint** $F_v R_d = \text{interf} \times n_{cj} \times k_{mod} \times F_v R_k / (1000 \times \gamma_{M}) = 7.737$ kN
- **Design load at joint** $F_d = 7.4813$ kN

Since $F_v R_d \geq F_d$ ($7.737$ kN $\geq 7.4813$ kN ) design capacity of joint exceeds the design load at joint hence connection is satisfactory.

### Minimum toothed-plate connector spacings - EC5 Table 8.8

- **Spacing within one row parallel to grain:**
  \[ a_1 = (1.2 + 0.3 \times \cos(\alpha')) \times d_c = 88.964 \text{ mm} \]
- **Spacing between connector rows perpendicular to grain:**
  \[ a_2 = 1.2 \times d_c = 75.6 \text{ mm} \]
- **End distance (loaded)**
  \[ a_{3t} = 1.5 \times d_c = 94.5 \text{ mm} \]
- **End distance (unloaded)**
  \[ a_{3c} = (0.9 + 0.6 \times \sin(\alpha')) \times d_c = 83.429 \text{ mm} \]
- **Edge distance (loaded)**
  \[ a_{4t} = (0.6 + 0.2 \times \sin(\alpha')) \times d_c = 46.71 \text{ mm} \]
- **Edge distance (unloaded)**
  \[ a_{4c} = 0.6 \times d_c = 37.8 \text{ mm} \]

### Joint slip check - EC5 Section 7

- **Stiffness of joint final condition at the SLS:**
  \[ K_{serfj} = K_{serj} / (1 + 2 \times k_{def}) = 3780 \text{ N} \]
- **Final deflection of the joint at SLS:**
  \[ u_{ser} = F_{d_{sls}} / K_{serfj} + 2 = 3.3889 \text{ mm} \]
The connector bolts hold the members together and are assumed not to contribute to the load-carrying capacity of the connection. Use proformas 273 and 274 to evaluate the bolt contribution.
Toothed-plate connector joint design to EC5

Toothed-plate connectors should comply with BS EN 912 & BS EN 14545 and be used in conjunction with connector bolts having a min tensile strength of 400 N/mm². Bolts should comply with BS EN 20898-2. Washers should have a minimum thickness of 0.3d and a minimum diameter or side length of 3d, where d is the bolt diameter. Washers need to be used under both the head and nut. Bolt holes in timber should have a diameter not more than 1.0 mm larger than the bolt diameter.

Characteristic permanent load \( G_k = 2.475 \, \text{kN} \)
Characteristic variable load \( Q_k = 2.475 \, \text{kN} \)
Design axial load at joint \( F_d = 1.35 \times G_k + 1.5 \times Q_k = 7.0538 \, \text{kN} \)
Number of bolts to be used \( n_{bolt} = 5 \)
Angle between grain and load \( \alpha' = 37.5^\circ \)
Connector side length \( d_c = 38 \, \text{mm} \)
Timber strength class C22.
Thickness of thinner member \( t_1 = 47 \, \text{mm} \)
Thickness of other member \( t_2 = 47 \, \text{mm} \)
Thickness of metal in connector \( t_{met} = 1 \, \text{mm} \)
Height of toothed-plate connector \( h_c = 11 \, \text{mm} \)
Material factor for connection \( \gamma_M = 1.3 \)
Design capacity of joint \( F_{vRd} = \text{interf} \times n_{cj} \times k_{mod} \times F_{vRk} / (1000 \times \gamma_M) = 7.8769 \, \text{kN} \)
Design load at joint \( F_d = 7.0538 \, \text{kN} \)

Since \( F_{vRd} \geq F_d \ (7.8769 \, \text{kN} \geq 7.0538 \, \text{kN}) \) design capacity of joint exceeds the design load at joint hence connection is satisfactory.

Minimum toothed-plate connector spacings - EC5 Table 8.8

Spacing within one row parallel to grain:
\[ a_1 = (1.2 + 0.3 \times \cos(\alpha')) \times d_c = 54.644 \, \text{mm} \]
Spacing between connector rows perpendicular to grain:
\[ a_2 = 1.2 \times d_c = 45.6 \, \text{mm} \]
End distance (loaded)
\[ a_{3t} = 1.5 \times d_c = 57 \, \text{mm} \]
End distance (unloaded)
\[ a_{3c} = (0.9 + 0.6 \times \sin(\alpha')) \times d_c = 48.08 \, \text{mm} \]
Edge distance (loaded)
\[ a_{4t} = (0.6 + 0.2 \times \sin(\alpha')) \times d_c = 27.427 \, \text{mm} \]
Edge distance (unloaded)
\[ a_{4c} = 0.6 \times d_c = 22.8 \, \text{mm} \]

Joint slip check - EC5 Section 7

Stiffness of joint final condition at the SLS:
\[ K_{serfj} = K_{serj} / (1 + 2 \times k_{def}) = 4845 \, \text{N} \]
Final deflection of the joint at SLS:
\[ u_{ser} = F_{d_{sls}} / K_{serfj} + 2 = 3.0217 \, \text{mm} \]
Design load at joint: 7.0538 kN
Design capacity of joint: 7.8769 kN

**SUMMARY**
- Number of connectors in joint: 5
- Number of bolts in joint: 5
- Toothed-plate connector type: C8
- Provide a toothed-plate connector on one side of timber only

The connector bolts hold the members together and are assumed not to contribute to the load-carrying capacity of the connection. Use proformas 273 and 274 to evaluate the bolt contribution.
Location: Test case 6

Toothed-plate connector joint design to EC5

Toothed-plate connectors should comply with BS EN 912 & BS EN 14545 and be used in conjunction with connector bolts having a min tensile strength of 400 N/mm². Bolts should comply with BS EN 20898-2. Washers should have a minimum thickness of 0.3d and a minimum diameter or side length of 3d, where d is the bolt diameter. Washers need to be used under both the head and nut. Bolt holes in timber should have a diameter not more than 1.0 mm larger than the bolt diameter.

Characteristic permanent load \( G_k = 1.6 \text{ kN} \)

Characteristic variable load \( Q_k = 1.6 \text{ kN} \)

Design axial load at joint \( F_d = 1.35 \times G_k + 1.5 \times Q_k = 4.56 \text{ kN} \)

Number of bolts to be used \( n_{bolt} = 1 \)

Angle between grain and load \( \alpha' = 90^\circ \)

Connector nominal diameter \( d_c = 75 \text{ mm} \)

Timber strength class C16.

Outer member thickness \( t_1 = 50 \text{ mm} \)

Inner member thickness \( t_2 = 50 \text{ mm} \)

Thickness of metal in connector \( t_{met} = 1.25 \text{ mm} \)

Height of toothed-plate connector \( h_c = 10.4 \text{ mm} \)

Material factor for connection \( \gamma_{M} = 1.3 \)

Design capacity of joint \( F_{vRd} = \frac{\text{interf} \times n_{cj} \times k_{mod} \times F_{vRk}}{1000 \times \gamma_{M}} = 10.706 \text{ kN} \)

Design load at joint \( F_d = 4.56 \text{ kN} \)

Since \( F_{vRd} \geq F_d \geq 4.56 \text{ kN} \), design capacity of joint exceeds the design load at joint hence connection is satisfactory.

Minimum toothed-plate connector spacings - EC5 Table 8.8

Spacing within one row parallel to grain: \( a_1 = (1.2 + 0.3 \times \cos(\alpha')) \times d_c = 90 \text{ mm} \)

Spacing between connector rows perpendicular to grain: \( a_2 = 1.2 \times d_c = 90 \text{ mm} \)

End distance (loaded) \( a_3t = 1.5 \times d_c = 112.5 \text{ mm} \)

End distance (unloaded) \( a_3c = (0.9 + 0.6 \times \sin(\alpha')) \times d_c = 112.5 \text{ mm} \)

Edge distance (loaded) \( a_4t = (0.6 + 0.2 \times \sin(\alpha')) \times d_c = 60 \text{ mm} \)

Edge distance (unloaded) \( a_4c = 0.6 \times d_c = 45 \text{ mm} \)

Joint slip check - EC5 Section 7

Stiffness of joint final condition at the SLS: \( K_{serfj} = K_{serj} / (1 + 2 \times k_{def}) = 3963.1 \text{ N} \)

Final deflection of the joint at SLS: \( u_{ser} = F_{dls} / K_{serfj} + 2 = 2.8075 \text{ mm} \)
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
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<td>Design capacity of joint</td>
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<td>1</td>
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<td>1</td>
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<tr>
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<td>C7</td>
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</table>

Provide toothed-plate connectors on both sides of timber & on same bolt

The connector bolts hold the members together and are assumed not to contribute to the load-carrying capacity of the connection. Use proformas 273 and 274 to evaluate the bolt contribution.
**Location: Forces in rafter 1**

- Top of rafter is pinned and restrained against horizontal movement, but unrestrained against vertical movement.
- Bottom of rafter is pinned and restrained against horizontal movement.

**Rafter projection onto horizontal** \( b = 3 \, \text{m} \)

**Rafter projection onto vertical** \( c = 1 \, \text{m} \)

- Elastic modulus \( E = 9 \times 10^6 \, \text{kN/m}^2 \)
- Moment of inertia of rafter \( I = 14.063 \times 10^{-6} \, \text{m}^4 \)
- Slope of roof \( \alpha = \text{ATN}(c/b) = 0.32175 \, \text{radians} \)
  \( \alpha' = \text{DEG}(\alpha) = 18.435 \, \text{degrees} \)
- Length of rafter \( r = \sqrt{b^2 + c^2} = 3.1623 \, \text{m} \)

Loads and reactions are positive in the directions shown.

- Vertical load per unit length \( w_1 = 2 \, \text{kN/m} \)
- Total load on rafter \( W = w_1 \times r = 6.3246 \, \text{kN} \)
- Vertical reaction at 1 (+ve up) \( V_1 = W = 6.3246 \, \text{kN} \)
- Maximum moment (+ve sagging) \( M = W \times \cos(\alpha) \times r / 8 = 2.3717 \, \text{kNm} \)
- Max defln in rafter (+ve is sag) \( D_1 = 5 \times W \times \cos(\alpha) \times r^3 / (384 \times E \times I) = 0.01952 \, \text{m} \)
- Span/deflection ratio \( s' = r / D_1 = 162.01 \)
- Deflection/span ratio \( d' = D_1 / r = 0.0061726 \)
- Horizontal react at 1 (+ve right) \( H_1 = W \times b / (2 \times c) = 9.4868 \, \text{kN} \)
- Horizontal react at 2 (+ve left) \( H_2 = 9.4868 \, \text{kN} \)
- Axial force at 1 (+ve compressn) \( F_1 = V_1 \times \sin(\alpha) + H_1 \times \cos(\alpha) = 11 \, \text{kN} \)
- Shear force at 1 \( S_1 = V_1 \times \cos(\alpha) - H_1 \times \sin(\alpha) = 3 \, \text{kN} \)
Location: Forces in rafter 2

Top of rafter is pinned and restrained against vertical and horizontal movement.

Bottom of rafter is pinned and unrestrained against horizontal movement.

Rafter projection onto horizontal \( b = 3 \text{ m} \)
Rafter projection onto vertical \( c = 1 \text{ m} \)
Elastic modulus \( E = 9E6 \text{ kN/m}^2 \)
Moment of inertia of rafter \( I = 14.063E-6 \text{ m}^4 \)
Slope of roof \( \alpha = \text{ATN}(c/b) = 0.32175 \text{ radians} \)
\( \alpha' = \text{DEG}(\alpha) = 18.435 \text{ degrees} \)
Length of rafter \( r = \text{SQR}(b^2+c^2) = 3.1623 \text{ m} \)

Loads and reactions are positive in the directions shown.

Load is UDL applied vertically per unit length of rafter.

Vertical load per unit length \( w_1 = 2 \text{ kN/m} \)
Total load on rafter \( W = w_1 \times r = 6.3246 \text{ kN} \)
Maximum moment (+ve sagging) \( M = W \times \cos(\alpha) \times r/8 = 2.3717 \text{ kNm} \)
Max defln in rafter (+ve is sag) \( D_1 = 5 \times W \times \cos(\alpha) \times r^3/(384 \times E \times I) = 0.01952 \text{ m} \)
Span/deflection ratio \( s' = r/D_1 = 162.01 \)
Deflection/span ratio \( d' = D_1/r = 0.0061726 \)
Horizontal react at 1 (+ve right) \( H_1 = 0 \text{ kN as joint 1 is on rollers} \)
Horizontal react at 2 (+ve left) \( H_2 = 0 \text{ kN resolving horizontally} \)
Vertical reaction at 1 (+ve up) \( V_1 = W/2 = 3.1623 \text{ kN} \)
Vertical reaction at 2 (+ve up) \( V_2 = 3.1623 \text{ kN} \)
Axial force at 1 (+ve compress.) \( F_1 = V_1 \times \sin(\alpha) = 1 \text{ kN} \)
Shear force at 1 \( S_1 = V_1 \times \cos(\alpha) = 3 \text{ kN} \)
Location: Forces in rafter 3

Top of rafter is pinned and restrained against horizontal movement, but unrestrained against vertical movement.

Bottom of rafter is pinned and restrained against horizontal movement.

Rafter projection onto horizontal \( b = 3 \) m
Rafter projection onto vertical \( c = 1 \) m
Elastic modulus \( E = 9E6 \) kN/m²
Moment of inertia of rafter \( I = 14.063E-6 \) m⁴
Slope of roof \( \alpha = \text{ATN}(c/b) = 0.32175 \) radians
\( \alpha' = \text{DEG}(\alpha) = 18.435 \) degrees
Length of rafter \( r = \sqrt{b^2+c^2} = 3.1623 \) m

Loads and reactions are positive in the directions shown.

Load is UDL applied vertically on plan.

Plan load per unit length \( w_2 = 1 \) kN/m
Total load on rafter \( W = w_2 \cdot b = 3 \) kN
Vertical reaction at 1 (+ve up) \( V_1 = W = 3 \) kN
Maximum moment (+ve sagging) \( M = W \cdot \cos(\alpha) \cdot r / 8 = 1.125 \) kNm
Max defln in rafter (+ve is sag) \( D_1 = 5 \cdot W \cdot \cos(\alpha) \cdot r^3 / (384 \cdot E \cdot I) = 0.0092589 \) m

Span/deflection ratio \( s'd = r / D_1 = 341.54 \)
Deflection/span ratio \( d's = D_1 / r = 0.0029279 \)
Horizontal react at 1 (+ve right) \( H_1 = W \cdot b / (2 \cdot c) = 4.5 \) kN
Horizontal react at 2 (+ve left) \( H_2 = 4.5 \) kN
Axial force at 1 (+ve compress.) \( F_1 = V_1 \cdot \sin(\alpha) + H_1 \cdot \cos(\alpha) = 5.2178 \) kN
Shear force at 1 \( S_1 = V_1 \cdot \cos(\alpha) - H_1 \cdot \sin(\alpha) = 1.423 \) kN
**Location: Forces in rafter 4**

Top of rafter is pinned and restrained against vertical and horizontal movement.

Bottom of rafter is pinned and unrestrained against horizontal movement.

Rafter projection onto horizontal $b = 3\, \text{m}$
Rafter projection onto vertical $c = 1\, \text{m}$
Elastic modulus $E = 9 \times 10^6\, \text{kN/m}^2$
Moment of inertia of rafter $I = 14.063 \times 10^{-6}\, \text{m}^4$
Slope of roof
  - $\alpha = \text{ATN}(c/b) = 0.32175\, \text{radians}$
  - $\alpha' = \text{DEG}(\alpha) = 18.435\, \text{degrees}$
Length of rafter $r = \sqrt{b^2 + c^2} = 3.1623\, \text{m}$

Loads and reactions are positive in the directions shown.

- Plan load per unit length $w_2 = 2\, \text{kN/m}$
- Total load on rafter $W = w_2 \times b = 6\, \text{kN}$
- Maximum moment (+ve sagging) $M = W \times \cos(\alpha) \times r^3 / (384 \times E \times I)$
  - $= 0.018518\, \text{m}$
- Span/deflection ratio $s'd = r / D_1 = 170.77$
- Deflection/span ratio $d's = D_1 / r = 0.0058559$
- Horizontal react at 1 (+ve right) $H_1 = 0\, \text{kN}$ as joint 1 is on rollers
- Horizontal react at 2 (+ve left) $H_2 = 0\, \text{kN}$ resolving horizontally
- Vertical reaction at 1 (+ve up) $V_1 = W / 2 = 3\, \text{kN}$
- Vertical reaction at 2 (+ve up) $V_2 = 3\, \text{kN}$
- Axial force at 1 (+ve compress.) $F_1 = V_1 \times \sin(\alpha) = 0.94868\, \text{kN}$
- Shear force at 1 $S_1 = V_1 \times \cos(\alpha) = 2.846\, \text{kN}$

Load is UDL applied vertically on plan.
Location: Forces in rafter 5

Top of rafter is pinned and restrained against horizontal movement, but unrestrained against vertical movement.

Bottom of rafter is pinned and restrained against horizontal movement.

Rafter projection onto horizontal $b=3\,\text{m}$
Rafter projection onto vertical $c=1\,\text{m}$
Elastic modulus $E=9\times 10^6\,\text{kN/m}^2$
Moment of inertia of rafter $I=1.4063\times 10^{-4}\,\text{m}^4$
Slope of roof $\alpha=\text{ATN}(c/b)=0.32175\,\text{radians}$
$\alpha'=\text{DEG}(\alpha)=18.435\,\text{degrees}$
Length of rafter $r=\sqrt{b^2+c^2}=3.1623\,\text{m}$

Loads and reactions are positive in the directions shown.

Suction load per unit length $w_3=2\,\text{kN/m}$
Total load on rafter $W=w_3\times r=6.3246\,\text{kN}$
Maximum moment (+ve sagging) $M=-W\times r/8=-2.5\,\text{kNm}$
Max defln in rafter (+ve is sag) $D_1=-5\times W\times r^3/(384\times E\times I)=-0.020575\,\text{m}$
Span/deflection ratio $s'd=r/\text{ABS}(D_1)=153.69$
Deflection/span ratio $d'=\text{ABS}(D_1)/r=0.0065065$
Horizontal react at 2 (+ve left) $H_2=-W\times r/(2\times c)=-10\,\text{kN}$
Horizontal react at 1 (+ve right) $H_1=W\times \text{SIN}(\alpha)+H_2=-8\,\text{kN}$
Vertical reaction at 1 (+ve up) $V_1=-W\times \text{COS}(\alpha)=-6\,\text{kN}$
Axial force at 1 (+ve compress.) $F_1=V_1\times \text{SIN}(\alpha)+H_1\times \text{COS}(\alpha)=-9.4868\,\text{kN}$
Shear force at 1 $S_1=V_1\times \text{COS}(\alpha)-H_1\times \text{SIN}(\alpha)=-3.1623\,\text{kN}$
Location: Forces in rafter 2

Top of rafter is pinned and restrained against vertical and horizontal movement.

Bottom of rafter is pinned and unrestrained against horizontal movement.

Rafter projection onto horizontal \( b = 3 \) m
Rafter projection onto vertical \( c = 1 \) m
Elastic modulus \( E = 9E6 \) kN/m²
Moment of inertia of rafter \( I = 14.063E-6 \) m⁴
Slope of roof \( \alpha = \text{ATN}(c/b) = 0.32175 \) radians
\( \alpha' = \text{DEG}(\alpha) = 18.435 \) degrees
Length of rafter \( r = \text{SQR}(b^2+c^2) = 3.1623 \) m

Loads and reactions are positive in the directions shown.

Suction (+ve) per unit length \( w_3 = 2 \) kN/m
Total load on rafter \( W = w_3 * r = 6.3246 \) kN
Maximum moment (+ve sagging) \( M = -W * r / 8 = -2.5 \) kNm
Max defln in rafter (+ve is sag) \( D_1 = -5 * W * r^3 / (384 * E * I) = -0.020575 \) m
Span/deflection ratio \( s'd = r / \text{ABS}(D_1) = 153.69 \)
Deflection/span ratio \( d's = \text{ABS}(D_1) / r = 0.0065065 \)
Horizontal react at 1 (+ve right) \( H_1 = 0 \) kN as joint 1 is on rollers
Horizontal react at 2 (+ve left) \( H_2 = -W * \text{SIN}(\alpha) = -2 \) kN
Vertical reaction at 2 (+ve up) \( V_2 = (-W * r / 2 - H_2 * c) / b = -2.6667 \) kN
Vertical reaction at 1 (+ve up) \( V_1 = -W * \text{COS}(\alpha) - V_2 = -3.3333 \) kN
Axial force at 1 (+ve compress.) \( F_1 = V_1 * \text{SIN}(\alpha) = -1.0541 \) kN
Shear force at 1 \( S_1 = V_1 * \text{COS}(\alpha) = -3.1623 \) kN
Shear force at 2 \( S_2 = V_2 * \text{COS}(\alpha) + H_2 * \text{SIN}(\alpha) = -3.1623 \) kN
Location: Forces in couple roof

NOTE: At joints 1 and 3 there must be either two horizontal restraints, or a tie between joints 1 and 3, to prevent the formation of a mechanism.

Span of couple: \( a = 3 \text{ m} \)
Height of couple: \( c = 1 \text{ m} \)
Elastic modulus: \( E = 9 \times 10^6 \text{ kN/m}^2 \)
Moment of inertia of rafters: \( I = 1 \text{ m}^4 \)

Vertical load per unit length: \( w_1 = 2 \text{ kN/m} \)
Total load on truss: \( W = w_1 \times 2 \times r = 7.2111 \text{ kN} \)
Vertical reaction at 1 (+ve up): \( V_1 = W / 2 = 3.6056 \text{ kN} \)
Vertical reaction at 3 (+ve up): \( V_3 = 3.6056 \text{ kN} \)
Horizontal react at 1 (+ve right): \( H_1 = V_1 \times a / (4 \times c) = 2.7042 \text{ kN} \)
Horizontal react at 3 (+ve left): \( H_3 = 2.7042 \text{ kN} \)
Max moment in 1-2 (+ve sagging): \( M = V_1 \times \cos(\alpha) \times r / 8 = 0.67604 \text{ kNm} \)
Max defln in 1-2 (+ve sagging): \( D_1 = 5 \times V_1 \times \cos(\alpha) \times r^3 / (384 \times E \times I) = 25.43 \times 10^{-9} \text{ m} \)
Span/deflection ratio: \( s \cdot d / r = D_1 / L = 70.892 \times 10^6 \)
Deflection/span ratio: \( d \cdot s / L = 14.106 \times 10^6 \)
Axial force at 1 (+ve compress.): \( F_1 = V_1 \sin(\alpha) + H_1 \cos(\alpha) = 4.25 \text{ kN} \)
Shear force at 1: \( S_1 = V_1 \cos(\alpha) - H_1 \sin(\alpha) = 1.5 \text{ kN} \)
Location: Forces in collar tie roof

Span of truss \( a = 3 \text{ m} \)
Height of truss \( c = 1 \text{ m} \)
Height to tie \( d = 0.3 \text{ m} \)
Elastic modulus \( E = 9 \times 10^6 \text{ kN/m}^2 \)
Moment of inertia of rafters \( I = 1 \text{ m}^4 \)

Vertical load per unit length \( w_1 = 2 \text{ kN/m} \)
Total load on truss \( W = w_1 \times 2 \times r = 7.2111 \text{ kN} \)
Vertical reaction at 1 (+ve up) \( V_1 = W/2 = 3.6056 \text{ kN} \)
Vertical reaction at 5 (+ve up) \( V_5 = 3.6056 \text{ kN} \)
Horizontal react at 1 (+ve right) \( H_1 = 0 \text{ kN} \)
Axial force 1->2 (+ve compress.) \( F_1 = V_1 \times \sin(\alpha) = 2 \text{ kN} \)
Shear force at 1 (or 5) \( S_1 = V_1 \times \cos(\alpha) = 3 \text{ kN} \)
Force in tie (-ve tension) \( T = -(V_1 \times a/2 - w_1 \times r \times a/4) / (c-d) = -3.8631 \text{ kN} \)
Axial force 2->3 (+ve compress.) \( F_2 = F_1 - e \times w_1 \times \sin(\alpha) - T \times \cos(\alpha) = 4.6143 \text{ kN} \)
Mmt at 2 (+ve sagging) \( M_2 = V_1 \times f - w_1 \times e \times f/2 = 1.3791 \text{ kNm} \)
Constant \( n = N/L = 0.42857 \)
Magnitude of udl normal to rafter \( w = w_1 \times \cos(\alpha) = 1.6641 \text{ kN/m} \)
The deflection due to spreading is calculated from bending strain energy only, i.e. axial shortening and shear deformation are ignored.
Normal defln at 5 due to UDL \( D_5u = -w_1 \times L^3 \times N^2 \times (3 \times n^3 + 4 \times n^2 - 1) / (24 \times E \times I) = 0.24413 \times 10^{-9} \text{ m} \)
Normal defln at 5 due to shear \( D_5s = S_1 \times N^2 \times (N+L) / (3 \times E \times I) = 58.59 \times 10^{-9} \text{ m} \)
Defln at 5 (+ve is rightwards) \( D_5 = 2 \times (D_5u + D_5s) \times \sin(\alpha) = 65.27 \times 10^{-9} \text{ m} \)
Location: Forces and moments in lean-to: Mark A7

- Height of post: a = 2.4 m
- Width of lean-to: b = 2.1 m
- Rafter projection onto vertical: c = 1 m
- Moment of inertia of rafter: \( I_r = 14.063 \times 10^{-6} \) m\(^4\)
- Moment of inertia of post: \( I_p = 14.063 \times 10^{-6} \) m\(^4\)
- Elastic modulus: \( E = 9 \times 10^6 \) kN/m\(^2\)
- Vertical load per unit length: \( w_1 = 2 \) kN/m
- Max defln in rafter (+ve is sag): \( D_2 = \frac{5W\cos(\alpha)r^3}{384EIr} = 0.0054371 \) m
- Span/deflection ratio: \( s'd = \frac{r}{D_2} = 427.79 \)
- Deflection/span ratio: \( d's = D_2/r = 0.0023376 \)
- Horizontal react at 1 (+ve right): \( H_1 = 0 \) kN as 1-2 is free to articulate
- Horizontal react at 3 (+ve left): \( H_3 = 0 \) kN resolving horizontally
- Vertical reaction at 1 (+ve up): \( V_1 = W/2 = 2.3259 \) kN
- Vertical reaction at 3 (+ve up): \( V_3 = 2.3259 \) kN
- Axial force 1->2 (+ve compress.): \( F_1 = V_1 = 2.3259 \) kN
- Axial force 2->3 (+ve compress.): \( F_2 = V_1 \sin(\alpha) = 1 \) kN
- Shear force 2->3: \( S_2 = V_1 \cos(\alpha) = 2.1 \) kN
Location: Trussed beam: support in main room

Length of beam AB  \( l_4 = 6.096 \text{ m} \)
UDL on beam  \( w = 13.076 \text{ kN/m} \)
Point load on beam  \( W = 0 \text{ kN} \)
Distance to point load  \( a = 0 \text{ m} \)
Width of beam AB  \( b = 0.2286 \text{ m} \)
Depth of beam AB  \( d = 0.2286 \text{ m} \)
Elastic modulus for beam AB  \( E_4 = 9.2666 \times 10^6 \text{ kN/m}^2 \)
Length of strut CD  \( l_3 = 0.6096 \text{ m} \)
Area of strut CD  \( A_3 = 0.015484 \text{ m}^2 \)
Elastic modulus for strut CD  \( E_3 = 92.666 \times 10^6 \text{ kN/m}^2 \)
Area of ties AD and BD  \( A_1 = 0.50671 \times 10^{-3} \text{ m}^2 \)
Elastic modulus for ties AD & BD  \( E_1 = 185.33 \times 10^6 \text{ kN/m}^2 \)

Trussed beam properties

Area of beam cross-section  \( A_4 = b \times d = 0.052258 \text{ m}^2 \)
Section modulus of beam  \( z_b = b \times d^2 / 6 = 0.001991 \text{ m}^3 \)
Inertia of beam  \( I_4 = b \times d^3 / 12 = 0.22757 \times 10^{-3} \text{ m}^4 \)
Length of ties AD and BD  \( l_1 = \sqrt{(l_4/2 \times l_4/2 + l_3 \times l_3)} = 3.1084 \text{ m} \)
Elasticity for strut  \( e_3 = l_3 / (E_3 \times A_3) = 0.42486 \times 10^{-6} \text{ m/kN} \)
Elasticity for two ties  \( e_1 = l_1^3 / (2 \times l_3^2 \times E_1 \times A_1) = 0.4303 \times 10^{-3} \text{ m/kN} \)
Elasticity for beam in compress.  \( e_4 = l_4^3 / (16 \times l_3^2 \times A_4 \times E_4) = 78.678 \times 10^{-6} \text{ m/kN} \)
Elasticity for beam in bending  \( e_4' = l_4^3 / (48 \times E_4 \times I_4) = 0.0022379 \text{ m/kN} \)
Total elasticity  \( t_e = e_1 + e_3 + e_4 + e_4' = 0.0027473 \text{ m/kN} \)

Forces due to UDL

Deflection at C for unbraced beam  \( \delta_{C} = 5 \times w \times l_4^4 / (384 \times E_4 \times I_4) = 0.11149 \text{ m} \)
Hence thrust in the strut  \( F_s = \delta_{C} / t_e = 40.582 \text{ kN} \)
Pull in each tie-rod  \( F_t = F_s \times 0.5 \times 11 / 13 = 103.46 \text{ kN} \)
Thrust in beam  \( F_b = 0.5 \times F_s \times (l_4 / 13) = 101.46 \text{ kN} \)
End reactions  \( R = w \times l_4 / 2 = 39.856 \text{ kN} \)
Position for zero shear  \( x = 0.5 \times (2 \times R - F_s) / w = 1.4962 \text{ m} \)
Shear in beam at ends  \( s_B = R - F_t \times 13 / 11 = 19.565 \text{ kN} \)
Shear in beam at centre  \( s_C = s_B - (l_4 / 2) \times w = -20.291 \text{ kN} \)
Maximum sagging bending moment  \( M = s_b \times x = 0.5 \times w \times x^2 = 14.636 \text{ kNm} \)
Bending moment in beam at C  \( M_c = s_b \times l_4 / 2 - R \times l_4 / 4 = -1.1073 \text{ kNm} \)
Maximum bending moment in beam  \( M_m = A_B \times x = 14.636 \text{ kNm} \)
Extreme fibre bending stress  \( f_b = M_m / z_b = 7351.2 \text{ kN/m}^2 \)
Axial stress in beam  \( f_a = F_b / A_4 = 1941.4 \text{ kN/m}^2 \)
Maximum compressive stress  \( f_{mc} = f_a + f_b = 9232.6 \text{ kN/m}^2 \)
Maximum tensile stress  \( f_{mt} = f_b - f_a = 5409.7 \text{ kN/m}^2 \)

 SCALE 5.48  Office 1007  Proforma 290
Location: Forces in braced shed type GH-4

Members AB and CD are similar and represent vertical columns connected by a cross-beam BC having pinned ends. Members KG and EF are braces with pinned ends.

Column feet are pinned.

Span of frame \( l = 2.4 \) m
Height of frame \( L = 3 \) m
Height to knee brace \( h = 2.4 \) m
Distance B to G (also E to C) \( d = 0.8 \) m
Horizontal force at B \( W = 2.5 \) kN
Horizontal reactions at A & D \( H = W/2 = 1.25 \) kN
Vertical reaction at D \( V = W \times L / l = 3.125 \) kN
Moments at K and F are equal \( \text{Mk} = H \times h = 3 \) kNm
Moments at G and E are equal \( \text{Mg} = W \times L \times (0.5 - d/l) = 1.25 \) kNm
Slope of knee brace with horiz. \( \theta = \text{ATN}((L-h)/d) = 0.6435 \) radians
Tension in KG \( T = 0.5 \times W \times L / ((L-h) \times \text{COS}(\theta)) = 7.8125 \) kN
Thrust in EF \( T' = T = 7.8125 \) kN
Thrust in BG \( t = 0.5 \times W \times (2 \times L - h) / (L-h) = 7.5 \) kN
Tension in EC \( t' = t = 7.5 \) kN
Thrust in GE \( t_g = 0.5 \times W = 1.25 \) kN
Shearing force from B to G \( v_{bg} = W \times L \times (1/l - 1/(2 \times d)) = -1.5625 \) kN
Shearing force from G to E \( v_{ge} = W \times L / l = 3.125 \) kN
**Location:** Proformas to BS5400:Part 3:2000

**Units for input of data and output of results**

<table>
<thead>
<tr>
<th>Element</th>
<th>Unit</th>
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<tbody>
<tr>
<td>Moments</td>
<td>kNm</td>
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<tr>
<td>Forces</td>
<td>kN</td>
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<tr>
<td>Stresses</td>
<td>N/mm²</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>N/mm²</td>
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<tr>
<td>Dimensions including spans</td>
<td>mm</td>
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<tr>
<td>Areas</td>
<td>mm²</td>
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<tr>
<td>Second moments of area</td>
<td>mm⁴</td>
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</tbody>
</table>

Numbers may be input in exponent form as shown below:

- 20900000 may be input as 209E5
- 0.00209 may be input as 2.09E-3
Location: Proformas to BS EN 1993-2:2006

Units for input of data and output of results

- Moments: kNm
- Forces: kN
- Stresses: N/mm²
- Young's modulus: N/mm²
- Dimensions including spans: mm
- Areas: mm²
- Second moments of area: mm⁴

Numbers may be input in exponent form as shown below:

20900000 may be input as 209E5
0.00209 may be input as 2.09E-3
Location: BS 5400:Part 3:2000

Design objectives and partial safety factors

The proforma is in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.
Values of partial safety factors (Clause 4.3.3 and Table 2)

gfL - the values of gfL are given in Departmental Standard BD 37/01 for each type and combination of loading.

gf3 - the factor gf3 in this part of BS 5400 shall be taken as 1.1 for the ultimate limit state and 1.0 for the serviceability limit state.

a) Ultimate limit state

Where explicitly expressed in a strength requirement in this part of BS5400 gm shall be taken as 1.05 except in the following Clauses:

<table>
<thead>
<tr>
<th>Structural component and behaviour</th>
<th>Clauses</th>
<th>gm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength of longitudinal stiffeners</td>
<td>9.10.2.3(a) and (b)</td>
<td>1.20</td>
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<tr>
<td></td>
<td>9.11.5.2</td>
<td>(Comp)</td>
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<td>(Tens)</td>
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<td>Buckling resistance of stiffeners</td>
<td>9.13.5.3, 9.13.6, 9.14.4.3, 9.17.6.7, 9.17.7.3.2, 9.17.8</td>
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</tr>
<tr>
<td>Fasteners in stiffeners</td>
<td>14.5.3.2, 14.5.3.3, 14.5.3.5</td>
<td>1.20</td>
</tr>
<tr>
<td>Fasteners in shear</td>
<td>14.5.3.4</td>
<td>1.10</td>
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<tr>
<td>Friction capacity of HSFG bolts</td>
<td>14.5.4.2</td>
<td>1.30</td>
</tr>
<tr>
<td>Welds</td>
<td>14.6.3.11.1, 14.6.3.11.2, 14.6.3.11.3</td>
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b) Serviceability limit state

Where explicitly expressed in a strength requirement in this part of BS5400 gm shall be taken as 1.0 except in the following Clause:

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<th>Structural component and behaviour</th>
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<tr>
<td>Friction capacity of HSFG bolts</td>
<td>14.5.4.2</td>
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</table>
**Location:** Partial factors to BS EN 1993-2

Partial factors for materials - (BS EN 1993-2 Table 6.1)

<table>
<thead>
<tr>
<th>Resistance type</th>
<th>Factor</th>
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<tr>
<td>(a) Resistance of members and cross-section</td>
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<tr>
<td>- resistance of cross-sections to excessive yielding including local buckling</td>
<td>$\gamma M_0 = 1.0$</td>
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<tr>
<td>- resistance of members to instability assessed by member checks</td>
<td>$\gamma M_1 = 1.1$</td>
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<tr>
<td>- resistance to fracture of cross sections in tension</td>
<td>$\gamma M_2 = 1.25$</td>
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<tr>
<td>(b) Resistance of joints</td>
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</tr>
<tr>
<td>- resistance of bolts</td>
<td>$\gamma M_2 = 1.25$</td>
</tr>
<tr>
<td>- resistance of rivets</td>
<td>$\gamma M_2 = 1.25$</td>
</tr>
<tr>
<td>- resistance of pins</td>
<td>$\gamma M_2 = 1.25$</td>
</tr>
<tr>
<td>- resistance of welds</td>
<td>$\gamma M_2 = 1.25$</td>
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<tr>
<td>- resistance of plates in bending</td>
<td>$\gamma M_2 = 1.25$</td>
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<tr>
<td>- slip resistance at ULS</td>
<td>$\gamma M_3 = 1.25$</td>
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<tr>
<td>- slip resistance at SLS</td>
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<tr>
<td>- bearing resistance of an injection bolt</td>
<td>$\gamma M_4 = 1.10$</td>
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<tr>
<td>- resistance of joints in hollow section lattice girders</td>
<td>$\gamma M_5 = 1.10$</td>
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<tr>
<td>- resistance of pins at SLS</td>
<td>$\gamma M_{6ser} = 1.00$</td>
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<tr>
<td>- preload of high-strength bolts</td>
<td>$\gamma M_7 = 1.10$</td>
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## Partial factors on actions - (BS EN 1990)

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<tr>
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Location: BS5400:Part 3:2000

**Steel grades and standards**

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<th>Impact Quality</th>
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**Longitudinal Charpy V-notch**

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<th>G  - 27J @ -15° C</th>
<th>M - 40J @ -20° C</th>
<th>Q  - 30J @ -20° C</th>
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<tbody>
<tr>
<td>J0 - 27J @ 0° C</td>
<td>M - 27J @ -50° C</td>
<td>QL - 30J @ -40° C</td>
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<td>J2 - 27J @ -20° C</td>
<td>N - 40J @ -20° C</td>
<td>QL1 - 30J @ -60° C</td>
</tr>
<tr>
<td>K2 - 40J @ -20° C</td>
<td>NL - 27J @ -50° C</td>
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</tr>
</tbody>
</table>

Note: 40J @ -20° C = 27J @ -30° C  Clause 6.5.4
30J @ -xx° C = 27J @ -xx° C
Location: BS EN 1993-2:2006

Steel grades and standards

<table>
<thead>
<tr>
<th>Standard BS EN</th>
<th>Strength Grade</th>
<th>Impact Quality</th>
<th>Standard BS EN</th>
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<th>Impact Quality</th>
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<tr>
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<td>ML</td>
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</table>

Charpy V-notch

<table>
<thead>
<tr>
<th>Longitudinal Charpy V-notch</th>
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<tbody>
<tr>
<td>G - 27J @ -15°C</td>
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<tr>
<td>J0 - 27J @ 0°C</td>
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<tr>
<td>J2 - 27J @ -20°C</td>
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<tr>
<td>K2 - 40J @ -20°C</td>
</tr>
<tr>
<td>M - 40J @ -20°C</td>
</tr>
<tr>
<td>ML - 27J @ -50°C</td>
</tr>
<tr>
<td>N - 40J @ -20°C</td>
</tr>
<tr>
<td>NL - 27J @ -50°C</td>
</tr>
<tr>
<td>Q - 30J @ -20°C</td>
</tr>
<tr>
<td>QL - 30J @ -40°C</td>
</tr>
<tr>
<td>QL1 - 30J @ -60°C</td>
</tr>
</tbody>
</table>
Location: BS5400:Part 3:2000

The proforma is in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

For steel conforming to the standards in Clause 6.1.2, and supplied within the specified tolerances, the nominal yield stress, $\sigma_y$, should be the minimum yield strength specified for the appropriate thickness.

Clause 6.3 states the specified minimum ultimate tensile stress of steel plates and sections of steel should not be less than $1.2\sigma_y$ where $\sigma_y$ is the nominal yield stress defined in Clause 6.2.
Location: BS EN 1993-2:2006

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges".

Steel conforming to Cl. 3.1 and Table 3.1 of EN 1993-1-1 and supplied within the specified tolerances, the nominal yield stress, \( f_y \), should be the minimum yield strength specified for the appropriate thickness.

The ultimate tensile strength \( f_u \) for hot rolled structural steel should be in accordance with Table 3.1 of BS EN 1993-1-1:2005. Steel grades in Table 3.1 will automatically provide the levels of ductility required by Clause 3.3.2.
Location: Ex1 - Edge beam

Minimum effective bridge temperature  \( U_e = -10 \, ^\circ C \)

Test temperature  \( T_{27J} = -30 \, ^\circ C \)

Factor \( k_d \)  \( k_d = 1 \)

Ratio  \( s_{rat} = 0.4 \)

Factor \( k_{sigma} \)  \( k_{sigma} = 1 \)

Factor \( k_s \)  \( k_s = 1 \)

Nominal yield stress \( \sigma_y \)  \( \sigma_y = 355 \, N/mm^2 \)

Limiting thickness  \[ t = 50 \times k \times (355/\sigma_y)^{1.4} \times 1.2^{((U-T_{27J})/10)} = 65.727 \, mm \]

Limiting thickness for \( k = 1 \)  \[ t_1 = 50 \times (355/\sigma_y)^{1.4} \times 1.2^{((U-T_{27J})/10)} = 72 \, mm \]
Location: Ex1 - Worked example 3.2-2 Designers' Guide to EC3 Part 2

Element thickness t=40 mm
Minimum ambient air temperature Tmd=-12 °C
Radiation loss adjustment dTr=0 °C
Adjustment for stress dTs=0 °C
Safety allowance dTR=0 °C
Impact strain rate e=0.017 /s
Reference strain rate eo=0.4E-3 /s
Adjustment for cold forming dTecf=0 °C

SUMMARY OF RESULTS

Actual element thickness 40 mm
Maximum thickness permitted 43.545 mm
Steel grade S 355 (from Table 2.1)
Steel sub-grade J2 (from Table 2.1)
Steel strength grade required S355J2

NOTE: Further reference should be made to the National Annex to ensure that the grades will also meet any additional guidelines at welded details.
Location: Ex2 - Example 3.2-1 Designers' Guide to EC3 Pt 2 bridge 2

Element thickness \( t = 30 \text{ mm} \)
Minimum ambient air temperature \( T_{md} = -20 \degree C \)
Radiation loss adjustment \( d_T = 0 \degree C \)
Adjustment for stress \( d_T = 0 \degree C \)
Safety allowance \( d_T = 0 \degree C \)
Adjustment for cold forming \( d_{Tecf} = 0 \degree C \)

**SUMMARY OF RESULTS**

Actual element thickness 30 mm
Maximum thickness permitted 35 mm
Steel grade S 355 (from Table 2.1)
Steel sub-grade J0 (from Table 2.1)
Steel strength grade required S355J0

NOTE: Further reference should be made to the National Annex to ensure that the grades will also meet any additional guidelines at welded details.
Location: Ex3 - Example 3.2-1 Designers' Guide to EC3 Pt 2 bridge 6

Element thickness $t=63$ mm
Minimum ambient air temperature $T_{md}=-20$ °C
Radiation loss adjustment $d_{Tr}=0$ °C
Adjustment for stress $d_{Ts}=0$ °C
Safety allowance $d_{TR}=0$ °C
Adjustment for cold forming $d_{Tecf}=0$ °C

SUMMARY OF RESULTS

Actual element thickness $63$ mm
Maximum thickness permitted $90$ mm
Steel grade $S\ 355$ (from Table 2.1)
Steel sub-grade ML or NL (from Table 2.1)
Steel strength grade required S355ML or S355NL

NOTE: Further reference should be made to the National Annex to ensure that the grades will also meet any additional guidelines at welded details.
Location: Ex1 - Internal girder

Stress analysis - Allowance for shear lag (Clause 8.2)

The calculations are in accordance with BS5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

**Portion 1**

Portion of flange spans between two webs.

```
  +-------------------+
  |                  |
  |                  |
  +-------------------+
```

<table>
<thead>
<tr>
<th>Dimension b</th>
<th>b(1)=500 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span of beam</td>
<td>L=2000 mm</td>
</tr>
<tr>
<td>between centres</td>
<td></td>
</tr>
<tr>
<td>of support:</td>
<td></td>
</tr>
<tr>
<td>Effective breadth ratio for interior spans of continuous beam from Table 5 is 0.36.</td>
<td>be(1)=ψ(i)*b(i)=180 mm</td>
</tr>
</tbody>
</table>

**Portion 2**

Portion of flange projects beyond an outer web.

```
  +-------------------+
  |                  |
  |                  |
  +-------------------+
```

<table>
<thead>
<tr>
<th>Dimension b</th>
<th>b(2)=250 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective breadth ratio for interior spans of continuous beam from Table 5 is 0.615.</td>
<td>be(2)=k(i)*ψ(i)*b(i)=150.87 mm</td>
</tr>
<tr>
<td>Effective breadth</td>
<td>be(2)=k(i)*ψ(i)*b(i)=150.87 mm</td>
</tr>
<tr>
<td>Total effective breadth</td>
<td>be=be(1)+be(2)=330.87 mm</td>
</tr>
</tbody>
</table>
Location: Ex1 - Internal girder without longitudinal stiffeners

Stress analysis - Allowance for shear lag

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges".

The proforma calculates the effective widths for shear lag for members in bending as per Clause 6.2.2.3.

**Portion 1**

Portion of flange spans between two webs.

Dimension bo = 500 mm
Span of beam between centres of support: L = 2000 mm
Distance below is between points of zero bending moment from Fig. 3.1 of BS EN 1993-1-5:2006.
Effective length: Le = 1400 mm
Effective width at SLS: bes(1) = beta*bo(i) = 275.28 mm
Effective width at ULS: beu(1) = beta^k*bo(i) = 404.02 mm

**Portion 2**

Portion of flange projects beyond an outer web.

Dimension bo = 250 mm
Distance below is between points of zero bending moment from Fig. 3.1 of BS EN 1993-1-5:2006.
Effective length: Le = 1400 mm
Effective width at SLS: bes(2) = beta*bo(i) = 207.63 mm
Effective width at ULS: beu(2) = beta^k*bo(i) = 241.85 mm
Total effective width at SLS: beff = bes(1) + bes(2) = 482.91 mm
Total effective width at ULS: beff = beu(1) + beu(2) = 645.86 mm
Location: Ex2 - Internal girder with longitudinal stiffeners

Stress analysis - Allowance for shear lag

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges".

The proforma calculates the effective widths for shear lag for members in bending as per Clause 6.2.2.3.

Portion 1

Portion of flange spans between two webs.

- Dimension bo: bo(1)=500 mm
- Span of beams between centres of support:
  - Beam span on LHS of support: L1=2000 mm
  - Beam span on RHS of support: L2=2000 mm
- Distance below is between points of zero bending moment from Fig. 3.1 of BS EN 1993-1-5:2006.
- Effective length: Le=1000 mm
- Total area of long.stiffeners: Asl=500 mm²
- Flange plate thickness: t=15 mm
- Effective width at SLS: bes(1)=beta*bo(i)=110.61 mm
- Effective width at ULS: beu(1)=beta^k*bo(i)=229.42 mm

Portion 2

Portion of flange projects beyond an outer web.

- Dimension bo: bo(2)=250 mm
- Distance below is between points of zero bending moment from Fig. 3.1 of BS EN 1993-1-5:2006.
- Effective length: Le=1000 mm
- Effective width at SLS: bes(2)=beta*bo(i)=96.51 mm
- Effective width at ULS: beu(2)=beta^k*bo(i)=197.06 mm
- Total effective width at SLS: beff=bes(1)+bes(2)=207.12 mm
- Total effective width at ULS: beff=beu(1)+beu(2)=426.48 mm
Stress analysis - Allowance for shear lag

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges".

The proforma calculates the effective widths for shear lag for members in bending as per Clause 6.2.2.3.

Portion 1

Portion of flange spans between two webs.

\[
\text{Dimension } bo \quad bo(1) = 500 \text{ mm}
\]

Span of beam between centres of support:

\[
L = 2000 \text{ mm}
\]

Distance below is between points of zero bending moment from Fig. 3.1 of BS EN 1993-1-5:2006.

Effective length

\[
Le = 1400 \text{ mm}
\]

Effective width at SLS

\[
bes(1) = \beta \times bo(1) = 149.69 \text{ mm}
\]

Effective width at ULS

\[
beu(1) = \beta^k \times bo(1) = 325.02 \text{ mm}
\]

Portion 2

Portion of flange projects beyond an outer web.

\[
\text{Dimension } bo \quad bo(2) = 250 \text{ mm}
\]

Distance below is between points of zero bending moment from Fig. 3.1 of BS EN 1993-1-5:2006.

Effective length

\[
Le = 1400 \text{ mm}
\]

Effective width at SLS

\[
bes(2) = \beta \times bo(2) = 118.54 \text{ mm}
\]

Effective width at ULS

\[
beu(2) = \beta^k \times bo(2) = 218.81 \text{ mm}
\]

Total effective width at SLS

\[
beff = bes(1) + bes(2) = 268.23 \text{ mm}
\]

Total effective width at ULS

\[
beff = beu(1) + beu(2) = 543.83 \text{ mm}
\]
Location: Ex1 - Internal girder without longitudinal stiffeners

Stress analysis - Allowance for shear lag

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges".

The proforma calculates the effective widths for shear lag for members in bending as per Clause 6.2.2.3.

Portion 1

Portion of flange spans between two webs.

```
\[ \text{Dimension bo} \quad \text{bo(1)=500 mm} \]
\[ \text{Span of beams between centres of support:} \]
\[ \text{Beam span on LHS of support} \quad L1=2000 \text{ mm} \]
\[ \text{Beam span on RHS of support} \quad L2=2000 \text{ mm} \]
\[ \text{Distance below is between points of zero bending moment from Fig. 3.1 of BS EN 1993-1-5:2006.} \]
\[ \text{Effective length} \quad L_e=1000 \text{ mm} \]
\[ \text{Effective width at SLS} \quad b_e(1)=\beta_1\times b_0(1)=113.76 \text{ mm} \]
\[ \text{Effective width at ULS} \quad b_e(1)=\beta_2\times b_0(1)=238.5 \text{ mm} \]
```

Portion 2

Portion of flange projects beyond an outer web.

```
\[ \text{Dimension bo} \quad \text{bo(2)=250 mm} \]
\[ \text{Distance below is between points of zero bending moment from Fig. 3.1 of BS EN 1993-1-5:2006.} \]
\[ \text{Effective length} \quad L_e=1000 \text{ mm} \]
\[ \text{Effective width at SLS} \quad b_e(2)=\beta_1\times b_0(2)=96.51 \text{ mm} \]
\[ \text{Effective width at ULS} \quad b_e(2)=\beta_2\times b_0(2)=197.06 \text{ mm} \]
\[ \text{Total effective width at SLS} \quad b_{eff}=b_e(1)+b_e(2)=210.27 \text{ mm} \]
\[ \text{Total effective width at ULS} \quad b_{eff}=b_e(1)+b_e(2)=435.56 \text{ mm} \]

Location: Ex1 - Girder A

BS5400:Part 3:2000 - Shape limitations for flanges

Flanges and webs - Requirements for compact section

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks flanges for compliance with the shape limitations to Clause 9.3.2 and requirements for a compact section to Clause 9.3.7.3.1 or 9.3.7.3.

Flange outstands in compression (Clause 9.3.2.1).
Flange comprises multiple flange plates welded to web. Clause 9.3.2.2.
Flange plates welded along edges.

Where a flange consists of several flange plates built-up and connected to each other only by welds at their edges, an outer flange plate should not be thicker than an inner plate. For the flange plate connected to the web, bfoc should be taken as the width of the outstand measured from the edge to the surface of the web. For all other plates bfoc should be taken as half the width between the welds connecting it to an inner plate.

Maximum value of bfo/tfo ratio1=bfoc/tfoc=6.0606
Nominal comp yield stress σy syc=345 N/mm²

Flange outstands in tension. Clause 9.3.2.2.
Flange comprises multiple flange plates welded to web. Clause 9.3.2.2.
Flange plates welded along edges.

Where a flange consists of several flange plates built-up and connected to each other only by welds at their edges, an outer flange plate should not be thicker than an inner plate. For the flange plate connected to the web, bfo should be taken as the width of the outstand measured from the edge to the surface of the web. For all other plates bfo should be taken as half the width between the welds connecting it to an inner plate.
Maximum value of $b_f / t_f = \text{ratio} = 6.0606$

Nominal tensile yield stress $\sigma_y = 345 \text{ N/mm}^2$

**RESULTS FROM CALCULATION**

Flange outstands in compression comply with Clause 9.3.2.1 with nominal yield compressive stress of 345 N/mm².

Flange outstands in tension comply with Clause 9.3.2.2 with nominal yield tensile stress of 345 N/mm².

Compact sections. Clause 9.3.7.

Multiple flange plates welded to web:

- Compression flange
- Plastic Neutral Axis
- Depths $c_e$ and $d_w$ should be taken clear of the web to flange welds.

Thickness of web plate $t_w = 12 \text{ mm}$

Dimension $c_e = 620 \text{ mm}$

Depth of web $d_w = 1240 \text{ mm}$

Nominal tensile yield stress $\sigma_y$ $s_{yw} = 355 \text{ N/mm}^2$

Nominal tensile yield stress $\sigma_y$ $s_{yf} = 345 \text{ N/mm}^2$

**RESULTS SUMMARY**

Web does not comply with Clause 9.3.7.2.

Compression flange outstands comply with Clause 9.3.7.3.1.

Section is not compact.
Location: Ex1 - Girder A

Shape limitations for flanges

Flanges and webs - Requirements for compact section

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006

Flange outstands in compression. Flange plates welded to web along edges:

\[
\begin{align*}
2 \times C_f
\end{align*}
\]

Mean thickness \( t_f \) of outer plate

\[
\begin{align*}
C_f
\end{align*}
\]

Mean thickness \( t_f \) of inner plate

Where a flange consists of several flange plates built-up and connected to each other only by welds at their edges, an outer flange plate should not be thicker than an inner plate. For the flange plate connected to the web, \( C_f \) should be taken as the width of the outstand measured from the edge to the toe of root fillet. For all other plates, \( C_f \) should be taken as half the width between the welds connecting it to an inner plate.

- Dimension of outstand: \( C_f = 200 \text{ mm} \)
- Mean thickness of plate: \( t_f = 33 \text{ mm} \)
- Web depth between root fillets: \( C_w = 1240 \text{ mm} \)
- Thickness of web: \( t_w = 12 \text{ mm} \)
- Yield strength: \( f_y = 345 \text{ N/mm}^2 \)
- Design axial load: \( N_{Ed} = 0 \text{ kN} \)

Section classification

Classify outstand element of compression flange:

- Outstand: \( c = C_f = 200 \text{ mm} \)
- Ratio: \( c'/t_f = 6.0606 \)

As \( c/t_f \leq 9 \text{e} (6.0606 \leq 7.4279) \), outstand element of compression flange is classified as Class 1 plastic.

Classify web element of section:

- Ratio: \( C'/t_w = 103.33 \)

As \( C/t_w > 124 \text{e} (103.33 > 102.34) \), web element in bending is classified as Class 4 slender.
Location: Ex2 - Girder A

Shape limitations for flanges

Flanges and webs - Requirements for compact section

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006

Flange outstands in compression. Flange plates bolted to the web:

![Diagram of flange outstand with labels:
- Thickness tf is the mean thickness of the outstand,
- Dimension Cf is the width of the outstand measured from the edge to the nearest line of bolts connecting it to the supporting part of the member.
]

Dimension of outstand: Cf = 200 mm
Mean thickness of plate: tf = 33 mm
Web depth between root fillets: Cw = 1000 mm
Thickness of web: tw = 12 mm
Yield strength: fy = 345 N/mm²
Design axial load: NEd = 0 kN

Section classification

Classify outstanding element of compression flange:
Outstand: c = Cf = 200 mm
Ratio: c' = c/tf = 6.0606
As c/tf ≤ 9e (6.0606 ≤ 7.4279), outstanding element of compression flange is classified as Class 1 plastic.

Classify web element of section:
Ratio: C' = Cw/tw = 83.333
As Cw/tw ≤ 124e (83.333 ≤ 102.34), web element in bending is classified as Class 3 semi-compact.
Location: Ex3 - Girder A

Shape limitations for flanges

Flanges and webs - Requirements for compact section

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006

Flange outstands in compression. Single flange plate welded to web:

- Thickness $t_f$ is the mean thickness of the outstand
- Dimension $C_f$ is the width of the outstand measured from the edge to the toe of root fillet
- Dimension $C_w$ is the web depth between root fillets

<table>
<thead>
<tr>
<th>Dimension of outstand</th>
<th>$C_f=200$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of flange plate</td>
<td>$t_f=25$ mm</td>
</tr>
<tr>
<td>Web depth between root fillets</td>
<td>$C_w=1000$ mm</td>
</tr>
<tr>
<td>Thickness of web</td>
<td>$t_w=12$ mm</td>
</tr>
<tr>
<td>Yield strength</td>
<td>$f_y=345$ N/mm²</td>
</tr>
<tr>
<td>Design axial load</td>
<td>$N_{Ed}=50$ kN</td>
</tr>
</tbody>
</table>

Section classification

Classify outstand element of compression flange:
- Parameter (Table 5) $e=(235/f_y)^{0.5}=0.82532$
- Outstand $c=C_f=200$ mm
- Ratio $c'/t=8$

Classify web element of section:
- Depth between fillets $d=C_w=1000$ mm
- Ratio $c'/t_w=83.333$
- Area of section $A=5800$ mm²
- Limiting ratio $c'/t^3=101.9$
- Web is Class 3 semi-compact
**Location: Ex4 - Girder A**

**Shape limitations for flanges**

**Flanges and webs - Requirements for compact section**

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006

Flange outstands in compression. Rolled section:

![Diagram of flange outstand](image)

- Thickness $t_f$ is the thickness of plate
- Dimension $C_f$ is the width of the outstand measured from the edge to the toe of root fillet
- Dimension $C_w$ is the web depth between root fillets

<table>
<thead>
<tr>
<th>Dimension of outstand</th>
<th>$C_f=200$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of flange plate</td>
<td>$t_f=25$ mm</td>
</tr>
<tr>
<td>Web depth between root fillets</td>
<td>$C_w=600$ mm</td>
</tr>
<tr>
<td>Thickness of web</td>
<td>$t_w=12$ mm</td>
</tr>
<tr>
<td>Yield strength</td>
<td>$f_y=345$ N/mm$^2$</td>
</tr>
<tr>
<td>Design axial load</td>
<td>$N_{Ed}=100$ kN</td>
</tr>
</tbody>
</table>

**Section classification**

Classify outstanding element of compression flange:

- Parameter (Table 5): $e=(235/f_y)^{0.5}=0.82532$
- Outstand: $c=C_f=200$ mm
- Ratio: $c'/t_c=t_f=8$

Classify web element of section:

- Depth between fillets: $d=C_w=600$ mm
- Ratio: $c'/t_w=d/t_w=50$
- Limiting ratio: $c'/t_1=396*e/(13*ad-1)=56.725$

Web is Class 1 plastic.
Location: Ex1 - Girder W

Openings in webs and compression flanges

The calculations are in accordance with BS5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

The proforma outlines the requirements from Clause 9.3.3.

Openings in webs or compression members (Clause 9.3.3)

Clause 9.3.3.1 (General)

Any openings in webs or compression flanges should be framed and the stiffened section designed for local load effects, including secondary bending. Alternatively, openings in webs may be unstiffened provided that they meet the provisions of Clause 9.3.3.2.

All corners should be rounded with a radius of at least one-quarter of the least dimension of the hole.

Openings in webs or compression members (Clause 9.3.3)

Clause 9.3.3.2 (Unstiffened openings in webs)

Openings in a web may be unstiffened provided the following conditions are met:

- the overall greatest internal dimension does not exceed one-tenth of the depth of the web, nor, for longitudinally stiffened webs, one-third of the depth of the panel containing the opening

- the longitudinal distance between the boundaries of adjacent openings is at least three times the maximum internal dimension

- not more than one opening is provided at any cross-section.

Cut outs provided for transverse stiffeners should either have at least one-third of their perimeters welded to stiffeners, or the stiffeners should be cleated to the web with at least two bolts or rivets per side of the connection or by full perimeter welding of the cleat.
Location: Ex1 - Web opening

Openings in beam webs


Openings in beam webs based on "Access Steel" NCCI SN019b-EN-EU

Reinforced openings

Any openings in beam webs should be reinforced and the stiffened section designed for local load effects, including secondary bending. Reinforced openings should satisfy the following conditions:

1. The minimum projection of the stiffeners past the opening should be at least 20 times the thickness of the stiffener.
2. The stiffener projection from the face of the web should not exceed 10 x thickness of the stiffener.
3. The distance between edges of openings should be at least 1.5 times the length of the longer opening.
4. All corners should be rounded with a radius of at least one-quarter of the least dimension of the hole.
5. The length of the opening should not exceed 2.5 x opening depth.
6. The distance of any part of the opening to the nearest point load should not be less than half the depth of the member.

Openings in webs based on "Access Steel" NCCI SN019b-EN-EU

Unreinforced openings

Openings in a web may be unreinforced (unstiffened) provided the following conditions are met:

1. The depth of the opening should not exceed 0.6 x beam depth.
2. The length of the opening should not exceed 1.5 x opening depth.
3. The distance between edges of openings should be at least 1.5 times the length of the longer opening.
4. Not more than one opening is provided at any cross-section.
5. Opening not to be closer than twice the beam depth from support.
6. The distance of any part of the opening to the nearest point load should not be less than the depth of the member.

Cut outs provided for transverse stiffeners should either have at least one-third of their perimeters welded to stiffeners, or the stiffeners should be cleated to the web with at least two bolts or rivets per side of the connection or by full perimeter welding of the cleat.
Location: Ex1 - Internal beam

Shape limitations for flat stiffeners (Clause 9.3.4.1.2)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.
Stiffeners to webs and compression flanges. Clause 9.3.4.

Flat stiffeners (Clause 9.3.4.1.2):

```
hs

|--|<-- ts

b
```

Dimension b is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener
Stiffener thickness
Spacing of stiffeners
Plate thickness (tf or tw)
Nominal yield stress of plate
Nominal yield stress of stiffener
Ratio (b/t) (sy/355)=9.8581 is less than 31.
Ratio (hs/ts) (syy/355)=12.5 is less than limit 19.624.
Stiffener complies with Clause 9.3.4.1.2.
Location: Internal beam

Shape limitations for flat stiffeners (Clause 9.3.4.1.2)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges. Clause 9.3.4.

Flat stiffeners (Clause 9.3.4.1.2):

![Diagram of Geometric Notation Figure 1]

Dimension b is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener \( hs = 100 \text{ mm} \)
Stiffener thickness \( ts = 8 \text{ mm} \)
Spacing of stiffeners \( b = 200 \text{ mm} \)
Plate thickness (tf or tw) \( t = 8 \text{ mm} \)
Nominal yield stress of plate \( sy = 355 \text{ N/mm}^2 \)
Nominal yield stress of stiffener \( sys = 355 \text{ N/mm}^2 \)
Ratio \( (b/t) \ (sy/355) = 25 \) is less than 31.
Ratio \( (hs/ts) \ (sys/355) = 12.5 \) exceeds limit 11.025.
Stiffener does not comply with Clause 9.3.4.1.2.
Location: Internal beam

Shape limitations for flat stiffeners (Clause 9.3.4.1.2)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges. Clause 9.3.4.

Flat stiffeners (Clause 9.3.4.1.2):

\[ b \quad \text{is the thickness of flange (tf) or web (tw)} \]

\[ h_s \]

\[ \rightarrow \quad \text{ts} \quad \leftarrow \]

Geometric Notation Figure 1

Dimension \( b \) is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener \( h_s = 100 \text{ mm} \)

Stiffener thickness \( t_s = 12 \text{ mm} \)

Spacing of stiffeners \( b = 200 \text{ mm} \)

Plate thickness (tf or tw) \( t = 8 \text{ mm} \)

Nominal yield stress of plate \( \sigma_y = 355 \text{ N/mm}^2 \)

Nominal yield stress of stiffener \( \sigma_{ys} = 355 \text{ N/mm}^2 \)

Ratio \( (h_s / t_s) \cdot (\sigma_{ys} / 355) = 8.3333 \) is not greater than limit 10.

Stiffener complies with Clause 9.3.4.1.2.
**Location: Internal beam**

**Shape limitations for flat stiffeners (Clause 9.3.4.1.2)**

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. Stiffeners to webs and compression flanges. Clause 9.3.4.

Flat stiffeners (Clause 9.3.4.1.2):

![Geometric Notation Figure 1](image)

Dimension b is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

- Depth of stiffener \( hs = 120 \text{ mm} \)
- Stiffener thickness \( ts = 3 \text{ mm} \)
- Spacing of stiffeners \( b = 200 \text{ mm} \)
- Plate thickness (tf or tw) \( t = 5 \text{ mm} \)
- Nominal yield stress of plate \( sy = 355 \text{ N/mm}^2 \)
- Nominal yield stress of stiffener \( sys = 355 \text{ N/mm}^2 \)
- Ratio \( (hs/ts) \cdot (sys/355) = 40 \) exceeds limiting Fig. 2 value 33.

Stiffener does not comply with Clause 9.3.4.1.2.
Location: Internal beam

Shape limitations for flat stiffeners (Clause 9.3.4.1.2)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges. Clause 9.3.4.

Flat stiffeners (Clause 9.3.4.1.2):

[Diagram of geometric notation figure 1]

Dimension b is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener

Stiffener thickness

Spacing of stiffeners

Plate thickness (tf or tw)

Nominal yield stress of plate

Nominal yield stress of stiffener

Ratio (bt/ts^2) (sys/355)=1.6 is less than limiting Fig. 2 value 5.

Stiffener does not comply with Clause 9.3.4.1.2.
Shape limitations for flat stiffeners (Clause 9.3.4.1.2)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges. Clause 9.3.4.

Flat stiffeners (Clause 9.3.4.1.2):

\[
\text{Depth of stiffener} \quad hs=100 \text{ mm}
\]

\[
\text{Stiffener thickness} \quad ts=5 \text{ mm}
\]

\[
\text{Spacing of stiffeners} \quad b=1000 \text{ mm}
\]

\[
\text{Plate thickness (tf or tw)} \quad t=5 \text{ mm}
\]

\[
\text{Nominal yield stress of plate} \quad sy=355 \text{ N/mm}^2
\]

\[
\text{Nominal yield stress of stiffener} \quad sys=355 \text{ N/mm}^2
\]

\[
\text{Ratio (bt/ts}^2) \quad (sys/355)=200 \text{ exceeds limiting Fig. 2 value 160.}
\]

Stiffener does not comply with Clause 9.3.4.1.2.
**Location: Internal beam**

**Shape limitations for flat stiffeners (Clause 9.3.4.1.2)**

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges. Clause 9.3.4.

Flat stiffeners (Clause 9.3.4.1.2):

![Geometric Notation Figure 1](image)

Dimension b is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener \( hs = 100 \text{ mm} \)

Stiffener thickness \( ts = 8 \text{ mm} \)

Spacing of stiffeners \( b = 200 \text{ mm} \)

Plate thickness (tf or tw) \( t = 5 \text{ mm} \)

Nominal yield stress of plate \( sy = 355 \text{ N/mm}^2 \)

Nominal yield stress of stiffener \( sys = 355 \text{ N/mm}^2 \)

Ratio \( (b/t) \) \( (sy/355) = 40 \) exceeds 31.

Ratio \( (hs/ts) \) \( (sys/355) = 12.5 \) exceeds 10

Stiffener does not comply with Clause 9.3.4.1.2.
Shape limitations for flat stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Flat stiffeners (Clause 9.2.1(8)):

\[
\begin{align*}
\text{tp} & \quad \text{(thickness of flange (tf) or web (tw))} \\
\text{hs} & \quad \text{(depth of stiffener)} \\
\text{ts} & \quad \text{(thickness of stiffener)} \\
a & \quad \text{(spacing of stiffeners)}
\end{align*}
\]

Dimension a is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

- Depth of stiffener: \( hs = 100 \text{ mm} \)
- Stiffener thickness: \( ts = 10 \text{ mm} \)
- Spacing of stiffeners: \( a = 200 \text{ mm} \)
- Plate thickness (tf or tw): \( tp = 20 \text{ mm} \)
- Plate yield strength: \( f_{yp} = 345 \text{ N/mm}^2 \)
- Stiffener yield strength: \( f_{ys} = 355 \text{ N/mm}^2 \)

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

- Ratio (hs/ts): \( p = hs/ts = 10 \)
- Limit to ratio \( p = (hs/ts) \) is 13\(r=10.577 \).
- Ratio (hs/ts) = 10 is not greater than limit 13\(r=10.577 \).
- Stiffener complies with Clause 9.2.1(8).

**Stiffness check - Clause 9.3.3(3)**

- Stiffener length/depth: \( hw = 225 \text{ mm} \)
- 2nd moment of area provided: \( I_{pro} = 3.8467E6 \text{ mm}^4 \)
- 2nd moment of area required: \( I_{st} = 1.5*hw^3*tp^3/a^2 = 3.4172E6 \text{ mm}^4 \)
- As \( I_{st} \leq I_{pro} \) \( 3.4172E6 \text{ mm}^4 \leq 3.8467E6 \text{ mm}^4 \), the stiffener is OK.
Location: Internal beam

Shape limitations for flat stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Flat stiffeners (Clause 9.2.1(8)):

\[ \text{tp is the thickness of flange (tf) or web (tw)} \]
\[ \text{hs = depth of stiffener} \]
\[ \text{ts = thickness of stiffener} \]
\[ \text{a = spacing of stiffeners} \]

Dimension a is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener \( hs=100 \text{ mm} \)
Stiffener thickness \( ts=10 \text{ mm} \)
Spacing of stiffeners \( a=200 \text{ mm} \)
Plate thickness (tf or tw) \( tp=8 \text{ mm} \)
Plate yield strength \( f_{yp}=355 \text{ N/mm}^2 \)
Stiffener yield strength \( f_{ys}=355 \text{ N/mm}^2 \)

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Ratio (hs/ts) \( p=\frac{hs}{ts}=10 \)
Limit to ratio p=(hs/ts) is \( 13\pi=10.577 \).
Ratio (hs/ts)=10 is not greater than limit \( 13\pi=10.577 \).
Stiffener complies with Clause 9.2.1(8).

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth \( hw=500 \text{ mm} \)
2nd moment of area provided \( I_{pro}=2.6363E6 \text{ mm}^4 \)
2nd moment of area required \( I_{st}=1.5\times hw^3\times tp^3/a^2=2.4E6 \text{ mm}^4 \)
As \( I_{st} \leq I_{pro} \ (2.4E6 \text{ mm}^4 \leq 2.6363E6 \text{ mm}^4) \), the stiffener is OK.
**Location: Internal beam**

**Shape limitations for flat stiffeners**


Stiffeners to webs and compression flanges (Clause 9.2.1).

Flat stiffeners (Clause 9.2.1(8)):

```
  a
hs
  ts
```

*tp is the thickness of flange (tf) or web (tw)*

*hs = depth of stiffener*

*ts = thickness of stiffener*

*a = spacing of stiffeners*

Geometric Notation Figure 1

Dimension a is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

- Depth of stiffener: $hs=100$ mm
- Stiffener thickness: $ts=10$ mm
- Spacing of stiffeners: $a=200$ mm
- Plate thickness (tf or tw): $tp=8$ mm
- Plate yield strength: $f_{yp}=355$ N/mm$^2$
- Stiffener yield strength: $f_{ys}=355$ N/mm$^2$

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

- Ratio (hs/ts): $p=hs/ts=10$
- Limit to ratio $p=hs/ts$ is 13ε=10.577.
- Ratio (hs/ts)=10 is not greater than limit 13ε=10.577.
- Stiffener complies with Clause 9.2.1(8).

**Stiffness check - Clause 9.3.3(3)**

- Stiffener length/depth: $hw=400$ mm
- 2nd moment of area provided: $I_{pro}=2.6363E6$ mm$^4$
- 2nd moment of area required: $I_{st}=1.5*hw^3*tp^3/a^2=1.2288E6$ mm$^4$
- As $I_{st} \leq I_{pro}$ (1.2288E6 mm$^4 \leq 2.6363E6$ mm$^4$), the stiffener is OK.
Location: Internal beam

Shape limitations for flat stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Flat stiffeners (Clause 9.2.1(8)):

<table>
<thead>
<tr>
<th>hs</th>
<th>ts</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>tp</td>
</tr>
</tbody>
</table>

tp is the thickness of flange (tf) or web (tw)
hs = depth of stiffener
ts = thickness of stiffener
a = spacing of stiffeners

Geometric Notation Figure 1

Dimension a is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener hs=120 mm
Stiffener thickness ts=12 mm
Spacing of stiffeners a=200 mm
Plate thickness (tf or tw) tp=5 mm
Plate yield strength fyp=355 N/mm²
Stiffener yield strength fys=355 N/mm²

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Ratio (hs/ts) p=hs/ts=10
Limit to ratio p=(hs/ts) is 13ε=10.577.
Ratio (hs/ts)=10 is not greater than limit 13ε=10.577.
Stiffener complies with Clause 9.2.1(8).

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth hw=600 mm
2nd moment of area provided Ipro=3.5159E6 mm⁴
2nd moment of area required Ist=1.5*hw³*tp³/a²=1.0125E6 mm⁴
As Ist ≤ Ipro (1.0125E6 mm⁴ ≤ 3.5159E6 mm⁴), the stiffener is OK.
Shape limitations for flat stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Flat stiffeners (Clause 9.2.1(8)):

\[ \text{tp is the thickness of flange (tf) or web (tw)} \]

\[ \text{hs} = \text{depth of stiffener} \]

\[ \text{ts} = \text{thickness of stiffener} \]

\[ \text{a} = \text{spacing of stiffeners} \]

Geometric Notation Figure 1

Dimension a is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener \( hs = 100 \text{ mm} \)
Stiffener thickness \( ts = 10 \text{ mm} \)
Spacing of stiffeners \( a = 20 \text{ mm} \)
Plate thickness (tf or tw) \( tp = 2 \text{ mm} \)
Plate yield strength \( f_{yp} = 355 \text{ N/mm}^2 \)
Stiffener yield strength \( f_{ys} = 355 \text{ N/mm}^2 \)

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Ratio (hs/ts) \( p = \frac{hs}{ts} = 10 \)
Limit to ratio \( p = (hs/ts) \) is \( 13\pi = 10.577 \).
Ratio (hs/ts) = 10 is not greater than limit \( 13\pi = 10.577 \).
Stiffener complies with Clause 9.2.1(8).

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth \( hw = 300 \text{ mm} \)
2nd moment of area provided \( I_{pro} = 933385 \text{ mm}^4 \)
2nd moment of area required \( I_{st} = 1.5 \times hw^3 \times tp \times a^2 = 810000 \text{ mm}^4 \)
As \( I_{st} \leq I_{pro} \) ( \( 810000 \text{ mm}^4 \leq 933385 \text{ mm}^4 \) ), the stiffener is OK.
Location: Internal beam

Shape limitations for flat stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Flat stiffeners (Clause 9.2.1(8)):

\[
\text{tp is the thickness of flange (tf) or web (tw)}
\]

\[
\text{hs = depth of stiffener}
\]

\[
\text{ts = thickness of stiffener}
\]

\[
\text{a = spacing of stiffeners}
\]

Geometric Notation Figure 1

Dimension a is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener \( \text{hs}=100 \text{ mm} \)
Stiffener thickness \( \text{ts}=10 \text{ mm} \)
Spacing of stiffeners \( \text{a}=1000 \text{ mm} \)
Plate thickness (tf or tw) \( \text{tp}=5 \text{ mm} \)
Plate yield strength \( \text{fyp}=355 \text{ N/mm}^2 \)
Stiffener yield strength \( \text{fys}=355 \text{ N/mm}^2 \)

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Ratio (hs/ts) \( p=\text{hs/ts}=10 \)
Limit to ratio \( p=(\text{hs/ts})=13\pi=10.577. \)
Ratio (hs/ts)=10 is not greater than limit 13\pi=10.577. Stiffener complies with Clause 9.2.1(8).

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth \( \text{hw}=700 \text{ mm} \)
2nd moment of area provided \( \text{Ipro}=1.9308\text{E6 mm}^4 \)
2nd moment of area required \( \text{Ist}=0.75*\text{hw}*\text{tp}^3=65625 \text{ mm}^4 \)
As \( \text{Ist} \leq \text{Ipro} (65625 \text{ mm}^4 \leq 1.9308\text{E6 mm}^4) \), the stiffener is OK.
Shape limitations for flat stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Flat stiffeners (Clause 9.2.1(8)):

\[ hs \text{ is the depth of stiffener} \]
\[ ts \text{ is the thickness of stiffener} \]
\[ a \text{ is the spacing of stiffeners} \]

\[ \text{Geometric Notation Figure 1} \]

Dimension \( a \) is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener \( hs=100 \text{ mm} \)
Stiffener thickness \( ts=10 \text{ mm} \)
Spacing of stiffeners \( a=200 \text{ mm} \)
Plate thickness (tf or tw) \( tp=5 \text{ mm} \)
Plate yield strength \( f_{yp}=355 \text{ N/mm}^2 \)
Stiffener yield strength \( f_{ys}=355 \text{ N/mm}^2 \)

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Ratio \( (hs/ts) \)
Limit to ratio \( p=(hs/ts) \) is \( 13\pi=10.577 \).
Ratio \( (hs/ts)=10 \) is not greater than limit \( 13\pi=10.577 \).
Stiffener complies with Clause 9.2.1(8).

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth \( hw=700 \text{ mm} \)
2nd moment of area provided \( I_{pro}=1.9308E6 \text{ mm}^4 \)
2nd moment of area required \( I_{st}=1.5*hw^3*tp^3/a^2=1.6078E6 \text{ mm}^4 \)
As \( I_{st} \leq I_{pro} \) (\( 1.6078E6 \text{ mm}^4 \leq 1.9308E6 \text{ mm}^4 \)), the stiffener is OK.
Shape limitations for flat stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Flat stiffeners (Clause 9.2.1(8)):

```
   a
```

tp is the thickness of flange (tf) or web (tw)

```
hs
```

hs = depth of stiffener
ts = thickness of stiffener

```
a
```

a = spacing of stiffeners

Dimension a is the spacing of stiffeners or distance between stiffener and the beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener hs=200 mm
Stiffener thickness ts=20 mm
Spacing of stiffeners a=1967 mm
Plate thickness (tf or tw) tp=14 mm
Plate yield strength fyp=355 N/mm²
Stiffener yield strength fys=345 N/mm²

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Ratio (hs/ts) p=hs/ts=10
Limit to ratio p=(hs/ts) is 13ε=10.729.
Ratio (hs/ts)=10 is not greater than limit 13ε=10.729.
Stiffener complies with Clause 9.2.1(8).

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth hw=1000 mm
2nd moment of area provided Ipro=39.002E6 mm⁴
2nd moment of area required Ist=0.75*hw*tp^3=2.058E6 mm⁴
As Ist ≤ Ipro ( 2.058E6 mm⁴ ≤ 39.002E6 mm⁴ ), the stiffener is OK.
Shape limitations for bulb flat stiffeners (Clause 9.3.4.1.3)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges (Clause 9.3.4).

Bulb flat stiffeners (Clause 9.3.4.1.3):

\[ t \text{ is the thickness of flange (tf) or web (tw)} \]

\[ b \]

\[ hs \rightarrow \]

\[ <- ts \]

Geometric Notation Figure 1

Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener \( hs = 100 \text{ mm} \)

Span of stiffener between members \( ls = 250 \text{ mm} \)

Spacing of stiffeners \( b = 200 \text{ mm} \)

Plate thickness (tf or tw) \( t = 10 \text{ mm} \)

Nominal yield stress of plate \( sy = 355 \text{ N/mm}^2 \)

Nominal yield stress of stiffener \( sys = 355 \text{ N/mm}^2 \)

Factor \( ks = 0.4 \)

Ratio \( ls/hs = 2.5 \) does not exceed 3 \( (355/sys)=3 \).

Ratio \( ((b+ks*hs)/t) \times (sy/355)) = 24 \) does not exceed 30.0.

Stiffener complies with Clause 9.3.4.1.3.
Shape limitations for bulb flat stiffeners (Clause 9.3.4.1.3)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges (Clause 9.3.4).

Bulb flat stiffeners (Clause 9.3.4.1.3):

\[
\text{b} \quad \text{t is the thickness of flange (tf) or web (tw)}
\]

\[
\text{hs} \quad \rightarrow \quad \leftarrow \text{ts}
\]

Geometric Notation Figure 1

Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener \( \text{hs}=100 \text{ mm} \)

Span of stiffener between members \( \text{ls}=250 \text{ mm} \)

Spacing of stiffeners \( \text{b}=300 \text{ mm} \)

Plate thickness (tf or tw) \( \text{t}=10 \text{ mm} \)

Nominal yield stress of plate \( \text{sy}=355 \text{ N/mm}^2 \)

Nominal yield stress of stiffener \( \text{sys}=355 \text{ N/mm}^2 \)

Factor \( \text{ks}=0.4 \)

Ratio \( \text{ls}/\text{hs}=2.5 \) does not exceed 3 \( (355/\text{sys})=3 \).

Ratio \( \left( \text{b}+\text{ks}\times \text{hs}/\text{t} \right) \times (\text{sy}/355)=34 \) exceeds 30.0.

Stiffener complies with Clause 9.3.4.1.3.
Shape limitations for bulb flat stiffeners (Clause 9.3.4.1.3)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges (Clause 9.3.4).

Bulb flat stiffeners (Clause 9.3.4.1.3):

![Geometric Notation Figure 1]

Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

- Depth of stiffener: hs=20 mm
- Span of stiffener between members: ls=100 mm
- Spacing of stiffeners: b=200 mm
- Plate thickness (tf or tw): t=10 mm
- Nominal yield stress of plate: $\sigma_y=355 \text{ N/mm}^2$
- Nominal yield stress of stiffener: $\sigma_{ys}=355 \text{ N/mm}^2$
- Factor $k_s=0.4$

Ratio $ls/hs=5$ exceeds 3 ($355/\sigma_{ys}=3$).
Ratio $((b+k_s*hs)/t)\times (\sigma_y/355)=20.8$ does not exceed 30.0.
Stiffener complies with Clause 9.3.4.1.3.
Location: Stiffeners to plate A

Shape limitations for bulb flat stiffeners (Clause 9.3.4.1.3)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges (Clause 9.3.4).

Bulb flat stiffeners (Clause 9.3.4.1.3):

\[
\text{hs} \rightarrow \text{t} \rightarrow \text{ts}
\]

Geometric Notation Figure 1

Dimension \( b \) is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

Depth of stiffener \( \text{hs} = 20 \text{ mm} \)
Span of stiffener between members \( \text{ls} = 100 \text{ mm} \)
Spacing of stiffeners \( b = 300 \text{ mm} \)
Plate thickness (tf or tw) \( t = 10 \text{ mm} \)

Nominal yield stress of plate \( \text{sy} = 355 \text{ N/mm}^2 \)
Nominal yield stress of stiffener \( \text{sys} = 355 \text{ N/mm}^2 \)
Factor \( k_s \) \( k_s = 0.4 \)

Ratio \( \text{ls}/\text{hs} = 5 \) exceeds 3 \((355/\text{sys}) = 3.5 \)
Ratio \((b+ks*hs)/t) \times (\text{sy}/355) = 30.8 \) exceeds 30.0.
Stiffener does not comply with Clause 9.3.4.1.3.
Location: Ex1 - Bulb flat stiffeners

Transverse bulb flat stiffeners to BS EN 1993-1-5

Bulb flat stiffeners are not recommended by the author especially if warping is significant. The rules of BS EN 1993-1-5 are considered conservative and bulb flat stiffeners are unlikely to comply. Adopt one of the following alternative stiffener options instead:

- flat stiffeners see proforma 364
- angle stiffeners see proforma 366
- T-Stiffeners see proforma 367
Location: Ex1 - Angle stiffeners to compression flange

Shape limitations for angle stiffeners (Clause 9.3.4.1.4)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.4.1.4.

Stiffeners to webs and compression flanges. Clause 9.3.4

Angle stiffeners (Clause 9.3.4.1.4):

\[ \begin{align*}
\text{hs} & \quad (\rightarrow) \quad <t_s \\
\text{bs} & \quad (\downarrow)
\end{align*} \]

Dimension \( b \) is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

Nominal yield stress of plate \( \sigma_y = 355 \text{ N/mm}^2 \)
Nominal yield stress of stiffener \( \sigma_{ys} = 355 \text{ N/mm}^2 \)

Dimension \( bs=40 \) does not exceed \( hs=100 \)
Clause 9.3.4.1.4 Condition (a) is satisfied.

Factor \( (bs/ts) \ (\sigma_{ys}/355) = 10 \) does not exceed 11.0
Clause 9.3.4.1.4 Condition (b) is satisfied.

Factor \( (b/t) \ (\sigma_{ys}/355) = 20 \) does not exceed 30.0
There is no limitation on the value of the factor \( (hs/ts) \ (\sigma_{ys}/355) \)
Clause 9.3.4.1.4 Condition (c) is satisfied.
Stiffener complies with Clause 9.3.4.1.4.
Location: Ex2 - Angle stiffeners to compression flange

Shape limitations for angle stiffeners (Clause 9.3.4.1.4)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.4.1.4.

Stiffeners to webs and compression flanges. Clause 9.3.4

Angle stiffeners (Clause 9.3.4.1.4):

![Geometric Notation Figure 1]

Dimension $b$ is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

Nominal yield stress of plate $\sigma_y = 355\, \text{N/mm}^2$
Nominal yield stress of stiffener $\sigma_{ys} = 355\, \text{N/mm}^2$

Dimension $bs = 50$ does not exceed $hs = 100$
Clause 9.3.4.1.4 Condition (a) is satisfied.

Factor $(bs/t)$ $(\sigma_{ys}/355) = 12.5$ exceeds 11.0
Clause 9.3.4.1.4 Condition (b) is not satisfied.
Stiffener does not comply with Clause 9.3.4.1.4.
Location: Ex3 - Angle stiffeners to compression flange

Shape limitations for angle stiffeners (Clause 9.3.4.1.4)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.4.1.4.

Stiffeners to webs and compression flanges. Clause 9.3.4

Angle stiffeners (Clause 9.3.4.1.4):

\[ b \]

[Geometric Notation Figure 1]

Dimension \( b \) is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

Nominal yield stress of plate \( \sigma_y = 355 \text{ N/mm}^2 \)
Nominal yield stress of stiffener \( \sigma_{ys} = 355 \text{ N/mm}^2 \)

Dimension \( bs = 110 \) exceeds \( hs = 100 \)
Clause 9.3.4.1.4 Condition (a) is not satisfied.

Factor \( (bs/ts) (\sigma_{ys}/355) = 27.5 \) exceeds 11.0
Clause 9.3.4.1.4 Condition (b) is not satisfied.
Stiffener does not comply with Clause 9.3.4.1.4.
Location: Ex4 - Angle stiffeners to compression flange

Shape limitations for angle stiffeners (Clause 9.3.4.1.4)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.4.1.4.

Stiffeners to webs and compression flanges. Clause 9.3.4

Angle stiffeners (Clause 9.3.4.1.4):

\[ b \]

\[ t \]

\[ t \]

\[ h_s \rightarrow <t_s \]

\[ b_s \]

Geometric Notation Figure 1

Dimension \( b \) is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

Nominal yield stress of plate \( \sigma_y \) \( s_y = 355 \) N/mm\(^2\)
Nominal yield stress of stiffener \( s_y = 355 \) N/mm\(^2\)

Dimension \( b_s = 40 \) does not exceed \( h_s = 100 \)
Clause 9.3.4.1.4 Condition (a) is satisfied.

Factor \( (b_s/t_s) (s_y/355) = 10 \) does not exceed 11.0
Clause 9.3.4.1.4 Condition (b) is satisfied.

From Appendix G.3.
With \( (L_s/b_s) (s_y/355) = 12 \) the upper limit to factor \( (h_s/t_s) (s_y/355) \) is 22
Slenderness ratio \( (h_s/t_s) (s_y/355) = 25 \) exceeds the upper limit 22
Clause 9.3.4.1.4 Condition (c)(2) is not satisfied.
Stiffener does not comply with Clause 9.3.4.1.4.
Shape limitations for angle stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Angle stiffeners:

```
+-----------------+     +-----------------+
|                 |     |                 |
|                 |     |                 |
| b               |     |                   |
|                 |     |                   |
|                 |     |                   |
|                 |     |                   |
```

Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

Plate yield strength \( f_{yp} = 355 \text{ N/mm}^2 \)

Stiffener yield strength \( f_{ys} = 355 \text{ N/mm}^2 \)

**Stiffness check - Clause 9.3.3(3)**

- Stiffener length/depth \( h_w = 450 \text{ mm} \)
- Second moment of area provided \( I_{pro} = 6.2152E6 \text{ mm}^4 \)
- Second moment of area required \( I_{st} = 1.5 \ast h_w^3 \ast t_p^3 / a^2 = 1.4762E6 \text{ mm}^4 \)

As \( I_{st} \leq I_{pro} \) \((1.4762E6 \text{ mm}^4 \leq 6.2152E6 \text{ mm}^4)\), the stiffener is OK.
Shape limitations for angle stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Angle stiffeners:

\[
\begin{align*}
\text{Dimension } b & \text{ is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.} \\
\text{Plate yield strength} & = f_{yp} = 355 \text{ N/mm}^2 \\
\text{Stiffener yield strength} & = f_{ys} = 355 \text{ N/mm}^2 \\
\text{Stiffness check - Clause 9.3.3(3)} \\
\text{Stiffener length/depth} & = hw = 450 \text{ mm} \\
\text{Second moment of area provided} & = I_{pro} = 6.9785 \times 10^6 \text{ mm}^4 \\
\text{Second moment of area required} & = I_{st} = 1.5 \times hw^3 \times tp^3 / a^2 = 1.4762 \times 10^6 \text{ mm}^4 \\
\text{As } I_{st} & \leq I_{pro} (1.4762 \times 10^6 \text{ mm}^4 \leq 6.9785 \times 10^6 \text{ mm}^4), \text{ the stiffener is OK.}
\end{align*}
\]
Shape limitations for angle stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Angle stiffeners:

Dimension $b$ is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

Plate yield strength

Stiffener yield strength

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth
Second moment of area provided
Second moment of area required
As $I_{st} \leq I_{pro}$ (5.1258E6 mm$^4 \leq 13.523E6$ mm$^4$), the stiffener is OK.
Location: Ex4 - Angle stiffeners to compression flange

Shape limitations for angle stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Angle stiffeners:

\[
\text{tp} \quad \text{is the thickness of flange (tf) or web (tw)}
\]

\[
\begin{align*}
\text{hs} & \rightarrow < - \text{ts} \\
\downarrow & \\
\text{bs} & \rightarrow \text{ts}
\end{align*}
\]

Geometric Notation Figure 1

Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

Plate yield strength \( f_{yp} = 355 \text{ N/mm}^2 \)
Stiffener yield strength \( f_{ys} = 355 \text{ N/mm}^2 \)

**Stiffness check - Clause 9.3.3(3)**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiffener length/depth</td>
<td>( hw = 400 \text{ mm} )</td>
</tr>
<tr>
<td>Second moment of area provided</td>
<td>( I_{pro} = 6.965 \times 10^6 \text{ mm}^4 )</td>
</tr>
<tr>
<td>Second moment of area required</td>
<td>( I_{st} = 1.5 \times hw^3 \times tp^3 / a^2 = 2.6449 \times 10^6 \text{ mm}^4 )</td>
</tr>
</tbody>
</table>

As \( I_{st} \leq I_{pro} \) (\( 2.6449 \times 10^6 \text{ mm}^4 \leq 6.965 \times 10^6 \text{ mm}^4 \)), the stiffener is OK.
Shape limitations for angle stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Angle stiffeners:

```
hs   ->   <-ts
      /  \
      b   
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Shape limitations for angle stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Angle stiffeners:

![Diagram of angle stiffener](image)

Geometric Notation Figure 1

Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of non-uniform spacing the average value on the two sides may be taken.

- Plate yield strength $f_{yp} = 355 \text{ N/mm}^2$
- Stiffener yield strength $f_{ys} = 355 \text{ N/mm}^2$
- $L = 1400 \text{ mm}$

**Stiffness check - Clause 9.3.3(3)**

- Stiffener length/depth $h_w = 600 \text{ mm}$
- Second moment of area provided $I_{pro} = 9.9923E6 \text{ mm}^4$
- Second moment of area required $I_{st} = 1.5 \times h_w^3 \times t_p^3 / a^2 = 3.6E6 \text{ mm}^4$

As $I_{st} \leq I_{pro}$ ($3.6E6 \text{ mm}^4 \leq 9.9923E6 \text{ mm}^4$), the stiffener is OK.
Location: Ex1 - Web stiffener

Shape limitations for tee stiffeners (Clause 9.3.4.1.5)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.4.1.5.

Stiffeners to webs and compression flanges. Clause 9.3.4

Tee stiffeners. Clause 9.3.4.1.5

\[ t \text{ is the thickness of flange (tf) or web (tw)} \]

\[ \text{Dimension } b \text{ is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.} \]

Effective stiffener depth is measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee stiffener.

Nominal yield stress of plate \( \sigma_y \) \( \text{sy}=355 \text{ N/mm}^2 \)

Nominal yield stress of stiffener \( \text{syst}=355 \text{ N/mm}^2 \)

Factor \( \left( \frac{bso}{tso} \right) \left( \frac{\text{syst}}{355} \right) = 8 \) does not exceed 10

Condition (a) is satisfied.

Factor \( \left( \frac{ds}{ts} \right) \left( \frac{\text{sy}+\text{sa}}{355} \right) = 7.0711 \) does not exceed 41

Condition (b) is satisfied.

Factor \( \left( \frac{ds}{ts} \right) \left( \frac{\text{sy}}{355} \right) = 5 \) does not exceed 7

Condition (c) is satisfied.

Stiffener complies with the requirements of Clause 9.3.4.1.5.
Shape limitations for tee stiffeners (Clause 9.3.4.1.5)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.4.1.5.

Stiffeners to webs and compression flanges. Clause 9.3.4

Tee stiffeners. Clause 9.3.4.1.5

Effective stiffener depth is measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee stiffener.

Nominal yield stress of plate \( \sigma_y = 355 \text{ N/mm}^2 \)
Nominal yield stress of stiffener \( \sigma_{ys} = 355 \text{ N/mm}^2 \)
Factor (bso/tso) \( (\sigma_{ys}/355) = 12 \) exceeds 10
Condition (a) is not satisfied.
Factor (ds/ts) \( ((\sigma_{ys}+\sigma_a)/355) = 7.0711 \) does not exceed 41
Condition (b) is satisfied.
Factor (ds/ts) \( (\sigma_{ys}/355) = 5 \) does not exceed 7
Condition (c) is satisfied.
Stiffener does not comply with the requirements of Clause 9.3.4.1.5.
Location: Web stiffener

Shape limitations for tee stiffeners (Clause 9.3.4.1.5)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.4.1.5.

Stiffeners to webs and compression flanges. Clause 9.3.4

Tee stiffeners. Clause 9.3.4.1.5

\[
\text{t is the thickness of flange (tf) or web (tw)}
\]

\[
\text{Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.}
\]

Effective stiffener depth is measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee stiffener.

Nominal yield stress of plate \( \sigma_y \) \( sy=355 \text{ N/mm}^2 \)

Nominal yield stress of stiffener \( \sigma_y=355 \text{ N/mm}^2 \)

Factor \( (bso/tso) (\sigma_y/355)=8 \) does not exceed 10

Condition (a) is satisfied

Factor \( (ds/ts) ((\sigma_y+\sigma_a)/355)=38.184 \) does not exceed 41

Condition (b) is satisfied.

Condition (c) (1) is not satisfied.

Factor \( (b/t) (\sigma_y/355)=25 \) does not exceed 32

Factor \( (ds/bs)=3 \) does not exceed 4

Limiting value of factor \( (ds/ts) (\sigma_y/355) \) from Figure 4b is 29.384

Factor \( (ds/ts) (\sigma_y/355)=27 \) does not exceed 29.384

Condition (c) (2) is satisfied.

Stiffener complies with the requirements of Clause 9.3.4.1.5.
Location: Web stiffener

Shape limitations for tee stiffeners (Clause 9.3.4.1.5)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.4.1.5.

Stiffeners to webs and compression flanges. Clause 9.3.4

Tee stiffeners. Clause 9.3.4.1.5

Effective stiffener depth is measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee stiffener.

Nominal yield stress of plate $\sigma_y$ $\text{sy}=355 \text{ N/mm}^2$

Nominal yield stress of stiffener $\text{sys}=355 \text{ N/mm}^2$

Factor ($bso/tso$) ($\text{sys}/355$)$=8$ does not exceed 10

Condition (a) is satisfied

Factor ($ds/ts$) ($\text{(sys+sa)}/355$)$=10.607$ does not exceed 41

Condition (b) is satisfied.

Factor ($ls/bs$) ($\text{sys}/355$)$=21.778$ is greater than 12 and less than 25

Limiting value of factor ($ds/ts$) ($\text{sys}/355$) from Figure 4a is 7.5472

Factor ($ds/ts$) ($\text{sys}/355$)$=7.5$ does not exceed 7.5472

Condition (c) (1) is satisfied.

Factor ($b/t$) ($\text{sys}/355$)$=25$ does not exceed 32

Factor ($ds/bs$)$=0.83333$ does not exceed 4

Limiting value of factor ($ds/ts$) ($\text{sys}/355$) from Figure 4b is 29.384

Factor ($ds/ts$) ($\text{sys}/355$)$=7.5$ does not exceed 29.384

Condition (c) (2) is satisfied.

Stiffener complies with the requirements of Clause 9.3.4.1.5.
Location: Web stiffener

Shape limitations for tee stiffeners (Clause 9.3.4.1.5)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.4.1.5.

Stiffeners to webs and compression flanges. Clause 9.3.4

Tee stiffeners. Clause 9.3.4.1.5

\[ t \text{ is the thickness of flange (tf) or web (tw)} \]

Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Effective stiffener depth is measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee stiffener.

Nominal yield stress of plate \( \sigma_y \) \( sy = 355 \text{ N/mm}^2 \)

Nominal yield stress of stiffener \( sy = 355 \text{ N/mm}^2 \)

Factor \( (bso/tso) (sys/355) = 8 \) does not exceed 10
Condition (a) is satisfied

Factor \( (ds/ts) ((sys+oa)/355) = 10.889 \) does not exceed 41
Condition (b) is satisfied.

Condition (c) (1) is not satisfied.
Factor \( (b/t) (\sigma_y/355) = 25 \) does not exceed 32
Factor \( (ds/bs) = 0.85556 \) does not exceed 4

Limiting value of factor \( (ds/ts) (sys/355) \) from Figure 4b is 29.384
Factor \( (ds/ts) (sys/355) = 7.7 \) does not exceed 29.384
Condition (c) (2) is satisfied.

Stiffener complies with the requirements of Clause 9.3.4.1.5.
Shape Limitations for T-Stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Tee stiffeners:

\[ b \]

\[ t_p \] is the thickness of flange (tf) or web (tw)

\[ d_s \] \[ \rightarrow \] \[ \leftarrow t_s \]

Dimension \( b \) is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Effective stiffener depth is measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee stiffener.

Plate yield strength \( f_{yp} = 345 \) N/mm\(^2\)

Stiffener yield strength \( f_{ys} = 355 \) N/mm\(^2\)

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth \( h_w = 300 \) mm

Second moment of area provided \( I_{pro} = 2.6239E6 \) mm\(^4\)

Second moment of area required \( I_{st} = 0.75 \times h_w \times t_p^3 = 1.8E6 \) mm\(^4\)

As \( I_{st} \leq I_{pro} \) \( (1.8E6 \text{ mm}^4 \leq 2.6239E6 \text{ mm}^4) \), the stiffener is OK.
Location: Ex2 - Web stiffener

Shape Limitations for T-Stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Tee stiffeners:

\[ \text{tp is the thickness of flange (tf) or web (tw)} \]

Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Effective stiffener depth is measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee stiffener.

Plate yield strength \( f_{yp} = 345 \text{ N/mm}^2 \)
Stiffener yield strength \( f_{ys} = 355 \text{ N/mm}^2 \)
\( L = 3000 \text{ mm} \)
**Shape Limitations for T-Stiffeners**


Stiffeners to webs and compression flanges (Clause 9.2.1).

Tee stiffeners:

![Geometric Notation Figure 1](image)

Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate. In the case of a non-uniform spacing the average value on the two sides may be taken.

Effective stiffener depth is measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee stiffener.

- Plate yield strength \( f_{yp} = 345 \text{ N/mm}^2 \)
- Stiffener yield strength \( f_{ys} = 345 \text{ N/mm}^2 \)
Location: Ex4 - Web stiffener

Shape Limitations for T-Stiffeners


Stiffeners to webs and compression flanges (Clause 9.2.1).

Tee stiffeners:

\[ b \]

\[ t_p \] is the thickness of flange (tf) or web (tw)

\[ d_s \]

\[ t_s \]

\[ t_s \]

\[ b_s \]

Dimension b is the spacing of stiffeners or distance between the stiffener and beam flange/web boundary, as appropriate.

In the case of a non-uniform spacing the average value on the two sides may be taken.

Effective stiffener depth is measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee stiffener.

Plate yield strength \( f_{yp} = 345 \text{ N/mm}^2 \)

Stiffener yield strength \( f_{ys} = 355 \text{ N/mm}^2 \)

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth \( h_w = 600 \text{ mm} \)

Second moment of area provided \( I_{pro} = 11.84E6 \text{ mm}^4 \)

Second moment of area required \( I_{st} = 1.5 \times h_w^3 \times t_p^2 / a^2 = 10.368E6 \text{ mm}^4 \)

As \( I_{st} \leq I_{pro} \) (10.368E6 mm\(^4\) \leq 11.84E6 mm\(^4\)), the stiffener is OK.
Location: Ex5 - Web stiffener

Shape Limitations for T-Stiffeners


Stiffeners towebs and compression flanges (Clause 9.2.1).

Tee stiffeners:

\[
\begin{align*}
\text{ds} & \rightarrow \text{ts} \\
tso & \rightarrow \text{bs}
\end{align*}
\]

Effective stiffener depth is measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee stiffener.

Plate yield strength \( f_{yp} = 345 \text{ N/mm}^2 \)
Stiffener yield strength \( f_{ys} = 355 \text{ N/mm}^2 \)
\( L = 1500 \text{ mm} \)

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth \( hw = 450 \text{ mm} \)
Second moment of area provided \( I_{pro} = 5.5936E6 \text{ mm}^4 \)
Second moment of area required \( I_{st} = 1.5 \times hw^3 \times tp^3 / a^2 = 4.374E6 \text{ mm}^4 \)
As \( I_{st} \leq I_{pro} \) ( \( 4.374E6 \text{ mm}^4 \leq 5.5936E6 \text{ mm}^4 \)), the stiffener is OK.
Shape limitations for closed stiffeners (Clause 9.3.4.1.5)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges (Clause 9.3.4).

Closed stiffeners (Clause 9.3.4.2):

Width of wall ds1=75 mm
Width of wall ds2=75 mm
Thickness of stiffener ts=3 mm

Stiffener:
Nominal yield stress of stiffener sys=355 N/mm²
Stress at centroid of section sa=sys=355 N/mm²
Factor (ds1/ts) (sys/355)=25 does not exceed 29.0
Clause 9.3.4.2 Condition (a) is satisfied.
Factor (ds2/ts) ((sys+oa)/355)=35.355 does not exceed 41.0
Clause 9.3.4.2 Condition (b) is satisfied.
Stiffener complies with Clause 9.3.4.2.
Location: Stiffened compression flange

Shape limitations for closed stiffeners (Clause 9.3.4.1.5)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges (Clause 9.3.4).

Closed stiffeners (Clause 9.3.4.2):

\[ b_1 \] \[ b_2 \] \[ b_1 \]

\[ \text{Geometric Notation Figure 1} \]

Width of wall \( ds_1 = 100 \text{ mm} \)
Width of wall \( ds_2 = 75 \text{ mm} \)
Thickness of stiffener \( ts = 3 \text{ mm} \)

Stiffener:

Nominal yield stress of stiffener \( sys = 355 \text{ N/mm}^2 \)
Stress \( sa \) at centroid of section \( sa = sys = 355 \text{ N/mm}^2 \)
Factor (\( ds_1/ ts \)) \((sys/355) = 33.333 \) is greater than 29.0
Stiffener does not comply with Clause 9.3.4.2 Condition (a).

Factor (\( ds_2/ ts \)) \(((sys+oa)/355) = 35.355 \) does not exceed 41.0
Clause 9.3.4.2 Condition (b) is satisfied.
Stiffener does not comply with Clause 9.3.4.2.
Location: Stiffened compression flange

Shape limitations for closed stiffeners (Clause 9.3.4.1.5)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Stiffeners to webs and compression flanges (Clause 9.3.4).

Closed stiffeners (Clause 9.3.4.2):

![Geometric Notation Figure 1]

Width of wall
- $d_{s1}=75$ mm
- $d_{s2}=95$ mm

Thickness of stiffener
- $t_{s}=3$ mm

Stiffener:
Nominal yield stress of stiffener $s_{ys}=355$ N/mm$^2$
Stress at centroid of section $s_a=s_{ys}=355$ N/mm$^2$
Factor $(d_{s1}/t_{s}) (s_{ys}/355)=25$ does not exceed 29.0
Clause 9.3.4.2 Condition (a) is satisfied.
Factor $(d_{s2}/t_{s}) ((s_{ys}+s_a)/355)=44.783$ is greater than 41.0
Stiffener does not comply with Clause 9.3.4.2 Condition (b).
Stiffener does not comply with Clause 9.3.4.2.
Location: Ex1 - Stiffened compression flange

Shape limitations for closed stiffeners


Stiffeners to webs and compression flanges (Clause 9.3.3).

Closed stiffeners:

\[
\begin{array}{c|c|c|c}
\hline
& b1 & b2 & b1 \\
\hline
ds & / & / & /
\hline
ds2 & / & / & /
\hline
ds1 & is ts and & measures on & centreline
\hline
tp & be constant & & \\
\hline
\end{array}
\]

Geometric Notation

Width of wall \( ds1 = 100 \text{ mm} \)
Width of wall \( ds2 = 100 \text{ mm} \)
Distance \( b1 = 200 \text{ mm} \)
Distance \( b2 = 200 \text{ mm} \)
Distance \( ds = 80 \text{ mm} \)
Plate thickness \( tp = 15 \text{ mm} \)
Thickness of stiffener \( ts = 10 \text{ mm} \)
Plate yield strength \( f_{yp} = 355 \text{ N/mm}^2 \)
Stiffener yield strength \( f_{ys} = 355 \text{ N/mm}^2 \)

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth \( hw = 600 \text{ mm} \)
compute \( \text{comp1} = A1 \cdot y1^2 + 2 \cdot A2 \cdot y2^2 + A3 \cdot y3^2 \)
= \( 8.4681 \times 10^6 \)
compute \( \text{comp2} = fw \cdot tp^3 / 12 + 2 \cdot ts \cdot ds^3 / 12 + ds1 \cdot ts^3 / 12 = 974167 \)
Second moment of area provided \( I_{pro} = \text{comp1} + \text{comp2} = 9.4422 \times 10^6 \text{ mm}^4 \)
Second moment of area required \( I_{st} = 1.5 \cdot hw \cdot tp^3 / a^2 = 6.8344 \times 10^6 \text{ mm}^4 \)
As \( I_{st} \leq I_{pro} \) (\( 6.8344 \times 10^6 \text{ mm}^4 \leq 9.4422 \times 10^6 \text{ mm}^4 \)), the stiffener is OK.
Shape limitations for closed stiffeners


Stiffeners to webs and compression flanges (Clause 9.3.3).

Closed stiffeners:

\[
\begin{array}{ccc}
  & b_1 & b_2 & b_1 \\
\hline
ds & 100 & 120 & 340 \\
ds_2 & 80 & 15 & 15 \\
\end{array}
\]

Geometric Notation

- Width of wall: ds1=100 mm
- Width of wall: ds2=120 mm
- Distance: b1=280 mm
- Distance: b2=280 mm
- Distance: ds=80 mm
- Plate thickness: tp=15 mm
- Thickness of stiffener: ts=15 mm
- Plate yield strength: fyp=275 N/mm\(^2\)
- Stiffener yield strength: fys=275 N/mm\(^2\)

Stiffness check - Clause 9.3.3(3)

Stiffener length/depth: hw=900 mm

\[
\begin{align*}
\text{comp1} &= A_1 y_1^2 + A_2 y_2^2 + A_3 y_3^2 \\
&= 12.84E6 \\
\text{comp2} &= f_w t_p^3/12 + 2 t_s d_s^3/12 + d_{s1} t_s^3/12 \\
&= 1.4656E6
\end{align*}
\]

Second moment of area provided: Ipro=comp1+comp2=14.305E6 mm\(^4\)

Second moment of area required: Ist=1.5 * hw \times tp^3/a^2 = 11.768E6 mm\(^4\)

As Ist \leq Ipro (11.768E6 mm\(^4\) \leq 14.305E6 mm\(^4\)), the stiffener is OK.
**Location: Ex1 - Beam F**

**Shape limitations for flanges curved in elevation (Clause 9.3.5)**

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Flanges curved in elevation (Clause 9.3.5).
Flange outstand:
Section comprises flange plates connected to the web by riveted/bolted connections.

```
  tfo1
  >|< bfo1
```

Case 1: bfo=bfo1, tfo=tfo1
Case 2: bfo=bfo2, tfo=tfo2

Thickness tfo is the mean thickness of the outstand.

Dimension bfo is the width of outstand measured from the edge to the nearest line of rivets or bolts connecting it to the supporting part of the member.

```
  tfo2
  >|< bfo2
```

Dimension of outstand bfo=200 mm
Thickness of plate tfo=10 mm

Radius of curvature of the flange Rf=25000 mm

Factor bfo²/tfo=4000 does not exceed factor Rf/6=4166.7
Flange complies with Clause 9.3.5 Condition (a).
Shape limitations for flanges curved in elevation (Clause 9.3.5)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Flanges curved in elevation (Clause 9.3.5).
Flange outstand:
Section comprises flange plates welded to web.
Flange is a single plate.

Thickness \( t_{fo} \) is the mean thickness of the outstand

Dimension \( b_{fo} \) is the width of the outstand measured from the edge to the surface of the web.

Dimension of outstand \( b_{fo} = 200 \) mm
Thickness of plate \( t_{fo} = 8 \) mm

Radius of curvature of the flange \( R_f = 25000 \) mm

Factor \( b_{fo}^2/t_{fo} = 5000 \) is greater than factor \( R_f/6 = 4166.7 \)

Flange does not comply with Clause 9.3.5 Condition (a).
Shape limitations for flanges curved in elevation (Clause 9.3.5)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Flanges curved in elevation (Clause 9.3.5).

Section is the flange of an internal panel.

\[
\begin{align*}
\text{Dimension of plate} & \quad b=200 \text{ mm} \\
\text{Thickness of plate} & \quad t_f=10 \text{ mm} \\
\text{Radius of curvature of the flange} & \quad R_f=25000 \text{ mm} \\
\text{Factor } b^2/t_f & \leq 4000 \text{ does not exceed factor } R_f/2=12500 \\
\text{Flange complies with Clause 9.3.5 Condition (b).}
\end{align*}
\]
Shape limitations for flanges curved in elevation


Flanges are curved in elevation (Clause 8(2)).
Section comprises of flange plates bolted to the web:

Comp flange radius of curvature \( r = 5000 \text{ mm} \)
Clear web depth between flanges \( hw = 800 \text{ mm} \)
Thickness of web \( tw = 16 \text{ mm} \)
Compression flange width \( bcf = 450 \text{ mm} \)
Compression flange thickness \( tcf = 35 \text{ mm} \)
Yield strength of flange \( f_yf = 345 \text{ N/mm}^2 \)

From BS EN 1993-1-5:2006, Clause 8(2):
LHS term of expression 8.2 \( \text{ratio1} = hw/tw = 50 \)
compute \( \text{term1} = \sqrt{Aw/Afc} = 0.9015 \)
compute \( \text{term2} = hw^2E/(3r^2fyf) = 32.464 \)
RHS term of expression 8.2 \( \text{ratio2} = kE/f_yf*\text{term1}/(1+\text{term2})^{0.5} \)
\[ = 52.172 \]

As \( hw/tw \leq \text{ratio2} \ (50 \leq 52.172) \), the criterion of Clause 8(2) has been met. Hence satisfactory.
Shape limitations for flanges curved in elevation


Flanges are curved in elevation (Clause 8(2)).
Section comprises of single flange plates welded to web:

```
 bcf          tcf
  hw
```

Comp flange radius of curvature  \(r=4000 \text{ mm}\)
Clear web depth between flanges  \(hw=900 \text{ mm}\)
Thickness of web  \(tw=15 \text{ mm}\)
Compression flange width  \(bcf=400 \text{ mm}\)
Compression flange thickness  \(tcf=15 \text{ mm}\)
Yield strength of flange  \(fyf=355 \text{ N/mm}^2\)

From BS EN 1993-1-5:2006, Clause 8(2):
LHS term of expression 8.2  \(\text{ratio}_1=hw/tw=60\)
compute  \(\text{term}_1=\sqrt{Aw/A\text{fc}}=1.5\)
compute  \(\text{term}_2=hw*E/(3*r*fyf)=44.366\)
RHS term of expression 8.2  \(\text{ratio}_2=k*E/fyf*\text{term}_1/(1+\text{term}_2)^0.5=72.457\)

As \(hw/tw \leq \text{ratio}_2\) (60 \(\leq 72.457\)), the criterion of Clause 8(2) has been met. Hence satisfactory.
Location: Ex3 - Box girder

Shape limitations for flanges curved in elevation


Flanges are curved in elevation (Clause 8(2)).

Section is the flange of an internal panel:

\[
\begin{align*}
&bcf \quad bcf \\
&\text{tcf} = \text{thickness of panel between box webs} \\
&bcf = \text{half the width of effective flange between box webs} \\
&tw \quad tw
\end{align*}
\]

Comp flange radius of curvature \( r = 4000 \text{ mm} \)
Clear web depth between flanges \( hw = 900 \text{ mm} \)
Thickness of web \( tw = 15 \text{ mm} \)
Compression flange width \( bcf = 450 \text{ mm} \)
Compression flange thickness \( tcf = 15 \text{ mm} \)
Yield strength of flange \( f_yf = 355 \text{ N/mm}^2 \)

From BS EN 1993-1-5:2006, Clause 8(2):

LHS term of expression 8.2 \( \text{ratio1} = hw/tw = 60 \)
compute \( \text{term1} = \sqrt{(Aw/Afc)} = 1.4142 \)
compute \( \text{term2} = hwE/(3*r*fyf) = 44.366 \)
RHS term of expression 8.2 \( \text{ratio2} = k*E/fyf*term1/(1+term2)^{0.5} = 68.313 \)

As \( hw/tw \leq \text{ratio2} \ (60 \leq 68.313) \), the criterion of Clause 8(2) has been met. Hence satisfactory.
Shape limitations for flanges curved in elevation


Flanges are curved in elevation (Clause 8(2)).
Section comprises of flange plates welded to compression web:

\[
\begin{align*}
2*b_{fo} & = \text{width of outstand measured from the edge to the surface of the web.} \\
& \text{For all other plates } "b_{fo}" \text{ should be taken as half the width between the welds connecting it to an inner plate.}
\end{align*}
\]

Comp flange radius of curvature \( r = 5000 \text{ mm} \)
Clear web depth between flanges \( h_w = 400 \text{ mm} \)
Thickness of web \( t_w = 15 \text{ mm} \)
Dimension of outstand \( b_{fo} = 200 \text{ mm} \)
Thickness of inner plate \( t_{cf1} = 20 \text{ mm} \)
Thickness of outer plate \( t_{cf2} = 15 \text{ mm} \)
Yield strength of flange \( f_{yf} = 345 \text{ N/mm}^2 \)

From BS EN 1993-1-5:2006, Clause 8(2):

LHS term of expression 8.2 \( \text{ratio1} = \frac{h_w}{t_w} = 26.667 \)
compute \( \text{term1} = \sqrt{\frac{A_{w}}{A_{fc}}} = 0.64775 \)
compute \( \text{term2} = \frac{h_w E}{3 r f_{yf}} = 16.232 \)
RHS term of expression 8.2
\[
\begin{align*}
\text{ratio2} & = \frac{k E}{f_{yf} \text{term1}/(1+\text{term2})^{0.5}} \\
& = 52.24
\end{align*}
\]

As \( \frac{h_w}{t_w} \leq \text{ratio2} \) (26.667 \leq 52.24), the criterion of Clause 8(2) has been met. Hence satisfactory.
Location: Ex5 - Rolled section

Shape limitations for flanges curved in elevation


Flanges are curved in elevation (Clause 8(2)).
Rolled section:

\[
\begin{align*}
&\text{bcf} \\
&\text{tcf} \\
&\text{hw} \\
&\text{bcf = comp flange width} \\
&\text{tcf = comp flange thickness}
\end{align*}
\]

Comp flange radius of curvature \( r = 5000 \text{ mm} \)
Clear web depth between flanges \( hw = 1000 \text{ mm} \)
Thickness of web \( tw = 15 \text{ mm} \)
Compression flange width \( bcf = 400 \text{ mm} \)
Compression flange thickness \( tcf = 15 \text{ mm} \)
Yield strength of flange \( fyf = 355 \text{ N/mm}^2 \)

From BS EN 1993-1-5:2006, Clause 8(2):

LHS term of expression 8.2 \[ \text{ratio1}=hw/tw=66.667 \]
compute \[ \text{term1}=\sqrt{Aw/Afc}=1.5811 \]
compute \[ \text{term2}=hw*E/(3*r*fyf)=39.437 \]
RHS term of expression 8.2 \[ \text{ratio2}=k*E/fyf*term1/(1+term2)^0.5 = 80.898 \]

As \( hw/tw \leq \text{ratio2} (66.667 \leq 80.898)\), the criterion of Clause 8(2) has been met. Hence satisfactory.
Shape limitations for circular hollow sections (Clause 9.3.6)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.6.

Circular hollow sections. Clause 9.3.6.

The ratio of outside diameter to wall thickness of a circular hollow section should not exceed 60 \((355/\sigma)\).

Nominal yield stress \(\sigma_y\) \(\text{sys}=355 \text{ N/mm}^2\)
Outside diameter of section \(\text{od}=170 \text{ mm}\)
Wall thickness \(\text{wt}=5 \text{ mm}\)
Ratio of outside diameter to wall thickness \(34\) is less than \(60 \ (355/\sigma)=60\)
Section complies with Clause 9.3.6.

Compact sections. Clause 9.3.7.4.
Nominal yield stress \(\sigma_y\) \(\text{syc}=355 \text{ N/mm}^2\)
Ratio of outside diameter to wall thickness \(34\) is less than \(46 \ (355/\sigma)=46\)
Section complies with Clause 9.3.7.4.
Section is compact.
Shape limitations for circular hollow sections (Clause 9.3.6)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.6.

Circular hollow sections. Clause 9.3.6.

The ratio of outside diameter to wall thickness of a circular hollow section should not exceed 60 \((355/\sigma)\).

Nominal yield stress \(\sigma_y\) \(s_{ys}=300\) N/mm\(^2\)
Outside diameter of section \(od=240\) mm
Wall thickness \(wt=5\) mm
Ratio of outside diameter to wall thickness \(48\) is less than \(60 (355/\sigma)=65.269\)
Section complies with Clause 9.3.6.

Compact sections. Clause 9.3.7.4.

Nominal yield stress \(\sigma_y\) \(s_{yc}=355\) N/mm\(^2\)
Ratio of outside diameter to wall thickness \(48\) exceeds \(46 (355/\sigma y)=46\)
Section does not comply with Clause 9.3.7.4.
Section is not compact.
Shape limitations for circular hollow sections (Clause 9.3.6)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma checks for compliance with the shape limitations to Clause 9.3.6.

Circular hollow sections. Clause 9.3.6.

The ratio of outside diameter to wall thickness of a circular hollow section should not exceed 60 \( \left( \frac{355}{\sigma} \right) \).

Nominal yield stress \( \sigma_y \) \( \text{sys}=355 \text{ N/mm}^2 \)

Outside diameter of section \( \text{od}=300 \text{ mm} \)

Wall thickness \( \text{wt}=4 \text{ mm} \)

Ratio of outside diameter to wall thickness 75 exceeds 60 \( \left( \frac{355}{\sigma} \right)=60 \)

Section does not comply with Clause 9.3.6.


For compliance \( \sigma_y= \left( \frac{60}{\gamma} \right)^2 \times 355=227.2 \text{ N/mm}^2 \)

Compact sections. Clause 9.3.7.4.

Nominal yield stress \( \sigma_y \) \( \text{syc}=355 \text{ N/mm}^2 \)

Ratio of outside diameter to wall thickness 75 exceeds 46 \( \left( \frac{355}{\sigma_y} \right)=46 \)

Section does not comply with Clause 9.3.7.4.

Section is not compact.
Location: Ex1 - Tube X

Shape limitations for circular hollow sections

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006

The proforma checks the section classification to Clause 5.5.2 of BS EN 1993-2:2006 and Table 5.2 of BS EN 1193-1-1:2005 relating to circular hollow sections.

The ratio of outside diameter of section to wall thickness (D/t) of a circular hollow section should not exceed 90. When d/t > 90 the section classification is "slender". Reference to BS EN 1993-1-6 is made in Table 5.2 of BS EN 1193-1-1:2005 for slender sections.

Properties of circular hollow sections

- Outside diameter of section: D=170 mm
- Wall thickness: t=5 mm
- Yield strength: fy=355 N/mm²
Shape limitations for circular hollow sections

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma checks the section classification to Clause 5.5.2 of BS EN 1993-2:2006 and Table 5.2 of BS EN 1193-1-1:2005 relating to circular hollow sections.

The ratio of outside diameter of section to wall thickness (D/t) of a circular hollow section should not exceed 90. When d/t > 90, the section classification is "slender". Reference to BS EN 1993-1-6 is made in Table 5.2 of BS EN 1193-1-1:2005 for slender sections.

Properties of circular hollow sections

- Outside diameter of section: D=240 mm
- Wall thickness: t=5 mm
- Yield strength: fy=355 N/mm²
Location: X3 - Tube X

Shape limitations for circular hollow sections

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006

The proforma checks the section classification to Clause 5.5.2 of BS EN 1993-2:2006 and Table 5.2 of BS EN 1193-1-1:2005 relating to circular hollow sections.

The ratio of outside diameter of section to wall thickness (D/t) of a circular hollow section should not exceed 90\(\pi\). When d/t > 90\(\pi\) the section classification is "slender". Reference to BS EN 1993-1-6 is made in Table 5.2 of BS EN 1193-1-1:2005 for slender sections.

Properties of circular hollow sections

Outside diameter of section \(D=300\) mm
Wall thickness \(t=4\) mm
Yield strength \(f_y=355\) N/mm\(^2\)

As D/t > 90\(\pi\) ( 75 > 59.577 ), the section classification is "slender" in accordance with Table 5.2 of EC3 Part 1-1. Refer to BS EN 1993-1-6 for further information.
Location: Ex1 - Effective section Panel A

Effective section (Clause 9.4)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma calculates the effective sections in accordance with Clause 9.4.

Compression flange is stiffened by three or more open stiffeners.

\[ a = \text{panel dimension in direction of stress considered} \]
\[ b = \text{panel dimension normal to the direction of stress} \]

Panel dimension a: 1500 mm
Breadth between stiffeners: 300 mm
Thickness of flange plate: 12 mm
Nominal yield stress of plate: 355 N/mm²
Buckling coefficient Kc: 0.9798

Effective web (Clause 9.4.2.5)

Thickness of web: 20 mm

Web without effective long. stiffeners in accordance with Cl. 9.11.5.

Depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web. Depth yc.

Depth yc: 750 mm
Nominal yield stress of web: 355 N/mm²
Effective web thickness \( t_{we} = t_w = 20 \text{ mm} \)

The area of longitudinal stiffeners should be included for stress analysis only if they are continuous on either side of the section under consideration over a depth equal to the beam. Clause 9.4.2.6.
Location: Effective section Panel A

Effective section (Clause 9.4)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma calculates the effective sections in accordance with Clause 9.4.

Compression flange is stiffened by two open stiffeners.

```
                  tf
<---b--><---b--><---b-->  
```

\[a = \text{panel dimension in direction of stress considered}
\]
\[b = \text{panel dimension normal to the direction of stress}\]

- Panel dimension \(a\) = 1500 mm
- Breadth between stiffeners \(b\) = 300 mm
- Thickness of flange plate \(t_f\) = 12 mm
- Nominal yield stress of plate \(S_{Yf}\) = 355 N/mm²
- Buckling coefficient \(K_c\) = 0.97482

Effective web (Clause 9.4.2.5)

- Thickness of web \(t_w\) = 20 mm

Web without effective long. stiffeners in accordance with Cl. 9.11.5.

```
Compression flange
\[\text{yc}\]
\[---\] Elastic NA
```

Depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web. Depth \(yc\).

For a beam built in stages, \(yc\) should be calculated for the cross section of the beam appropriate to the stage considered.

- Depth \(yc\) = 750 mm
- Nominal yield stress of web \(S_{Yw}\) = 355 N/mm²
Effective web thickness \( t_{we} = t_w = 20 \text{ mm} \)

The area of longitudinal stiffeners should be included for stress analysis only if they are continuous on either side of the section under consideration over a depth equal to the beam. Clause 9.4.2.6.
Location: Effective section Panel A

Effective section (Clause 9.4)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma calculates the effective sections in accordance with Clause 9.4.

Compression flange is stiffened by one open stiffener.

\[ \text{Depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web. Depth } \text{yc.} \]

- Panel dimension a = 1500 mm
- Breadth between stiffeners b = 300 mm
- Thickness of flange plate \(tf=12 \text{ mm}\)
- Nominal yield stress of plate \(syf=355 \text{ N/mm}^2\)
- Buckling coefficient Kc = 0.96985

Effective web (Clause 9.4.2.5)

| Thickness of web \(tw=20 \text{ mm}\) |

- Depth yc = 750 mm
- Nominal yield stress of web \(syw=355 \text{ N/mm}^2\)
Effective web thickness \( t_{we} = t_w = 20 \text{ mm} \)

The area of longitudinal stiffeners should be included for stress analysis only if they are continuous on either side of the section under consideration over a depth equal to the beam. Clause 9.4.2.6.
Location: Effective section Panel A

Effective section (Clause 9.4)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06. The proforma calculates the effective sections in accordance with Clause 9.4.

Compression flange is stiffened by two or more closed stiffeners.

\[
\begin{align*}
\text{Dimension } b_1 & = 500 \text{ mm} \\
\text{Dimension } b_2 & = 300 \text{ mm} \\
\text{Thickness of flange plate } & = 12 \text{ mm} \\
\text{Nominal yield stress of plate } & = 355 \text{ N/mm}^2 \\
\end{align*}
\]

Calculate effective area for each panel

For panel width 500

Buckling coefficient \( K_c \) = 0.75895

For panel width 300

Buckling coefficient \( K_c \) = 0.9798

Effective web (Clause 9.4.2.5)

\[
\text{Thickness of web } = 20 \text{ mm}
\]
Web without effective long. stiffeners in accordance with Cl. 9.11.5.

Depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web. Depth \( y_c \).

For a beam built in stages, \( y_c \) should be calculated for the cross section of the beam appropriate to the stage considered.

Depth \( y_c \) \( y_c = 750 \text{ mm} \)
Nominal yield stress of web \( s_yw = 355 \text{ N/mm}^2 \)
Effective web thickness \( t_w = 20 \text{ mm} \)

The area of longitudinal stiffeners should be included for stress analysis only if they are continuous on either side of the section under consideration over a depth equal to the beam. Clause 9.4.2.6.
Effective section

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the reduction factor $\rho$ for plate buckling which is needed to evaluate the effective area $A_{eff}$ in accordance with Clause 4.4 of BS EN 1193-1-5, where $A_{eff}=\rho A_c$.

Compression flange is stiffened by three or more open stiffeners.

\[ a = \text{panel dimension in direction of stress considered} \]
\[ b = \text{panel dimension normal to the direction of stress} \]

Panel dimension $a$ $= 1500 \text{ mm}$
Breadth between stiffeners $b = 300 \text{ mm}$
Thickness of flange plate $t_f = 12 \text{ mm}$
Yield strength (flange) $f_y = 355 \text{ N/mm}^2$
Buckling factor (Table 4.1) $k_s = 1$

Effective web thickness

Web without effective longitudinal stiffeners.

Depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web. Depth $z_c$.

For a beam built in stages, $z_c$ should be calculated for the cross section of the beam appropriate to the stage considered.
<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth zc</td>
<td>zc=750 mm</td>
</tr>
<tr>
<td>Thickness of web</td>
<td>tw=20 mm</td>
</tr>
<tr>
<td>Yield strength (web)</td>
<td>fyw=355 N/mm²</td>
</tr>
<tr>
<td>Factor</td>
<td>p=(zc/tw)*SQR(fyw/355)=37.5</td>
</tr>
<tr>
<td>Effective web thickness</td>
<td>twe=tw=20 mm</td>
</tr>
</tbody>
</table>

**NOTE:**
The area of longitudinal stiffeners should be included for stress analysis only if they are continuous on either side of the section under consideration over a depth equal to the beam.
Location: Effective section Panel A

Effective section


The proforma calculates the reduction factor $\beta$ for plate buckling which is needed to evaluate the effective area $A_{eff}$ in accordance with Clause 4.4 of BS EN 1193-1-5, where $A_{eff} = \beta A_c$.

Compression flange is stiffened by two open stiffeners.

\[ \begin{align*}
\text{a} &= \text{panel dimension in direction of stress considered} \\
\text{b} &= \text{panel dimension normal to the direction of stress} \\
\text{a} &= 1500 \text{ mm} \\
\text{b} &= 300 \text{ mm} \\
\text{tf} &= 12 \text{ mm} \\
\text{fy} &= 355 \text{ N/mm}^2 \\
\text{ks} &= 1
\end{align*} \]

Effective web thickness

Web without effective longitudinal stiffeners.

Depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web. Depth $z_c$.

For a beam built in stages, $z_c$ should be calculated for the cross section of the beam appropriate to the stage considered.
Depth $zc$ = 750 mm
Thickness of web $tw$ = 20 mm
Yield strength (web) $fyw$ = 355 N/mm²
Factor $p$ = $(zc/tw) \times \sqrt{fyw/355}$ = 37.5
Effective web thickness $twe$ = $tw$ = 20 mm

NOTE:
The area of longitudinal stiffeners should be included for stress analysis only if they are continuous on either side of the section under consideration over a depth equal to the beam.
Location: Effective section Panel A

Effective section

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the reduction factor $\rho$ for plate buckling which is needed to evaluate the effective area $A_{eff}$ in accordance with Clause 4.4 of BS EN 1193-1-5, where $A_{eff} = \rho A_c$.

Compression flange is stiffened by one open stiffener.

```
\[ a = \text{panel dimension in direction of stress considered} \]
\[ b = \text{panel dimension normal to the direction of stress} \]
```

Panel dimension $a$ = 1500 mm
Breadth between stiffeners $b$ = 300 mm
Thickness of flange plate $t_f$ = 12 mm
Yield strength (flange) $f_y$ = 355 N/mm$^2$
Buckling factor (Table 4.1) $k_s$ = 1

Effective web thickness

Web without effective longitudinal stiffeners.

Depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web. Depth $z_c$.

For a beam built in stages, $z_c$ should be calculated for the cross section of the beam appropriate to the stage considered.
Depth $zc$                          $zc=750$ mm
Thickness of web                  $tw=20$ mm
Yield strength (web)              $f_{yw}=355$ N/mm$^2$
Factor                            $p=(zc/tw)\times\sqrt{f_{yw}/355}=37.5$
Effective web thickness           $t_{we}=tw=20$ mm

NOTE:
The area of longitudinal stiffeners should be included for stress analysis only if they are continuous on either side of the section under consideration over a depth equal to the beam.
Location: Effective section Panel A

Effective section

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the reduction factor $\rho$ for plate buckling which is needed to evaluate the effective area $A_{ceff}$ in accordance with Clause 4.4 of BS EN 1193-1-5, where $A_{ceff}=\rho.A_c$.

Compression flange is stiffened by two or more closed stiffeners.

\[ \text{Panel dimension } a = 1500 \text{ mm} \]
\[ \text{Dimension } b_1 = 500 \text{ mm} \]
\[ \text{Dimension } b_2 = 350 \text{ mm} \]
\[ \text{Yield strength (flange) } f_y = 355 \text{ N/mm}^2 \]
\[ \text{Buckling factor (Table 4.1) } k_s = 1 \]
Effective web thickness

Web without effective longitudinal stiffeners.

<table>
<thead>
<tr>
<th>Z</th>
<th>Compression flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>zc</td>
<td>Elastic NA</td>
</tr>
<tr>
<td>tw</td>
<td></td>
</tr>
</tbody>
</table>

Depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web. Depth zc.

For a beam built in stages, zc should be calculated for the cross section of the beam appropriate to the stage considered.

Depth zc: zc=750 mm
Thickness of web: tw=20 mm
Yield strength (web): fyw=355 N/mm²
Factor: p=(zc/tw)*SQR(fyw/355)=37.5
Effective web thickness: tew=tw=20 mm

NOTE:
The area of longitudinal stiffeners should be included for stress analysis only if they are continuous on either side of the section under consideration over a depth equal to the beam.
Effective section

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the reduction factor ϒ for plate buckling which is needed to evaluate the effective area \( A_{ceff} \) in accordance with Clause 4.4 of BS EN 1193-1-5, where \( A_{ceff} = \rho \cdot A_c \).

Compression flange is unstiffened.

\[ a = \text{panel dimension in direction of stress considered} \]
\[ b = \text{panel dimension normal to the direction of stress} \]

Panel dimension \( a = 1500 \text{ mm} \)
Panel dimension \( b = 300 \text{ mm} \)
Thickness of flange plate \( t_f = 12 \text{ mm} \)
Yield strength (flange) \( f_y = 355 \text{ N/mm}^2 \)
Buckling factor (Table 4.1) \( k_s = 1 \)

Effective web thickness

Web without effective longitudinal stiffeners.

\[ z_c = 750 \text{ mm} \]
\[ t_w = 20 \text{ mm} \]
Yield strength (web) \( f_yw = 355 \text{ N/mm}^2 \)
Factor \( p = (z_c/t_w) \cdot \sqrt{f_yw/355} = 37.5 \)
Effective web thickness \( t_{we} = t_w = 20 \text{ mm} \)

NOTE:
The area of longitudinal stiffeners should be included for stress analysis only if they are continuous on either side of the section.
under consideration over a depth equal to the beam.
Location: Ex1 - Girder S

Radius of curvature of flange  \( R_f = 10000 \text{ mm} \)
Flange outstand  \( b_{fo} = 150 \text{ mm} \)
Flange thickness  \( t_{fo} = 15 \text{ mm} \)
Longitudinal stress in flange of  \( \sigma_{f} = 300 \text{ N/mm}^2 \)
Flange panel width  \( b = 400 \text{ mm} \)
Flange thickness  \( t_{f} = 15 \text{ mm} \)
Longitudinal stress in flange of  \( \sigma_{f} = 300 \text{ N/mm}^2 \)
Location: Girder S

<table>
<thead>
<tr>
<th>Stress</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>σ1</td>
<td>σ1=150 N/mm²</td>
</tr>
<tr>
<td>σ2</td>
<td>σ2=125 N/mm²</td>
</tr>
</tbody>
</table>
Location: Girder S

Stress σ₁  
\[ \sigma_1 = 250 \text{ N/mm}^2 \]

Stress σ₂  
\[ \sigma_2 = 200 \text{ N/mm}^2 \]
Location: Girder S
Location: Girder S
Location: Girder S
Location: Girder S

Slope of flange to vertical $\alpha$  alpha=80°
Force in flange $P=1000$ kN
Location: Ex1 - Girder S

Radius of curvature of flange \( R_f = 10000 \text{ mm} \)
Flange outstand \( b_{fo} = 150 \text{ mm} \)
Flange thickness \( t_{fo} = 15 \text{ mm} \)
Longitudinal stress in flange \( \sigma_f = 300 \text{ N/mm}^2 \)
Flange panel width \( b = 400 \text{ mm} \)
Flange thickness \( t_f = 15 \text{ mm} \)
Longitudinal stress in flange \( \sigma_f = 300 \text{ N/mm}^2 \)
Location: Ex2 - Girder S

Stress σ1: \( \sigma_1 = 150 \text{ N/mm}^2 \)
Stress σ2: \( \sigma_2 = 125 \text{ N/mm}^2 \)
Location: Ex3 - Girder S

Stress $\sigma_1$  $\quad \sigma_{\text{1}} = 250 \text{ N/mm}^2$
Stress $\sigma_2$  $\quad \sigma_{\text{2}} = 200 \text{ N/mm}^2$
Location: Ex4 - Girder S
Location: Ex5 - Girder S
Location: Ex6 - Girder S
Location: Ex7 - Girder S

Slope of flange to vertical $\alpha$  $\alpha=80^\circ$
Force in flange $P=1000$ kN
Location: Ex1 - Edge beam

Effective length for lateral torsional buckling (Clause 9.6)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

The proforma calculates the effective length for lateral torsional buckling in accordance with Clause 9.6. Beams (other than cantilevers) without intermediate lateral restraint. Clause 9.6.2.

Single span beams

Span of beam between lateral restraints at supports.
Span $L$ $L=25000$ mm
All beams should be restrained at their supports against rotation

Where there is no intermediate lateral restraint to a compression flange, the effective length $l_e$ should be taken as:

$$l_e = k_1 \cdot k_2 \cdot k_e \cdot L$$

Factor $k_1=1.0$
Factor $k_2=1.2$

Effective length factor $k_e$ :

$$k_e = \frac{1}{0.5} \cdot \frac{60 \cdot E \cdot tf_{\text{max}} \cdot \beta \cdot \delta t \cdot R_v}{1 - \frac{W \cdot (L/ry)^3 \cdot v^4}{W \cdot (L/ry)^3 \cdot v^4}}$$

Consider end 1:
Max thickness of comp. flange in span $tf_{\text{max}}=20$ mm
Vertical reaction at support $R_v1=1500$ kN
Total vertical load on span $W=3500$ kN
Deflection $\delta t$ $\delta t=0.01$ mm
Coefficient $\beta$ $\beta_1=1$
Radius of gyration $ry=100$ mm
Factor $v$ $v=0.9$

Consider end 2:
Vertical reaction at support $R_v2=2000$ kN
Coefficient $\beta$ $\beta_2=1$

Effective length (Cl.9.6.2(a))

$$l_e = k_1 \cdot k_2 \cdot k_e \cdot L = 32296$$ mm
**Location: Edge beam**

**Effective length for lateral torsional buckling (Clause 9.6)**

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

The proforma calculates the effective length for lateral torsional buckling in accordance with Clause 9.6.

Cantilevers without intermediate lateral restraints. Clause 9.6.3.

Length of cantilever $L = 10000$ mm

Factor from Table 8 $q = 1.4$

Effective length $l_e = q \times L = 14000$ mm
Location: Edge beam

Effective length for lateral torsional buckling (Clause 9.6)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

The proforma calculates the effective length for lateral torsional buckling in accordance with Clause 9.6.

Beams with fully effective lateral restraints at the level of the compression flange. Clause 9.6.4.1.1.1.

Number of intermediate restraints nr=2

<table>
<thead>
<tr>
<th>Length</th>
<th>L(1)=10000 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>L(2)=15000 mm</td>
</tr>
<tr>
<td>Length</td>
<td>L(3)=12500 mm</td>
</tr>
</tbody>
</table>

Effective length le 15000 mm
Effective length for lateral torsional buckling (Clause 9.6)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

The proforma calculates the effective length for lateral torsional buckling in accordance with Clause 9.6. Beam restrained by discrete U-frame restraints. Clause 9.6.4.1.3. Intermediate restraint to the compression flange is provided only by U-frames in accordance with Clause 9.12.3, le should be calculated in accordance with Clause 9.6.4.1.1.2.

Span \( L = 15000 \) mm

Second moment of area \( I_c \) \( I_c = 360E6 \) mm^4

Young's modulus of steel \( E = 205000 \) N/mm²

Factor \( k_2 = 1.2 \)

U-frame spacing \( l_R = 3000 \) mm

U-frame is symmetrical with cross members and verticals of constant second moment of area throughout their lengths.

Distance \( B = 5000 \) mm

Distance \( d_1 = 1500 \) mm

Distance \( d_2 = 1250 \) mm

Second moment of area \( I_1 = 400E6 \) mm^4

Second moment of area \( I_2 = 500E6 \) mm^4

Factor \( u = 0.5 \)

Factor \( f = 20E-12 \) rd/Nmm

Total lateral deflection \( \Delta R = \Delta_{tav} + \Delta_{tab} + \Delta_{tar} \)

=83.079E-6 mm

Deflection \( \Delta_1 \) \( \Delta_{tav} = 0.21E-3 \) mm

Deflection \( \Delta_2 \) \( \Delta_{tab} = 0.22E-3 \) mm

Effective length from Cl.9.6.2(a) \( l_{ea} = 7450.6 \) mm

Effective length from Cl.9.6.2(b) \( l_{eb} = 5483.4 \) mm

Effective length when \( k_3 \) is 1.0 \( l_e = l_{ea} = 7450.6 \) mm
Effective length for lateral torsional buckling (Clause 9.6)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

The proforma calculates the effective length for lateral torsional buckling in accordance with Clause 9.6.

Deck is connected to an unstiffened web, either directly or via the tension flange, over the length of the beam in accordance with Clause 9.12.4.2. The effect of the restraint should be allowed for by assuming that the deck and webs of the main beams comprise a continuous series of effective U-frames of unit length longitudinally.

The effective length should be calculated in accordance with Clause 9.6.4.1.3 with length lR equal to unity and deflection deltaR equal to the lateral deflection which would occur in an effective U-frame at the level of the centroid of the flange being considered when a unit force acts laterally to the effective U-frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same effective U-frame.

Effective length should be calculated in accordance with Clause 9.6.4.1.1.2

Span \( L = 15000 \text{ mm} \)
Second moment of area \( I_c = 360E6 \text{ mm}^4 \)
Young's modulus of steel \( E = 205000 \text{ N/mm}^2 \)

Factor \( k_2 = 1.2 \)

The deck and webs of the beam are of constant thickness throughout the span and the beam of constant depth.

Distance \( B = 5000 \text{ mm} \)
Distance \( d_1 = 1500 \text{ mm} \)
Distance \( d_2 = 1250 \text{ mm} \)
Second moment of area $I_1$  
$I_1 = 400 \times 10^6 \text{ mm}^4$

Second moment of area $I_2$  
$I_2 = 500 \times 10^6 \text{ mm}^4$

Factor $u$  
$u = 0.5$

Total lateral deflection  
$\delta_R = \delta_{av} + \delta_{ab} = 51.829 \times 10^{-6} \text{ mm}$

Deflection $\delta_1$  
$\delta_{1} = 0.21 \times 10^{-3} \text{ mm}$

Deflection $\delta_2$  
$\delta_{2} = 0.22 \times 10^{-3} \text{ mm}$

Effective length:

$$le_b = \pi k_2 \sqrt{E I_c (\delta_1 + \delta_2)/L} = 5483.4 \text{ mm}$$

Effective length from Cl.9.6.2(a)  
$le_a = 1073.8 \text{ mm}$

Effective length from Cl.9.6.2(b)  
$le_b = 5483.4 \text{ mm}$

Effective length when $k_3$ is 1.0  
$le = le_b = 5483.4 \text{ mm}$
Effective length for lateral torsional buckling (Clause 9.6)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

The proforma calculates the effective length for lateral torsional buckling in accordance with Clause 9.6.

Beams (other than cantilevers) without intermediate lateral restraint. Clause 9.6.2.

Continuous beams

Span of beam between lateral restraints at supports. For continuous beams it is the larger of the adjacent spans.

Span L = 25000 mm

All beams should be restrained at their supports against rotation.

Where there is no intermediate lateral restraint to a compression flange, the effective length le should be taken as:

\[ le = k_1 \cdot k_2 \cdot k_e \cdot L \]

Factor \( k_1 = 1.0 \)

Factor \( k_2 = 1.2 \)

Effective length factor \( k_e \):

\[ k_e = \left[ \frac{1}{\frac{60 \cdot E \cdot t_{\text{max}} \cdot \beta \cdot \delta \cdot R_v}{W \cdot (L/ry)^3 \cdot v^4}} \right]^{0.5} \]

Consider end 1:

Max thickness of comp. flange in span \( t_{\text{max}} = 20 \text{ mm} \)

Vertical reaction at support \( R_v1 = 1500 \text{ kN} \)

Total vertical load on span \( W = 3500 \text{ kN} \)

Deflection \( \delta \)

Coefficient \( \beta \)

Radius of gyration \( ry = 100 \text{ mm} \)

Factor \( v \)

Consider end 2:

Vertical reaction at support \( R_v2 = 2000 \text{ kN} \)

Coefficient \( \beta \)

Effective length (Cl.9.6.2(a)) \( le = k_1 \cdot k_2 \cdot k_e \cdot L = 32296 \text{ mm} \)

\[ le = \left[ \frac{1}{\frac{2 \cdot (\Sigma L)^3}{\pi^4 \cdot E \cdot I_c \cdot (\delta_i + (\delta_e/2))}} \right]^{0.5} \]
Sum of lengths of adjacent spans \( \text{sumL}=50000 \text{ mm} \)
Second moment of area \( I_c \) \( I_c=360E6 \text{ mm}^4 \)
Deflection \( \delta_i \) \( \delta_i=23 \text{ mm} \)
Deflection \( \delta_e \) \( \delta_e=24 \text{ mm} \)

Effective length:
\[
le_b = k_1 k_2 \times \text{sumL} \times SQR\left(\frac{1}{1 + \left(\frac{2 \times \text{sumL}^3}{\pi^4 \times E \times I_c \times (\delta_i + \delta_e/2)}\right)}\right)
\]
\[
=59970 \text{ mm}
\]

Effective length (\( le_b \) governs) \( le=59970 \text{ mm} \)
Effective length for lateral torsional buckling

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the effective length for lateral torsional buckling using the same approach used in BS 5400-3:2000.

Beams (other than cantilevers) without intermediate lateral restraint.

Single span beams

Span of beam between lateral restraints at supports. Span L = 25000 mm
All beams should be restrained at their supports against rotation

Where there is no intermediate lateral restraint to a compression flange, the effective length le should be taken as:

\[ le = k_1 \cdot k_2 \cdot k_e \cdot L \]

Factor \( k_1 = 1.0 \)
Factor \( k_2 = 1.2 \)

Effective length factor \( k_e \):

\[ k_e = \left[ \frac{1}{1 - \frac{60 \cdot E \cdot t_{\text{max}} \cdot \beta \cdot \delta_t \cdot R_v}{W \cdot (L/r_z)^3 \cdot v^4}} \right]^{0.5} \]

Consider end 1:
Max thickness of comp. flange in span \( t_{\text{max}} = 20 \text{ mm} \)
Vertical reaction at support \( R_{v1} = 1500 \text{ kN} \)
Total vertical load on span \( W = 3500 \text{ kN} \)
Deflection \( \delta_t \) \( \text{deltat} = 0.01 \text{ mm} \)
Coefficient \( \beta \) \( \beta_{1} = 1 \)
Radius of gyration \( r_z = 100 \text{ mm} \)
Slenderness factor \( v \) \( v = 0.9 \)

Consider end 2:
Vertical reaction at support \( R_{v2} = 2000 \text{ kN} \)
Coefficient \( \beta \) \( \beta_{2} = 1 \)
Value of \( k_e2 \) governs \( k_e = k_e2 = 1.0765 \)
Effective length \( le = k_1 \cdot k_2 \cdot k_e \cdot L = 32296 \text{ mm} \)
Location: Edge beam

Effective length for lateral torsional buckling

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the effective length for lateral torsional buckling using the same approach used in BS 5400-3:2000.

Cantilevers without intermediate lateral restraints.
Length of cantilever       L=10000 mm
Factor from table              q=1.4
Effective length       le=q*L=14000 mm
Location: Edge beam

Effective length for lateral torsional buckling

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the effective length for lateral torsional buckling using the same approach used in BS 5400-3:2000.

Beams with fully effective lateral restraints at the level of the compression flange.
Number of intermediate restraints nr=2

<table>
<thead>
<tr>
<th>Length</th>
<th>L(1)=10000 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>L(2)=15000 mm</td>
</tr>
<tr>
<td>Length</td>
<td>L(3)=12500 mm</td>
</tr>
</tbody>
</table>

Effective length le 15000 mm
Location: Ex4- Edge beam

Effective length for lateral torsional buckling

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the effective length for lateral torsional buckling using the same approach used in BS 5400-3:2000.

Beam restrained by discrete U-frame restraints. Intermediate restraint to the compression flange is provided only by U-frames.

Span \( L = 15000 \text{ mm} \)

Second moment of area \( I_c = 360 \times 10^6 \text{ mm}^4 \)

Young's modulus of steel \( E = 210000 \text{ N/mm}^2 \)

Factor \( k_2 = 1.2 \)

U-frame spacing \( l_R = 3000 \text{ mm} \)

U-frame is symmetrical with cross members and verticals of constant second moment of area throughout their lengths.

Distance \( B = 5000 \text{ mm} \)

Distance \( d_1 = 1500 \text{ mm} \)

Distance \( d_2 = 1250 \text{ mm} \)

Second moment of area \( I_1 = 400 \times 10^6 \text{ mm}^4 \)

Second moment of area \( I_2 = 500 \times 10^6 \text{ mm}^4 \)

Factor \( u = 0.5 \)

Factor \( f = 20 \times 10^{-12} \text{ rd/Nmm} \)

Total lateral deflection \( \Delta R = \Delta v + \Delta b + \Delta t = 81.845 \times 10^{-6} \text{ mm} \)

Deflection \( \Delta_1 = 0.21 \times 10^{-3} \text{ mm} \)

Deflection \( \Delta_2 = 0.22 \times 10^{-3} \text{ mm} \)

Effective length (case 1) \( l_a = 7482.2 \text{ mm} \)

Effective length (case 2) \( l_b = 5549.8 \text{ mm} \)

Effective length when \( k_3 \) is 1.0 \( l_e = l_a = 7482.2 \text{ mm} \)
Effective length for lateral torsional buckling

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the effective length for lateral torsional buckling using the same approach used in BS 5400-3:2000.

Deck is connected to an unstiffened web, either directly or via the tension flange, over the length of the beam. The effect of the restraint should be allowed for by assuming that the deck and webs of the main beams comprise a continuous series of effective U-frames of unit length longitudinally.

The effective length should be calculated with length \( l_R \) equal to unity and deflection \( \delta_R \) equal to the lateral deflection which would occur in an effective U-frame at the level of the centroid of the flange being considered when a unit force acts laterally to the effective U-frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same effective U-frame.

Span \( L = 15000 \text{ mm} \)

Second moment of area \( I_c \) \( I_c = 360E6 \text{ mm}^4 \)

Young's modulus of steel \( E = 210000 \text{ N/mm}^2 \)

Factor \( k_2 = 1.2 \)

The deck and webs of the beam are of constant thickness throughout the span and the beam of constant depth.

Distance \( B \) \( B = 5000 \text{ mm} \)

Distance \( d_1 \) \( d_1 = 1500 \text{ mm} \)

Distance \( d_2 \) \( d_2 = 1250 \text{ mm} \)
Second moment of area $I_1$ $I_1=400E6$ mm$^4$

Second moment of area $I_2$ $I_2=500E6$ mm$^4$

Factor u $u=0.5$

Total lateral deflection $\delta_R=\delta_{1}+\delta_{2}=50.595E-6$ mm

Deflection $\delta_1$ $\delta_{1}=0.21E-3$ mm

Deflection $\delta_2$ $\delta_{2}=0.22E-3$ mm

Effective length:

$$Ie_b=\pi*k_2*\sqrt{E*Ic*(\delta_{1}+\delta_{2})/L}=5549.8$$ mm

Effective length (case 1) $Ie_a=1073.8$ mm

Effective length (case 2) $Ie_b=5549.8$ mm

Effective length when $k_3$ is 1.0 $Ie=Ie_b=5549.8$ mm
Location: Ex6 - As Ex1 but continuous beam

Effective length for lateral torsional buckling

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the effective length for lateral torsional buckling using the same approach used in BS 5400-3:2000.

Beams (other than cantilevers) without intermediate lateral restraint.

Continuous beams

Span of beam between lateral restraints at supports.
For continuous beams it is the larger of the adjacent spans.
Span L = 25000 mm
All beams should be restrained at their supports against rotation

Where there is no intermediate lateral restraint to a compression flange, the effective length le should be taken as:

\[ le = k_1 \cdot k_2 \cdot k_e \cdot L \]

Factor \( k_1 = 1.0 \)
Factor \( k_2 = 1.2 \)

Effective length factor \( k_e \):

\[
k_e = \frac{1}{1 - \frac{60 \cdot E \cdot t_{f_{\text{max}}} \cdot \beta \cdot \delta t \cdot R_v}{W \cdot (L/r_z)^3 \cdot v^4}}^{0.5} \]

Consider end 1:
Max thickness of comp. flange in span \( t_{f_{\text{max}}} = 20 \) mm
Vertical reaction at support \( R_v1 = 1500 \) kN
Total vertical load on span \( W = 3500 \) kN
Deflection \( \delta t \) \( \delta t = 0.01 \) mm
Coefficient \( \beta \) \( \beta = 2 \)
Radius of gyration \( r_z = 100 \) mm
Slenderness factor \( v \) \( v = 0.9 \)

Consider end 2:
Vertical reaction at support \( R_v2 = 2000 \) kN
Coefficient \( \beta \) \( \beta = 2 \)
Value of \( k_e2 \) governs \( k_e = k_e2 = 1.1793 \)
Effective length \( le = k_1 \cdot k_2 \cdot k_e \cdot L = 35378 \) mm

\[
le = k_1 \cdot k_2 \cdot \frac{1}{1 + \frac{2 \cdot (\Sigma L)^3}{\pi^4 \cdot E \cdot I_c \cdot (\delta_i + (\delta e/2))}}^{0.5} \]
Effective length:

\[ le_b = k_1 \times k_2 \times \text{sumL} \times SQR\left(\frac{1}{1 + \frac{2 \times \text{sumL}^3}{\pi^4 E I_c (\delta_i + \delta_e/2)}}\right) \]

\[ = 33215 \text{ mm} \]

Effective length (lea governs) \[ le = 35378 \text{ mm} \]
Location: Ex1 - Slenderness, moment and shear resistance

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

**Slenderness lamLT**

The proforma calculates the slenderness parameter lambdaLT in accordance with Clause 9.7.

I, Channel, Tee or Angle sections (Clause 9.7.2).

Half wavelength of buckling \( lw = 10400 \) mm

Effective length to Clause 9.6 \( le = 10400 \) mm

2nd moment of area \( @ \) minor axis \( I_{yy} = 133E6 \) mm\(^4\)

Cross sectional area \( A = 32300 \) mm\(^2\)

Factor \( k4 = 0.9 \)

Simply supported mid-span moment \( M_m = -236.98 \) kNm

Overall depth of section \( D = 918.5 \) mm

Mean flange thickness \( t_f = 27.9 \) mm

For a uniform section

Slenderness parameter \( \lambda_{mLT} = (le/ry)*k4*\eta*v = 119.61 \)

**Limiting moment of resistance**

The proforma calculates the limiting moment of resistance \( MR \) in accordance with Clause 9.8.

Ratio \( le/lw = 1 \)

Section is compact in accordance with Clause 9.3.7

Moment of resistance of section \( \text{Mult} = 3770 \) kNm

Nominal yield stress comp. flange \( sy_c = 345 \) N/mm\(^2\)

Nominal yield stress tens. flange \( sy_t = 345 \) N/mm\(^2\)

Nominal yield stress of web \( sy_w = 345 \) N/mm\(^2\)

Slenderness ratio \( \lambda_{mLT} = 119.61 \)

\( n = 0.008*lw/le*(\beta-30) = 0.7033 \)

\( MR = ((1+(1+n)*5700/\beta^2)-\sqrt{(1+(1+n)*5700/\beta^2)^2-22800/\beta^2})) = 0.5828 \)

Limiting moment of resistance \( MR = \text{Mult} \times 0.5 = 1098.6 \) kNm

**Bending resistance**

The proforma calculates the bending resistance of beams without longitudinal stiffeners in accordance with Clause 9.9.

Partial factor on yield strength \( g_m = 1.05 \)

Partial factor gammaf3 \( g_f3 = 1.1 \)

Bending resistance \( MD = MR/(g_m*gf3) = 951.15 \) kNm

Calculations for the bending resistance ignoring web contribution \( M_f \) from Clause 9.9.3.1.

Area of effective tension flange \( A_{te} = 8523 \) mm\(^2\)

Partial factor on yield strength \( g_{mt} = 1.05 \)

Partial factor gammaf3 \( g_{f3} = 1.1 \)

Area of effective compr. flange \( A_{ce} = 8523 \) mm\(^2\)

Elastic modulus of the section \( Z_{xc} = 9.5E6 \) mm\(^3\)
Limiting stress (comp. flange) \( \sigma_{yc} = \sigma_{yc2} = 115.64 \text{ N/mm}^2 \)
Partial factor on comp. strength \( \gamma_{mc} = 1.05 \)
Partial factor gammaf3 \( \gamma_f3 = 1.1 \)
Force in compression flange \( F_{fc} = \frac{(A*\sigma_{yc})}{(\gamma_{mc}\gamma_{f3}1E3)} = 853.33 \text{ kN} \)
Bending resistance is based on flange with lower value of force.
Bending resistance \( M_f = F_{fc} \times df/1E3 = 759.98 \text{ kNm} \)

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The result is the shear capacity of a web panel under pure shear calculated using Clause 9.9.2 Shear Resistance.
Depth of web \( d_{we} = 862.7 \text{ mm} \)
Clear length of panel \( a = 10400 \text{ mm} \)
Web thickness \( t_w = 17.3 \text{ mm} \)
Half eff. breadth of flanges \( b_{fe} \) is the smaller of the following:
1. The result of the expression \( b_{fe1} = 10*tf*SQRT(355/s_yf) = 283.01 \text{ mm} \)
2. The distance from the mid-plane of the web to the nearer edge of the flange \( b_{fe2} = 152.75 \text{ mm} \)
Non-dimensional value of plastic moment of resistance of flange:
\( m_{fw} = \frac{(s_yf \times b_{fe} \times tf \times tf)}{(2 \times s_yw \times d_{we} \times d_{we} \times t_w)} = 0.0046174 \)

Shear resistance results from analysis

Shear resistance of web panel under pure shear.
Limiting shear stress \( (\tau_l) \) \( 199.19 \text{ N/mm}^2 \)
Resistance \( V_D = \frac{(t_w \times (d_w-h_h) \times \tau_{ul1})}{(\gamma_m \gamma_f3 \times 1000)} = 2740.3 \text{ kN} \)

Shear resistance of web panel under pure shear ignoring flange capacity i.e. taking \( m_{fw} = 0 \).
Limiting shear stress \( (\tau_l) \) \( 199.19 \text{ N/mm}^2 \)
Resistance \( V_R = \frac{(t_w \times (d_w-h_h) \times \tau_{ul2})}{(\gamma_m \gamma_f3 \times 1000)} = 2740.3 \text{ kN} \)

Combined bending and shear in web sections

Web with intermediate transverse stiffeners. Clause 9.9.3.1.
Maximum shear force in panel \( V = 182.3 \text{ kN} \)
Maximum bending moment \( M = 947.9 \text{ kNm} \)
RESULTS FROM ANALYSIS

MD design capacity in bending     VD design capacity in shear
Mf bending capacity ignoring web  VR shear capacity ignoring flanges

MD=951.15 kNm
Mf=759.98 kNm
V=182.3 kN
VD=2740.3 kN
VR=2740.3 kN

Maximum bending moment 947.9 kNm is within interaction curve.
Maximum shear force 182.3 kN is within interaction curve.
Interaction relationship satisfied.
Location: Ex2 - SHS edge beam (500x500x16 SHS)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

**Slenderness lamLT**

The proforma calculates the slenderness parameter lambdaLT in accordance with Clause 9.7.

Rectangular or trapezoidal box sections (Clause 9.7.3.1).

- Half wavelength of buckling \( lw = 9000 \text{ mm} \)
- Effective length to Clause 9.6 \( le = 9000 \text{ mm} \)
- 2nd moment of area @ major axis \( I_{xx} = 1.21 \times 10^9 \text{ mm}^4 \)
- 2nd moment of area @ minor axis \( I_{yy} = 1.21 \times 10^9 \text{ mm}^4 \)
- Cross sectional area \( A = 31000 \text{ mm}^2 \)
- Simply supported mid-span moment \( M_m = -220 \text{ kNm} \)
- Plastic modulus \( Z_{pe} = 5.6 \times 10^6 \text{ mm}^3 \)

- Area \( Ao = 234000 \text{ mm}^2 \)
- Depth of section \( D = 500 \text{ mm} \)
- Width of section \( B = 500 \text{ mm} \)
- Thickness of flange \( t = 16 \text{ mm} \)
- Summation \( B/B/t = 4 \times B/t = 125 \)
- Torsional constant \( J = 4 \times Ao^2/B/Byt = 1.7522 \times 10^9 \text{ mm}^4 \)

For uniform section

- Slenderness parameter \( \lambda_{LT} = 2.25 \times \eta \times \xi \times \sqrt{(Z_{pe} \times le)/(ry \times SQR(A \times J))} = 0 \)

**Limiting moment of resistance**

The proforma calculates the limiting moment of resistance \( MR \) in accordance with Clause 9.8.

- Ratio \( le/lw \) \( \text{rat2} = le/lw = 1 \)
- Section is compact in accordance with Clause 9.3.7
- Moment of resistance of section \( Mult = 1900 \text{ kNm} \)
- \( M_{pe} = Mult = 1900 \text{ kNm} \)
- Nominal yield stress comp.flange \( syc = 355 \text{ N/mm}^2 \)
- Nominal yield stress tens.flange \( syt = 355 \text{ N/mm}^2 \)
- Nominal yield stress of web \( syw = 355 \text{ N/mm}^2 \)
- Slenderness ratio \( \lambda_{adlt} = \lambda_{LT} = 0 \)

\[ n = 0.008 \times lw/le \times (\beta - 30) = -0.24 \]

- \( MR = Mult = 1900 \text{ kNm} \)

**Bending resistance**

The proforma calculates the bending resistance of beams without longitudinal stiffeners in accordance with Clause 9.9.

- Partial factor on yield strength \( gm = 1.05 \)
- Partial factor gammaf3 \( gf3 = 1.1 \)
- Bending resistance \( MD = MR/(gm \times gf3) = 1645 \text{ kNm} \)

Calculations for the bending resistance ignoring web contribution \( M_f \) from Clause 9.9.3.1.

- Area of effective tension flange \( A_{te} = 8000 \text{ mm}^2 \)
- Partial factor on yield strength \( gmt = 1.05 \)
Partial factor $\gamma_{f3}$  $gf3=1.1$

Area of effective compr. flange  $Ace=8000 \text{ mm}^2$

Elastic modulus of the section  $Zxc=4.84E6 \text{ mm}^3$

Limiting stress (comp. flange)  $s_{yc}=355 \text{ N/mm}^2$

Partial factor on comp. strength  $gm=1.05$

Partial factor $\gamma_{f3}$  $gf3=1.1$

Force in compression flange  $F_{fc}=(Ace*s_{yc})/(gmc*gf3*1E3)=2458.9 \text{ kN}$

Bending resistance is based on flange with lower value of force.

Bending resistance  $Mf=Fft*df/1E3=1190.1 \text{ kNm}$

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The result is the shear capacity of a web panel under pure shear calculated using Clause 9.9.2 Shear Resistance.

Depth of web  $dwe=468 \text{ mm}$

Clear length of panel  $a=9000 \text{ mm}$

Web thickness  $tw=16 \text{ mm}$

Half eff.breadth of flanges $b_{fe}$ is the smallest of the following:

1. The result of the expression  $b_{fe1}=10*tf*SQR(355/syf)=160 \text{ mm}$

2. The distance from the mid-plane of the web to the nearer edge of the flange  $b_{fe2}=8 \text{ mm}$

3. Half the clear distance  $b_{fe3}=234 \text{ mm}$

Non-dimensional value of plastic moment of resistance of flange:

$mfw=(syf*b_{fe}*tf*tf)/(2*syw*dwe*dwe*tw)=0.29221E-3$

Shear resistance results from analysis

Shear resistance of web panel under pure shear.

Limiting shear stress ( $\tau_1$ )  $204.96 \text{ N/mm}^2$

Resistance  $VD=(tw*(dwe-hh)*taul1)/(gm*gf3*1000)$

$=1419.6 \text{ kN}$

Shear resistance of web panel under pure shear ignoring flange capacity i.e. taking $mfw=0$.

Limiting shear stress ( $\tau_1$ )  $204.96 \text{ N/mm}^2$

Resistance  $VR=(tw*(dwe-hh)*taul2)/(gm*gf3*1000)$

$=1419.6 \text{ kN}$

Combined bending and shear in web sections

Web with transverse stiffeners at the supports only. Clause 9.9.3.2.

Shear force  $V=182.3 \text{ kN}$

Co-existent bending moment  $M=947.9 \text{ kNm}$
RESULTS FROM ANALYSIS

MD design capacity in bending     VD design capacity in shear
Mf bending capacity ignoring web  VR shear capacity ignoring flanges

Interaction between shear and bending resistances

Maximum bending moment 947.9 kNm is within interaction curve.
Maximum shear force 182.3 kN is within interaction curve.
Interaction relationship satisfied.
Location: Ex3 - Solid rectangular section 100x100

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Slenderness lamLT

The proforma calculates the slenderness parameter lambdaLT in accordance with Clause 9.7.
Solid rectangular sections (Clause 9.7.3.2).
Half wavelength of buckling \( lw = 2000 \) mm
Effective length to Clause 9.6 \( le = 2000 \) mm
Depth D \( D = 100 \) mm
Width of section \( B = 100 \) mm

Simply supported mid-span moment \( M_m = 30 \) kNm

Slenderness parameter \( \lambda_{lt} = 2.8*\eta SQR(l_e*D)/B = 8.4752 \)

Limiting moment of resistance

The proforma calculates the limiting moment of resistance \( MR \) in accordance with Clause 9.8.

Ratio \( le/lw \) \( \text{rat2}=le/lw=1 \)
Section is compact in accordance with Clause 9.3.7
Moment of resistance of section \( \text{Mult}=78 \) kNm
Mpe as defined in Clause 9.7.1 \( \text{Mpe} = \text{Mult} = 78 \) kNm
Nominal yield stress comp.flange \( \text{syc}=315 \) N/mm²
Nominal yield stress of web \( \text{syw}=315 \) N/mm²
Slenderness ratio \( \lambda_{lt} \) \( \lambda = \lambda_{lt} = 8.4752 \)
\( n=0.008*lw/le*(\beta-30)=-0.17613 \)
MR=Mult=78 kNm

Bending resistance

The proforma calculates the bending resistance of beams without longitudinal stiffeners in accordance with Clause 9.9.
Partial factor on yield strength \( gm=1.05 \)
Partial factor gammaf3 \( gf3=1.1 \)
Bending resistance \( \text{MD}=MR/(gm*gf3)=67.532 \) kNm
Location: Ex4 - Other cases

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

Slenderness lamLT

The proforma calculates the slenderness parameter lambdaLT in accordance with Clause 9.7.

Half wavelength of buckling \( l_w = 9000 \text{ mm} \)
Effective length to Clause 9.6 \( l_e = 9000 \text{ mm} \)
Young's modulus of steel \( E = 210000 \text{ N/mm}^2 \)
Plastic modulus \( Z_{pe} = 5.6E6 \text{ mm}^3 \)

Bending moment \( M_{cr} \)

\[ \text{M}_{cr} = 604E6 \text{ kNm} \]

Slenderness parameter \( \lambda_mL_{T} = \sqrt{\frac{\pi^2 E Z_{pe}}{M_{cr} 1E6}} \)

\[ \lambda_mL_{T} = 0.13862 \]

Limiting moment of resistance

The proforma calculates the limiting moment of resistance \( M_R \) in accordance with Clause 9.8.

Ratio \( l_e/l_w \)

\[ \text{rat}_2 = \frac{l_e}{l_w} = 1 \]

Section is compact in accordance with Clause 9.3.7

Moment of resistance of section \( \text{Mult} = 1900 \text{ kNm} \)

Mpe as defined in Clause 9.7.1 \( \text{M}_{pe} = \text{Mult} = 1900 \text{ kNm} \)
Nominal yield stress comp.flange \( \text{s}_{yc} = 355 \text{ N/mm}^2 \)
Nominal yield stress tens.flange \( \text{s}_{yt} = 355 \text{ N/mm}^2 \)
Nominal yield stress of web \( \text{s}_{yw} = 355 \text{ N/mm}^2 \)

Slenderness ratio \( \lambda_m = \lambda_mL_{T} = 0.13862 \)

\[ n = 0.008 \times l_w / l_e \times (\beta - 30) = -0.23889 \]

\[ M_R = \text{Mult} = 1900 \text{ kNm} \]

Bending resistance

The proforma calculates the bending resistance of beams without longitudinal stiffeners in accordance with Clause 9.9.

Partial factor on yield strength \( g_m = 1.05 \)
Partial factor gammaf3 \( g_f3 = 1.1 \)

Bending resistance \( \text{MD} = \frac{M_R}{(g_m * g_f3)} = 1645 \text{ kNm} \)

Calculations for the bending resistance ignoring web contribution \( M_f \) from Clause 9.9.3.1.

Area of effective tension flange \( A_{te} = 8000 \text{ mm}^2 \)
Partial factor on yield strength \( g_m t = 1.05 \)
Partial factor gammaf3 \( g_f3 = 1.1 \)

Area of effective compr. flange \( A_{ce} = 8000 \text{ mm}^2 \)

Elastic modulus of the section \( Z_{xc} = 4.84E6 \text{ mm}^3 \)
Limiting stress (comp. flange) \( \text{s}_{yc} = \text{s}_{ycl} = 355 \text{ N/mm}^2 \)

Partial factor on comp. strength \( g_m c = 1.05 \)
Partial factor gammaf3 \( g_f3 = 1.1 \)

Force in compression flange \( F_{fc} = (A_{ce} \times \text{s}_{yc}) / (g_m c * g_f3 \times 1E3) = 2458.9 \text{ kN} \)
Distance between centroids \( d_f = 484 \text{ mm} \)

Bending resistance is based on flange with lower value of force.
Bending resistance \[ M_f = F_f t^2 / 1E3 = 1190.1 \text{ kNm} \]
Location: Ex5 - Assessment to Standard BD 56/10 (Equal flanges)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

**Slenderness lamLT**

The proforma calculates the slenderness parameter lambdaLT in accordance with Clause 9.7.

I, Channel, Tee or Angle sections (Clause 9.7.2).

- Half wavelength of buckling $l_w = 15000$ mm
- Effective length to Clause 9.6 $l_e = 15000$ mm

2nd moment of area @ minor axis $I_{yy} = 156E6$ mm$^4$

Cross sectional area $A = 36800$ mm$^2$

Factor $k_4 = 0.9$

Overall depth of section $D = 926.6$ mm

Mean flange thickness $t_f = 32$ mm

For a uniform section

**Slenderness parameter**

$\lambda_{LT} = \left(\frac{l_e}{r_y}\right)k_4\eta_v = 207.35$

**Limiting moment of resistance**

The proforma calculates the limiting moment of resistance $M_R$ in accordance with Clause 9.8.

- Ratio $l_e/l_w$: rat2 = $l_e/l_w = 1$
- Section is non-compact.
- Multiplier for non-compact section $Mult = 4340$ kNm
- $M_{pe}$ as defined in Clause 9.7.1 $M_{pe} = 4000$ kNm

Nominal yield stress comp. flange $s_{yc} = 355$ N/mm$^2$

Nominal yield stress tens. flange $s_{yt} = 355$ N/mm$^2$

Nominal yield stress of web $s_{yw} = 355$ N/mm$^2$

Slenderness ratio $\lambda = \lambda_{LT} = 207.35$

$n = 0.008\frac{l_w}{l_e}(\beta - 30) = 1.4878$

$M_R = \left(1 + (1+n)\frac{5700}{\beta^2}\right) - \sqrt{\left(1 + (1+n)\frac{5700}{\beta^2}\right)^2 - 22800/\beta^2})$

$= 0.20326$

Limiting moment of resistance $M_R = M_R \times Mult \times 0.5 = 441.07$ kNm

**Bending resistance**

The proforma calculates the bending resistance of beams without longitudinal stiffeners in accordance with Clause 9.9.

- Partial factor on yield strength $g_m = 1.05$
- Partial factor gammaf3 $g_f3 = 1.1$
- Bending resistance $M_D = M_R/(g_m g_f3) = 381.88$ kNm

Calculations for the bending resistance ignoring web contribution $M_f$ from Clause 9.9.3.1.

- Area of effective tension flange $A_{te} = 9824$ mm$^2$
- Partial factor on yield strength $g_{mt} = 1.05$
- Partial factor gammaf3 $g_f3 = 1.1$
- Area of effective compr. flange $A_{ce} = 9824$ mm$^2$
- Elastic modulus of the section $E_{xc} = 10.9E6$ mm$^3$
Limiting stress (comp. flange) \( s_{yc} = s_{yc2} = 40.465 \text{ N/mm}^2 \)
Partial factor on comp. strength \( g_{mc} = 1.05 \)
Partial factor gammaf3 \( \gamma_f3 = 1.1 \)
Force in compression flange \( F_{fc} = \frac{A_c e s_{yc}}{g_{mc} \gamma_f3 \times 1E3} = 344.18 \text{ kN} \)

Bending resistance is based on flange with lower value of force.
Bending resistance \( M_f = F_{fc} d_f / 1E3 = 307.9 \text{ kNm} \)

**Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners**

The assessment calculations are in accordance with Standard BD 56/10 "The Assessment of Steel Highway Bridges and Structures". The result is the shear capacity of a web panel under pure shear calculated using Clause 9.9.2 Shear Resistance.

- Depth of web \( d_{we} = 824 \text{ mm} \)
- Clear length of panel \( a = 15000 \text{ mm} \)
- Web thickness \( t_w = 19.5 \text{ mm} \)
- Half eff. breadth of flanges \( b_{fe} \) is the smaller of the following:
  1. The result of the expression \( b_{fe1} = 10 \times t_f \times \sqrt{355/s_yf} = 320 \text{ mm} \)
  2. The distance from the mid-plane of the web to the nearer edge of the flange \( b_{fe2} = 153 \text{ mm} \)

Non-dimensional value of plastic moment of resistance of flange:
\[ m_{fw} = \frac{s_yf b_{fe} t_f t_f}{2 s_yw d_{we} d_{we} t_w} = 0.0059166 \]

**Shear resistance results from analysis**

- Shear resistance of web panel under pure shear.
  - Limiting shear stress \( (\tau_1) \) \( 204.96 \text{ N/mm}^2 \)
  - Resistance \( V_D = \frac{t_w (d_w - h_h) \tau_{aul1}}{g_m \gamma_f3 \times 1000} \)
    \[ = 2984.9 \text{ kN} \]

- Shear resistance of web panel under pure shear ignoring flange capacity i.e. taking \( m_{fw} = 0 \).
  - Limiting shear stress \( (\tau_1) \) \( 204.96 \text{ N/mm}^2 \)
  - Resistance \( V_R = \frac{t_w (d_w - h_h) \tau_{aul2}}{g_m \gamma_f3 \times 1000} \)
    \[ = 2984.9 \text{ kN} \]

**Combined bending and shear in web sections**

Web with transverse stiffeners at the supports only. Clause 9.9.3.2.
- Shear force \( V = 2100 \text{ kN} \)
- Co-existent bending moment \( M = 2300 \text{ kNm} \)
RESULTS FROM ANALYSIS

MD design capacity in bending   VD design capacity in shear
Md bending capacity ignoring web VR shear capacity ignoring flanges

MD=381.88 kNm
Md=307.9 kNm
M=2300 kNm
VD=2984.9 kN
VR=2984.9 kN
V=2100 kN

Interaction between shear and bending resistances

Maximum bending moment 2300 kNm is outside interaction curve.
Maximum shear force 2100 kN is within interaction curve.
Interaction relationship exceeded.
Location: Ex6 - Assessment to Standard BD 56/10) (Unequal flanges)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

**Slenderness lamLT**

The proforma calculates the slenderness parameter lambdaLT in accordance with Clause 9.7.

I, Channel, Tee or Angle sections (Clause 9.7.2).

Half wavelength of buckling $l_w=15000$ mm

Effective length to Clause 9.6 $l_e=15000$ mm

2nd moment of area @ minor axis $I_{yy}=156E6$ mm$^4$

Cross sectional area $A=36800$ mm$^2$

Factor $k_4=0.9$

Overall depth of section $D=926.6$ mm

Mean flange thickness $t_f=27.5$ mm

For a uniform section

Slenderness parameter $\lambda_{LT}=(l_e/\gamma_1)*k_4*\eta_*v=207.35$

**Limiting moment of resistance**

The proforma calculates the limiting moment of resistance MR in accordance with Clause 9.8.

Ratio $l_e/l_w$ $\text{rat}_2=l_e/l_w=1$

Section is non-compact.

Mult for non-compact section $\text{Mult}=4340$ kNm

$M_p$ as defined in Clause 9.7.1 $M_p=4000$ kNm

Nominal yield stress comp.flange $\gamma_{cyc}=355$ N/mm$^2$

Nominal yield stress tens.flange $\gamma_{cyc}=355$ N/mm$^2$

Nominal yield stress of web $\gamma_{cyc}=355$ N/mm$^2$

Slenderness ratio $\lambda_{LT}=207.35$

$n=0.008*l_w/l_e*(\beta-30)=1.4878$

$MR=((1+(1+n)*5700/\beta^2)-(SQR((1+(1+n)*5700/\beta^2)^2-22800/\beta^2)))$

$=0.20326$

Limiting moment of resistance $MR=MR*\text{Mult}*0.5=441.07$ kNm

**Bending resistance**

The proforma calculates the bending resistance of beams without longitudinal stiffeners in accordance with Clause 9.9.

Partial factor on yield strength $\gamma_m=1.05$

Partial factor $\gamma_f3=1.1$

Bending resistance $\text{MD}=MR/(\gamma_m*\gamma_f3)=381.88$ kNm

Calculations for the bending resistance ignoring web contribution $M_f$ from Clause 9.9.3.1.

Area of effective tension flange $A_{te}=9824$ mm$^2$

Partial factor on yield strength $\gamma_{mt}=1.05$

Partial factor $\gamma_f3=1.1$

Area of effective compr. flange $A_{ce}=9824$ mm$^2$

Elastic modulus of the section $Z_{xc}=10.9E6$ mm$^3$
Limiting stress (comp. flange) $syc = syc2 = 40.465 \text{ N/mm}^2$
Partial factor on comp. strength $gmc = 1.05$
Partial factor $\gamma_f3$ $gf3 = 1.1$
Force in compression flange $Ffc = (Ace \times syc)/(gmc \times gf3 \times 1E3) = 344.18 \text{ kN}$
Bending resistance is based on flange with lower value of force.
Bending resistance $Mf = Ffc \times df/1E3 = 309.45 \text{ kNm}$

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The assessment calculations are in accordance with Standard BD 56/10 "The Assessment of Steel Highway Bridges and Structures". The result is the shear capacity of a web panel under pure shear calculated using Clause 9.9.2 Shear Resistance.

Depth of web $dwe = 400 \text{ mm}$
Clear length of panel $a = 15000 \text{ mm}$
Web thickness $tw = 19.5 \text{ mm}$
Thickness of flange plates $tft = 25 \text{ mm}$
Nominal yield stress comp. flange $syf1 = syc = 40.465 \text{ N/mm}^2$
Half eff. breadth of flange $bfet$ is the smaller of the following:
1. The result of the expression $bfet1 = 10 \times tft \times \sqrt{355/syf1} = 740.48 \text{ mm}$
2. The distance from the mid-plane of the web to the nearer edge of the flange $bfet2 = 153 \text{ mm}$

Non-dimensional value plastic moment of resistance of top flange:
$mfwt = \frac{syf1 \times bfet \times tft \times tft}{2 \times syw \times dwe \times dwe \times tw} = 0.0017468$

Thickness of plate $tfb = 30 \text{ mm}$
Nominal yield stress bottom flange $syf2 = syt = 355 \text{ N/mm}^2$
Half eff. breadth of flange $bfeb$ is the smaller of the following:
1. The result of the expression $bfeb1 = 10 \times tfb \times \sqrt{355/2} = 3996.9 \text{ mm}$
2. The distance from the mid-plane of the web to the nearer edge of the flange $bfeb2 = 153 \text{ mm}$

Non-dimensional value of plastic moment of resistance of bottom flange.
Value $mfwb$
$mfwb = \frac{syf2 \times bfeb \times tfb \times tfb}{2 \times syw \times dwe \times dwe \times tw} = 0.022067$

Shear resistance results from analysis

Shear resistance of web panel under pure shear.
Limiting shear stress ($\tau_1$) $204.96 \text{ N/mm}^2$
Resistance $VD = \frac{tw \times (dw-hh) \times taul1}{(gm \times gf3 \times 1000)} = 3016 \text{ kN}$

Shear resistance of web panel under pure shear ignoring flange capacity i.e. taking $mfw=0$.
Limiting shear stress ($\tau_1$) $204.96 \text{ N/mm}^2$
Resistance $VR = \frac{tw \times (dw-hh) \times taul2}{(gm \times gf3 \times 1000)} = 3016 \text{ kN}$

Combined bending and shear in web sections

Web with transverse stiffeners at the supports only. Clause 9.9.3.2.
Shear force $V = 2100 \text{ kN}$
Co-existent bending moment $M = 2300 \text{ kNm}$
RESULTS FROM ANALYSIS

MD design capacity in bending       VD design capacity in shear
Mf bending capacity ignoring web   VR shear capacity ignoring flanges

\[ M_f = 309.45 \text{ kNm} \quad V_f = 3016 \text{ kN} \]

Interaction between shear and bending resistances

Maximum bending moment 2300 kNm is outside interaction curve.
Maximum shear force 2100 kN is within interaction curve.
Interaction relationship exceeded.
Location: Ex7 - Slenderness, moment and shear resistance

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

**Slenderness lamLT**

The proforma calculates the slenderness parameter lambdaLT in accordance with Clause 9.7. 

I, Channel, Tee or Angle sections (Clause 9.7.2).

Half wavelength of buckling \( lw = 10400 \text{ mm} \)

Effective length to Clause 9.6 \( le = 10400 \text{ mm} \)

2nd moment of area @ minor axis \( I_{yy} = 133E6 \text{ mm}^4 \)

Cross sectional area \( A = 32300 \text{ mm}^2 \)

Factor \( k4 = 0.9 \)

Overall depth of section \( D = 918.5 \text{ mm} \)

Mean flange thickness \( tf = 27.9 \text{ mm} \)

For a uniform section

Slenderness parameter \( \lambda_{LT} = (le/ry)*k4*\eta*v = 145.87 \)

**Limiting moment of resistance**

The proforma calculates the limiting moment of resistance MR in accordance with Clause 9.8.

Ratio \( le/lw \)

\( \text{rat}2 = le/lw = 1 \)

Section is compact in accordance with Clause 9.3.7

Moment of resistance of section \( \text{Mult} = 3770 \text{ kNm} \)

Mpe as defined in Clause 9.7.1 \( \text{Mpe} = \text{Mult} = 3770 \text{ kNm} \)

Nominal yield stress comp.flange \( \text{syc} = 345 \text{ N/mm}^2 \)

Nominal yield stress tens.flange \( \text{syt} = 345 \text{ N/mm}^2 \)

Nominal yield stress of web \( \text{syw} = 345 \text{ N/mm}^2 \)

Slenderness ratio \( \lambda = \lambda_{LT} = 145.87 \)

\( n = 0.008*lw/le*(\beta - 30) = 0.91037 \)

\( \text{MR} = ((1+(1+n)*5700/\beta^2)-\text{SQR}((1+(1+n)*5700/\beta^2)^2-22800/\beta^2)) \)

\( = 0.41851 \)

Limiting moment of resistance \( \text{MR} = \text{MR} \times \text{Mult} \times 0.5 = 788.89 \text{ kNm} \)

**Bending resistance**

The proforma calculates the bending resistance of beams without longitudinal stiffeners in accordance with Clause 9.9.

Partial factor on yield strength \( \text{gm} = 1.05 \)

Partial factor gammaf3 \( \text{gf3} = 1.1 \)

Bending resistance \( \text{MD} = \text{MR} / (\text{gm} \times \text{gf3}) = 683.02 \text{ kNm} \)

**Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners**

The result is the shear capacity of a web panel under pure shear calculated using Clause 9.9.2 Shear Resistance.
Depth of web  dwe=862 mm
Clear length of panel  a=10400 mm
Web thickness  tw=18 mm
Half eff breadth of flanges bfe is the smaller of the following:
1. The result of the expression  bfe1=10*tf*SQR(355/syf)=283.01 mm
2. The distance from the mid plane of the web to the nearer edge of the flange  bfe2=152 mm
Non-dimensional value of plastic moment of resistance of flange:  
\[
mfw=\frac{(syf*bfe*tf*tf)}{(2*syw*dwe*dwe*tw)}=0.0044232
\]

Shear resistance results from analysis

Shear resistance of web panel under pure shear.
Limiting shear stress ( \( {t_l} \) )  199.19 N/mm²
Resistance  
\[
VD=(tw*(dw-hh)*taul1)/(gm*gf3*1000)
\]
=2851.2 kN

Shear resistance of web panel under pure shear ignoring flange capacity i.e. taking mfw=0.
Limiting shear stress ( \( {t_l} \) )  199.19 N/mm²
Resistance  
\[
VR=(tw*(dw-hh)*taul2)/(gm*gf3*1000)
\]
=2851.2 kN

Combined bending and shear in web sections

Web with transverse stiffeners at the supports only. Clause 9.9.3.2.
Shear force  V=182 kN
Co-existent bending moment M=947 kNm

RESULTS FROM ANALYSIS

MD design capacity in bending  VD design capacity in shear
Mf bending capacity ignoring web  VR shear capacity ignoring flanges

Interaction between shear and bending resistances

Maximum bending moment 947 kNm is outside interaction curve.
Maximum shear force 182 kN is within interaction curve.
Interaction relationship exceeded.
Location: Ex1 - Welded I section

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (λLT) and utilises Appendix C of SCI publication P356 or Clause 6.3.2.2 of BS EN 1993-2:2006.

Compression flange classification will be taken as Class 3.
The web classification will be taken as Class 3.
The overall section classification is Class 3.

Geometrical parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Half wavelength of buckling</td>
<td>Lw=28000 mm</td>
</tr>
<tr>
<td>Eff.length between restraints</td>
<td>LT=8200 mm</td>
</tr>
<tr>
<td>Depth of section</td>
<td>h=1100 mm</td>
</tr>
<tr>
<td>Flange width</td>
<td>b=500 mm</td>
</tr>
<tr>
<td>Flange thickness</td>
<td>tf=40 mm</td>
</tr>
<tr>
<td>Web thickness</td>
<td>tw=10 mm</td>
</tr>
<tr>
<td>Distance between flange centroids</td>
<td>df=1060 mm</td>
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</tbody>
</table>

Design parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coefficient C1</td>
<td>C1=1</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>E=210000 N/mm²</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>G=80768 N/mm²</td>
</tr>
<tr>
<td>Yield strength</td>
<td>fy=345 N/mm²</td>
</tr>
</tbody>
</table>

Expression for lamLT (λLT)

The clause references in this section are as per Appendix C of SCI P356. The following are the 2nd moments of area of the compression and tension flanges, respectively, about the z-z axes:
For buckling over a half wavelength Lw the parameter V is as given by the expression below:

\[ V = \left(4*a*(1-a)+0.05*lamF^2+psia^2\right)^{0.5}+psia \]

Parameter from C.4.5
\[ V_{eq}=V=0.70179 \]

The restraint parameter is calculated from the following expression as given in SCI publication P356:

\[ rp = \frac{V_{eq}^4*Lw^3}{E*Izc*\theta_R*d^2*(1-a)} \]

Rotation of restraint \( \theta_R \)
\[ \theta_R=27.06E-12 \text{ rad/Nmm} \]

Restraint parameter:
\[ rp=V_{eq}^4*Lw^3/(E*Izc*\theta_R*d^2*(1-a))=4003.1 \text{ rad/Nmm} \]

Longest unbraced length
\[ L_r=LT=8200 \text{ mm} \]

Plastic section modulus (yy axis)
\[ W_{ply}=23.88E6 \text{ mm}^3 \]
Elastic section modulus (yy axis) \( W_{ely} = 22.87 \times 10^6 \text{ mm}^3 \)

Beta \( \beta_w \) \( \beta_w = W_{ely}/W_{ply} = 0.95771 \)

Section property from C.1 \( U = 1 \)

Non-dimensional slenderness \( \hat{\lambda}_{LT} \):

\[
\hat{\lambda}_{LT} = 1/(C_1)^{0.5} \cdot U \cdot \sqrt{\beta_w} \cdot \sqrt{\lambda} = 0.90719
\]

**Partial factors on strength**

- Partial factor \( \gamma_{M0} \) \( \gamma_{M0} = 1.0 \)
- Partial factor \( \gamma_{M1} \) \( \gamma_{M1} = 1.1 \)

**Lateral torsional buckling**

The design lateral torsional buckling resistance moment \( M_{bRd} \) will be calculated. Section under consideration is a welded section and EC3 Part 1-1 Clause 6.3.2.2 expressions will be adopted.

Limiting slenderness value \( \lambda_{LT0} = 0.2 \)

According to the NA to BS EN 1993-1-1, for welded doubly symmetrical sections the imperfection value is as follows:

- Imperfection value \( a_{LT} = 0.76 \) (curve d)
- Modified chiLT factor \( \chi_{LT} = \chi_{LT}/f = 0.51675 \)

Design buckling resistance moment \( M_{bRd} = \chi_{LT} \cdot W_{ely} \cdot f_y / (\gamma_{M1} \cdot 10^6) \)

\[ = 3706.6 \text{ kNm} \]

**Interaction equation for moments**

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

Design moment resistance \( M_{cRd} = W_{ely} \cdot f_y / (\gamma_{M0} \cdot 10^6) = 7890.2 \text{ kNm} \)

As \( M_{bRd} \leq M_{cRd} \) \( (3706.6 \text{ kNm} \leq 7890.2 \text{ kNm}) \), the lateral torsional buckling resistance will be adopted.

Design moment from analysis \( M_{Ed} = 3132 \text{ kNm} \)

Unity factor \( unity = M_{Ed}/M_{bRd} = 0.84499 \)

As \( M_{Ed} \leq M_{bRd} \) \( (3132 \text{ kNm} \leq 3706.6 \text{ kNm}) \), section chosen is considered to be suitable.

**Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners**

The shear resistance of a web under pure shear will be evaluated. Beam will be stable against shear buckling due to the presence of closely spaced web transverse stiffeners.

**Shear-moment interaction**

Design shear force (coexisting) \( V_{Ed} = 500 \text{ kN} \)

The value of parameter \( \eta \) from Clause 5.1(2) \( \eta = 1 \)

Plastic shear resistance \( V_{plRd} = \eta \cdot A_w \cdot f_y / (\gamma_{M0} \cdot \sqrt{3}) / 1000 \)

\[ = 2031.7 \text{ kN} \]

As \( V_{Ed} \leq 0.5 \cdot V_{plRd} \), the design shear will not reduce the design resistance moment. Hence OK.
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design moment</td>
</tr>
<tr>
<td></td>
<td>Design shear</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>345 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>1100 mm</td>
</tr>
<tr>
<td>Flange width</td>
<td>500 mm</td>
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<tr>
<td>Classification</td>
<td>Class 3</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>3706.6 kNm</td>
</tr>
<tr>
<td>Plastic shear resistance</td>
<td>2031.7 kN</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.84499 ≤ 1</td>
</tr>
</tbody>
</table>
Location: Ex2 - As Ex1 but with symmetry about both axes

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter \( \lambda_{LT} \) and utilises Appendix C of SCI publication P356 or Clause 6.3.2.2 of BS EN 1993-2:2006.

Compression flange classification will be taken as Class 3.
The web classification will be taken as Class 3.
The overall section classification is Class 3.

Geometrical parameters

- Half wavelength of buckling \( L_w = 28000 \text{ mm} \)
- Eff.length between restraints \( L_T = 8200 \text{ mm} \)
- Depth of section \( h = 1100 \text{ mm} \)
- Flange width \( b = 500 \text{ mm} \)
- Flange thickness \( t_f = 40 \text{ mm} \)
- Web thickness \( t_w = 10 \text{ mm} \)
- Distance between flange centroids \( d_f = 1060 \text{ mm} \)

Design parameters

- Coefficient \( C_1 \) \( C_1 = 1 \)
- Young's modulus \( E = 210000 \text{ N/mm}^2 \)
- Shear modulus \( G = 80768 \text{ N/mm}^2 \)
- Yield strength \( f_y = 345 \text{ N/mm}^2 \)

Expression for \( \lambda_{LT} \) (\( \lambda_{LT} \))

The clause references in this section are as per Appendix C of SCI P356. The following are the 2nd moments of area of the compression and tension flanges, respectively, about the z-z axes:
For buckling over a half wavelength \( L_w \) the parameter \( V \) is as given by the expression below:
\[
V = \left( \frac{(4*a^2*(1-a)+0.05*lamF^2+psia^2)^0.5+psia}{0.5+psia} \right)^{-0.5} = 0.70179
\]
Parameter from C.4.5 \( V = 0.70179 \)
The restraint parameter is calculated from the following expression as given in SCI publication P356:
\[
rp = \left[ \frac{Veq^4*Lw^3}{E*Izc*thetaR*d_f^2*(1-a)} \right]
\]
Rotation of restraint \( \theta_R \) \( \theta_R = 27.06E-12 \text{ rad/Nmm} \)
Restraint parameter:
\[
rp = Veq^4*Lw^3/(E*Izc*thetaR*d_f^2*(1-a)) = 4003.1 \text{ rad/Nmm}
\]
Longest unbraced length \( L_r = 8200 \text{ mm} \)
Plastic section modulus (yy axis) \( W_{ply} = 23.88E6 \text{ mm}^3 \)
Elastic section modulus (yy axis) \( W_{ely} = 22.87 \times 10^6 \text{ mm}^3 \)

Beta \( \beta_w \) 

Section property from C.1 

Non-dimensional slenderness \( \lambda_{LT} \):

\[
\lambda_{LT} = 1 / (C_1)^{0.5} \times U \times V \times D \times \lambda_{LT} \times \text{SQR}(\beta_w) = 0.90719
\]

**Partial factors on strength**

- Partial factor \( \gamma_{M0} \) \( \gamma_{M0} = 1.0 \)
- Partial factor \( \gamma_{M1} \) \( \gamma_{M1} = 1.1 \)

**Lateral torsional buckling**

The design lateral torsional buckling resistance moment \( (M_{bRd}) \) will be calculated. Section under consideration is a welded section and EC3 Part 1-1 Clause 6.3.2.2 expressions will be adopted.

- Limiting slenderness value \( \lambda_{LT0} = 0.2 \)
- According to the NA to BS EN 1993-1-1, for welded doubly symmetrical sections the imperfection value is as follows:
  - Imperfection value \( a_{LT} = 0.76 \) (curve d)
  - Modified \( \chi_{LT} \) factor \( \chi_{LT} = \chi_{LT}/f = 0.51675 \)

Design buckling resistance moment 
\[
M_{bRd} = \chi_{LT} \times W_{ely} \times f_y / (\gamma_{M1} \times 10^6) = 3706.6 \text{ kNm}
\]

**Interaction equation for moments**

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

- Design moment resistance \( M_{cRd} = W_{ely} \times f_y / (\gamma_{M0} \times 10^6) = 7890.2 \text{ kNm} \)
- As \( M_{bRd} \leq M_{cRd} \) (7306.6 \text{ kNm} \leq 7890.2 \text{ kNm}), the lateral torsional buckling resistance will be adopted.

- Design moment from analysis \( M_{Ed} = 3132 \text{ kNm} \)
- Unity factor \( \text{unity} = M_{Ed} / M_{bRd} = 0.84499 \)
- As \( M_{Ed} \leq M_{bRd} \) (3132 \text{ kNm} \leq 3706.6 \text{ kNm}), section chosen is considered to be suitable.

**Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners**

The shear resistance of a web under pure shear will be evaluated. Beam will be stable against shear buckling due to the presence of closely spaced web transverse stiffeners.

**Shear-moment interaction**

- Design shear force (coexisting) \( V_{Ed} = 500 \text{ kN} \)
- Value of parameter \( \eta \) from Clause 5.1(2) \( \eta = 1 \)
- Plastic shear resistance 
  \[
  V_{plRd} = \eta \times A_w \times f_y / (\gamma_{M0} \times \text{SQR}(3)) / 1000 = 2031.7 \text{ kN}
  \]
- As \( V_{Ed} \leq 0.5 \times V_{plRd} \), the design shear will not reduce the design resistance moment. Hence OK.
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>3132 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>500 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>345 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>1100 mm</td>
</tr>
<tr>
<td>Flange width</td>
<td>500 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Class 3</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>3706.6 kNm</td>
</tr>
<tr>
<td>Plastic shear resistance</td>
<td>2031.7 kN</td>
</tr>
<tr>
<td>Unity factor</td>
<td>$0.84499 \leq 1$</td>
</tr>
</tbody>
</table>
Location: Ex3 - Beam 500x300x20 RHS (Hot Rolled section)

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter \( \lambda_{LT} \) and utilises Clause 6.3.2.2 of BS EN 1993-2:2006.

Expression for \( \lambda_{LT} \) (\( \lambda_{LT} \))

The non-dimensional slenderness \( \lambda_{LT} \) could be derived from the following expression:

\[
\lambda_{LT} = \frac{\frac{f_y W_y}{M_{cr}}}{\frac{1}{10^6}}
\]

where,

- \( M_{cr} \) = Elastic critical buckling moment from the analysis
- \( f_y \) = Yield strength
- \( W_y \) = Plastic modulus of the section about y-y axis

Compression flange classification will be taken as Class 2.
The web classification will be taken as Class 2.
The overall section classification is Class 2.

Geometrical parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eff.length between restraints</td>
<td>LT=6833.3 mm</td>
</tr>
<tr>
<td>Depth of section</td>
<td>h=500 mm</td>
</tr>
<tr>
<td>Flange width</td>
<td>b=300 mm</td>
</tr>
<tr>
<td>Flange thickness</td>
<td>tf=20 mm</td>
</tr>
<tr>
<td>Plastic section modulus (yy axis)</td>
<td>W_{pl}=4.89E6 mm³</td>
</tr>
</tbody>
</table>

Design parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus</td>
<td>E=210000 N/mm²</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>G=81000 N/mm²</td>
</tr>
<tr>
<td>Yield strength</td>
<td>f_y=345 N/mm²</td>
</tr>
</tbody>
</table>

Non-dimensional slenderness (\( \lambda_{LT} \))

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective length parameter</td>
<td>K=1</td>
</tr>
<tr>
<td>Length for LTB</td>
<td>LT=LT/1000=8.2 m</td>
</tr>
<tr>
<td>Critical moment</td>
<td>M_{cr}=C1<em>E</em>Gt^0.5/10^6=32465 kNm</td>
</tr>
<tr>
<td>Non-dimensional slenderness</td>
<td>( \lambda_{LT}=(W_{pl}*f_y/(M_{cr}*10^6))^0.5=0.22796 )</td>
</tr>
</tbody>
</table>
Partial factors on strength

Partial factor $\gamma_M0$  $\gamma_M0=1.0$
Partial factor $\gamma_M1$  $\gamma_M1=1.1$

Lateral torsional buckling

The design lateral torsional buckling resistance moment ($MbRd$) will be calculated. Section under consideration is a Hot Rolled section and EC3 Part 1-1 Clause 6.3.2.3 expressions will be adopted.

Limiting slenderness value $\lambda_{LT0}=0.4$
As $\lambda_{LT} \leq \lambda_{LT0}$, the segment of the beam is not subject to lateral torsional buckling.

The design buckling resistance $MbRd$ will therefore be the same as the design resistance for bending $McRd$.

Design buckling resistance moment $MbRd=W_{ply}*f_y/(\gamma_M0*10^6)=1687.1$ kNm

Design moment resistance $McRd=MbRd=1687.1$ kNm

Interaction equation for moments

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

Design moment resistance $McRd=W_{ply}*f_y/(\gamma_M0*10^6)=1687.1$ kNm
As $MbRd \leq McRd$ ($1687.1$ kNm $\leq 1687.1$ kNm ), the lateral torsional buckling resistance will be adopted.

Design moment from analysis $MEd=1600$ kNm
Unity factor $unity=MEd/MbRd=0.9484$
As $MEd \geq MbRd$ ($1600$ kNm $\geq 1687.1$ kNm ), section chosen is considered to be suitable.

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The shear resistance of a web under pure shear will be evaluated. As $hw/tw \leq 72\varepsilon/\eta$ (23 $\leq 59.423$ ), the beam is considered to be stable against shear buckling.

Shear-moment interaction

Design shear force (coexisting) $VEd=500$ kN
The value of parameter $\eta$ is as per the NA to BS EN 1993-1-5.
Value of $\eta$ from Clause 5.1(2) $\eta=1$
Plastic shear resistance $V_{plRd}=\eta*A*w*f_y/(\gamma_M0*SQR(3))*1000 =3784.5$ kN
As $VEd \leq 0.5*V_{plRd}$, the design shear will not reduce the design resistance moment. Hence OK.
<table>
<thead>
<tr>
<th><strong>DESIGN</strong></th>
<th><strong>SUMMARY</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Design moment</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Design shear</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Yield strength of steel</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Depth of beam</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Flange width</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Classification</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Design buckling resistance</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Plastic shear resistance</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Unity factor</strong></td>
</tr>
</tbody>
</table>
Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (ÂLT) from an elastic buckling analysis of the structure.

Expression for lamLT (ÂLT)

The non-dimensional slenderness lamLT (ÂLT) could be derived from the following expression:

\[
\text{ÂLT} = \frac{fy \cdot Wy}{Mcr}
\]

where,

- \(Mcr\) = Elastic critical buckling moment from the analysis
- \(fy\) = Yield strength
- \(Wy\) = Plastic modulus of the section about y-y axis

Geometrical and design parameters

Section is not symmetrical about either axis (Tee section).
Section under consideration is a Hot Rolled section.
Eff.length between restraints \(LT=5000\) mm
Depth of section \(h=317.9\) mm
Width of section \(b=311.4\) mm
Plate thickness \(tf=31.4\) mm
Web thickness \(tw=18.4\) mm
Yield strength \(fy=345\) N/mm²
Section classification \(\text{class}=2\)
The overall section classification will be taken as Class 2.
Plastic section modulus (yy axis) \(Wply=894000\) mm³

Bending moment \(Mcr\)
\(Mcr=2180\) kNm
Non-dimensional slenderness \(\text{ÂLT}\)
\(\text{lamLT}=\text{SQR}(fy \cdot Wply/(Mcr \cdot 1E6))=0.37614\)

Partial factors on strength

Partial factor \(\gamma_M0\) \(\text{gamM0}=1.0\)
Partial factor \(\gamma_M1\) \(\text{gamM1}=1.1\)

Lateral torsional buckling

The design lateral torsional buckling resistance moment (MbRd) will be calculated. Section under consideration is a Hot Rolled section and EC3 Part 1-1 Clause 6.3.2.3 expressions will be adopted.
Limiting slenderness value \(\text{lamLT0}=0.4\)
As \(\text{lamLT} \leq \text{lamLT0}\), the segment of the beam is not subject to lateral torsional buckling.
The design buckling resistance MbRd will therefore be the same as the design resistance for bending McRd.

Design buckling resistance moment MbRd=Wply*fy/(gamM0*10^6)=308.43 kNm
Design moment resistance McRd=MbRd=308.43 kNm

Interaction equation for moments

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

Design moment resistance McRd=Wply*fy/(gamM0*10^6)=308.43 kNm

As MbRd ≤ McRd (308.43 kNm ≤ 308.43 kNm), the lateral torsional buckling resistance will be adopted.

Design moment from analysis MEd=275 kNm
Unity factor unity=MEd/MbRd=0.89161

As MEd ≤ MbRd (275 kNm ≤ 308.43 kNm), section chosen is considered to be suitable.

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The shear resistance of a web under pure shear will be evaluated.

As hw/tw ≤ 72ε/η (15.571 ≤ 59.423), the beam is considered to be stable against shear buckling.

Shear-moment interaction

Design shear force (coexisting) VEd=600 kN

The value of parameter η is as per the NA to BS EN 1993-1-5.
Value of η from Clause 5.1(2) eta=1

Plastic shear resistance VplRd=eta*Aw*fy/(gamM0*SQR(3))/1000 =1050 kN

Plastic moment resistance MyvRd=300 kNm
Design moment resistance McRd=308.43 kNm

Hence the moment resistance of the section reduces from 308.43 kNm to 300 kNm with coexistent shear of 600 kN.
As MEd ≤ MyvRd (275 kNm ≤ 300 kNm), the design moment is less than the design moment resistance. Hence OK.

Provide web transverse stiffeners at supports.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>275 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>600 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>345 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>317.9 mm</td>
</tr>
<tr>
<td>Width of section</td>
<td>311.4 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Class 2</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>308.43 kNm</td>
</tr>
<tr>
<td>Plastic shear resistance</td>
<td>1050 kN</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.89161 ≤ 1</td>
</tr>
</tbody>
</table>
Location: Ex5 - Composite section (SCI/P357 Sect. 9.1) welded sect.

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter \( \lambda_{LT} \) using the simplified method of EN 1993-2, as allowed by BS EN 1994-2:2005.

Expression for \( \lambda_{LT} \) (\( \lambda_{ALT} \)) - composite construction

The section under consideration is assumed to be at an internal support of a continues girder where hogging bending moment with axial force are present. The non-dimensional slenderness \( \lambda_{LT} \) (\( \lambda_{ALT} \)) could be derived from the following expression as per Clause 6.3.4.2(4):

\[
\lambda_{ALT} = \frac{fy \cdot A_{eff}}{N_{crit}}
\]

where,

- \( N_{crit} \) = Elastic critical load
- \( fy \) = Yield strength
- \( A_{eff} \) = Effective area of the chord

Geometrical and design parameters

Compression flange classification will be taken as Class 3.
Section symmetrical only about the minor axis (unequal flanges).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eff.length between restraints</td>
<td>LT=5900 mm</td>
</tr>
<tr>
<td>Depth of section</td>
<td>h=1100 mm</td>
</tr>
<tr>
<td>Top flange width</td>
<td>bt=500 mm</td>
</tr>
<tr>
<td>Bottom flange width</td>
<td>bb=600 mm</td>
</tr>
<tr>
<td>Top flange thickness</td>
<td>tft=40 mm</td>
</tr>
<tr>
<td>Bottom flange thickness</td>
<td>tfb=60 mm</td>
</tr>
<tr>
<td>Web thickness</td>
<td>tw=14 mm</td>
</tr>
<tr>
<td>Yield strength</td>
<td>fy=335 N/mm²</td>
</tr>
<tr>
<td>Total design moment</td>
<td>M₁=11950 kNm</td>
</tr>
<tr>
<td>Total shear</td>
<td>V₁=1528 kNm</td>
</tr>
<tr>
<td>Design moment at brace position</td>
<td>M₂=4000 kNm</td>
</tr>
<tr>
<td>Design shear at brace position</td>
<td>V₂=1080 kNm</td>
</tr>
<tr>
<td>Non-dimensional slenderness ( \lambda_{ALT} )</td>
<td>lamLT=SQR(fy*Aeff/(N_{crit}*1E3))=0.3635</td>
</tr>
</tbody>
</table>

Partial factors on strength

| Partial factor \( \gamma_{M0} \) | \( \gamma_{M0}=1.0 \) |
| Partial factor \( \gamma_{M1} \) | \( \gamma_{M1}=1.1 \) |
**Lateral torsional buckling**

The design lateral torsional buckling resistance moment \( (M_{brd}) \) will be calculated. Section under consideration is a welded section and EC3 Part 1-1 Clause 6.3.2.2 expressions will be adopted.

Limiting slenderness value \( \lambda_{LT0}=0.2 \)

According to the NA to BS EN 1993-1-1, Tables 6.3 and 6.4 could be used with Clause 6.3.2.2 to determine the imperfection factor \( \alpha_{LT} \).

Imperfection value \( \alpha_{LT}=0.76 \) (curve d)

Modified chiLT factor \( \chi_{LT}=\chi_{LT}/f=0.87678 \)

Design moment on steel section \( M_{Ed}=2573 \text{ kNm} \)

Composite cracked section moment \( M_{cEd}=M_{max}-M_{Ed}=9377 \text{ kNm} \)

Design yield strength of steel \( f_{yd}=f_{y}/\gamma_{M1}=304.55 \text{ N/mm}^2 \)

Construction stage:

- Stress in steel bottom flange \( s_{sb1}=67 \text{ N/mm}^2 \)
- Composite stage:
  - Stress in steel bottom flange \( s_{sb2}=205 \text{ N/mm}^2 \)
  - Factor \( k \) (EN 1994-2 Cl. 6.2.1.4) \( k=(f_{yd}-s_{sb1})/s_{sb2}=1.1588 \)
  - Design elastic resistance moment \( M_{elRd}=M_{Ed}+k*M_{cEd}=13439 \text{ kNm} \)
  - Modification factor \( \chi_{LT}=0.87678 \)
  - Design buckling resistance moment \( M_{brd}=\chi_{LT}*M_{elRd}=11783 \text{ kNm} \)

**Interaction equation for moments**

- Distance \( L_{1}=0.25*L/\sqrt{m}=1197.6 \text{ mm} \)
- Design moment from analysis \( M_{Ed}=M_{1}-(M_{1}-M_{2})*L_{1}/L=10336 \text{ kNm} \)
- Axial stress from the analysis \( \text{stress}=5 \text{ N/mm}^2 \)
- Axial load \( N_{Ed}=A_{eff}*\text{stress}/1000=192.37 \text{ kN} \)
- Axial resistance of Tee section \( N_{brd}=\chi_{LT}*A_{eff}*f_{yd}/1000=10273 \text{ kN} \)
- Unity factor \( \text{unity}=M_{Ed}/M_{brd}+N_{Ed}/N_{brd}=0.89597 \)

As unity \( \leq 1 \), section chosen is considered to be suitable.

**Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners**

The shear resistance of a web under pure shear will be evaluated. Beam will be stable against shear buckling due to the presence of closely spaced web transverse stiffeners.

**Shear-moment interaction**

- Design shear force (coexisting) \( V_{Ed}=600 \text{ kN} \)
- The value of parameter \( \eta \) is as per the NA to BS EN 1993-1-5.
- Value of \( \eta \) from Clause 5.1(2) \( \eta=1 \)
- Plastic shear resistance \( V_{plRd}=\eta*A*w*f_{y}/(\gamma_{M0}*(\sqrt{3}))/1000 =2707.8 \text{ kN} \)

As \( V_{Ed} \leq 0.5*V_{plRd} \), the design shear will not reduce the design resistance moment. Hence OK.
<table>
<thead>
<tr>
<th><strong>DESIGN</strong></th>
<th><strong>SUMMARY</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>10336 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>600 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>335 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>1100 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Class 3</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>11783 kNm</td>
</tr>
<tr>
<td>Plastic shear resistance</td>
<td>2707.8 kN</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.89597 ≤ 1</td>
</tr>
</tbody>
</table>
Location: Ex6 - 430x100 Channel, simplified expression (Hot Rolled)

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (\( \hat{\alpha}L_{T} \)) and utilises Appendix C of SCI publication P356 or Clause 6.3.2.2 of BS EN 1993-2:2006.

Compression flange classification will be taken as Class 1.
The web classification will be taken as Class 1.
The overall section classification is Class 1.

Geometrical parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Half wavelength of buckling (Lw)</td>
<td>8000 mm</td>
</tr>
<tr>
<td>Eff.length between restraints (LT)</td>
<td>4000 mm</td>
</tr>
<tr>
<td>Depth of section (h)</td>
<td>430 mm</td>
</tr>
<tr>
<td>Flange width (b)</td>
<td>100 mm</td>
</tr>
<tr>
<td>Flange thickness (tf)</td>
<td>19 mm</td>
</tr>
<tr>
<td>Web thickness (tw)</td>
<td>11 mm</td>
</tr>
<tr>
<td>Second moment of area (z-z axis)</td>
<td>7.22E6 mm⁴</td>
</tr>
<tr>
<td>St.Venant torsional constant (It)</td>
<td>630000 mm⁴</td>
</tr>
<tr>
<td>Distance between flange centroids (df)</td>
<td>411 mm</td>
</tr>
</tbody>
</table>

Design parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>1</td>
</tr>
<tr>
<td>E</td>
<td>210000 N/mm²</td>
</tr>
<tr>
<td>G</td>
<td>81000 N/mm²</td>
</tr>
<tr>
<td>fy</td>
<td>345 N/mm²</td>
</tr>
</tbody>
</table>

Expression for lamLT (\( \hat{\alpha}L_T \))

The clause references in this section are as per Appendix C of SCI P356.

2nd moment of area (comp flange) \( I_{zc} = 1.5833E6 \) mm⁴
2nd moment of area (tens. flange) \( I_{zt} = 1.5833E6 \) mm⁴

For buckling over a half wavelength \( Lw \) the parameter \( V \) is as given by the expression below:

\[
V = (4\times a \times (1-a) + 0.05 \times \lambda_{FM}^2 + 0.1 \times \psi_a^2)^{-0.5} = 0.59424
\]

Parameter from C.4.5

\[
\omega = \pi^2 \times \frac{df^2 \times E \times Iz}{G \times It \times Lw^2}
\]

compute

\[
compl = SQR(4 \times tee \times omegaa) + \psi_a \\
\times SQR(omegaa) = 2.1849
\]

Parameter from C.4.5

\[
V_{eq} = (2 \times a \times \omega \times compl^2)^{0.25} = 0.63455
\]

The restraint parameter is calculated from the following expression as given in SCI publication P356:

\[
\frac{Ve_{eq}^4 \times Lw^3}{E \times Iz_{c} \times 0R \times df^2 \times (1-a)}
\]
Rotation of restraint $\theta_R$  
\[
\theta_R = 27 \times 10^{-12} \text{ rad/Nmm}
\]

Restrainment parameter:  
\[
\rho_p = \frac{V_{eq}^4 \cdot L_w^3}{(E \cdot I_{zc} \cdot \theta_R \cdot df^2 \cdot (1-a))} = 109477 \text{ rad/Nmm}
\]

Longest unbraced length  
\[L_r = L_T = 4000 \text{ mm}\]

Beta $\beta_w$  
\[\beta_w = 1.0\]

Plastic section modulus (yy axis)  
\[W_{ply} = 1.22 \times 10^6 \text{ mm}^3\]

Section property from C.1  
\[U = 0.9\]

Non-dimensional slenderness $\hat{\lambda}_{LT}$:  
\[
\hat{\lambda}_{LT} = \frac{1}{(C_1)^{0.5} \cdot U \cdot V \cdot D \cdot \lambda_b \cdot \sqrt{(\beta_w)}} = 1.3942
\]

Partial factors on strength

Partial factor $\gamma_{M0}$  
\[\gamma_{M0} = 1.0\]

Partial factor $\gamma_{M1}$  
\[\gamma_{M1} = 1.1\]

Lateral torsional buckling

The design lateral torsional buckling resistance moment ($M_{bRd}$) will be calculated. Section under consideration is a Hot Rolled section and EC3 Part 1-1 Clause 6.3.2.3 expressions will be adopted.  

Limiting slenderness value  
\[\hat{\lambda}_{LT0} = 0.4\]

According to the NA to BS EN 1993-1-1, for rolled doubly symmetrical sections the imperfection value is as follows:  
Imperfection value  
\[a_{LT} = 0.76 \text{ (curve d)}\]

According to the NA to BS EN 1993-1-1, for not doubly symmetrical sections the imperfection value is as follows:  
Imperfection value  
\[a_{LT} = 0.76 \text{ (curve d)}\]

Modified chi$_{LT}$ factor  
\[\text{Chi}_{LT} = \frac{\chi_{LT}}{f} = 0.37499\]

Design buckling resistance moment  
\[M_{bRd} = \text{Chi}_{LT} \cdot W_{ply} / (\gamma_{M1} \cdot 10^6) = 143.49 \text{ kNm}\]

Interaction equation for moments

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:  

Design moment resistance  
\[M_{cRd} = W_{ply} / (\gamma_{M0} \cdot 10^6) = 420.9 \text{ kNm}\]

As $M_{bRd} \leq M_{cRd} \leq 420.9 \text{ kNm}$, the lateral torsional buckling resistance will be adopted.  

Design moment from analysis  
\[M_{Ed} = 135 \text{ kNm}\]

Unity factor  
\[\text{unity} = M_{Ed} / M_{bRd} = 0.94085\]

As $M_{Ed} \leq M_{bRd} \leq 135 \text{ kNm}$, section chosen is considered to be suitable.

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The shear resistance of a web under pure shear will be evaluated. As $h_w/t_w \leq 72/\eta$ (35.636 \leq 59.423), the beam is considered to be stable against shear buckling.
Shear-moment interaction

Design shear force (coexisting) \( V_{Ed} = 450 \text{ kN} \)
The value of parameter \( \eta \) is as per the NA to BS EN 1993-1-5.
Value of \( \eta \) from Clause 5.1(2) \( \eta = 1 \)
Plastic shear resistance \( V_{plRd} = \frac{\eta \cdot A_w \cdot f_y}{(\gamma M_0 \cdot \sqrt{3})/1000} \)
\[ = 858.89 \text{ kN} \]
The beam is not symmetrical and EC3 Part 1-1 expression 6.30 cannot be used. The plastic neutral axis at height \( x_1 \) from the top of the bottom flange is found by balancing forces.
Allowable stress in the web \( f_{yw} = (1 - \rho_o) \cdot f_y = 344.21 \text{ N/mm}^2 \)
Reduced design plastic moment resistance:
\[ M_{vRd} = \frac{(A_1 \cdot L_1 + A_2 \cdot L_2) \cdot f_y + (A_3 \cdot L_3 + A_4 \cdot L_4) \cdot f_{yw}}{1E6} = 414.87 \text{ kNm} \]
Design moment resistance \( M_{Rd} = 420.9 \text{ kNm} \)
Hence the moment resistance of the section reduces from 420.9 kNm to 414.87 kNm with coexistent shear of 450 kN.
As \( M_{Ed} \leq M_{vRd} \ (135 \text{ kNm} \leq 414.87 \text{ kNm}) \), the design moment is less than the design moment resistance. Hence OK.

Provide web transverse stiffeners at supports.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>135 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>450 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>345 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>430 mm</td>
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<tr>
<td>Top flange width</td>
<td>100 mm</td>
</tr>
<tr>
<td>Bottom flange width</td>
<td>100 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Class 1</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>143.49 kNm</td>
</tr>
<tr>
<td>Plastic shear resistance</td>
<td>858.89 kN</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.94085 ≤ 1</td>
</tr>
</tbody>
</table>
Location: Ex7 - Channel section using Mcr expression (Hot Rolled)

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (λLT) and utilises Appendix C of SCI publication P356 or Clause 6.3.2.2 of BS EN 1993-2:2006.

Compression flange classification will be taken as Class 1.
The web classification will be taken as Class 1.
The overall section classification is Class 1.

Geometrical parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Half wavelength of buckling</td>
<td>Lw=8000 mm</td>
</tr>
<tr>
<td>Eff. length between restraints</td>
<td>LT=4000 mm</td>
</tr>
<tr>
<td>Depth of section</td>
<td>h=430 mm</td>
</tr>
<tr>
<td>Flange width</td>
<td>b=100 mm</td>
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<tr>
<td>Flange thickness</td>
<td>tf=19 mm</td>
</tr>
<tr>
<td>Web thickness</td>
<td>tw=11 mm</td>
</tr>
<tr>
<td>Second moment of area (z-z axis)</td>
<td>Iz=7.22E6 mm⁴</td>
</tr>
<tr>
<td>St. Venant torsional constant</td>
<td>It=630000 mm⁴</td>
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</table>

Design parameters

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<th>Parameter</th>
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</thead>
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<td>Coefficient C1</td>
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<tr>
<td>Young's modulus</td>
<td>E=210000 N/mm²</td>
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<tr>
<td>Shear modulus</td>
<td>G=81000 N/mm²</td>
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<tr>
<td>Yield strength</td>
<td>fy=345 N/mm²</td>
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</table>

Expression for lamLT (λLT)

<table>
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<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Beam width</td>
<td>b=100 mm</td>
</tr>
<tr>
<td>Web thickness</td>
<td>tw=11 mm</td>
</tr>
<tr>
<td>Warping constant</td>
<td>Iw=219E9 mm⁶</td>
</tr>
<tr>
<td>Plastic section modulus (yy axis)</td>
<td>Wply=1.22E6 mm³</td>
</tr>
</tbody>
</table>

Partial factors on strength

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partial factor γM0</td>
<td>gamM0=1.0</td>
</tr>
<tr>
<td>Partial factor γM1</td>
<td>gamM1=1.1</td>
</tr>
</tbody>
</table>

Lateral torsional buckling

The design lateral torsional buckling resistance moment (MbRd) will be calculated. Section under consideration is a Hot Rolled section and EC3 Part 1-1 Clause 6.3.2.3 expressions will be adopted.

Limiting slenderness value lamLT0=0.4

According to the NA to BS EN 1993-1-1, for rolled doubly symmetrical sections the imperfection value is as follows:
Imperfection value \( a_{LT} = 0.76 \) (curve d)

According to the NA to BS EN 1993-1-1, for not doubly symmetrical sections the imperfection value is as follows:

Imperfection value \( a_{LT} = 0.76 \) (curve d)

Modified chiLT factor \( \chi_{LT} = \frac{\chi_{LT}}{f} = 0.43593 \)

Design buckling resistance moment

\[
Mb_{Rd} = \chi_{LT} \cdot W_{pily} \cdot f_y / (\gamma_M1 \cdot 10^6)
\]

\( = 166.8 \) kNm

**Interaction equation for moments**

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

Design moment resistance

\[
Mc_{Rd} = \frac{W_{pily} \cdot f_y}{(\gamma_M0 \cdot 10^6)} = 420.9 \text{ kNm}
\]

As \( Mb_{Rd} \leq Mc_{Rd} \) (166.8 kNm \( \leq 420.9 \) kNm), the lateral torsional buckling resistance will be adopted.

Design moment from analysis

\( M_{Ed} = 155 \) kNm

Unity factor

\( \text{unity} = \frac{M_{Ed}}{Mb_{Rd}} = 0.92925 \)

As \( M_{Ed} \leq Mb_{Rd} \) (155 kNm \( \leq 166.8 \) kNm), section chosen is considered to be suitable.

**Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners**

The shear resistance of a web under pure shear will be evaluated.

As \( hw/tw \leq 72 \epsilon/\eta \) (35.636 \( \leq 59.423 \)), the beam is considered to be stable against shear buckling.

**Shear-moment interaction**

Design shear force (coexisting)

\( V_{Ed} = 450 \) kN

The value of parameter \( \eta \) is as per the NA to BS EN 1993-1-5.

Value of \( \eta \) from Clause 5.1(2)

\( \eta = 1 \)

Plastic shear resistance

\[
V_{pplRd} = \frac{\eta \cdot A_w \cdot f_y}{(\gamma_M0 \cdot SQR(3))} / 1000
\]

\( = 858.89 \) kN

The beam is not symmetrical and EC3 Part 1-1 expression 6.30 cannot be used. The plastic neutral axis at height \( x1 \) from the top of the bottom flange is found by balancing forces.

Allowable stress in the web

\( f_{yw} = (1 - \rho) \cdot f_y = 344.21 \text{ N/mm}^2 \)

Reduced design plastic moment resistance:

\[
My_{vRd} = \left( (A1 \cdot L1 + A2 \cdot L2) \cdot f_y + (A3 \cdot L3 + A4 \cdot L4) \cdot f_{yw} \right) / 1E6 = 414.87 \text{ kNm}
\]

Design moment resistance

\( Mc_{Rd} = 420.9 \) kNm

Hence the moment resistance of the section reduces from 420.9 kNm to 414.87 kNm with coexistent shear of 450 kN.

As \( M_{Ed} \leq My_{vRd} \) (155 kNm \( \leq 414.87 \) kNm), the design moment is less than the design moment resistance. Hence OK.

Provide web transverse stiffeners at supports.
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>155 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>450 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>345 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>430 mm</td>
</tr>
<tr>
<td>Top flange width</td>
<td>100 mm</td>
</tr>
<tr>
<td>Bottom flange width</td>
<td>100 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Class 1</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>166.8 kNm</td>
</tr>
<tr>
<td>Plastic shear resistance</td>
<td>858.89 kN</td>
</tr>
<tr>
<td>Unity factor</td>
<td>$0.92925 \leq 1$</td>
</tr>
</tbody>
</table>
Location: Ex8 - Hot Rolled I section using Mcr expression

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (ALT) and utilises Appendix C of SCI publication P356 or Clause 6.3.2.2 of BS EN 1993-2:2006.

Compression flange classification will be taken as Class 1.
The web classification will be taken as Class 1.
The overall section classification is Class 1.

Geometrical parameters

Half wavelength of buckling \( L_w = 9000 \text{ mm} \)
Eff.length between restraints \( L_T = 3000 \text{ mm} \)
Depth of section \( h = 528.3 \text{ mm} \)
Flange width \( b = 208.8 \text{ mm} \)
Flange thickness \( t_f = 13.2 \text{ mm} \)
Web thickness \( t_w = 9.6 \text{ mm} \)
Distance between flange centroids \( d_f = 515 \text{ mm} \)

Design parameters

Coefficient \( C_1 \) \( C_1 = 1 \)
Young's modulus \( E = 210000 \text{ N/mm}^2 \)
Shear modulus \( G = 81000 \text{ N/mm}^2 \)
Yield strength \( f_y = 275 \text{ N/mm}^2 \)

Expression for lamLT (ALT)

Warping constant \( I_w = 1.3333E12 \text{ mm}^6 \)
Plastic section modulus (yy axis) \( W_{ply} = 2.06E6 \text{ mm}^3 \)

Partial factors on strength

Partial factor \( \gamma_0 \) \( \gamma_0M_0 = 1.0 \)
Partial factor \( \gamma_1 \) \( \gamma_1M_1 = 1.1 \)

Lateral torsional buckling

The design lateral torsional buckling resistance moment \( (M_{BRd}) \) will be calculated. Section under consideration is a Hot Rolled section and EC3 Part 1-1 Clause 6.3.2.3 expressions will be adopted.
Limiting slenderness value \( \text{lamLT}_0 = 0.4 \)
According to the NA to BS EN 1993-1-1, for rolled doubly symmetrical sections the imperfection value is as follows:
Imperfection value \( a_{LT} = 0.49 \) (curve c)
Modified chiLT factor \( \text{ChiLT} = \text{chiLT}/f = 0.84391 \)
Design buckling resistance moment \( M_{BRd} = \text{ChiLT} \cdot W_{ply} \cdot f_y / (\gamma_1M_1 \cdot 10^6) \)
\[ = 434.61 \text{ kNm} \]
Interaction equation for moments

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

Design moment resistance $McRd = Wply*fy/(\gamma_M0*10^6) = 566.5 \text{ kNm}$

As $MbRd \leq McRd$ (434.61 kNm $\leq 566.5 \text{ kNm}$), the lateral torsional buckling resistance will be adopted.

Design moment from analysis $MEd = 425 \text{ kNm}$

Unity factor $\text{unity} = MEd/MbRd = 0.97788$

As $MEd \leq MbRd$ (425 kNm $\leq 434.61 \text{ kNm}$), section chosen is considered to be suitable.

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The shear resistance of a web under pure shear will be evaluated. As $hw/tw \leq 72\varepsilon/\eta$ (52.281 $\leq 66.558$), the beam is considered to be stable against shear buckling.

Shear-moment interaction

Design shear force (coexisting) $VEd = 400 \text{ kN}$

The value of parameter $\eta$ is as per the NA to BS EN 1993-1-5.

Value of $\eta$ from Clause 5.1(2) $\eta = 1$

Plastic shear resistance $VplRd = \eta*Aw*fy/(\gamma_M0*SQR(3))/1000$

$= 765 \text{ kN}$

As the beam is symmetric EC3 Part 1-1 expression 6.30 will be used to evaluate the reduced design plastic moment resistance:

$MyvRd = (Wply-ro*Aw^2/(4*tw))*fy/(\gamma_M0*1E6) = 566.15 \text{ kNm}$

Design moment resistance $McRd = 566.5 \text{ kNm}$

Hence the moment resistance of the section reduces from 566.5 kNm to 566.15 kNm with coexistent shear of 400 kN.

As $MEd \leq MyvRd$ (425 kNm $\leq 566.15 \text{ kNm}$), the design moment is less than the design moment resistance. Hence OK.

Provide web transverse stiffeners at supports.

| DESIGN | Design moment | 425 kNm |
| SUMMARY | Design shear | 400 kN |
|         | Yield strength of steel | 275 N/mm² |
|         | Depth of beam | 528.3 mm |
|         | Flange width | 208.8 mm |
|         | Classification | Class 1 |
|         | Design buckling resistance | 434.61 kNm |
|         | Plastic shear resistance | 765 kN |
|         | Unity factor | 0.97788 $\leq 1$ |
Location: Ex9 - Beam 914x305x253 UKB (Hot Rolled section)

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (ALT) and utilises Appendix C of SCI publication P356 or Clause 6.3.2.2 of BS EN 1993-2:2006.

Compression flange classification will be taken as Class 1.
The web classification will be taken as Class 1.
The overall section classification is Class 1.

Geometrical parameters

Half wavelength of buckling \( L_w=10000 \) mm
Effective length between restraints \( L_T=2000 \) mm
Depth of section \( h=918.5 \) mm
Flange width \( b=305.5 \) mm
Flange thickness \( t_f=27.9 \) mm
Web thickness \( t_w=17.3 \) mm
Distance between flange centroids \( d_f=890 \) mm

Design parameters

Coefficient \( C_1 \)
Young's modulus \( E=210000 \) N/mm\(^2\)
Shear modulus \( G=81000 \) N/mm\(^2\)
Yield strength \( f_y=345 \) N/mm\(^2\)

Expression for lamLT (ALT)

Warping constant \( I_w=26.4E12 \) mm\(^4\)
Plastic section modulus (yy axis) \( W_{ply}=10.9E6 \) mm\(^3\)

Partial factors on strength

Partial factor \( \gamma M_0 \) \( \gamma M_0=1.0 \)
Partial factor \( \gamma M_1 \) \( \gamma M_1=1.1 \)

Lateral torsional buckling

The design lateral torsional buckling resistance moment \( (M_bRd) \) will be calculated. Section under consideration is a Hot Rolled section and EC3 Part 1-1 Clause 6.3.2.3 expressions will be adopted.

Limiting slenderness value \( \text{lamLT}_0=0.4 \)
As \( \text{lamLT} \leq \text{lamLT}_0 \), the segment of the beam is not subject to lateral torsional buckling.

The design buckling resistance \( M_bRd \) will therefore be the same as the design resistance for bending \( M_cRd \).

Design buckling resistance moment \( M_bRd=W_{ply}*f_y/(\gamma M_0*10^6)=3760.5 \) kNm
Design moment resistance  \( \text{McRd}=\text{MbRd}=3760.5 \text{ kNm} \)

**Interaction equation for moments**

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

Design moment resistance  \( \text{McRd}=W_{\text{ply}}*f_y/(\gamma_M*10^6)=3760.5 \text{ kNm} \)

As \( \text{MbRd} \leq \text{McRd} \ (3760.5 \text{ kNm} \leq 3760.5 \text{ kNm}) \), the lateral torsional buckling resistance will be adopted.

Design moment from analysis  \( \text{MEd}=3500 \text{ kNm} \)

Unity factor  \( \text{unity}=\text{MEd}/\text{MbRd}=0.93073 \)

As \( \text{MEd} \leq \text{MbRd} \ (3500 \text{ kNm} \leq 3760.5 \text{ kNm}) \), section chosen is considered to be suitable.

**Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners**

The shear resistance of a web under pure shear will be evaluated. As \( hw/\ell \leq 72\varepsilon/\eta \ (49.867 \leq 59.423) \), the beam is considered to be stable against shear buckling.

**Shear-moment interaction**

Design shear force (coexisting)  \( \text{VED}=182.3 \text{ kN} \)

The value of parameter \( \eta \) is as per the NA to BS EN 1993-1-5.  
Value of \( \eta \) from Clause 5.1(2)  \( \eta=1 \)

Plastic shear resistance  \( \text{VplRd}=\eta*Aw*f_y/(\gamma_M*\text{SQR}(3))/1000 =2972.8 \text{ kN} \)

As \( \text{VED} \leq 0.5*\text{VplRd} \), the design shear will not reduce the design resistance moment. Hence OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
<th>3500 kNm</th>
<th>182.3 kN</th>
<th>345 N/mm²</th>
<th>918.5 mm</th>
<th>305.5 mm</th>
<th>Class 1</th>
<th>3760.5 kNm</th>
<th>2972.8 kN</th>
<th>0.93073 ≤ 1</th>
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</thead>
<tbody>
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<td>Depth of beam</td>
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<tr>
<td>Unity factor</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex10 - ICE book worked example 6.2-12 (no shear buckling)

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (ΛLT) and utilises Appendix C of SCI publication P356 or Clause 6.3.2.2 of BS EN 1993-2:2006.

Compression flange classification will be taken as Class 2.
The web classification will be taken as Class 2.
The overall section classification is Class 2.

Geometrical parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Half wavelength of buckling</td>
<td>Lw=10000 mm</td>
</tr>
<tr>
<td>Eff.length between restraints</td>
<td>LT=1000 mm</td>
</tr>
<tr>
<td>Depth of section</td>
<td>h=1200 mm</td>
</tr>
<tr>
<td>Top flange width</td>
<td>bt=400 mm</td>
</tr>
<tr>
<td>Bottom flange width</td>
<td>bb=500 mm</td>
</tr>
<tr>
<td>Top flange thickness</td>
<td>tft=30 mm</td>
</tr>
<tr>
<td>Bottom flange thickness</td>
<td>tbf=30 mm</td>
</tr>
<tr>
<td>Web thickness</td>
<td>tw=20 mm</td>
</tr>
<tr>
<td>St.Venant torsional constant</td>
<td>It=11.1E6 mm^4</td>
</tr>
<tr>
<td>Second moment of area (z-z axis)</td>
<td>Iz=472E6 mm^4</td>
</tr>
<tr>
<td>Distance between flange centroids</td>
<td>df=1170 mm</td>
</tr>
</tbody>
</table>

Design parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coefficient C1</td>
<td>C1=1.13</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>E=210000 N/mm^2</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>G=81000 N/mm^2</td>
</tr>
<tr>
<td>Yield strength</td>
<td>fy=355 N/mm^2</td>
</tr>
</tbody>
</table>

Expression for lamLT (ΛLT)

The clause references in this section are as per Appendix C of SCI P356.

2nd moment of area (comp flange) Izc=312.5E6 mm^4
2nd moment of area (tens.flange) Izt=160E6 mm^4

For buckling over a half wavelength Lw the parameter V is as given by the expression below:

V=((4*a*(1-a)+0.05*lamF^2+psia^2)^0.5+psia)^-0.5=0.84668

Parameter from C.4.5

omega=PI^2*dt^2*E*Iz/(G*It*Lw^2)
omega=14.894

compute

comp1=SQR(4+tee*omega)+psia*SQR(omega)=5.2785

Parameter from C.4.5

Veq=(2*a*omega/comp1^2)^0.25=0.917
The restraint parameter is calculated from the following expression as given in SCI publication P356:

\[\text{rp} = \frac{V_{eq}^4 L_w^3}{E I_{zc} \theta R df^2 (1-a)}\]

Rotation of restraint \(\theta R\) \(\theta R = 1 \times 10^{-15}\) rad/Nmm

Restraint parameter:
\[\text{rp} = \frac{V_{eq}^4 L_w^3}{E I_{zc} \theta R df^2 (1-a)} = 23.244 \times 10^6\] rad/Nmm

Longest unbraced length \(L_r = L_T = 1000\) mm

Beta \(\beta_w\) \(\beta_w = 1.0\)

Plastic section modulus (yy axis) \(W_{ply} = 22E6\) mm³

Section property from C.1 \(U = 0.9\)

Non-dimensional slenderness \(\lambda_{LT}\):
\[\lambda_{LT} = \frac{1}{(C1)^{0.5} U V D \lambda_{SQR}(\beta_w)} = 0.10196\]

Partial factors on strength

Partial factor \(\gamma M_0\) \(\gamma M_0 = 1.0\)
Partial factor \(\gamma M_1\) \(\gamma M_1 = 1.1\)

Lateral torsional buckling

The design lateral torsional buckling resistance moment \(M_{Brd}\) will be calculated. Section under consideration is a welded section and EC3 Part 1-1 Clause 6.3.2.2 expressions will be adopted. Limiting slenderness value \(\lambda_{LT0} = 0.2\)

As \(\lambda_{LT} \leq \lambda_{LT0}\), the segment of the beam is not subject to lateral torsional buckling.

The design buckling resistance \(M_{Brd}\) will therefore be the same as the design resistance for bending \(M_{Cr}\).

Design buckling resistance moment \(M_{Brd} = W_{ply} f_y / (\gamma M_0 \times 10^6) = 7810\) kNm

Design moment resistance \(M_{Cr} = M_{Brd} = 7810\) kNm

Interaction equation for moments

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

Design moment resistance \(M_{Cr} = W_{ply} f_y / (\gamma M_0 \times 10^6) = 7810\) kNm

As \(M_{Brd} \leq M_{Cr}\) \((7810\) kNm \(\leq 7810\) kNm\), the lateral torsional buckling resistance will be adopted.

Design moment from analysis \(M_{Ed} = 7000\) kNm

Unity factor \(\text{unity} = M_{Ed} / M_{Brd} = 0.89629\)

As \(M_{Ed} \leq M_{Brd}\) \((7000\) kNm \(\leq 7810\) kNm\), section chosen is considered to be suitable.
Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The shear resistance of a web under pure shear will be evaluated. As \( \frac{hw}{tw} \leq 72\eta/\varepsilon \) ( 57 \leq 58.58 ), the beam is considered to be stable against shear buckling.

Shear-moment interaction

Design shear force (coexisting) \( V_{Ed} = 4486 \text{ kN} \)
The value of parameter \( \eta \) is as per the NA to BS EN 1993-1-5.
Value of \( \eta \) from Clause 5.1(2) \( \eta = 1.2 \)
Plastic shear resistance \( V_{plRd} = \eta \cdot \frac{Aw \cdot fy}{(\Gamma M0 \cdot SQR(3))}/1000 \)
\( = 5607.7 \text{ kN} \)
The beam is not symmetrical and EC3 Part 1-1 expression 6.30 cannot be used. The plastic neutral axis at height \( x_1 \) from the top of the bottom flange is found by balancing forces.
Allowable stress in the web \( f_{yw} = (1-\rho_0) \cdot fy = 227.22 \text{ N/mm}^2 \)
Reduced design plastic moment resistance:
\[
M_{yvRd} = \frac{(A_1 \cdot L_1 + A_2 \cdot L_2) \cdot fy + (A_3 \cdot L_3 + A_4 \cdot L_4) \cdot f_{yw}}{1E6} = 7021.3 \text{ kNm}
\]
Design moment resistance \( M_{crd} = 7810 \text{ kNm} \)
Hence the moment resistance of the section reduces from 7810 kNm to 7021.3 kNm with coexistent shear of 4486 kN.
As \( M_{Ed} \leq M_{yvRd} \) ( 7000 kNm \leq 7021.3 kNm ), the design moment is less than the design moment resistance. Hence OK.

Provide web transverse stiffeners at supports.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>7000 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>4486 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>355 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>1200 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Class 2</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>7810 kNm</td>
</tr>
<tr>
<td>Plastic shear resistance</td>
<td>5607.7 kN</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.89629 ≤ 1</td>
</tr>
</tbody>
</table>
Location: Ex11 - ICE book worked example 6.2-13 (no shear buckling)

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (ALT) and utilises Appendix C of SCI publication P356 or Clause 6.3.2.2 of BS EN 1993-2:2006.

Compression flange classification will be taken as Class 3.
The web classification will be taken as Class 3.
The overall section classification is Class 3.

Geometrical parameters

- Half wavelength of buckling: \( L_w = 10000 \text{ mm} \)
- Eff.length between restraints: \( L_T = 1000 \text{ mm} \)
- Depth of section: \( h = 2060 \text{ mm} \)
- Flange width: \( b = 400 \text{ mm} \)
- Flange thickness: \( t_f = 30 \text{ mm} \)
- Web thickness: \( t_w = 20 \text{ mm} \)
- Distance between flange centroids: \( d_f = 2030 \text{ mm} \)

Design parameters

- Coefficient \( C_1 \): \( C_1 = 1.13 \)
- Young's modulus: \( E = 210000 \text{ N/mm}^2 \)
- Shear modulus: \( G = 81000 \text{ N/mm}^2 \)
- Yield strength: \( f_y = 355 \text{ N/mm}^2 \)

Expression for lamLT (ALT)

The clause references in this section are as per Appendix C of SCI P356. The following are the 2nd moments of area of the compression and tension flanges, respectively, about the z-z axes:

For buckling over a half wavelength \( L_w \) the parameter V is as given by the expression below:

\[
V = ((4a^2 + 0.05\lambda F^2 + p_{sia}^2)^0.5 + p_{sia})^{-0.5} = 0.95305
\]

Parameter from C.4.5 \( V_{eq} = V = 0.95305 \)

The restraint parameter is calculated from the following expression as given in SCI publication P356:

\[
\rho_p = \frac{V_{eq}^4 L_w^3}{E I_{zc} \theta_R d_f^2 (1-a)}
\]

Rotation of restraint \( \theta_R \): \( \theta_R = 1E-15 \text{ rad/Nmm} \)

Restraint parameter:

\[
\rho_p = V_{eq}^4 L_w^3 / (E I_{zc} \theta_R d_f^2 (1-a)) = 11.917E6 \text{ rad/Nmm}
\]

Longest unbraced length: \( L_r = L_T = 1000 \text{ mm} \)

Plastic section modulus (yy axis): \( W_{ply} = 44.36E6 \text{ mm}^3 \)
Elastic section modulus (yy axis) \( W_{ely} = 37.5 \times 10^6 \text{ mm}^3 \)
Beta \( \beta_w \)
Section property from C.1 \( U = 0.9 \)
Non-dimensional slenderness \( \hat{\lambda}_{LT} \):
\[
\hat{\lambda}_{LT} = \frac{1}{(C_1)^{0.5} \cdot U \cdot V \cdot D \cdot \sqrt{\beta_w}} = 0.14182
\]

Partial factors on strength

- Partial factor \( \gamma_{M0} \): \( \gamma_{M0} = 1.0 \)
- Partial factor \( \gamma_{M1} \): \( \gamma_{M1} = 1.1 \)

Lateral torsional buckling

The design lateral torsional buckling resistance moment \( (M_{bRd}) \) will be calculated. Section under consideration is a welded section and EC3 Part 1-1 Clause 6.3.2.2 expressions will be adopted.

Limiting slenderness value \( \hat{\lambda}_{LT0} = 0.2 \)
As \( \hat{\lambda}_{LT} \leq \hat{\lambda}_{LT0} \), the segment of the beam is not subject to lateral torsional buckling.
The design buckling resistance \( M_{bRd} \) will therefore be the same as the design resistance for bending \( M_{cRd} \).

Design buckling resistance moment \( M_{bRd} = \frac{W_{ely} \cdot f_y}{(\gamma_{M0} \cdot 10^6)} = 13313 \text{ kNm} \)

Design moment resistance \( M_{cRd} = M_{bRd} = 13313 \text{ kNm} \)

Interaction equation for moments

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

Design moment resistance \( M_{cRd} = \frac{W_{ely} \cdot f_y}{(\gamma_{M0} \cdot 10^6)} = 13313 \text{ kNm} \)
As \( M_{bRd} \leq M_{cRd} \) (13313 kNm \leq 13313 kNm), the lateral torsional buckling resistance will be adopted.

Design moment from analysis \( M_{Ed} = 10000 \text{ kNm} \)
Unity factor \( Unity = \frac{M_{Ed}}{M_{bRd}} = 0.75117 \)
As \( M_{Ed} \leq M_{bRd} \) (10000 kNm \leq 13313 kNm), the section chosen is considered to be suitable.

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The shear resistance of a web under pure shear will be evaluated. Beam will be stable against shear buckling due to the presence of closely spaced web transverse stiffeners.

Shear-moment interaction

Design shear force (coexisting) \( V_{Ed} = 7871 \text{ kN} \)
The value of parameter \( \eta \) is as per the NA to BS EN 1993-1-5.
Value of \( \eta \) from Clause 5.1(2) \( \eta = 1.2 \)
Plastic shear resistance \( V_{plRd} = \frac{\eta \cdot A_w \cdot f_y}{(\gamma_{M0} \cdot \sqrt{3})} / 1000 = 9838 \text{ kN} \)
As the beam is symmetric EC3 Part 1-1 expression 6.30 will be used to evaluate the reduced design plastic moment resistance:

\[ \text{MyvRd} = \left( \frac{W_{\text{ply}} - r_0 \cdot A_w^2}{4 \cdot t_w} \right) \cdot \frac{f_y}{\gamma_M} \cdot 10^6 = 13191 \text{ kNm} \]

Design moment resistance \( M_{\text{Cd}} = 13313 \text{ kNm} \)

Hence the moment resistance of the section reduces from 13313 kNm to 13191 kNm with coexistent shear of 7871 kN.

As \( M_{\text{Ed}} \leq M_{\text{vRd}} \) (10000 kNm ≤ 13191 kNm), the design moment is less than the design moment resistance. Hence OK.

Beam is assumed to be stable against shear buckling due to the presence of closely spaced intermediate web transverse stiffeners.

Provide web transverse stiffeners at supports.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>10000 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>7871 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>355 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>2060 mm</td>
</tr>
<tr>
<td>Flange width</td>
<td>400 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Class 3</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>13313 kNm</td>
</tr>
<tr>
<td>Plastic shear resistance</td>
<td>9838 kN</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.75117 ≤ 1</td>
</tr>
</tbody>
</table>
Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter \( \text{lam}\text{LT} (\hat{\text{LT}}) \) and utilises Appendix C of SCI publication P356 or Clause 6.3.2.2 of BS EN 1993-2:2006.

Compression flange classification will be taken as Class 3.
The web classification will be taken as Class 3.
The overall section classification is Class 3.

Geometrical parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Half wavelength of buckling</td>
<td>( L_w = 9000 \text{ mm} )</td>
</tr>
<tr>
<td>Eff.length between restraints</td>
<td>( L_T = 1000 \text{ mm} )</td>
</tr>
<tr>
<td>Depth of section</td>
<td>( h = 2060 \text{ mm} )</td>
</tr>
<tr>
<td>Flange width</td>
<td>( b = 400 \text{ mm} )</td>
</tr>
<tr>
<td>Flange thickness</td>
<td>( t_f = 30 \text{ mm} )</td>
</tr>
<tr>
<td>Web thickness</td>
<td>( t_w = 20 \text{ mm} )</td>
</tr>
<tr>
<td>Distance between flange centroids</td>
<td>( d_f = 2030 \text{ mm} )</td>
</tr>
</tbody>
</table>

Design parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coefficient C1</td>
<td>( C_1 = 1.13 )</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>( E = 210000 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>( G = 81000 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Yield strength</td>
<td>( f_y = 355 \text{ N/mm}^2 )</td>
</tr>
</tbody>
</table>

Expression for \( \text{lam}\text{LT} (\hat{\text{LT}}) \)

The clause references in this section are as per Appendix C of SCI P356. The following are the 2nd moments of area of the compression and tension flanges, respectively, about the z-z axes:

For buckling over a half wavelength \( L_w \) the parameter \( V \) is as given by the expression below:

\[
V = (4*a*(1-a) + 0.05*\text{lam}F^2 + \text{psia}^2)^0.5 + \text{psia} \leq 0.96114
\]

Parameter from C.4.5

\[
\text{Veq} = V = 0.96114
\]

The restraint parameter is calculated from the following expression as given in SCI publication P356:

\[
rp = \left[ \frac{\text{Veq}^4*\text{Lw}^3}{E*\text{Izc}*\theta_R*\text{df}^2*(1-a)} \right]
\]

Rotation of restraint \( \theta_R \): \( \theta_R = 1E-15 \text{ rad/Nmm} \)

Restraint parameter:

\[
\text{rp} = \text{Veq}^4*\text{Lw}^3/(E*\text{Izc}*\theta_R*\text{df}^2*(1-a)) = 8.9862E6 \text{ rad/Nmm}
\]

Longest unbraced length \( L_r = L_T = 1000 \text{ mm} \)

Plastic section modulus (yy axis) \( W_{p\text{ly}} = 44.36E6 \text{ mm}^3 \)
Elastic section modulus (yy axis) \( W_{ely} = 37E6 \) mm\(^3\)
Beta \( \beta_w \)
Section property from C.1
Non-dimensional slenderness \( \lambda_{LT} \):
\[
\lambda_{LT} = 1/(C1)^{0.5} \times U \times V \times D \times \lambda_{SQR}(\beta_w) = 0.14129
\]

Partial factors on strength

Partial factor \( \gamma_M0 \) \( \gamma_M0 = 1.0 \)
Partial factor \( \gamma_M1 \) \( \gamma_M1 = 1.1 \)

Lateral torsional buckling

The design lateral torsional buckling resistance moment \( (M_{Brd}) \) will be calculated. Section under consideration is a welded section and EC3 Part 1-1 Clause 6.3.2.2 expressions will be adopted.
Limiting slenderness value \( \lambda_{LT0} = 0.2 \)
As \( \lambda_{LT} \leq \lambda_{LT0} \), the segment of the beam is not subject to lateral torsional buckling.
The design buckling resistance \( M_{Brd} \) will therefore be the same as the design resistance for bending \( M_{Cr} \).

Design buckling resistance moment \( M_{Brd} = W_{ely} \times f_y / (\gamma_M0 \times 10^6) = 13135 \text{ kNm} \)
Design moment resistance \( M_{Cr} = M_{Brd} = 13135 \text{ kNm} \)

Interaction equation for moments

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:
Design moment resistance \( M_{Cr} = W_{ely} \times f_y / (\gamma_M0 \times 10^6) = 13135 \text{ kNm} \)
As \( M_{Brd} \leq M_{Cr} \) (13135 kNm \leq 13135 kNm), the lateral torsional buckling resistance will be adopted.
Design moment from analysis \( M_{Ed} = 10000 \text{ kNm} \)
Unity factor \( \text{unity} = M_{Ed} / M_{Brd} = 0.76132 \)
As \( M_{Ed} \leq M_{Brd} \) (10000 kNm \leq 13135 kNm), section chosen is considered to be suitable.

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The shear resistance of a web under pure shear will be evaluated. As \( hw/tw > 72\varepsilon/\eta \) (100 > 58.58), beam is susceptible to shear buckling as there are no web transverse stiffeners except at supports.

Shear-moment interaction

Design shear force (coexisting) \( V_{Ed} = 4000 \text{ kN} \)
Plastic moment resistance \( M_{plRd} = W_{ply} \times f_y / (\gamma_M1 \times 1E6) = 15748 \text{ kNm} \)
Design shear buckling resistance assuming no contribution from the flanges is given by expression 5.2 (EC3 Part 1-5):
\[
V_{bwRd} = \chi_w \times f_y \times hw/tw / (\gamma_M1 \times SQR(3)) / 1000 = 4810.6 \text{ kN}
\]
Shear ratio \( \eta_1 \)
\( \text{eta1} = M_{Ed} / M_{plRd} = 0.63501 \)
Plastic moment resistance ignoring the web is:
MfRd=(Wply-Aw^2/(4*tw))*fy/(gamM0*1E6)=8647.8 kNm

Expression 7.1  \( \eta_1 = \eta_1 + (1 - \frac{MfRd}{MplRd}) \times (2 \times \eta_3 - 1)^2 \approx 0.83319 \)

\( \leq 1 \) hence OK.

Provide web transverse stiffeners at supports.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>10000 kNm</td>
<td></td>
</tr>
<tr>
<td>Design shear</td>
<td>4000 kN</td>
<td></td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>355 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Depth of beam</td>
<td>2060 mm</td>
<td></td>
</tr>
<tr>
<td>Flange width</td>
<td>400 mm</td>
<td></td>
</tr>
<tr>
<td>Classification</td>
<td>Class 3</td>
<td></td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>13135 kNm</td>
<td></td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.76132 ( \leq 1 )</td>
<td></td>
</tr>
</tbody>
</table>
Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.


Expression for lamLT (\(\hat{\lambda}_T\)) - composite construction

The section under consideration is assumed to be at an internal support of a continues girder where hogging bending moment with axial force are present. The non-dimensional slenderness lamLT (\(\hat{\lambda}_T\)) could be derived from the following expression as per Clause 6.3.4.2(4):

\[
\hat{\lambda}_T = \frac{fy \cdot A_{eff}}{N_{crit}}
\]

where,

\(N_{crit}\) = Elastic critical load
\(fy\) = Yield strength
\(A_{eff}\) = Effective area of the chord

Geometrical and design parameters

Compression flange classification will be taken as Class 3.
Section symmetrical about both major and minor axis (equal flanges).
Eff.length between restraints \(LT=5000\) mm
Depth of section \(h=2060\) mm
Flange width \(b=400\) mm
Web thickness \(tw=20\) mm
Flange thickness \(tf=30\) mm
Yield strength \(fy=345\) N/mm²
Non-dimensional slenderness \(\hat{\lambda}_T\) \(\text{lamLT} = \sqrt{\frac{fy \cdot A_{eff}}{N_{crit} \cdot 10^3}} = 0.69678\)

Partial factors on strength

Partial factor \(\gamma_M0\) \(\text{gamM0}=1.0\)
Partial factor \(\gamma_M1\) \(\text{gamM1}=1.1\)

Lateral torsional buckling

The design lateral torsional buckling resistance moment (\(Mb_{Rd}\)) will be calculated. Section under consideration is a welded section and EC3 Part 1-1 Clause 6.3.2.2 expressions will be adopted.

Limiting slenderness value \(\text{lamLT0}=0.2\)
According to the NA to BS EN 1993-1-1, for welded doubly symmetrical sections the imperfection value is as follows:
Imperfection value \( aLT = 0.76 \) (curve d)

Modified chiLT factor \( \text{ChiLT} = \text{chiLT}/f = 0.64525 \)

Total design hogging moment \( M_{\text{max}} = 10000 \) kNm

Design moment on steel section \( M_{\text{Ed}} = 3000 \) kNm

Composite cracked section moment \( M_{cEd} = M_{\text{max}} - M_{\text{Ed}} = 7000 \) kNm

Design yield strength of steel \( f_{yd} = f_y/g_{\text{M1}} = 313.64 \) N/mm²

Construction stage:

**Stress in steel bottom flange**

\( s_{sb f1} = 45 \) N/mm²

**Composite stage**:

**Stress in steel bottom flange**

\( s_{sb f2} = 150 \) N/mm²

**Factor k (EN 1994-2 Cl. 6.2.1.4)**

\( k = (f_{yd} - s_{sb f1})/s_{sb f2} = 1.7909 \)

**Design elastic resistance moment**

\( M_{elRd} = M_{\text{Ed}} + k \times M_{cEd} = 15536 \) kNm

Hello. 

**Imperfection value**

\( aLT = 0.76 \) (curve d)

**Modified chiLT factor**

\( \text{ChiLT} = \text{chiLT}/f = 0.64525 \)

**Total design hogging moment**

\( M_{\text{max}} = 10000 \) kNm

**Design moment on steel section**

\( M_{\text{Ed}} = 3000 \) kNm

**Composite cracked section moment**

\( M_{cEd} = M_{\text{max}} - M_{\text{Ed}} = 7000 \) kNm

**Design yield strength of steel**

\( f_{yd} = f_y/g_{\text{M1}} = 313.64 \) N/mm²

**Construction stage:**

**Stress in steel bottom flange**

\( s_{sb f1} = 45 \) N/mm²

**Composite stage:**

**Stress in steel bottom flange**

\( s_{sb f2} = 150 \) N/mm²

**Factor k (EN 1994-2 Cl. 6.2.1.4)**

\( k = (f_{yd} - s_{sb f1})/s_{sb f2} = 1.7909 \)

**Design elastic resistance moment**

\( M_{elRd} = M_{\text{Ed}} + k \times M_{cEd} = 15536 \) kNm

**Design buckling resistance moment**

\( M_{bRd} = \text{ChiLT} \times M_{elRd} = 10025 \) kNm

**Interaction equation for moments**

**Distance**

\( L_1 = 0.25 \times L/\text{SQR}(m) = 1250 \) mm

**Design moment at brace position**

\( M_2 = 5000 \) kNm

**Design moment from analysis**

\( M_{Ed} = M_1 - (M_1 - M_2) \times L_1/L = 8750 \) kNm

**Axial stress from the analysis**

\( \text{stress} = 4 \) N/mm²

**Axial load**

\( \text{NEd} = A_{\text{eff}} \times \text{stress}/1000 = 74.667 \) kN

**Axial resistance of Tee section**

\( N_{bRd} = \text{ChiLT} \times A_{\text{eff}} \times f_{yd}/1000 = 3777.7 \) kN

**Unity factor**

\( \text{unity} = M_{Ed}/M_{bRd} + \text{NEd}/N_{bRd} = 0.89259 \)

As unity ≤ 1, section chosen is considered to be suitable.

**Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners**

The shear resistance of a web under pure shear will be evaluated. As \( hw/tw > 72\varepsilon/\eta \) (100 > 59.423), beam is susceptible to shear buckling as there are no web transverse stiffeners except at supports. Plastic section modulus (yy axis) \( W_{\text{ply}} = 44.36E6 \) mm²

**Shear-moment interaction**

**Design shear force (coexisting)**

\( V_{Ed} = 4000 \) kN

**Plastic moment resistance**

\( M_{plRd} = W_{\text{ply}} \times f_y/(\varepsilon_{\text{M0}} \times 1E6) = 15304 \) kNm

**Design shear buckling resistance assuming no contribution from the flanges is given by expression 5.2 (EC3 Part 1-5):**

\( V_{bwRd} = \text{chiw} \times f_y \times hw/tw/(\varepsilon_{\text{M1}} \times \text{SQR}(3)) \times 1000 = 4720 \) kN

**Shear ratio \( \eta \)**

\( \eta_1 = \text{MEd}/M_{plRd} = 0.57174 \)

**Area of top reinforcement**

\( A_1 = 12000 \) mm²

**Area of bottom reinforcement**

\( A_2 = 12000 \) mm²

**Axial resistance of the top bars and top flange:**

\( F_1 = ((A_1 + A_2) \times 500/1.15 + tf \times b \times f_y)/(\varepsilon_{\text{M0}} \times 1000) = 14575 \) kN

**Axial resistance of the bottom flange:**

\( F_2 = tf \times b \times f_y/(\varepsilon_{\text{M0}} \times 1000) = 4140 \) kN

**Lever arm**

\( d_{fc} = 2100 \) mm

**Plastic moment resistance ignoring the web is:**

\( M_{fRd} = F_{\text{min}} \times d_{fc}/(\varepsilon_{\text{M0}} \times 1E3) = 8694 \) kNm

**Expression 7.1**

\( u_1 = \eta_1 + (1 - M_{fRd}/M_{plRd}) \times (2 \times \eta_3 - 1)^2 = 0.57175 \)

As \( u_1 \leq 1 \), hence OK.

Provide web transverse stiffeners at supports.
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>8750 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>4000 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>345 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>2060 mm</td>
</tr>
<tr>
<td>Flange width</td>
<td>400 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Class 3</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>10025 kNm</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.89259 ≤ 1</td>
</tr>
</tbody>
</table>
Location: Ex14 - Non-symmetrical composite sect. (w. shear buckling)

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter \( \lambda_{LT} \) using the simplified method of EN 1993-2, as allowed by BS EN 1994-2:2005.

Expression for \( \lambda_{LT} \) - composite construction

The section under consideration is assumed to be at an internal support of a continuous girder where hogging bending moment with axial force are present. The non-dimensional slenderness \( \lambda_{LT} \) could be derived from the following expression as per Clause 6.3.4.2(4):

\[
\frac{f_y \cdot A_{eff}}{N_{crit}}
\]

where,
- \( N_{crit} \) = Elastic critical load
- \( f_y \) = Yield strength
- \( A_{eff} \) = Effective area of the chord

Geometrical and design parameters

Compression flange classification will be taken as Class 3.
Section symmetrical only about the minor axis (unequal flanges).
Eff.length between restraints \( LT = 5000 \) mm
Depth of section \( h = 2060 \) mm
Top flange width \( b_t = 400 \) mm
Bottom flange width \( b_b = 600 \) mm
Top flange thickness \( t_f = 30 \) mm
Bottom flange thickness \( t_b = 40 \) mm
Web thickness \( t_w = 20 \) mm
Yield strength \( f_y = 345 \) N/mm²

Non-dimensional slenderness \( \lambda_{LT} \) \( \lambda_{LT} = \sqrt{f_y \cdot A_{eff}/(N_{crit} \times 1E3)} \) = 0.41081

Partial factors on strength

Partial factor \( \gamma_M0 \) \( \gamma_M0 = 1.0 \)
Partial factor \( \gamma_M1 \) \( \gamma_M1 = 1.1 \)

Lateral torsional buckling

The design lateral torsional buckling resistance moment (\( M_{Brd} \)) will be calculated. Section under consideration is a welded section and EC3 Part 1-1 Clause 6.3.2.2 expressions will be adopted.
Limiting slenderness value \( \lambda_{LT0} = 0.2 \)
According to the NA to BS EN 1993-1-1, Tables 6.3 and 6.4 could be used with Clause 6.3.2.2 to determine the imperfection factor \( \alpha_{LT} \).
Imperfection value  
Modified chiLT factor  
Total design hogging moment  
Design moment on steel section  
Composite cracked section moment  
Design yield strength of steel  
Construction stage:  
Stress in steel bottom flange  
Composite stage:  
Stress in steel bottom flange  
Factor k (EN 1994-2 Cl. 6.2.1.4)  
Design elastic resistance moment  
Modification factor  
Design buckling resistance moment  

Interaction equation for moments

Distance  
Design moment at brace position  
Design moment from analysis  
Axial stress from the analysis  
Axial load  
Axial resistance of Tee section  
Unity factor  
As unity ≤ 1, section chosen is considered to be suitable.

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The shear resistance of a web under pure shear will be evaluated. As hw/tw > 72ε/η (99.5 > 59.423), beam is susceptible to shear buckling as there are no web transverse stiffeners except at supports. Plastic section modulus (yy axis) Wply=52.68E6 mm^3

Shear-moment interaction

Design shear force (coexisting)  
Plastic moment resistance  
Design shear buckling resistance assuming no contribution from the flanges is given by expression 5.2 (EC3 Part 1-5):  
Shear ratio η1  
Area of top reinforcement  
Area of bottom reinforcement  
Axial resistance of the top bars and top flange:  
Axial resistance of the bottom flange:  
Lever arm  
Plastic moment resistance ignoring the web is:  
Expression 7.1

Provide web transverse stiffeners at supports.
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>Design moment 8750 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>Design shear 4000 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>Yield strength of steel 345 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>Depth of beam 2060 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Classification Class 3</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>Design buckling resistance 13091 kNm</td>
</tr>
<tr>
<td>Unity factor</td>
<td>Unity factor 0.68352 ≤ 1</td>
</tr>
</tbody>
</table>
Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (ALT) and utilises Clause 6.3.2.2 of BS EN 1993-2:2006.

Expression for lamLT (ALT)

The non-dimensional slenderness lamLT (ALT) could be derived from the following expression:

$$\lambda_{LT} = \left( \frac{f_y W_{pl,y}}{M_{cr}} \right)^{0.5}$$

where,

- $M_{cr}$ = Elastic critical buckling moment from the analysis
- $f_y$ = Yield strength
- $W_{pl,y}$ = Plastic modulus of the section about y-y axis

Compression flange classification will be taken as Class 2.
The web classification will be taken as Class 2.
The overall section classification is Class 2.

Geometrical parameters

- Eff.length between restraints: $LT=5000$ mm
- Depth of section: $h=400$ mm
- Flange thickness: $t_f=20$ mm
- Plastic section modulus (yy axis): $W_{pl,y}=4.25E6$ mm$^3$

Design parameters

- Young's modulus: $E=210000$ N/mm$^2$
- Shear modulus: $G=81000$ N/mm$^2$
- Yield strength: $f_y=345$ N/mm$^2$

Non-dimensional slenderness (ALT)

- Effective length parameter: $K=1$
- Length for LTB: $LT=LT/1000=5$ m
- Critical moment: $M_{cr}=C_1 E_t G_t^{0.5}/10^6=72772$ kNm
- Non-dimensional slenderness: $\lambda_{LT}=(W_{pl,y}/(f_y/(M_{cr}10^6)))^{0.5}=0.14195$

Partial factors on strength

- Partial factor $\gamma_M0$: $\gamma_M0=1.0$
- Partial factor $\gamma_M1$: $\gamma_M1=1.1$
**Lateral torsional buckling**

The design lateral torsional buckling resistance moment \( Mb_{Rd} \) will be calculated. Section under consideration is a Hot Rolled section and EC3 Part 1-1 Clause 6.3.2.3 expressions will be adopted. 

Limiting slenderness value \( \lambda_{MLT0} = 0.4 \)

As \( \lambda_{MLT} \leq \lambda_{MLT0} \), the segment of the beam is not subject to lateral torsional buckling.

The design buckling resistance \( Mb_{Rd} \) will therefore be the same as the design resistance for bending \( Mc_{Rd} \).

Design buckling resistance moment \( Mb_{Rd} = W_{ply} * f_y / (\gamma_{M0} * 10^6) = 1466.3 \text{ kNm} \)

Design moment resistance \( Mc_{Rd} = Mb_{Rd} = 1466.3 \text{ kNm} \)

**Interaction equation for moments**

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

Design moment resistance \( Mc_{Rd} = W_{ply} * f_y / (\gamma_{M0} * 10^6) = 1466.3 \text{ kNm} \)

As \( Mb_{Rd} \leq Mc_{Rd} (1466.3 \text{ kNm} \leq 1466.3 \text{ kNm}) \), the lateral torsional buckling resistance will be adopted.

Design moment from analysis \( M_{Ed} = 1100 \text{ kNm} \)

Unity factor \( unity = M_{Ed} / Mb_{Rd} = 0.75021 \)

As \( M_{Ed} \leq Mb_{Rd} (1100 \text{ kNm} \leq 1466.3 \text{ kNm}) \), section chosen is considered to be suitable.

**Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners**

The shear resistance of a web under pure shear will be evaluated. As \( hw/tw \leq 72\varepsilon/\eta (18 \leq 59.423) \), the beam is considered to be stable against shear buckling.

**Shear-moment interaction**

Design shear force (coexisting) \( V_{Ed} = 2300 \text{ kN} \)

The value of parameter \( \eta \) is as per the NA to BS EN 1993-1-5.

Value of \( \eta \) from Clause 5.1(2) \( \eta = 1 \)

Plastic shear resistance \( V_{plRd} = \eta * A_w * f_y / (\gamma_{M0} * SQR(3)) / 1000 = 3027.6 \text{ kN} \)

As the beam is symmetric EC3 Part 1-1 expression 6.30 will be used to evaluate the reduced design plastic moment resistance:

\[ Myv_{Rd} = (W_{ply} - ro * A_w^2 / (4*tw)) * f_y / (\gamma_{M0} * 1E6) = 1197.5 \text{ kNm} \]

Design moment resistance \( Mc_{Rd} = 1466.3 \text{ kNm} \)

Hence the moment resistance of the section reduces from 1466.3 kNm to 1197.5 kNm with coexistent shear of 2300 kN.

As \( M_{Ed} \leq Myv_{Rd} (1100 \text{ kNm} \leq 1197.5 \text{ kNm}) \), the design moment is less than the design moment resistance. Hence OK.

Provide web transverse stiffeners at supports.
### DESIGN
- Design moment: 1100 kNm

### SUMMARY
- Design shear: 2300 kN
- Yield strength of steel: 345 N/mm²
- Depth of beam: 400 mm
- Flange width: 400 mm
- Classification: Class 2
- Design buckling resistance: 1466.3 kNm
- Plastic shear resistance: 3027.6 kN
- Unity factor: 0.75021 ≤ 1
Location: Ex16 - Welded RHS beam 1000x400 RHS (with shear buckling)

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (ALT) and utilises Clause 6.3.2.2 of BS EN 1993-2:2006.

Expression for lamLT (ALT)

The non-dimensional slenderness lamLT (ALT) could be derived from the following expression:

$$ ALT = \left[ \frac{fy \cdot Wy}{Mcr} \right] $$

where,

- Mcr = Elastic critical buckling moment from the analysis
- fy = Yield strength
- Wy = Plastic modulus of the section about y-y axis

Compression flange classification will be taken as Class 1.
The web classification will be taken as Class 1.
The overall section classification is Class 1.

Geometrical parameters

- Eff.length between restraints: LT=8000 mm
- Depth of section: h=1000 mm
- Flange width: b=400 mm
- Top flange thickness: tft=20 mm
- Bottom flange thickness: tfb=20 mm
- Web thickness: tw=16 mm
- Plastic section modulus (yy axis): Wply=18E6 mm³

Design parameters

- Young's modulus: E=210000 N/mm²
- Shear modulus: G=81000 N/mm²
- Yield strength: fy=345 N/mm²

Non-dimensional slenderness (ALT)

- Effective length parameter: K=1
- Length for LTB: LT=LT/1000=8 m
- Critical moment: Mcr=C1*Et*Gt^0.5/10^6=118342 kNm
- Non-dimensional slenderness: lamLT=(Wply*fy/(Mcr*10^6))^0.5=0.22907
Partial factors on strength

Partial factor $\gamma_M 0$  $\text{gamM0}=1.0$
Partial factor $\gamma_M 1$  $\text{gamM1}=1.1$

Lateral torsional buckling

The design lateral torsional buckling resistance moment ($MbRd$) will be calculated. Section under consideration is a welded section and EC3 Part 1-1 Clause 6.3.2.2 expressions will be adopted.

Limiting slenderness value  $\text{lamLT0}=0.2$
According to the NA to BS EN 1993-1-1, Tables 6.3 and 6.4 could be used with Clause 6.3.2.2 to determine the imperfection factor $\alphaLT$.  

Imperfection value  $aLT=0.76$ (curve d)
Modified $\text{ChiLT}$ factor  $\text{ChiLT}=\chiLT/f=0.97724$

Design buckling resistance moment $MbRd=\text{ChiLT}^*Wply^*fy/(\text{gamM1}*10^6)$

$$=5517 \text{ kNm}$$

Interaction equation for moments

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

Design moment resistance  $McRd=Wply^*fy/(\text{gamM0}*10^6)=6210 \text{ kNm}$
As $MbRd \leq McRd$ ( 5517 kNm $\leq$ 6210 kNm ), the lateral torsional buckling resistance will be adopted.

Design moment from analysis  $MEd=1600 \text{ kNm}$

Unity factor  $\text{unity}=MEd/MbRd=0.29001$
As $MEd \leq MbRd$ ( 1600 kNm $\leq$ 5517 kNm ), section chosen is considered to be suitable.

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The shear resistance of a web under pure shear will be evaluated. As $hw/tw > 72e/n$ ( 60 $> 59.423$ ), beam is susceptible to shear buckling as there are no web transverse stiffeners except at supports.

Shear-moment interaction

Design shear force (coexisting)  $VEd=2300 \text{ kN}$

Plastic moment resistance  $MplRd=Wply^*fy/(\text{gamM0}*1E6)=6210 \text{ kNm}$

Design shear buckling resistance assuming no contribution from the flanges is given by expression 5.2 (EC3 Part 1-5):

$$VbwRd=\chiw^*fy^*hw^*tw/(\text{gamM1}*SQR(3))/1000=2743.6 \text{ kN}$$

Shear ratio $n1$  $\text{eta1}=MEd/MplRd=0.25765$

Plastic moment resistance ignoring the web is:

$$Mfrd=(Wply-Aw^2/(4*tw))^*fy/(\text{gamM0}*1E6)=206.72 \text{ kNm}$$

Expression 7.1  $u1=\text{eta1}+(1-Mfrd/MplRd)*(2*eta3-1)^2=0.70023$

$\leq 1$ hence OK.

Provide web transverse stiffeners at supports.
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>Design moment: 1600 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>Design shear: 2300 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>Yield strength of steel: 345 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>Depth of beam: 1000 mm</td>
</tr>
<tr>
<td>Flange width</td>
<td>Flange width: 400 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Classification: Class 1</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>Design buckling resistance: 5517 kNm</td>
</tr>
<tr>
<td>Unity factor</td>
<td>Unity factor: 0.29001 ≤ 1</td>
</tr>
</tbody>
</table>
Location: Ex17- Hot Rolled Angle 200x150 UKA

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2 : Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (\(\lambdaLT\)) from an elastic buckling analysis of the structure.

Expression for lamLT (\(\lambdaLT\))

The non-dimensional slenderness lamLT (\(\lambdaLT\)) could be derived from the following expression:

\[
\lambdaLT = \frac{f_y W_y}{M_{cr}}
\]

where,

- \(M_{cr}\) = Elastic critical buckling moment from the analysis
- \(f_y\) = Yield strength
- \(W_y\) = Plastic modulus of the section about y-y axis

Geometrical and design parameters

Section is not symmetrical about either axis (Tee section).
Section under consideration is a Hot Rolled section.
Eff.length between restraints \(LT=4000\) mm
Depth of section \(h=200\) mm
Width of section \(b=150\) mm
Plate thickness \(t_f=18\) mm
Web thickness \(t_w=18\) mm
Yield strength \(f_y=345\) N/mm²
Section classification \(\text{class}=3\)
The overall section classification will be taken as Class 3.
Elastic section modulus \(W_{ley}\) \(=174000\) mm³

Bending moment \(M_{cr}\)
Non-dimensional slenderness \(\lambdaLT\)

\(\lambdaLT=SQR(f_y W_{ley}/(M_{cr}\times1E6))=0.63261\)

Partial factors on strength

Partial factor \(\gamma M_0\) \(\gamma M_0=1.0\)
Partial factor \(\gamma M_1\) \(\gamma M_1=1.1\)

Lateral torsional buckling

The design lateral torsional buckling resistance moment \((M_{Brd})\) will be calculated. Section under consideration is a Hot Rolled section and EC3 Part 1-1 Clause 6.3.2.3 expressions will be adopted.
Limiting slenderness value \(\lambdaLT_0=0.4\)
According to the NA to BS EN 1993-1-1, for all other Hot-Rolled sections the imperfection value is as follows:
Imperfection value  aLT=0.76 (curve d)
Modified chiLT factor  ChiLT=chiLT/f=0.81062
Design buckling resistance moment  MbRd=ChiLT*Wely*fy/(gamM1*10^6)
=44.238 kNm

Interaction equation for moments

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:
Design moment resistance  McRd=Wely*fy/(gamM0*10^6)=60.03 kNm
As MbRd ≤ McRd (44.238 kNm ≤ 60.03 kNm), the lateral torsional buckling resistance will be adopted.
Design moment from analysis  MEd=40 kNm
Unity factor  unity=MEd/MbRd=0.90421
As MEd ≤ MbRd (40 kNm ≤ 44.238 kNm), section chosen is considered to be suitable.

Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners

The shear resistance of a web under pure shear will be evaluated. As hw/tw ≤ 72r/η (10.111 ≤ 59.423), the beam is considered to be stable against shear buckling.

Shear-moment interaction

Design shear force (coexisting)  VEd=300 kN
The value of parameter η (η) is as per the NA to BS EN 1993-1-5.
Value of η from Clause 5.1(2)  η=1
Plastic shear resistance  VplRd=ηAw*fy/(gamM0*SQR(3))/1000
=652.53 kN
As VEd ≤ 0.5*VplRd, the design shear will not reduce the design resistance moment. Hence OK.

DESIGN                  SUMMARY
Design moment       40 kNm  Design shear      300 kN
Yield strength of steel  345 N/mm²  Depth of beam     200 mm
Width of section     150 mm  Classification  Class 3
Design buckling resistance 44.238 kNm  Plastic shear resistance  652.53 kN
Unity factor         0.90421 ≤ 1
Location: Ex18 - As Ex1 but using Class 4 section

Evaluation of non-dimensional slenderness

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma calculates the non-dimensional slenderness parameter lamLT (ALT) and utilises Appendix C of SCI publication P356 or Clause 6.3.2.2 of BS EN 1993-2:2006.

Compression flange classification will be taken as Class 3.
The web classification will be taken as Class 4.
The overall section classification is Class 4.

Geometrical parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Half wavelength of buckling</td>
<td>Lw=28000 mm</td>
</tr>
<tr>
<td>Eff.length between restraints</td>
<td>LT=8200 mm</td>
</tr>
<tr>
<td>Depth of section</td>
<td>h=1100 mm</td>
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<tr>
<td>Top flange width</td>
<td>bt=500 mm</td>
</tr>
<tr>
<td>Bottom flange width</td>
<td>bb=500 mm</td>
</tr>
<tr>
<td>Top flange thickness</td>
<td>tft=40 mm</td>
</tr>
<tr>
<td>Bottom flange thickness</td>
<td>tfb=40 mm</td>
</tr>
<tr>
<td>Web thickness</td>
<td>tw=10 mm</td>
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<tr>
<td>St.Venant torsional constant</td>
<td>It=21.67E6 mm^4</td>
</tr>
<tr>
<td>Second moment of area (z-z axis)</td>
<td>Iz=833E6 mm^4</td>
</tr>
<tr>
<td>Distance between flange centroids</td>
<td>df=1060 mm</td>
</tr>
</tbody>
</table>

Determine the effective section properties - EN 1992-1-5, Cl. 4.4

The process is iterative as the amount of web not considered is a function of the stresses at the top and bottom of the web.

Iteration 1:
Gross area of section A=2*b*tf+hw*tw=50200 mm^2
Gross 2nd moment of area Iy=(b*(hw+2*tf)^3-(b-tw)*hw^3)/12 =12.126E9 mm^4
Stress at top of web s1=MyEd*10^6*hw/2/Iy=131.73 N/mm^2
Stress at bottom of web s2=-MyEd*10^6*(zeff-tf)/Iyeff=-131.68 N/mm^2

Iteration 10 :
Position of effective centroid: zeff=(A*h/2-Aw*(zeffp+be2+lw/2))/Aeff=549.48 mm
Effective 2nd moment of area: Iyeff=Iy+A*(h/2-zeff)^2-tw*lw^3/12-Aw*(zeffp+be2+lw/2-zeff)^2 =12.118E9 mm^4
Top of web stress s1(10)=MyEd*10^6*(h-tf-zeff)/Iyeff =131.95 N/mm^2
Bottom of web stress s2(10)=-MyEd*10^6*(zeff-tf)/Iyeff =-131.68 N/mm^2
Net loss area Aw=lw*tw=85.988 mm^2
Effective area of section Aeff=A-Aw=50114 mm^2
Effective section modulus
Weffy=\text{Iyeff}/(1000*(h-zeff))=22011 \text{ cm}^3

Design parameters

Design moment (major axis) \quad MyEd=3132 \text{ kNm}
Coefficient C1 \quad C1=1
Young's modulus \quad E=210000 \text{ N/mm}^2
Shear modulus \quad G=80768 \text{ N/mm}^2
Yield strength \quad fy=345 \text{ N/mm}^2

Expression for \( \lambda_{\text{LT}} \) (\( \lambda_{\text{ALT}} \))

The clause references in this section are as per Appendix C of SCI P356.
2nd moment of area (comp flange) \quad Izc=416E6 \text{ mm}^4
2nd moment of area (tens. flange) \quad Izt=416E6 \text{ mm}^4
For buckling over a half wavelength \( L_w \) the parameter \( V \) is as given by the expression below:

\[
V = ((4*a*(1-a)+0.05*\lambda_{\text{F}}^2+\text{psia}^2)^{0.5}+\text{psia})^{-0.5}=0.70174
\]

Parameter from C.4.5 \quad \omega=\pi^2*df^2*E*Iz/(G*It*Lw^2)
compute \quad \text{comp}1=\text{SQR}(4+tee*\omega)+\text{psia} \\
\quad \text{SQR}(\omega)=2.3267
Parameter from C.4.5 \quad \text{Veq}=(2*a*\omega/\text{comp}1^2)^{0.25}=0.71485

The restraint parameter is calculated from the following expression as given in SCI publication P356:

\[
rp = \left( \frac{\text{Veq}^4*Lw^3}{E*Izc*\omega*R*df^2*(1-a)} \right)
\]

Rotation of restraint \( \theta_R \) \quad \theta_R=27.06E-12 \text{ rad/Nmm}
Restrainment parameter:

\[
rp=\text{Veq}^4*Lw^3/(E*Izc*\omega*R*df^2*(1-a))=4316.4 \text{ rad/Nmm}
\]
Longest unbraced length \quad Lr=LT=8200 \text{ mm}
Plastic section modulus (yy axis) \quad Wply=23.88E6 \text{ mm}^3
Effect section modulus (yy axis) \quad Weffy=22.011E6 \text{ mm}^3
Beta \( \beta_w \) \quad \text{betaw}=\text{Weffy}/Wply=0.92173
Section property from C.1 \quad U=1
Non-dimensional slenderness \( \lambda_{\text{LT}} \):

\[
\lambda_{\text{LT}}=1/(C1)^{0.5}*U*V*D*\lambda_{\text{SQR}}(\text{betaw})=0.87387
\]

Partial factors on strength

Partial factor \( \gamma_{M0} \) \quad \text{gamM0}=1.0
Partial factor \( \gamma_{M1} \) \quad \text{gamM1}=1.1

Lateral torsional buckling

The design lateral torsional buckling resistance moment (\( M_{\text{BrD}} \)) will be calculated. Section under consideration is a welded section and EC3 Part 1-1 Clause 6.3.2.2 expressions will be adopted.
Limiting slenderness value \( \lambda_{\text{LT0}}=0.2 \)
According to the NA to BS EN 1993-1-1, Tables 6.3 and 6.4 could be used with Clause 6.3.2.2 to determine the imperfection factor \( \alpha_{\text{LT}} \).
Imperfection value  
Modified chiLT factor  
Design buckling resistance moment $M_{bRd} = \frac{\text{ChiLT} \times W_{effy} \times f_y}{(\gamma_{M1} \times 10^6)}$

$$= 3698.2 \text{ kNm}$$

**Interaction equation for moments**

The beam is assumed to be without longitudinal stiffeners. The design resistance for bending about the major axis as per Clause 6.2.5(2) of BS EN 1993-2:2006 is as follows:

- **Design moment resistance** $M_{cRd} = \frac{W_{effy} \times f_y}{(\gamma_{M0} \times 10^6)} = 7593.8 \text{ kNm}$

As $M_{bRd} \leq M_{cRd}$ ($3698.2 \text{ kNm} \leq 7593.8 \text{ kNm}$), the lateral torsional buckling resistance will be adopted.

- **Unity factor** $\text{unity} = \frac{M_{Ed}}{M_{bRd}} = 0.84689$

As $M_{Ed} \leq M_{bRd}$ ($3132 \text{ kNm} \leq 3698.2 \text{ kNm}$), section chosen is considered to be suitable.

**Shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners**

The shear resistance of a web under pure shear will be evaluated. Beam will be stable against shear buckling due to the presence of closely spaced web transverse stiffeners.

**Shear-moment interaction**

- **Design shear force (coexisting)** $V_{Ed} = 500 \text{ kN}$

The value of parameter $\eta$ is as per the NA to BS EN 1993-1-5.

- **Value of $\eta$ from Clause 5.1(2)** $\eta = 1$

- **Plastic shear resistance** $V_{plRd} = \eta \times A_w \times f_y / (\gamma_{M0} \times SQR(3)) / 1000$

$$= 2031.7 \text{ kN}$$

As $V_{Ed} \leq 0.5 \times V_{plRd}$, the design shear will not reduce the design resistance moment. Hence OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design moment</td>
<td>3132 kNm</td>
</tr>
<tr>
<td>Design shear</td>
<td>500 kN</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>345 N/mm²</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>1100 mm</td>
</tr>
<tr>
<td>Classification</td>
<td>Class 4</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>3698.2 kNm</td>
</tr>
<tr>
<td>Plastic shear resistance</td>
<td>2031.7 kN</td>
</tr>
<tr>
<td>Unity factor</td>
<td>$0.84689 \leq 1$</td>
</tr>
</tbody>
</table>
Location: Ex1 - Two double-sided bearing stiffeners at support

Bearing stiffeners at end support

The proforma is in accordance with BS EN 1993-2:2006 "EC3 Design of Steel Structures - Part 2: Steel bridges", the NA to BS EN 1993-2:2006 and BS EN 1993-1-5 section 9. A rigid end post is assumed in the design which comprise of two double sided transverse stiffeners that form the flanges of a short beam of length hw, as highlighted in BS EN 1993-1-5 Clause 9.3.1(2).

Span of girder  
L=12 m

Depth of girder between flanges  
hw=1200 mm

Flange thickness  
\(t_f=30\) mm

Web thickness  
\(t_w=12.5\) mm

Assumed stiffener thickness  
\(t_s=20\) mm

Stiffener corner snipe  
\(snp=15\) mm

Partial factors for resistance

Partial factor  
gamM1=1.1

Partial factor (welds)  
gamM2=1.25

Strength of steel

Steel grade  
S 355

Yield strength (web)  
\(f_{yw}=355\) N/mm\(^2\)

Yield strength (stiffener)  
\(f_{ys}=345\) N/mm\(^2\)

Yield strength to be adopted  
\(f_y=345\) N/mm\(^2\)

Design strength of fillet weld  
\(f_{wcd}=f_u/(3^0.5*betw*gamM2)=241.2\) N/mm\(^2\)

Check web thickness

Limiting web thick for service  
\(t_{lim}=hw/250=4.8\) mm

Web thickness exceeds limiting value for serviceability, hence OK.

Limiting web thick flange buckl  
\(t_{lim}=(hw/250)*(f_y/345)=4.8\) mm

Web thickness exceeds limiting value for flange buckling, hence OK.

Bearing stiffeners double sided - BS EN 1993-1-5 section 9

Bearing stiffeners are assumed to be symmetric about the beam web and will be designed as a strut to BS EN 1993-1-5 Clause 9.4(2), resisting the bearing reaction together with any eccentricities.

compute \(k_t\) from Annex A  
\(k_t=5.34\)
Web shear resistance \( V_{bRd} = \chi_{iw} \cdot f_y \cdot h_w \cdot t_w / (\sqrt{3} \cdot \gamma_{M1} \cdot 1000) = 1801.4 \text{ kN} \)

Design bearing reaction \( N_{bEd} = 1800 \text{ kN} \)

**Action as a bearing stiffener**

![Diagram of stiffeners and bearing elements]

NOTE: Stiffeners to be equally spaced about the centreline of bearing.

Depth of bearing stiffener \( h_{sb} = 150 \text{ mm} \)

Thickness of bearing stiffener \( t_{sb} = 20 \text{ mm} \)

Distance \( c_1 = 325 \text{ mm} \)

Distance between stiffener CL's \( c_2 = 300 \text{ mm} \)

Eccentricity in each direction \( \epsilon_c = 10 \text{ mm} \)

Worst stress in web plate:

\[
sw_{p} = N_{bEd} \cdot 1E3 / A + M_{zEd} \cdot 1E6 \cdot (15 \cdot e_{w} \cdot t_{w} + t_{sb} / 2 + c_{2} / 2) / (I_{zz})
\]

\[
= 101.57 \text{ N/mm}^2
\]

Worst stress in stiffener:

\[
\text{comp}_1 = N_{bEd} \cdot 1E3 / A = 90.846 \text{ N/mm}^2
\]

\[
\text{comp}_2 = M_{zEd} \cdot 1E6 \cdot (c_{2} / 2 + t_{sb} / 2) / (I_{zz}) = 5.4915 \text{ N/mm}^2
\]

\[
\text{comp}_3 = M_{yEd} \cdot 1E6 \cdot (h_{sb} + t_{w} / 2) / (I_{yy}) = 27.648 \text{ N/mm}^2
\]

Stiffener stress \( s_{st} = \text{comp}_1 + \text{comp}_2 + \text{comp}_3 = 123.99 \text{ N/mm}^2 \)

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

Expression 9.3 in EN 1993-1-5 is further simplified by Table 2.6 in "Eurocode 3: Design of steel structures Part 1-5: Design of plated structures" ECCS Eurocode Design Manuals.

Ratio \( p = h_{sb} / t_{sb} = 7.5 \)

Limit to ratio \( p = h_{sb} / t_{sb} \) is \( 13 \epsilon = 10.729 \).

Ratio \( h_{sb} / t_{sb} = 7.5 \) is not greater than limit \( 13 \epsilon = 10.729 \).

Stiffener complies with Clause 9.2.1(8).

**Action as a rigid end post**

A rigid end post comprise of two double-sided transverse stiffeners that form the flanges of a short beam of length \( h_w \). The effective section below will be taken to exclude the outer parts of web plate.

![Diagram of stiffeners and web elements]
Minimum area  
Actual area provided  
As Asp ≥ Amin ( 6000 mm² ≥ 2500 mm² ), stiffener area is adequate.  
In-plane moment  
End post 2nd moment of area  
Worst stiffener stress from membrane action will be calculated below.  
Stress due to membrane action  

Cross-section resistance check

The maximum stresses from membrane action and bearing reaction will be added together here for simplicity, even though they occur at different heights on the stiffener.  
Total stress  
As sstT ≤ fy ( 211.29 N/mm² ≤ 345 mm² ), stiffener is OK.

Buckling check under combined bending and axial load

The stiffener will be checked for buckling under combined bending and axial load in accordance with BS EN 1993-1-1, Clause 6.3.3.  
Critical load  
Slenderness of the stiffener  
Design axial load resistance  
Design moment resistance yy axis  
Design moment resistance zz axis  
Design moment resistance zz axis  
Interaction equation: 

Connection of stiffeners to girder web

Net stiffener length  
Assumed weld size  
Effective length of weld  
Design load on weld  
Design weld resistance  

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DESIGN SUMMARY

Span of plate girder: L=12 m
All steel to be: Grade S 355

End stiffeners: c2

Posts 1 & 2 are double-sided:
- Post 1: 150 mm x 20 mm flats
- Post 2: 150 mm x 20 mm flats

Note: All welds to be 6 mm FW of design strength of 241 N/mm².
Assume rigid end posts to BS EN 1993-1-5, Fig 5.1 & Table 5.1.
Location: Ex2 - Two double-sided bearing stiffeners at support

Bearing stiffeners at end support

The proforma is in accordance with BS EN 1993-2:2006 " EC3 Design of Steel Structures - Part 2:Steel bridges ", the NA to BS EN 1993-2:2006 and BS EN 1993-1-5 section 9.
A rigid end post is assumed in the design which comprise of two double sided transverse stiffeners that form the flanges of a short beam of length hw, as highlighted in BS EN 1993-1-5 Clause 9.3.1(2).

Span of girder                    L=20 m
Depth of girder between flanges   hw=1950 mm
Flange thickness                  tf=30 mm
Web thickness                     tw=18 mm
Assumed stiffener thickness       ts=20 mm
Stiffener corner snipe            snp=15 mm

Partial factors for resistance

Partial factor                    gamM1=1.1
Partial factor (welds)            gamM2=1.25

Strength of steel

Steel grade                       S 275
Yield strength (web)              fyw=265 N/mm²
Yield strength (stiffener)        fys=265 N/mm²
Yield strength to be adopted      fy=265 N/mm²
Design strength of fillet weld    fvwd=fu/(3^0.5*bettw*gamM2)  =222.79 N/mm²

Check web thickness

Stiffener spacing of first panel  ae=1500 mm
Intermediate stiffener spacing   ai=3000 mm
Limiting web thick for service   tlim=hw/250=7.8 mm
Web thickness exceeds limiting value for serviceability, hence OK.
Limiting web thick flange buckl  tlim=(hw/250)*(fy/345)=5.9913 mm
Web thickness exceeds limiting value for flange buckling, hence OK.

Bearing stiffeners double sided - BS EN 1993-1-5 section 9

Bearing stiffeners are assumed to be symmetric about the beam web and will be designed as a strut to BS EN 1993-1-5 Clause 9.4(2), resisting the bearing reaction together with any eccentricities.
compute $kt$ from Annex A

$$kt = 4 + 5.34 \times (hw/a)^2 + ktsl = 13.025$$

Web shear resistance

$$V_{bRd} = \chi_w f_y \frac{hw \times tw}{(\sqrt{3}) \times \text{gamM1} \times 1000} = 4754.2 \text{ kN}$$

Design bearing reaction

$$Nb_{Ed} = 3590 \text{ kN}$$

**Action as a bearing stiffener**

Two double-sided stiffeners at

Support

**NOTE:** Stiffeners to be equally spaced about the centreline of bearing.

Depth of bearing stiffener $hsb = 225 \text{ mm}$

Thickness of bearing stiffener $tsb = 20 \text{ mm}$

Distance $c1 = 0 \text{ mm}$

Distance between stiffener CL's $c2 = 300 \text{ mm}$

Eccentricity in each direction $ecc = 10 \text{ mm}$

Worst stress in web plate:

$$swp = Nb_{Ed} \times 1E3/A + M_{zEd} \times 1E6 \times (15 \times ew \times tw + tsb/2 + c2/2)/(I_{zz})$$

$$= 148.27 \text{ N/mm}^2$$

Worst stress in stiffener:

$$\text{comp1} = Nb_{Ed} \times 1E3/A = 126.69 \text{ N/mm}^2$$

$$\text{comp2} = M_{zEd} \times 1E6 \times (c2/2 + tsb/2)/(I_{zz})$$

$$= 8.336 \text{ N/mm}^2$$

$$\text{comp3} = M_{yEd} \times 1E6 \times (hsb + tw/2)/(I_{yy})$$

$$= 24.586 \text{ N/mm}^2$$

Stiffener stress $sst = \text{comp1} + \text{comp2} + \text{comp3} = 159.61 \text{ N/mm}^2$

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

Expression 9.3 in EN 1993-1-5 is further simplified by Table 2.6 in "Eurocode 3: Design of steel structures Part 1-5: Design of plated structures" ECCS Eurocode Design Manuals.

Ratio $(hsb/tsb)$ $p = hsb/tsb = 11.25$

Limit to ratio $p = (hsb/tsb)$ is $13\varepsilon = 12.242$.

Ratio $(hsb/tsb) = 11.25$ is not greater than limit $13\varepsilon = 12.242$.

Stiffener complies with Clause 9.2.1(8).

**Action as a rigid end post**

A rigid end post comprise of two double-sided transverse stiffeners that form the flanges of a short beam of length $hw$. The effective section below will be taken to exclude the outer parts of web plate.

Two double-sided stiffeners at

Support
Minimum area
Actual area provided
As Asp ≥ Amin (9000 mm² ≥ 8424 mm²), stiffener area is adequate.
In-plane moment
End post 2nd moment of area
Worst stiffener stress from membrane action will be calculated below.
Stress due to membrane action

Cross-section resistance check

The maximum stresses from membrane action and bearing reaction will be added together here for simplicity, even though they occur at different heights on the stiffener.
Total stress
As sstT ≤ fy (159.61 N/mm² ≤ 265 mm²), stiffener is OK.

Buckling check under combined bending and axial load

The stiffener will be checked for buckling under combined bending and axial load in accordance with BS EN 1993-1-1, Clause 6.3.3.
Critical load
Slenderness of the stiffener
Design axial load resistance
Design moment resistance yy axis
Design moment resistance zz axis
Design moment resistance zz axis
Interaction equation:

Univ=NEd/(chiy*NplRd)+ratio*(MyEd/MyRd)+(MzEdb/MzRdb+MzEdm/MzRdm)
As Univ ≤ 1.0 (0.66477 ≤ 1.0), the interaction equation is OK.

Connection of stiffeners to girder web

Net stiffener length
Assumed weld size
Effective length of weld
Design load on weld
Design weld resistance
DESIGN SUMMARY

Span of plate girder  L = 20 m
All steel to be    Grade S 275

End stiffeners:  c2 ae

Posts 1 & 2 are double-sided c2=300 mm
stiffeners at ae=1500 mm end of girder.

Double-sided stiffeners: post 1  2No/225 mm x 20 mm flats
                      post 2  2No/225 mm x 20 mm flats

Note: All welds to be 6 mm FW of design strength of 222 N/mm². Assume rigid end posts to BS EN 1993-1-5, Fig 5.1 & Table 5.1.
Location: Ex3 - Single double-sided bearing stiffener at support

Bearing stiffeners at end support

The proforma is in accordance with BS EN 1993-2:2006 " EC3 Design of Steel Structures - Part 2:Steel bridges ", the NA to BS EN 1993-2:2006 and BS EN 1993-1-5 section 9.
A non-rigid end post is assumed in the design which comprise of a single double sided transverse stiffener of length hw, as highlighted in BS EN 1993-1-5 Clause 9.3.1(2).

Span of girder \( L = 12 \) m
Depth of girder between flanges \( hw = 1200 \) mm
Flange thickness \( tf = 30 \) mm
Web thickness \( tw = 12.5 \) mm
Assumed stiffener thickness \( ts = 20 \) mm
Design bearing reaction \( NbEd = 1825 \) kN
Stiffener corner snipe \( snp = 15 \) mm

Partial factors for resistance

Partial factor \( \gamma_{M1} = 1.1 \)
Partial factor (welds) \( \gamma_{M2} = 1.25 \)

Strength of steel

Steel grade \( S\ 355 \)
Yield strength (web) \( f_{yw} = 355 \) N/mm²
Yield strength (stiffener) \( f_{ys} = 345 \) N/mm²
Yield strength to be adopted \( f_y = 345 \) N/mm²
Design strength of fillet weld \( f_{vwd} = f_u / (3^0.5 * \text{betw} * \gamma_{M2}) = 241.2 \) N/mm²

Check web thickness

Stiffener spacing of first panel \( ae = 1500 \) mm
Intermediate stiffener spacing \( ai = 3000 \) mm
Limiting web thick for service \( t_{lim} = hw / 250 = 4.8 \) mm
Web thickness exceeds limiting value for serviceability, hence OK.
Limiting web thick flange buckl \( t_{lim} = (hw / 250) * (fy/345) = 4.8 \) mm
Web thickness exceeds limiting value for flange buckling, hence OK.
Bearing stiffener (double sided non-rigid end post)

It is assumed that the support reaction acts at the centroid of the effective stiffener section, such that there is no bending moment induced. Bearing stiffener consists a single double-sided stiffener. The stiffener effective cross section is as shown below:

One double-sided stiffener at support

Depth of bearing stiffener \( h_{sb} = 150 \text{ mm} \)
Thickness of bearing stiffener \( t_{sb} = 20 \text{ mm} \)
Distance \( c_1 = 300 \text{ mm} \)
Design buckling resistance \( N_{b,Rd} = \chi \ast A_{st} \ast f_{y,sb} / (\gamma_M 1 \ast 1000) = 3156.4 \text{ kN} \)

As \( N_{b,Ed} \leq N_{b,Rd} (1825 \text{ kN} \leq 3156.4 \text{ kN}) \), the design bearing reaction at the support is less than the design buckling resistance. Hence satisfactory.

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Expression 9.3 in EN 1993-1-5 is further simplified by Table 2.6 in "Eurocode 3: Design of steel structures Part 1-5: Design of plated structures" ECCS Eurocode Design Manuals.

Ratio \( p = h_{sb} / t_{sb} = 7.5 \)
Limit to ratio \( p = (h_{sb} / t_{sb}) \) is \( 13\gamma = 10.729 \).
Ratio \( (h_{sb} / t_{sb}) = 7.5 \) is not greater than limit \( 13\gamma = 10.729 \).
Stiffener complies with Clause 9.2.1(8).

Connection of stiffeners to girder web

Net stiffener length \( L_{sn} = h_{w} - 2 \ast s_{np} = 1170 \text{ mm} \)
Assumed weld size \( s_{w}' = 6 \text{ mm} \)
Effective length of weld \( L_{w} = 4 \ast (L_{sn} - 2 \ast s_{w}') = 4632 \text{ mm} \)
Design load on weld \( F_{w,Ed} = N_{b,Ed} \ast 10^{-3} / L_{w} = 394 \text{ N/mm} \)
Design weld resistance \( F_{w,Rd} = 0.7 \ast s_{w}'' \ast f_{w,d} = 1013.1 \text{ N/mm} \)

Plastic shear resistance - Clause 6.2.5

Shear area \( A_{vz} = h_{w} \ast t_{w} = 15000 \text{ mm}^2 \)
Plastic shear resistance of web \( V_{pl,Rd} = A_{vz} \ast f_{y} / SQR(3) / (\gamma_M 0 \ast 1000) = 2987.8 \text{ kN} \)

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

As \( h_{w} / t_{w} > 72 \ast ew (96 > 58.58) \), the web shear buckling resistance needs to be checked (EN 1993-1-5 Clause 5.2 and 5.3).
Buckling coefficient (Annex A) \( \kappa t = 5.34 + 4 \ast (h_{w} / a) \wedge 2 = 7.9 \)
Modified slenderness $\lambda$

$\lambda_{aw} = \frac{hw}{(37.4 \times tw \times ew \times SQR(kt))} = 1.1224$

Shear resistance of web $V_{plRd} = 2987.8 \text{ kN}$

Reduced design shear resistance at support (web contribution only will be considered):

$V_{bwRd} = \chi_{w} \times f_{yw} \times hw \times tw / (3^{0.5} \times \gamma_{M1} \times 1000) = 2066.7 \text{ kN}$

Design shear at support $V_{Ed(0)} = Nb_{Ed} = 1825 \text{ kN}$

As $V_{Ed(0)} \leq V_{bwRd}$ (1825 kN ≤ 2066.7 kN), the shear force is less than the shear resistance. Hence section OK.

**DESIGN SUMMARY**

Span of plate girder $L = 12 \text{ m}$

All steel to be Grade S 355

End stiffener:

<table>
<thead>
<tr>
<th>Post 1</th>
<th>c1</th>
<th>ae</th>
</tr>
</thead>
<tbody>
<tr>
<td>c1=300 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ae=1500 mm</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Double-sided stiffeners: post 1 1No/150 mm x 20 mm flats

Note: All welds to be 6 mm FW of design strength of 241 N/mm².

Assume non-rigid end posts to BS EN 1993-1-5, Fig 5.1 & Table 5.1.

Interaction between shear force, bending moment and axial force needs to be considered as the actual shear force exceeds $V_{bwRd}/2$. This is check is beyond the scope of this proforma.
**Location: Ex1 - Stiffener A**

**Transverse web stiffeners (Clause 9.13)**

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

The proforma is for transverse web stiffeners other than at supports for webs without longitudinal stiffeners (Clause 9.13).

![Diagram of web with transverse stiffeners]

Elevation on web without longitudinal stiffening

- Panel length 1: \(a(1)=2300\ mm\)
- Panel width: \(b=2274\ mm\)
- Web panels 1 and 2.

Longitudinal stresses in web calculated without any redistribution to the flanges.

- Stress at top of web plate \(\sigma_{top}\): \(stp(1)=73\ N/mm^2\)
- Stress at btm of web plate \(\sigma_{btm}\): \(sbm(1)=-57\ N/mm^2\)

**Axial force due to tension field action (Clause 9.13.3.2)**

- Depth \(d_w\) for panels: \(dw(1)=2274\ mm\)
- Height of any hole: \(hh(1)=0\ mm\)
- Shear force acting on panels: \(V(1)=1450\ kN\)
- Force due to tension field action: \(F_{tw}=F_{tw}(1)=893.18\ kN\)
Axial force representing the destabilizing influence of the web

\[ b_{we} \quad \text{b_{we}} \]

The effective stiffener section should comprise the stiffener section with a portion of web plate of width \( b_{we} \) on each side of the stiffener connection centreline taken as the lesser of:

- 16 times the web thickness = 176 mm
- Half the web panel length = 2300 mm

Geometric properties of effective stiffener section

Second moment of area about centroidal X-X axis.
Second moment of area \( I_{xx} = 38.512 \times 10^6 \) mm\(^4\)
Cross-sectional area \( A_{st} = 10280 \) mm\(^2\)
Nominal yield stress of stiffener \( f_{ys} = 345 \text{ N/mm}^2 \)
Nominal yield stress of web \( f_{yw} = 345 \text{ N/mm}^2 \)
Axial compressive force representing destabilising influence of web assumed to act along the centroidal axis of the stiffener.
Force \( F_{wi1} = l_s^1 l_s^1 t_w^1 k_s^1 f_{R}^1 / (a(3) \times 1000) \) = 137.98 kN

Strength of transverse web stiffeners (Clause 9.13.5)

Section 1:

Yielding of web plate (Clause 9.13.5.1)
Partial factor on yield strength \( \gamma_{mc} = 1.05 \)
Partial factor \( \gamma_{f3} \) \( \gamma_{f3} = 1.1 \)
Equivalent compressive stress in web \( 175.32 \, \text{N/mm}^2 \) does not exceed limiting stress \( 298.7 \, \text{N/mm}^2 \).
Web plate satisfactory for yielding.

**Yielding of stiffener (Clause 9.13.5.2)**

Distance from X-X axis to section at which stress is calculated:
- X-X axis to section on Side 1 \( dx_1(q) = 64.41 \, \text{mm} \)
- X-X axis to section on Side 2 \( dx_2(q) = 124.59 \, \text{mm} \)
- X-X axis to centre of web \( dx_w(q) = 58.91 \, \text{mm} \)
- Second moment of area (X-X axis) \( I_x(q) = 38.512 \times 10^6 \, \text{mm}^4 \)
- Cross-sectional area \( A(q) = 10280 \, \text{mm}^2 \)
- Force in stiffener \( F_{ts}(1) = 0 \, \text{kN} \)
- Applied moment about X-X axis \( M_t(1) = 0 \, \text{kNm} \)

**Transverse stresses in stiffener:**

**Stress \( \sigma_{es} \):**
- Side 1 \( \sigma_{es}(1) = \frac{F(q)}{A(q)} \times 1000 + \frac{M(q) \times dx_1(q)}{I_x(q)} \times 10^6 = 174.89 \, \text{N/mm}^2 \)
- Side 2 \( \sigma_{es}(1) = \frac{F(q)}{A(q)} \times 1000 - \frac{M(q) \times dx_2(q)}{I_x(q)} \times 10^6 = -83.337 \, \text{N/mm}^2 \)

Partial factor on yield strength \( \gamma_{mc} = 1.05 \)
Partial factor \( \gamma_{f3} \) \( \gamma_{f3} = 1.1 \)
Compressive transverse stress in stiffener \( 174.89 \, \text{N/mm}^2 \) does not exceed limiting stress \( 298.7 \, \text{N/mm}^2 \).
Stiffener satisfactory for yielding.
Buckling of effective stiffener section (Clause 9.13.5.3)

Geometric properties of effective stiffener section

Distance from X-X axis to section at which stress is calculated:
- X-X axis to section on Side 1: \( dx_1(q) = 64.41 \text{ mm} \)
- X-X axis to section on Side 2: \( dx_2(q) = 124.59 \text{ mm} \)
- X-X axis to centre of web: \( dx_w(q) = 58.91 \text{ mm} \)

Second moment of area (X-X axis): \( I_{x(q)} = 38.512 \times 10^6 \text{ mm}^4 \)

Cross-sectional area: \( A(q) = 10280 \text{ mm}^2 \)

Force from tension field action: \( F_{tw(1)} = 893.18 \text{ kN} \)

Force from destabilizing effects: \( F_{wi(1)} = 137.98 \text{ kN} \)

Force in stiffener: \( F_{ts(1)} = 0 \text{ kN} \)

Applied moment about X-X axis: \( M_{t(1)} = 0 \text{ kNm} \)

Inequality for buckling:
- Factor Side 1: \( v_1(1) = \frac{F(q) \times 10^3}{A(q) \times \sigma_{ls}} + \frac{M(q) \times 10^6}{Z_{x1(q)} \times \sigma_{sys}} = 0.61045 \)
- Factor Side 2: \( v_2(1) = \frac{F(q) \times 10^3}{A(q) \times \sigma_{ls}} - \frac{M(q) \times 10^6}{Z_{x2(q)} \times \sigma_{sys}} = -0.13802 \)

Partial factor on yield strength: \( g_{mbc} = 1.2 \)

Partial factor gamma3: \( g_f3 = 1.1 \)

Factor \( \frac{1}{(g_{mbc} \times g_f3)} = \frac{1}{(1.2 \times 1.1)} = 0.75758 \)

Hence critical factor for Side 1: 0.61045

Left hand side of inequality 0.61045 is not greater than right hand side 0.75758. Hence stiffener section is OK for buckling.
Location: Ex2 - Stiffener A

Transverse web stiffeners (Clause 9.13)

The calculations are in accordance with BS 5400:Part 3:2000 "Code of Practice for Design of Steel Bridges" as implemented by Departmental Standard BD 13/06.

The proforma is for transverse web stiffeners other than at supports for webs without longitudinal stiffeners (Clause 9.13).

Elevation on web without longitudinal stiffening

Panel length 1  a(1)=2300 mm
Panel length 2  a(2)=2000 mm
Panel width  b=2274 mm
Length of stiffener  l_s=2274 mm
Thickness of web  t_w=12 mm
Modulus of elasticity of web  E=205000 N/mm²

Web panel 1:
Longitudinal stresses in web calculated without any redistribution to the flanges.

Compression stresses are positive.

Stress at top of web plate  σ_{top}  stp(1)=80 N/mm²
Stress at btm of web plate  σ_{btm}  sbm(1)=-60 N/mm²
Axial force due to tension field action (Clause 9.13.3.2)

Depth dw for panel 1  
dw(1)=2274 mm

Height of any hole  
hh(1)=0 mm

Shear force acting on panel  
V(1)=1500 kN

Web panel 2 :

Stress at top of web plate åtop  
stp(2)=90 N/mm²

Stress at btm of web plate åbtm  
sbm(2)=-70 N/mm²

Axial force due to tension field action (Clause 9.13.3.2)

Depth dw for panel 2  
dw(2)=2274 mm

Height of any hole  
hh(2)=0 mm

Shear force acting on panel  
V(2)=1200 kN

Force due to tension field action taken as average value from two panels

Force due to tension field action  
Ftw=(Ftw(1)+Ftw(2))/2=572.99 kN

Axial force representing the destabilizing influence of the web

bwe  bwe
|     |     |
---|---|
web

16 times the web thickness =192 mm

Half the web panel length =2150 mm

Axial compressive force representing destabilising influence of web assumed to act along the centroidal axis of the stiffener.

Force  
Fwi1=ls*ls*tw*ks*sigR/(a(3)*1000)  
=172.95 kN

Reduced force  
Fwi1=Fwi1*ns1=172.12 kN

Axial compressive force representing destabilising influence of web assumed to act along the centroidal axis of the stiffener.

Force  
Fwi2=ls*ls*tw*ks*sigR/(a(3)*1000)  
=86.475 kN

Reduced force  
Fwi2=Fwi2*ns2=86.422 kN
Strength of transverse web stiffeners (Clause 9.13.5)

Section 1:

Yielding of web plate (Clause 9.13.5.1)

Geometric properties of effective stiffener section

Distance from:
Force in stiffener \( Fts(1) = 80 \text{ kN} \)
Applied moment about X-X axis \( Mt(1) = 20 \text{ kNm} \)
Dimension \( ys \) \( ys(1) = 1137 \text{ mm} \)
Equivalent long. stress \( \tau_R = 25.573 \text{ N/mm}^2 \)
Partial factor on yield strength \( gmc = 1.05 \)
Partial factor \( gf3 = 1.1 \)
Equivalent compressive stress in web 158.2 N/mm\(^2\) does not exceed limiting stress 298.7 N/mm\(^2\).
Web plate satisfactory for yielding.
Yielding of stiffener (Clause 9.13.5.2)

Distance from X-X axis to section at which stress is calculated:
- X-X axis to section on Side 1: $dx_1(q) = 60$ mm
- X-X axis to section on Side 2: $dx_2(q) = 100$ mm
- X-X axis to centre of web: $dx_w(q) = 60$ mm

Second moment of area (X-X axis): $I_x(q) = 40E6$ mm$^4$

Cross-sectional area: $A(q) = 9760$ mm$^2$

Force in stiffener: $F_{ts}(1) = 80$ kN

Applied moment about X-X axis: $M_t(1) = 20$ kNm

Transverse stresses in stiffener:
- Side 1: $\sigma_{t3}(1) = \frac{F(q)}{A(q)} \times 1000 + \frac{M(q) \times dx_1(q)}{I_x(q)} \times 1E6 = 148.47$ N/mm$^2$
- Side 2: $\sigma_{t4}(1) = \frac{F(q)}{A(q)} \times 1000 - \frac{M(q) \times dx_2(q)}{I_x(q)} \times 1E6 = -69.044$ N/mm$^2$

Partial factor on yield strength: $g_{mc} = 1.05$

Partial factor $\gamma_f3$: $g_f3 = 1.1$

Compressive transverse stress in stiffener 148.47 N/mm$^2$ does not exceed limiting stress 298.7 N/mm$^2$.

Stiffener satisfactory for yielding.
Buckling of effective stiffener section (Clause 9.13.5.3)

Distance from X-X axis to section at which stress is calculated:
- X-X axis to section on Side 1: \(dx_1(q) = 60 \text{ mm}\)
- X-X axis to section on Side 2: \(dx_2(q) = 100 \text{ mm}\)
- X-X axis to centre of web: \(dx_w(q) = 60 \text{ mm}\)

Second moment of area (X-X axis): \(I_x(q) = 40E6 \text{ mm}^4\)
Cross-sectional area: \(A(q) = 9760 \text{ mm}^2\)
Force from tension field action: \(F_{tw}(1) = 572.99 \text{ kN}\)
Force from destabilizing effects: \(F_{wi}(1) = 172.12 \text{ kN}\)
Force in stiffener: \(F_{ts}(1) = 80 \text{ kN}\)
Applied moment about X-X axis: \(M_t(1) = 20 \text{ kNm}\)

Inequality for buckling:
- Factor Side 1: \(v_1(1) = F(q) \times 1E3 / (A(q) \times \sigma_{l,s}) + M(q) \times 1E6 / (2 \times x_1(q) \times \sigma_{y,s}) = 0.53106\)
- Factor Side 2: \(v_2(1) = F(q) \times 1E3 / (A(q) \times \sigma_{l,s}) - M(q) \times 1E6 / (2 \times x_2(q) \times \sigma_{y,s}) = -0.099428\)

Partial factor on yield strength: \(g_{mbc} = 1.2\)
Partial factor \(\gamma_f3\): \(g_f3 = 1.1\)
Factor \(1 / (\gamma_{mbc} \times \gamma_{f3})\): \(w = 1 / (g_{mbc} \times g_f3) = 0.75758\)

Hence critical factor for Side 1: 0.53106
Left hand side of inequality 0.53106 is not greater than right hand side 0.75758. Hence stiffener section is OK for buckling.
Location: Ex1 - Transverse web stiffener

Transverse web stiffeners (BS EN 1993-1-5 section 9)

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma is for transverse web stiffeners other than at supports for webs without longitudinal stiffeners. No rules are given within BS EN 1993-2:2006 for the design of intermediate transverse stiffeners so reference has to be made to BS EN 1993-1-5 section 9.

```
          a(1)   a(2)                  Top flange

hw          Web panel 1          Web panel 2

          a(1)   a(2)                  Bottom flange

design for this stiffener

Elevation on web without longitudinal stiffening
```

Panel length 1  a(1)=4000 mm
Panel width      hw=1200 mm
Length of stiffener  ls=1200 mm
Thickness of web  tw=12 mm
Modulus of elasticity of web  E=210000 N/mm²
Stiffener corner snipe  snp=15 mm

Web panels 1 & 2:

```
          \sigma_{top}   \sigma_{btm}

ls          Longitudinal stresses in web with transverse stiffeners

ls=1200 mm

Compression stresses are positive

Stress at top of web plate (\sigma_{top}) \sigma_{top}=200 N/mm²
Stress at btm of web plate (\sigma_{btm}) \sigma_{btm}=-200 N/mm²
```


Design parameters

- Depth of stiffener: $h_s = 150$ mm
- Stiffener thickness: $t_s = 15$ mm
- Yield strength (web): $f_{yw} = 355$ N/mm$^2$
- Yield strength (stiffener): $f_{ys} = 355$ N/mm$^2$

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Expression 9.3 in EN 1993-1-5 is further simplified by Table 2.6 in "Eurocode 3: Design of steel structures Part 1-5: Design of plated structures" ECCS Eurocode Design Manuals.

Yield strength to be adopted $f_y = 355$ N/mm$^2$

Strain value $\varepsilon$ (EC3 Part 1-1) $\varepsilon = \sqrt{235/f_{yw}} = 0.81362$

Strain value $\varepsilon$ (EC3 Part 1-1) $\varepsilon = \sqrt{235/f_{ys}} = 0.81362$

Ratio $(h_s/t_s)$ $p = h_s/t_s = 10$

Limit to ratio $p = (h_s/t_s)$ is $13\varepsilon = 10.577$.

Ratio $(h_s/t_s) = 10$ is not greater than limit $13\varepsilon = 10.577$.

Stiffener complies with Clause 9.2.1(8).

Design shear resistance (single-sided transverse stiffener)

The effective cross section of the intermediate single-sided transverse stiffener is as shown below:

$$
\begin{align*}
&\text{tsi} & & 15.\varepsilon \cdot tw & & 15.\varepsilon \cdot tw \\
&tw & & & & \\
&hsi & & \text{PLAN ON INTERMEDIATE STIFFENER}
\end{align*}
$$

2nd moment of area $yy$ axis $I_{yy} = I_{pro} = 13.438E6$ mm$^4$

Design shear resistance $V_{bRd} = \chi \cdot A_e \cdot f_{yw} / (\gamma_M 1 \cdot 1000) = 1871.9$ kN

Axial force in the stiffener due to shear - BS EN 1993-1-5

Shear buckling coefficient for web panel $k_t$ is derived from Clause 5.2.

Buckling coefficient (Annex A) $k_t = 5.34 + 4 \cdot (h_w/a)^2 = 5.7$

Modified slenderness $\lambda_w = 0.76 \cdot (f_y/t_{cr}) \cdot 0.5 = 1.3767$

Max design shear force in panel $V_{Ed1} = 1700$ kN

Externally applied design load $P_{ext1} = 0$ kN

Option 1:

Longitudinal compressive stress $\sigma_{Ed} = 0$ for symmetrical sections.

Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):

$$PE_{d1}' = V_{Ed1} \cdot hw \cdot tw \cdot 0.8 \cdot t_{cr} / 1000 = 453.7 \text{ kN}$$

Option 2:

Vertical force generated in the stiffener as per BS EN 1993-1-5 (see NOTE, Clause 9.3.3(3)):

$$PE_{d1} = V_{Ed1} \cdot f_{yw} \cdot hw \cdot tw / (SQR(3) \cdot \gamma_M 1 \cdot \lambda_w^2 \cdot 1000) = 284.36 \text{ kN}$$
Generated force to be adopted \[ P_{Ed1} = 453.7 \text{ kN} \]

Design axial load in stiffener \[ P_{Ed} = P_{Ed1} + P_{ext1} = 453.7 \text{ kN} \]

As \( P_{Ed} \leq V_{bRd} \) (453.7 kN \leq 1871.9 kN), the design shear force does not exceed the shear resistance. Hence OK.

**Connection to the web**

Design shear load on weld \[
F_{wEd} = tw^2/(5*hs) + P_{Ed}/(hw-2*snp)
\]

\[
= 0.57978 \text{ kN/mm}
\]

Weld size required \[ s_{ww} = F_{wEd} * 10^3/(2*0.7*fvwd) = 1.7169 \text{ mm} \]

Weld size provided \[ s_{w1} = 6 \text{ mm} \]

**Stiffness check - Clause 9.3.3(3)**

2nd moment of area (effe. section) \[ I_{pro} = 13.438E6 \text{ mm}^4 \]

2nd moment of area required \[ I_{req} = 0.75*hw*tw^3 = 1.5552E6 \text{ mm}^4 \]

As \( I_{req} \leq I_{pro} \) (1.5552E6 mm^4 \leq 13.438E6 mm^4), the stiffener is OK.

**DESIGN**

<table>
<thead>
<tr>
<th>Panel length</th>
<th>4000 mm</th>
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**SUMMARY**

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<tr>
<th>Panel width</th>
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<tr>
<th>Beam web thickness</th>
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<th>Depth of stiffener</th>
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<th>Stiffener thickness</th>
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<tr>
<th>Design shear force</th>
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<tr>
<th>Axial load in stiffener</th>
<th>453.7 kN</th>
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<table>
<thead>
<tr>
<th>Design shear resistance</th>
<th>1871.9 kN</th>
</tr>
</thead>
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**INTERMEDIATE**

<table>
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<tr>
<th>Load carrying stiffener</th>
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**STIFFENER**

<table>
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<tr>
<th>1No/150 mm x 15 mm flats</th>
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<tr>
<th>Grade S 355 steel</th>
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<table>
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<tr>
<th>connected by 2/6 mm FW</th>
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</thead>
</table>
Location: Ex2 - Transverse web stiffener

Transverse web stiffeners (BS EN 1993-1-5 section 9)

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma is for transverse web stiffeners other than at supports for webs without longitudinal stiffeners. No rules are given within BS EN 1993-2:2006 for the design of intermediate transverse stiffeners so reference has to be made to BS EN 1993-1-5 section 9.

\[ a(1) \quad a(2) \]
\[ hw \]
\[ \text{Web panel 1} \quad \text{Web panel 2} \]
\[ a(1)=4000 \, \text{mm} \]
\[ hw=1200 \, \text{mm} \]
\[ ls=1200 \, \text{mm} \]
\[ tw=12 \, \text{mm} \]
\[ E=210000 \, \text{N/mm}^2 \]
\[ \text{snp}=15 \, \text{mm} \]

Web panels 1 & 2:

Longitudinal stresses in web with transverse stiffeners

\[ ls=1200 \, \text{mm} \]

Compression stresses are positive

Stress at top of web plate (\( \sigma_{\text{top}} \)) \( \text{stop}=200 \, \text{N/mm}^2 \)

Stress at btm of web plate (\( \sigma_{\text{btm}} \)) \( \text{sbtm}=-200 \, \text{N/mm}^2 \)
Design parameters

Depth of stiffener \( h_s = 150 \, \text{mm} \)
Stiffener thickness \( t_s = 15 \, \text{mm} \)
Yield strength (web) \( f_yw = 355 \, \text{N/mm}^2 \)
Yield strength (stiffener) \( f_y = 355 \, \text{N/mm}^2 \)

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Expression 9.3 in EN 1993-1-5 is further simplified by Table 2.6 in "Eurocode 3: Design of steel structures Part 1-5: Design of plated structures" ECCS Eurocode Design Manuals.

Yield strength to be adopted \( f_y = 355 \, \text{N/mm}^2 \)
Strain value \( \varepsilon \) (EC3 Part 1-1) \( \varepsilon_w = \sqrt{\frac{235}{f_yw}} = 0.81362 \)
Strain value \( \varepsilon \) (EC3 Part 1-1) \( \varepsilon_s = \sqrt{\frac{235}{f_y}} = 0.81362 \)
Ratio \( (h_s/t_s) \) \( p = h_s/t_s = 10 \)
Limit to ratio \( p = (h_s/t_s) \) is \( 13\varepsilon = 10.577 \).
Ratio \( (h_s/t_s) = 10 \) is not greater than limit \( 13\varepsilon = 10.577 \).
Stiffener complies with Clause 9.2.1(8).

Design shear resistance (double sided transverse stiffener)

The effective cross section of the intermediate double-sided transverse stiffener is as shown below:

\[
\begin{align*}
\text{tsi} & \quad \text{15}\varepsilon\cdot\text{tw} & \quad \text{15}\varepsilon\cdot\text{tw} \\
\text{hsi} & \quad \text{tw} & \quad \text{PLAN ON INTERMEDIATE} \\
& & \quad \text{hsi} \quad \text{STIFFENER}
\end{align*}
\]

2nd moment of area \( yy \) axis:
\[
I_{yy} = t_s i \times (2 \times h_s i \times t_w + 3 \times 12 + (30 \times \varepsilon_w \times t_w) \times t_w \times 3 \times 12
= 38,006E6 \, \text{mm}^4
\]

Design shear resistance \( V_{b Rd} = \frac{\chi_i \times A_e \times f_y w}{\gamma_M 1 \times 1000} = 2644.7 \, \text{kN} \)

Axial force in the stiffener due to shear - BS EN 1993-1-5

Shear buckling coefficient for web panel \( k_t \) is derived from Clause 5.2.
Buckling coefficient (Annex A) \( k_t = 5.34 + 4 \times (h_w / a) \times 2 = 5.7 \)
Modified slenderness \( \lambda_w \) \( \lambda_w = 0.76 \times (f_y / t_c r) \times 0.5 = 1.3767 \)
Max design shear force in panel \( V_{Ed1} = 1450 \, \text{kN} \)
Externally applied design load \( P_{ext1} = 0 \, \text{kN} \)

Option 1:
Longitudinal compressive stress \( \sigma_x Ed = 0 \) for symmetrical sections.
Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):
\[
P_{Ed1} = V_{Ed1} - h_w \times t_w \times 0.8 \times t_c r / 1000 = 203.7 \, \text{kN}
\]
Option 2:
Vertical force generated in the stiffener as per BS EN 1993-1-5
(see NOTE, Clause 9.3.3(3)):
\[
\text{PEd1}=\frac{VEd1-fyw*hw*tw}{(SQR(3))*gamM1*lamw^2*1000}=34.359 \text{ kN}
\]
Generated force to be adopted \( \text{PEd1}=203.7 \text{ kN} \)
Design axial load in stiffener \( \text{PEd}=\text{PEd1}+\text{Pext1}=203.7 \text{ kN} \)
As \( \text{PEd} \leq \text{VbRd} \) (203.7 kN \( \leq \)  2644.7 kN ), the design shear force does not exceed the shear resistance. Hence OK.

**Connection to the web**

Design shear load on weld
\[
\text{FwEd}=\frac{tw^2}{5*hs}+\frac{\text{PEd}}{2*(hw-2*snp)}=0.27905 \text{ kN/mm}
\]
Weld size required \( \text{sww}=\frac{\text{FwEd}*10^3}{2*0.7*fvwd}=0.82636 \text{ mm} \)
Weld size provided \( \text{sw1}=6 \text{ mm} \)

**Stiffness check - Clause 9.3.3(3)**

2nd moment of area (effe.section) \( \text{Ipro}=38.006E6 \text{ mm}^4 \)
2nd moment of area required \( \text{Ireq}=0.75*hw*tw^3=1.5552E6 \text{ mm}^4 \)
As \( \text{Ireq} \leq \text{Ipro} \) (1.5552E6 mm\(^4\) \( \leq \) 38.006E6 mm\(^4\) ), the stiffener is OK.

**DESIGN**

<table>
<thead>
<tr>
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<tbody>
<tr>
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<td>Depth of stiffener</td>
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<td>Stiffener thickness</td>
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<tr>
<td>Design shear force</td>
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<tr>
<td>Axial load in stiffener</td>
<td>203.7 kN</td>
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<tr>
<td>Design shear resistance</td>
<td>2644.7 kN</td>
</tr>
</tbody>
</table>

**INTERMEDIATE**

Load carrying stiffener

**STIFFENER**

2No/150 mm x 15 mm flats
Grade S 355 steel
connected by 4/6 mm FW
Transverse web stiffeners (BS EN 1993-1-5 section 9)

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma is for transverse web stiffeners other than at supports for webs without longitudinal stiffeners. No rules are given within BS EN 1993-2:2006 for the design of intermediate transverse stiffeners so reference has to be made to BS EN 1993-1-5 section 9.

---

Elevation on web without longitudinal stiffening

- Panel length 1: a(1) = 1967 mm
- Panel width: hw = 1000 mm
- Length of stiffener: ls = 1000 mm
- Thickness of web: tw = 14 mm
- Modulus of elasticity of web: E = 210000 N/mm²
- Stiffener corner snipe: snp = 15 mm

Web panels 1 & 2:

- Longitudinal stresses in web with transverse stiffeners
  - ls = 1000 mm
- Compression stresses are positive
- Stress at top of web plate (σtop) stop = 277 N/mm²
- Stress at btm of web plate (σbtm) sbtm = -242 N/mm²
Design parameters

Depth of stiffener         hs=200 mm
Stiffener thickness       ts=20 mm
Yield strength (web)      fyw=355 N/mm²
Yield strength (stiffener) fys=345 N/mm²

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Expression 9.3 in EN 1993-1-5 is further simplified by Table 2.6 in "Eurocode 3: Design of steel structures Part 1-5: Design of plated structures" ECCS Eurocode Design Manuals.

Yield strength to be adopted    fy=345 N/mm²
Strain value \( \varepsilon \) (EC3 Part 1-1)    \( \varepsilon_w = \sqrt{235/fyw} = 0.81362 \)
Strain value \( \varepsilon \) (EC3 Part 1-1)    \( \varepsilon_s = \sqrt{235/fys} = 0.82532 \)
Ratio (hs/ts)                   \( p = \frac{hs}{ts} = 10 \)
Limit to ratio \( p = \frac{hs}{ts} \) is 13\( \varepsilon \)=10.729.
Ratio (hs/ts)=10 is not greater than limit 13\( \varepsilon \)=10.729.
Stiffener complies with Clause 9.2.1(8).

Design shear resistance (double sided transverse stiffener)

The effective cross section of the intermediate double-sided transverse stiffener is as shown below:

2nd moment of area \( yy \) axis:
\[
I_{yy} = tsi * (2*hsi+tw)^3/12 + (30*ew*tw)*tw^3/12 \\
= 118.34E6 \text{ mm}^4
\]

Design shear resistance
\[
V_{brd} = \frac{\chi_i*Ae*fys1}{(\gamma_{M1}*1000)} \\
= 4097.4 \text{ kN}
\]

Axial force in the stiffener due to shear - BS EN 1993-1-5

Shear buckling coefficient for web panel \( k_t \) is derived from Clause 5.2.
Buckling coefficient (Annex A) \( k_t = 5.34 + 4*(hw/a)^2 = 6.3738 \)
Modified slenderness \( \lambda_w \) \( \lambda_w = hw/(37.4*tw*ew*\sqrt{kt}) = 0.92978 \)
Max design shear force in panel \( V_{Ed1} = 2511 \text{ kN} \)
Externally applied design load \( P_{ext1} = 0 \text{ kN} \)

Option 1:
Longitudinal compressive stress \( \sigma_{Ed} = 0 \) for symmetrical sections.
Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):
\[
P_{Ed1}' = V_{Ed1} - hw*tw*0.8*tcr/1000 = -144.65 \text{ kN}
\]
Option 2:  
Vertical force generated in the stiffener as per BS EN 1993-1-5 
(see NOTE, Clause 9.3.3(3)):  
\[ \text{PEd1} = \text{VEd1} - \text{fyw} \times \text{hw} \times \text{tw} / (\text{SQR}(3) \times \text{gamM1} \times \text{lamw}^2 \times 1000) = -506.47 \text{ kN} \]

Design axial load in stiffener  
\[ \text{PEd} = \text{PEd1} + \text{Pext1} = 0 \text{ kN} \]

As \( \text{PEd} \leq \text{VbRd} (0 \text{ kN} \leq 4097.4 \text{ kN}) \), the design shear force does not exceed the shear resistance. Hence OK.

### Connection to the web

- **Design shear load on weld**  
  \[ FwEd = \frac{\text{tw}^2}{2 \times (5 \times hs)} = 0.196 \text{ kN/mm} \]
- **Weld size required**  
  \[ sww = \frac{FwEd \times 10^3}{2 \times 0.7 \times \text{fvwd}} = 0.58042 \text{ mm} \]
- **Weld size provided**  
  \[ sw1 = 6 \text{ mm} \]

### Stiffness check - Clause 9.3.3(3)

- 2nd moment of area (effective section)  
  \[ I_{\text{pro}} = 118.34E6 \text{ mm}^4 \]
- 2nd moment of area required  
  \[ I_{\text{req}} = 0.75 \times \text{hw} \times \text{tw}^3 = 2.058E6 \text{ mm}^4 \]

As \( I_{\text{req}} \leq I_{\text{pro}} (2.058E6 \text{ mm}^4 \leq 118.34E6 \text{ mm}^4) \), the stiffener is OK.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Panel length</th>
<th>1967 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Panel width</td>
<td>1000 mm</td>
</tr>
<tr>
<td></td>
<td>Beam web thickness</td>
<td>14 mm</td>
</tr>
<tr>
<td></td>
<td>Depth of stiffener</td>
<td>200 mm</td>
</tr>
<tr>
<td></td>
<td>Stiffener thickness</td>
<td>20 mm</td>
</tr>
<tr>
<td></td>
<td>Design shear force</td>
<td>2511 kN</td>
</tr>
<tr>
<td></td>
<td>Axial load in stiffener</td>
<td>0 kN</td>
</tr>
<tr>
<td></td>
<td>Design shear resistance</td>
<td>4097.4 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>INTERMEDIATE STIFFENER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load carrying stiffener</td>
</tr>
<tr>
<td>2No/200 mm x 20 mm flats</td>
</tr>
<tr>
<td>Grade S 355 steel</td>
</tr>
<tr>
<td>connected by 4/6 mm FW</td>
</tr>
</tbody>
</table>
Transverse web stiffeners (BS EN 1993-1-5 section 9)

The proforma is in accordance with BS EN 1993-2:2006 "Eurocode 3 Design of Steel Structures - Part 2: Steel bridges" and the NA to BS EN 1993-2:2006.

The proforma is for transverse web stiffeners other than at supports for webs without longitudinal stiffeners. No rules are given within BS EN 1993-2:2006 for the design of intermediate transverse stiffeners so reference has to be made to BS EN 1993-1-5 section 9.

Elevation on web without longitudinal stiffening

Panel length 1  a(1)=1967 mm
Panel width  hw=1000 mm
Length of stiffener  ls=1000 mm
Thickness of web  tw=14 mm
Modulus of elasticity of web  E=210000 N/mm²
Stiffener corner snipe  snp=15 mm

Web panels 1 & 2:

Longitudinal stresses in web with transverse stiffeners

Stress at top of web plate (σtop)  σtop=277 N/mm²
Stress at btm of web plate (σbtm)  σbtm=-242 N/mm²
Design parameters

Depth of stiffener hs=200 mm
Stiffener thickness ts=20 mm
Yield strength (web) fyw=355 N/mm²
Yield strength (stiffener) fys=345 N/mm²

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Expression 9.3 in EN 1993-1-5 is further simplified by Table 2.6 in "Eurocode 3: Design of steel structures Part 1-5: Design of plated structures " ECCS Eurocode Design Manuals.

Yield strength to be adopted fy=345 N/mm²
Strain value ε (EC3 Part 1-1) ew=SQR(235/fyw)=0.81362
Strain value ε (EC3 Part 1-1) es=SQR(235/fys)=0.82532
Ratio (hs/ts) p=hs/ts=10
Limit to ratio p=(hs/ts) is 13ε=10.729.
Ratio (hs/ts)=10 is not greater than limit 13ε=10.729.
Stiffener complies with Clause 9.2.1(8).

Design shear resistance (double sided transverse stiffener)

The effective cross section of the intermediate double-sided transverse stiffener is as shown below:

2nd moment of area yy axis:
\[ I_{yy} = tsi \cdot (2 \cdot hsi + tw)^3 / 12 + (30 \cdot ew \cdot tw) \cdot tw^3 / 12 \]
\[ = 118.34 \text{E}6 \text{ mm}^4 \]

Design shear resistance
\[ V_{bRd} = \chi_{*} A_{e} \cdot f_{ysi} / (\gamma_{M1} \cdot 1000) \]
\[ = 4097.4 \text{ kN} \]

Axial force in the stiffener due to shear - BS EN 1993-1-5

Shear buckling coefficient for web panel kt is derived from Clause 5.2.
Buckling coefficient (Annex A) \[ k_{t} = 5.34 + 4 \cdot (hw/a)^2 \cdot 6.3738 \]
Modified slenderness λw \[ \lambda_{w} = hw / (37.4 \cdot tw \cdot ew \cdot SQR(kt)) = 0.92978 \]
Max design shear force in panel \[ V_{Ed1} = 2511 \text{ kN} \]
Externally applied design load \[ P_{ext1} = 0 \text{ kN} \]

Option 1:
Longitudinal compressive stress \( o_{xEd} = 0 \) for symmetrical sections.
Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):
\[ P_{Ed1}' = V_{Ed1} - hw \cdot tw \cdot 0.8 \cdot tcr / 1000 = -144.65 \text{ kN} \]
Option 2:

Vertical force generated in the stiffener as per BS EN 1993-1-5
(see NOTE, Clause 9.3.3(3)):

\[
PEd_1 = VEd_1 - f_{yw} \cdot h_w \cdot t_w / (SQR(3) \cdot \text{gamM1} \cdot \text{lamw}^2 \cdot 1000) = -506.47 \text{ kN}
\]

Design axial load in stiffener \( PE_d = PE_d_1 + P_{ext1} = 0 \text{ kN} \)

As \( PE_d \leq V_{bRd} \) (0 kN \( \leq 4097.4 \text{ kN} \)), the design shear force does not exceed the shear resistance. Hence OK.

**Connection to the web**

Design shear load on weld \( F_{wEd} = t_w^2 / (5 \cdot h_s) = 0.196 \text{ kN/mm} \)

Weld size required \( s_{ww} = F_{wEd} \cdot 10^3 / (2 \cdot 0.7 \cdot f_{vwd}) = 0.58042 \text{ mm} \)

Weld size provided \( s_{w1} = 6 \text{ mm} \)

**Stiffness check - Clause 9.3.3(3)**

2nd moment of area (effective section) \( I_{pro} = 118.34 \text{E6 mm}^4 \)

2nd moment of area required \( I_{req} = 0.75 \cdot h_w \cdot t_w^3 = 2.058 \text{E6 mm}^4 \)

As \( I_{req} \leq I_{pro} \) (2.058E6 mm\(^4\) \( \leq 118.34\text{E6 mm}^4 \)), the stiffener is OK.

**DESIGN**

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<td>4097.4 kN</td>
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</table>

**INTERMEDIATE**

**STIFFENER**

Load carrying stiffener

2No/200 mm x 20 mm flats

Grade S 355 steel

connected by 4/6 mm FW
Location:  Balustrade design with concentric base plate

Balustrade design with a concentric base plate

Calculations to BS5950

C = compression
T = tension

Yield strength     fy=275 N/mm²
Spacing between balusters     a=1.5 m
Unfactored udl at rail level     w=3 kN/m
Height of baluster     h=1.1 m
Base plate depth     D=400 mm
Base plate width     B=400 mm
Base plate thickness     t=15 mm
Distance of fixings from plate edge     edg=50 mm

DESIGN SUMMARY

(a)  Ultimate moment on baluster     Mu =7.92 kNm
(b)  2nd moment of area required     Ireq =157.91 cm⁴
(c)  Unfactored shear on fixings group     Q =4.5 kN
(d)  Base plate size  400 mm x 400 mm x 15 mm
(e)  Design strength for base plate     fy=275 N/mm²
(f)  Design strength for baluster     fy=275 N/mm²
(g)  Total unfactored tension on group of tension fixings     T=16.5 kN
Location: Balustrade design with non concentric base plate

Balustrade design with a non-concentric base plate

Calculations to BS5950

\[
\begin{align*}
C &= \text{compression} \\
T &= \text{tension}
\end{align*}
\]

Yield strength \( fy = 275 \text{ N/mm}^2 \)
Spacing between balusters \( a = 1.5 \text{ m} \)
Unfactored udl at rail level \( w = 3 \text{ kN/m} \)
Height of baluster \( h = 1.1 \text{ m} \)
Base plate depth \( D = 300 \text{ mm} \)
Base plate width \( B = 400 \text{ mm} \)
Base plate thickness \( t = 22 \text{ mm} \)

Distance of fixings from plate edge \( \text{edg} = 50 \text{ mm} \)

DESIGN SUMMARY

(a) Ultimate moment on baluster \( Mu = 7.92 \text{ kNm} \)
(b) 2nd moment of area required \( I_{req} = 157.91 \text{ cm}^4 \)
(c) Unfactored shear on fixings group \( Q = 4.5 \text{ kN} \)
(d) Base plate size \( 300 \text{ mm} \times 400 \text{ mm} \times 22 \text{ mm} \)
(e) Design strength for base plate \( fy = 275 \text{ N/mm}^2 \)
(f) Design strength for baluster \( fy = 275 \text{ N/mm}^2 \)
(g) Total unfactored tension on
   group of tension fixings \( T = 24.75 \text{ kN} \)
Location: Balustrade design with concentric base plate

Balustrade design with a concentric base plate

Calculations to EC3

\[
\begin{align*}
& C = \text{compression} \\
& T = \text{tension}
\end{align*}
\]

Yield strength (baluster & plate) \( f_y = 275 \text{ N/mm}^2 \)
Spacing between balusters \( a = 1.5 \text{ m} \)
Unfactored udl at rail level \( w = 3 \text{ kN/m} \)
Height of baluster \( h = 1.1 \text{ m} \)
Load factor for variable action \( \gamma_Q = 1.5 \)
Base plate depth \( D = 400 \text{ mm} \)
Base plate width \( B = 400 \text{ mm} \)
Base plate thickness \( t = 15 \text{ mm} \)
Distance of fixings from plate edge \( \text{edg} = 50 \text{ mm} \)

DESIGN SUMMARY

(a) Ultimate moment on baluster \( M_u = 7.425 \text{ kNm} \)
(b) 2nd moment of area required \( I_{req} = 157.91 \text{ cm}^4 \)
(c) Unfactored shear on fixings group \( Q = 4.5 \text{ kN} \)
(d) Base plate size \( 400 \text{ mm} \times 400 \text{ mm} \times 15 \text{ mm} \)
(e) Design strength for base plate \( f_y = 275 \text{ N/mm}^2 \)
(f) Design strength for baluster \( f_y = 275 \text{ N/mm}^2 \)
(g) Total unfactored tension on group of tension fixings \( T = 16.5 \text{ kN} \)
Location: Balustrade design with non concentric base plate

Balustrade design with a non-concentric base plate

Calculations to EC3

C = compression  
T = tension

Yield strength  \( fy = 275 \text{ N/mm}^2 \)
Spacing between balusters  \( a = 1.5 \text{ m} \)
Unfactored udl at rail level  \( w = 3 \text{ kN/m} \)
Height of baluster  \( h = 1.1 \text{ m} \)
Load factor for variable action  \( \gamma_{Q} = 1.5 \)
Base plate depth  \( D = 300 \text{ mm} \)
Base plate width  \( B = 400 \text{ mm} \)
Base plate thickness  \( t = 22 \text{ mm} \)

Distance of fixings from plate edge  \( edg = 50 \text{ mm} \)

DESIGN SUMMARY

(a) Ultimate moment on baluster  \( M_{u} = 7.425 \text{ kNm} \)
(b) 2nd moment of area required  \( I_{\text{req}} = 157.91 \text{ cm}^4 \)
(c) Unfactored shear on fixings group  \( Q = 4.5 \text{ kN} \)
(d) Base plate size  \( 300 \text{ mm} \times 400 \text{ mm} \times 22 \text{ mm} \)
(e) Design strength for base plate  \( fy = 275 \text{ N/mm}^2 \)
(f) Design strength for baluster  \( fy = 275 \text{ N/mm}^2 \)
(g) Total unfactored tension on group of tension fixings  \( T = 24.75 \text{ kN} \)
Location: Default 1

Steel runway beam design

Beam is designed in accordance with BS2853:1957 & BS 449.

Beam span L=3 m
Effective length of beam Le=3.6 m
Trolley capacity Wcap=1.5 tonnes
Number of wheels on trolley Nowheels=4
Load to be carried by trolley W=1.5 tonnes

Strength of steel - BS 449 Clause 3.a.(1)

Steel design grade (43 or 50) Grade=43
Maxm longitudinal bending stress F1max=162 N/mm²
Maxm transverse bending stress F2max=223 N/mm²
Maxm shear stress Fvmax=100 N/mm²

Cross flange bending

Transverse bending stresses
when trolley remote from end Ftr1=1.4*C*Wk*1E3*Dyn/(K1*T*T) =72.72 N/mm²
Transverse bending stresses when trolley at end of beam Ftr2=1.4*C*Wk*1E3*Dyn/(K2*T*T) =116.6 N/mm²

The actual transverse bending stress does not exceed the maximum permissible when load is remote from the end of the beam i.e Ftr1 ≤ F2max (72.72 N/mm² ≤ 223 N/mm²).
Trolley cannot approach end of beam and stress Ftr2 does not apply.

Combined stress check

Actual long'1 bending stress Fbc=41.19 N/mm²
Change back to Tonf/in² F1=Fbc/15.44=2.668 Tonf/in²
Allowable cross flange bending PK2=14.5 Tonf/in²
Convert to N/mm² PK2=PK2*15.44=223.9 N/mm²

The interaction of the longitudinal and transverse stresses at the centre of the span is satisfactory (i.e 72.72 N/mm² ≤ 223.9 N/mm²).
Check for deflection

\[
\text{Total deflection} \quad \text{DEL}=D_1+D_2=1.165 \text{ mm} \\
\text{Limiting deflection} \quad \text{DEL}_{\text{lim}}=L\times1E3/500=6 \text{ mm} \\
\text{Since } \text{DEL} \leq \text{DEL}_{\text{lim}} (1.165 \text{ mm} \leq 6 \text{ mm}), \text{ deflection is within limiting value.}
\]

**DESIGN SUMMARY**

Beam under consideration is simply supported.

- **Beam span is**: 3 m
- **Beam effective length**: 3.6 m
- **Trolley capacity**: 1.5 tonnes
- **Beam S.W.L.**: 1.5 tonnes
- **Hoist type**: Hand operated
- **Dynamic factor**: 1.1
- **Section size**: 254 x 102 x 28 Universal Beam

Section OK for long'l and transverse bending, shear and deflection.
Runway beam to be clearly marked for S.W.L. of 1.5 tonnes.
Location: Default 2

Steel runway beam design

Beam is designed in accordance with BS2853:1957 & BS 449.

Beam span \( L=1.5 \) m
Effective length of beam \( L_e=3 \) m
Trolley capacity \( W_{\text{cap}}=1.5 \) tonnes
Number of wheels on trolley \( N_{\text{wheels}}=4 \)
Load to be carried by trolley \( W=1.5 \) tonnes

Strength of steel - BS 449 Clause 3.a.(1)

Steel design grade (43 or 50) \( \text{Grade}=43 \)
Maxm longitudinal bending stress \( F_{1\text{max}}=162 \) N/mm²
Maxm transverse bending stress \( F_{2\text{max}}=223 \) N/mm²
Maxm shear stress \( F_{\text{vmax}}=100 \) N/mm²

Cross flange bending

Transverse bending stresses
when trolley remote from end \( F_{\text{tr1}}=1.4*C*W_k*1E3*\text{Dyn}/(K1*T*T) \)
\( =72.72 \) N/mm²
Transverse bending stresses
when trolley at end of beam \( F_{\text{tr2}}=1.4*C*W_k*1E3*\text{Dyn}/(K2*T*T) \)
\( =116.6 \) N/mm²

The actual transverse bending stress does not exceed the maximum permissible when load is remote from the end of the beam i.e \( F_{\text{tr1}} \leq F_{2\text{max}} \ (72.72 \text{ N/mm}² \leq 223 \text{ N/mm}²)\).
The actual transverse bending stress does not exceed the max. permissible when load is near end of beam (i.e. \( 116.6 \text{ N/mm}² \leq 223 \text{ N/mm}²\)).

Combined stress check

Beam is a cantilever and longitudinal and transverse stresses cannot be coincident, apart from a small bending moment caused by self weight of beam, so no check is required.
Check for deflection

Total deflection \[ \text{DEL}=D_1+D_2=2.279 \text{ mm} \]
Limiting deflection \[ \text{DELlim}=L\times10^3/250=6 \text{ mm} \]

Since \( \text{DEL} \leq \text{DELlim} \) (2.279 mm \( \leq \) 6 mm), deflection is within limiting value.

DESIGN SUMMARY

Beam under consideration is cantilevered.

- Beam span is 1.5 m
- Beam effective length 3 m
- Trolley capacity 1.5 tonnes
- Beam S.W.L. 1.5 tonnes
- Hoist type Hand operated
- Dynamic factor 1.1
- Section size 254 x 102 x 28 Universal Beam

Section OK for long’l and transverse bending, shear and deflection.
Runway beam to be clearly marked for S.W.L. of 1.5 tonnes.
Location: Default 3

Steel runway beam design

Beam span \( L = 3 \text{ m} \)
Effective length of beam \( L_e = 3.6 \text{ m} \)
Trolley capacity \( W_{\text{cap}} = 1.5 \text{ tonnes} \)
Number of wheels on trolley \( N_{\text{wheels}} = 2 \)
Load to be carried by trolley \( W = 1.5 \text{ tonnes} \)

Strength of steel - BS 449 Clause 3.a.(1)

Steel design grade (43 or 50) \( \text{Grade}=43 \)
Maxm longitudinal bending stress \( F_{1\text{max}} = 162 \text{ N/mm}^2 \)
Maxm transverse bending stress \( F_{2\text{max}} = 223 \text{ N/mm}^2 \)
Maxm shear stress \( F_{v\text{max}} = 100 \text{ N/mm}^2 \)

Cross flange bending

Transverse bending stresses
when trolley remote from end \( F_{t1} = 1.4 \times C \times W_k \times 10^3 \times \text{Dyn}/(K_1 \times T \times T) = 158.9 \text{ N/mm}^2 \)
Transverse bending stresses
when trolley at end of beam \( F_{t2} = 1.4 \times C \times W_k \times 10^3 \times \text{Dyn}/(K_2 \times T \times T) = 233.1 \text{ N/mm}^2 \)
The actual transverse bending stress does not exceed the maximum permissible when load is remote from the end of the beam
i.e \( F_{t1} \leq F_{2\text{max}} \) (158.9 N/mm\(^2\) \leq 223 N/mm\(^2\)).
Member overstressed due to cross flange bending when trolley near end of beam. Place stop ends well away from end of beam or strengthen beam by plating flanges.

Combined stress check

Actual long'\'l bending stress \( F_{bc} = 41.19 \text{ N/mm}^2 \)
Change back to Tonf/in\(^2\) \( F_1 = F_{bc}/15.44 = 2.668 \text{ Tonf/in}^2 \)
Allowable cross flange bending \( PK_2 = 14.5 \text{ Tonf/in}^2 \)
Convert to N/mm\(^2\) \( PK_2 = PK_2 \times 15.44 = 223.9 \text{ N/mm}^2 \)
The interaction of the longitudinal and transverse stresses at the centre of the span is satisfactory (i.e 158.9 N/mm\(^2\) \leq 223.9 N/mm\(^2\)).
Check for deflection

Total deflection \( \text{DEL} = D_1 + D_2 = 1.165 \text{ mm} \)
Limiting deflection \( \text{DEL}_{\text{lim}} = L \times 1 \times 10^3 / 500 = 6 \text{ mm} \)
Since \( \text{DEL} \leq \text{DEL}_{\text{lim}} \) (1.165 mm \( \leq \) 6 mm), deflection is within limiting value.

**DESIGN SUMMARY**

Beam under consideration is suspended.

- Beam span is 3 m
- Beam effective length 3.6 m
- Trolley capacity 1.5 tonnes
- Beam S.W.L. 1.5 tonnes
- Hoist type Hand operated
- Dynamic factor 1.1
- Section size 254 x 102 x 28 Universal Beam

Section OK for long'l and transverse bending, shear and deflection. Runway beam to be clearly marked for S.W.L. of 1.5 tonnes.

Member overstressed due to cross flange bending when trolley near end of beam. Place stop ends well away from the end of beam or strengthen flanges by plating.
Location: Ex1 - Simply supported runway beam

Steel runway beam design

Runway beam is designed in accordance with BS EN 1993-6:2007 and BS EN 1993-1-1:2005.

Beam span \( L = 3 \) m
Effective length of beam \( L_e = 3.6 \) m
Trolley capacity (unfactored) \( W_{cap} = 1.5 \) tonnes
Number of wheels on trolley \( N_{wheels} = 4 \)
Dynamic factor \( \phi = 1.1 \)

Dimensions (mm): \( h = 260.4 \) b=102.2 \( t_w = 6.3 \) tf=10 r=7.6
Properties (cm): \( I_y = 4010 \) \( I_z = 179 \) \( W_{ply} = 353 \) \( W_{plz} = 54.8 \) \( I_t = 9.57 \)
Partial factor (permanent action) \( \gamma_G = 1.35 \)
Partial factor (variable action) \( \gamma_Q = 1.5 \)

Resistance of bottom flange to wheel loads

(EC3 Part 6, Section 6.7)
Stress at mid-line of the flange due to the overall internal moment in the runway beam \( s_f E_d = \frac{M_{Ed} \times 10^3}{W_{ely}} = 61.56 \) N/mm²
Design resistance (bottom flange) \( F_{fRd} = \frac{L_{eff} \times t_f \times f_y \times \text{comp1}}{(4 \times m \times 1000)} = 108.34 \) kN
Vertical crane wheel load \( F_{zEd} = \frac{\gamma_Q \times W_k \times \phi}{N_{wheels}} = 6.1792 \) kN
As \( F_{zEd} \leq F_{fRd} \) (6.1792 \( kN \leq 108.34 \) \( kN \)), the design resistance of the bottom flange is satisfactory.

Lateral torsional buckling

Effective length of beam \( L_e = 3.6 \) m
Critical moment \( M_{cr} = C_1 \times E_t \times S_r / 10^6 = 80.026 \) kNm
Design buckling resistance moment \( M_{bRd} = \chi_{LT} \times W_{ply} \times f_y / (\gamma_M \times 10^3) = 59.642 \) kNm
Unity factor \( \text{unit}_b = \frac{M_{Ed}}{M_{bRd}} = 0.31789 \)
Section chosen is considered suitable.
Local bending stresses in the bottom flange due to wheel loads

Coefficient $C_x^0 = 0.05 - 0.58 \times \mu + 0.148 \times \exp(3.015 \times \mu) = 0.66321$
Coefficient $C_x^1 = 2.23 - 1.49 \times \mu + 1.39 \times \exp(-18.33 \times \mu) = 1.2978$
Coefficient $C_x^2 = 0.73 - 1.58 \times \mu + 2.91 \times \exp(-6.33 \times \mu) = -0.20308$
Coefficient $C_y^0 = -2.11 + 1.977 \times \mu + 0.0076 \times \exp(6.530 \times \mu) = -0.4211$
Coefficient $C_y^1 = 10.108 - 7.408 \times \mu - 10.108 \times \exp(-1.364 \times \mu) = 1.1675$
Coefficient $C_y^2 = 0$

Serviceability limit state stress check - EC3 Part 6, Section 7.5

Global shear stress $v_{Ed} = Vz_{Ed} \times 10^3 / (h \times tw) = 15.41 \text{ N/mm}^2$
Shear resistance (exp. 7.2b) $v_{Rd} = fy / (\sqrt{3} \times \gamma_{M1}) = 144.34 \text{ N/mm}^2$
As $v_{Ed} \leq v_{Rd}$ ($15.41 \text{ N/mm}^2 \leq 144.34 \text{ N/mm}^2$), shear stress OK.

Hence expression 7.2e is satisfied.

Combined bending and torsion check - EC3 Part 6, Annex A

Factored horizontal force $WH_1 = 2.247 \text{ kN}$
Total minor axis design moment $M_{zEd} = M_{zEd1} + M_{zEd2} = 1.9106 \text{ kNm}$
Design warping torsional moment $M_{wEd} = 0.44101 \text{ kNm}$
Minor axis resistance moment $M_{zRd} = fy \times Welz / 10^3 = 9.6331 \text{ kNm}$
Warping torsional resist. moment $M_{wRd} = M_{zRd} / 2 = 4.8165 \text{ kNm}$

\[
\begin{align*}
\frac{My_{Ed}}{Mb_{Rd}} &+ \frac{Cmz.Mz_{Ed}}{Mz_{Rd}} + \frac{kw.kzw.ka.Mw_{Ed}}{Mw_{Rd}/\gamma_{M1}} & \leq 1
\end{align*}
\]

Unity factor $uf = \frac{My_{Ed}}{Mb_{Rd}} + \frac{Cmz.Mz_{Ed}}{Mz_{Rd}} + \frac{kw.kzw.ka.Mw_{Ed}}{Mw_{Rd}/\gamma_{M1}} = 0.56197$
\leq 1

Hence the unity relationship is satisfied.

Check for deflection - EC6 Part 6, Section 7.3

Vertical deflection (payload) $DEL1 = D2 = 1.1007 \text{ mm}$
Vertical deflection (total load) $DEL2 = D1 + D2 = 1.1355 \text{ mm}$
Since $DEL1 \leq DEL11$ ($1.1007 \text{ mm} \leq 6 \text{ mm}$), deflection OK.
Since $DEL2 \leq DEL12$ ($1.1355 \text{ mm} \leq 5 \text{ mm}$), deflection OK.

Vibration of bottom flange

The slenderness ratio of the bottom flange $L/iz$ should be limited to 250, where $iz$ is the radius of gyration of the bottom flange and $L$ is the distance between lateral restraints. For simplicity, the inertia of the bottom flange will be taken as half of $Iz$.
Bottom flange inertia $Izf = I_z / 2 \times 1E4 = 895000 \text{ mm}^4$
Radius of gyration of the flange $iz = \sqrt{Izf / (b \times tf)} = 29.593 \text{ mm}$
Slenderness of bottom flange $sbf = L \times 1E3 / iz = 101.38$
As slenderness of bottom flange is $\leq 250$ vibration is satisfactory.
## DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section size</td>
<td>260 x 102 x 28 UKB</td>
</tr>
<tr>
<td>Beam actual span</td>
<td>3 m</td>
</tr>
<tr>
<td>Beam effective length</td>
<td>3.6 m</td>
</tr>
<tr>
<td>Trolley capacity</td>
<td>14.7 kN</td>
</tr>
<tr>
<td>Beam S.W.L.</td>
<td>14.7 kN</td>
</tr>
<tr>
<td>Hoist type</td>
<td>Hand operated.</td>
</tr>
<tr>
<td>Dynamic factor</td>
<td>1.1</td>
</tr>
<tr>
<td>Total load vertical deflection</td>
<td>1.03 mm (without dynamic effects)</td>
</tr>
<tr>
<td>Total load limiting deflection</td>
<td>5 mm</td>
</tr>
<tr>
<td>Bending and torsion interaction</td>
<td>0.562 ≤ 1</td>
</tr>
</tbody>
</table>

**NOTE:** The runway beam under consideration is simply supported. Longitudinal & transverse bending, shear and deflection are OK. Runway beam to be clearly marked for S.W.L. of 14.7 kN.
Location: Ex2 - Cantilever runway beam

Steel runway beam design

| / | / | Runway beam is designed |
| / | / | in accordance with |
| / | / | BS EN 1993-6:2007 and |

Beam span $L=1.5$ m
Effective length of beam $L_e=3$ m
Trolley capacity (unfactored) $W_{cap}=1.5$ tonnes
Number of wheels on trolley $N_{wheels}=4$
Dynamic factor $\phi=1.1$

Dimensions (mm): $h=260.4$ $b=102.2$ $t_w=6.3$ $t_f=10$ $r=7.6$
Properties (cm): $I_y=4010$ $I_z=179$ $W_{ply}=353$ $W_{plz}=54.8$ $I_t=9.57$
Partial factor (permanent action) $\gamma_G=1.35$
Partial factor (variable action) $\gamma_Q=1.5$

Resistance of bottom flange to wheel loads

(EC3 Part 6, Section 6.7)
Stress at mid-line of the flange due to the overall internal moment in the runway beam
$\sigma_{Ed}=\frac{M_{Ed} \times 10^3}{W_{ely}}=121.75$ N/mm²
Distance from LHS support to load $x_L=1.35$ m
Design resistance (bottom flange) $F_{fRd}=\frac{\text{leff} \times t_f \times f_y \times \text{comp}1}{(4 \times \text{m} \times 1000)}$
$=38.995$ kN
Vertical crane wheel load $F_{zEd}=\frac{\gamma_Q \times W_k \times \phi \times N_{wheels}}{4.1}$
As $F_{zEd} \leq F_{fRd}$ ($6.1792$ kN $\leq 38.995$ kN), the design resistance of the bottom flange is satisfactory.

Lateral torsional buckling

Effective length of beam $L_e=3$ m
Critical moment $M_{cr}=\frac{C_1 \times E \times Sr}{10^6}=103.49$ kNm
Design buckling resistance moment $M_{bRd}=\frac{\chi_{LT} \times W_{ply} \times f_y}{(4 \times \gamma_m \times 10^3)}$
$=68.427$ kNm
Unity factor $\text{unitb}=\frac{M_{Ed}}{M_{bRd}}=0.54799$
Section chosen is considered suitable.
Local bending stresses in the bottom flange due to wheel loads

Coefficient Cx0=0.05-0.58*mu+0.148*EXP(3.015*mu) =0.66321
Coefficient Cx1=2.23-1.49*mu+1.39*EXP(-18.33*mu) =1.2978
Coefficient Cx2=0.73-1.58*mu+2.91*EXP(-6.33*mu) =-0.20308
Coefficient Cy0=-2.11+1.977*mu+0.0076*EXP(6.530*mu) =-0.4211
Coefficient Cy1=10.108-7.408*mu-10.108*EXP(-1.364*mu) =1.1675
Coefficient Cy2=0 =0

Serviceability limit state stress check - EC3 Part 6, Section 7.5

Global shear stress vEd=VzEd*10^3/(h*tw)=15.41 N/mm²
Shear resistance (exp. 7.2b) vRd=fy/(SQR(3)*gamMs)=144.34 N/mm²
As vEd ≤ vRd ( 15.41 N/mm² ≤ 144.34 N/mm² ), shear stress OK.
Hence expression 7.2e is satisfied.

Combined bending and torsion check - EC3 Part 6, Annex A

Factored horizontal force WH1=2.247 kN
Total minor axis design moment MzEd=MzEd1+MzEd2=3.0462 kNm
Design warping torsional moment MwEd=0.13503 kNm
Minor axis resistance moment MzRd=fy*Welz/10^3=9.6331 kNm
Warping torsional resist. moment MwRd=MzRd/2=4.8165 kNm

\[
\frac{MyEd}{MbRd} + \frac{Cmz.MzEd}{MzRd} + \frac{kw.kzw.ka.MwEd}{MwRd/gamM1} \leq 1
\]

Unity factor uf=MyEd/MbRd+Cmz*MzEd/MzRd+kw*kzw*ka*MwEd/MwRd=0.85347 ≤ 1
Hence the unity relationship is satisfied.

Check for deflection - EC6 Part 6, Section 7.3

Vertical deflection (payload) DEL1=D2=2.2014 mm
Vertical deflection (total load) DEL2=D1+D2=2.2222 mm
Since DEL1 ≤ DEL11 ( 2.2014 mm ≤ 6 mm ), deflection OK.
Since DEL2 ≤ DEL12 ( 2.2222 mm ≤ 5 mm ), deflection OK.

Vibration of bottom flange

The slenderness ratio of the bottom flange L/iz should be limited to 250, where iz is the radius of gyration of the bottom flange and L is the distance between lateral restraints. For simplicity, the inertia of the bottom flange will be taken as half of Iz.
Bottom flange inertia Izf=Iz/b*1E4=895000 mm²
Radius of gyration of the flange iz=SQR(Izf/(b²tf))=29.593 mm
Slenderness of bottom flange sbf=L*1E3/iz=50.688
As slenderness of bottom flange is ≤ 250 vibration is satisfactory.
### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section size</td>
<td>260 x 102 x 28 UKB</td>
</tr>
<tr>
<td>Beam actual span</td>
<td>1.5 m</td>
</tr>
<tr>
<td>Beam effective length</td>
<td>3 m</td>
</tr>
<tr>
<td>Trolley capacity</td>
<td>14.7 kN</td>
</tr>
<tr>
<td>Beam S.W.L.</td>
<td>14.7 kN</td>
</tr>
<tr>
<td>Hoist type</td>
<td>Hand operated.</td>
</tr>
<tr>
<td>Dynamic factor</td>
<td>1.1</td>
</tr>
<tr>
<td>Total load vertical deflection</td>
<td>2.02 mm (without dynamic effects)</td>
</tr>
<tr>
<td>Total load limiting deflection</td>
<td>2.22 mm (with dynamic effects)</td>
</tr>
<tr>
<td>Total load limiting deflection</td>
<td>5 mm</td>
</tr>
<tr>
<td>Bending and torsion interaction</td>
<td>0.853 ≤ 1</td>
</tr>
</tbody>
</table>

**NOTE:** The runway beam under consideration is cantilevered. Longitudinal & transverse bending, shear and deflection are OK. Runway beam to be clearly marked for S.W.L. of 14.7 kN.
**Location: Ex3 - Suspended runway beam**

**Steel runway beam design**

Runway beam is designed in accordance with BS EN 1993-6:2007 and BS EN 1993-1-1:2005.

<table>
<thead>
<tr>
<th>Beam span</th>
<th>L=3 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective length of beam</td>
<td>Le=3.6 m</td>
</tr>
<tr>
<td>Trolley capacity (unfactored)</td>
<td>Wcap=1.5 tonnes</td>
</tr>
<tr>
<td>Number of wheels on trolley</td>
<td>Nowheels=2</td>
</tr>
<tr>
<td>Dynamic factor φ</td>
<td>Phi=1.1</td>
</tr>
</tbody>
</table>

254 x 102 x 28 UKB

**Resistance of bottom flange to wheel loads**

(EC3 Part 6, Section 6.7)

Stress at mid-line of the flange due to the overall internal moment in the runway beam

\[ sfEd = \frac{MyEd \times 10^3}{Wely} = 61.56 \text{ N/mm}^2 \]

Design resistance (bottom flange)

\[ FfRd = \frac{leff \times tf^2 \times fy \times \text{compl}}{4 \times m \times 1000} = 108.34 \text{ kN} \]

Vertical crane wheel load

\[ FzEd = \frac{\text{gamQ} \times Wk \times Phi}{\text{Nowheels}} = 12.358 \text{ kN} \]

As \( FzEd \leq FfRd \) (12.358 kN ≤ 108.34 kN), the design resistance of the bottom flange is satisfactory.

**Lateral torsional buckling**

<table>
<thead>
<tr>
<th>Effective length of beam</th>
<th>Le=3.6 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Critical moment</td>
<td>Mcr=C1<em>Et</em>Sr/10^6=80.026 kNm</td>
</tr>
<tr>
<td>Design buckling resistance moment</td>
<td>MbRd=ChiLT<em>Wply</em>fy/(gamM1*10^3) =59.642 kNm</td>
</tr>
</tbody>
</table>

Unity factor

\[ \text{unitb} = \frac{MyEd}{MbRd} = 0.31789 \]

Section chosen is considered suitable.
Local bending stresses in the bottom flange due to wheel loads

Coefficient $C_{x0} = 0.05 - 0.58 \mu + 0.148 \exp(3.015 \mu) = 0.66321$
Coefficient $C_{x1} = 2.23 - 1.49 \mu + 1.39 \exp(-18.33 \mu) = 1.2978$
Coefficient $C_{x2} = 0.73 - 1.58 \mu + 2.91 \exp(-6.33 \mu) = -0.20308$
Coefficient $C_{y0} = -2.11 + 1.977 \mu + 0.0076 \exp(6.530 \mu) = -0.4211$
Coefficient $C_{y1} = 10.108 - 7.408 \mu - 10.108 \exp(-1.364 \mu) = 1.1675$
Coefficient $C_{y2} = 0$

Serviceability limit state stress check - EC3 Part 6, Section 7.5

Global shear stress $\sigma_{Ed} = V_{zEd} \times 10^3 / (h \times t_w) = 15.41 \text{ N/mm}^2$
Shear resistance (exp. 7.2b) $\sigma_{Rd} = f_y / (\sqrt{3} \times \gamma_{Ms}) = 144.34 \text{ N/mm}^2$
As $\sigma_{Ed} \leq \sigma_{Rd}$ (15.41 N/mm ≤ 144.34 N/mm²), shear stress OK.

Combined bending and torsion check - EC3 Part 6, Annex A

Factored horizontal force $W_{H1} = 2.247 \text{ kN}$
Total minor axis design moment $M_{zEd} = M_{zEd1} + M_{zEd2} = 1.9106 \text{ kNm}$
Design warping torsional moment $M_{wEd} = 0.44101 \text{ kNm}$
Minor axis resistance moment $M_{zRd} = f_y \times W_{elz} / 10^3 = 9.6331 \text{ kNm}$
Warping torsional resist. moment $M_{wRd} = M_{zRd} / 2 = 4.8165 \text{ kNm}$

$$ \frac{M_{yEd}}{M_{wRd}} + \frac{C_{mz} \cdot M_{zEd}}{M_{zRd}} + \frac{kw \cdot kzw \cdot ka \cdot M_{wEd}}{M_{wRd} \times \gamma_{M1}} \leq 1 $$

Unity factor $uf = \frac{M_{yEd}}{M_{wRd}} + C_{mz} \cdot M_{zEd} / M_{zRd} + kw \cdot kzw \cdot ka \cdot M_{wEd} / M_{wRd} = 0.56197 \leq 1$
Hence the unity relationship is satisfied.

Check for deflection - EC6 Part 6, Section 7.3

Vertical deflection (payload) $D_2 = 1.1007 \text{ mm}$
Vertical deflection (total load) $D_2 = D_1 + D_2 = 1.1355 \text{ mm}$
Since $D_1 \leq D_2$ (1.1007 mm ≤ 6 mm), deflection OK.
Since $D_2 \leq D_{EL2}$ (1.1355 mm ≤ 5 mm), deflection OK.

Vibration of bottom flange

The slenderness ratio of the bottom flange $L / iz$ should be limited to 250, where $iz$ is the radius of gyration of the bottom flange and $L$ is the distance between lateral restraints. For simplicity, the inertia of the bottom flange will be taken as half of $Iz$.
Bottom flange inertia $Izf = Iz / 2 \times 1E4 = 895000 \text{ mm}^4$
Radius of gyration of the flange $iz = \sqrt{Izf / (b \times tf)} = 29.593 \text{ mm}$
Slenderness of bottom flange $sbf = L \times 1E3 / iz = 101.38$
As slenderness of bottom flange is ≤ 250 vibration is satisfactory.
DESIGN SUMMARY

Section size 260 x 102 x 28 UKB
Beam actual span 3 m
Beam effective length 3.6 m
Trolley capacity 14.7 kN
Beam S.W.L. 14.7 kN
Hoist type Hand operated.
Dynamic factor 1.1
Total load vertical deflection 1.03 mm (without dynamic effects)
Total load vertical deflection 1.14 mm (with dynamic effects)
Total load limiting deflection 5 mm
Bending and torsion interaction 0.562 ≤ 1

NOTE: The runway beam under consideration is suspended. Longitudinal & transverse bending, shear and deflection are OK. Runway beam to be clearly marked for S.W.L. of 14.7 kN.
Location: Default example

Stainless steel CHS

Calculations in accordance with BS5950-1:2000 and SCI publication "Structural Design of Stainless Steel"

All loads and moments are factored

Factored moment about z-z axis: Mz = 5.04 kNm
Factored SF in y direction: Fv = 5.6 kN
Axial load (comp. positive): F = 298 kN
Length of member: L = 3.6 m
Factored moment about axis yy: My = 0 kNm
168.3 O.D x 5 Circular Hollow Section
Young's Modulus: E = 200 kN/mm²

Length between restraints z axis: Lz = 3600 mm
Length between restraints y axis: Ly = 1800 mm
Quarter point: m2 = 0 kNm
Mid-span: m3 = 5.04 kNm
Three quarter point: m4 = 0 kNm
Quarter point: mz2 = 0 kNm
Mid-span: mz3 = 5.04 kNm
Three quarter point: mz4 = 0 kNm
Maximum moment on central half: M24 = 5.04 kNm

CIRCULAR HOLLOW SECTION & DESIGN
SUMMARY

168.3 mm O.D. x 5 mm CHS Grade 1.4404
Section is satisfactory for bending axial load, local capacity and overall buckling check.
Axial load: 298 kN
Compression resistance: 508.99 kN
Maximum moment z-axis: 5.04 kNm
Moment capacity: 29.343 kNm
Local capacity check: 0.69983 ≤ 1
Overall buckling checks: 0.76567 ≤ 1
0.70571 ≤ 1
Location: Tensile load and moment about one axis

Stainless steel CHS

Calculations in accordance with BS5950-1:2000 and SCI publication "Structural Design of Stainless Steel"

All loads and moments are factored

Factored moment about z-z axis $M_z=5.04$ kNm
Factored SF in y direction $F_v=5.6$ kN
Axial load (comp. positive) $F=-400$ kN
Length of member $L=3.6$ m
Factored moment about axis yy $M_y=0$ kNm

TENSILE LOAD APPLIED
168.3 O.D x 5 Circular Hollow Section
Young's Modulus $E=200$ kN/mm$^2$

Estimated net area (tension) $A_e=25.6$ cm$^2$

CIRCULAR HOLLOW SECTION & DESIGN

SUMMARY

168.3 mm O.D. x 5 mm CHS Grade 1.4404
Section is satisfactory for bending axial load, local capacity and overall buckling check.
Axial load $-400$ kN
Tensile capacity $563.2$ kN
Maximum moment z-axis $5.04$ kNm
Moment capacity $29.343$ kNm
Local capacity check $0.88199 \leq 1$
Location: Biaxial bending

Stainless steel CHS

Calculations in accordance with BS5950-1:2000 and SCI publication "Structural Design of Stainless Steel"

All loads and moments are factored

Factored moment about z-z axis \( M_z = 47 \text{ kNm} \)
Factored SF in y direction \( F_y = 46 \text{ kN} \)
Axial load (comp. positive) \( F = 48 \text{ kN} \)
Length of member \( L = 6 \text{ m} \)
Factored moment about axis yy \( M_y = 18 \text{ kNm} \)
219.1 O.D x 5 Circular Hollow Section
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)

Length between restraints z axis \( L_z = 6000 \text{ mm} \)
Length between restraints y axis \( L_y = 6000 \text{ mm} \)
Far end BM \( \beta M = 0 \text{ kNm} \)
Maximum moment on segment \( M_z = 47 \text{ kNm} \)
Far end BM \( \beta M_z = 0 \text{ kNm} \)
Far end BM \( \beta M_y = 0 \text{ kNm} \)
Far end BM \( \beta y = 0 \text{ kNm} \)

CIRCULAR HOLLOW SECTION & DESIGN

SUMMARY

219.1 mm O.D. x 5 mm CHS Grade 1.4362
Section is satisfactory for bending axial load, local capacity and overall buckling check.
Axial load \( 48 \text{ kN} \)
Compression resistance \( 825.94 \text{ kN} \)
Maximum moment z-axis \( 47 \text{ kNm} \)
Moment capacity \( 70.399 \text{ kNm} \)
Maximum moment y-axis \( 18 \text{ kNm} \)
Moment capacity \( 70.399 \text{ kNm} \)
Local capacity check \( 0.95899 \leq 1 \)
Overall buckling checks \( 0.6121 \leq 1 \)
**Location: Ex1 - Default example**

![Diagram of Stainless steel CHS]

**Stainless steel CHS**

Calculations in accordance with the SCI publication entitled "Design Manual For Structural Stainless Steel" (Third Edition) 18 April 2006. All loads and moments are factored.

- **Design moment about y-y axis** $M_y = 5.04$ kNm
- **Design shear force in z-direction** $V_z = 5.6$ kN
- **Design axial load (comp.positive)** $N_E = 298$ kN
- **Length of member** $L = 3.6$ m
- **Design moment about z-z axis** $M_z = 0$ kNm
- **168.3 x 5 thick CHS**
- **Young's Modulus** $E = 200$ kN/mm²
- **Length between restraints yy axis** $L_y = 3600$ mm
- **Length between restraints zz axis** $L_z = 1800$ mm

**CIRCULAR HOLLOW SECTION 168.3 mm x 5 mm CHS Grade 1.4404**

**DESIGN SUMMARY**

Section is satisfactory for bending axial load, local capacity and overall buckling check.

- **Design axial load** $298$ kN
- **Compression resistance** $462.72$ kN
- **Maximum moment y-y axis** $5.04$ kNm
- **Moment resistance** $26.675$ kNm
- **Overall buckling checks** $0.87075 \leq 1$
  
  $0.83296 \leq 1$

**SCALE 5.48**

Office 1007

Proforma 384
Location: Ex2 - Tensile load and moment about one axis

Stainless steel CHS

Calculations in accordance with the SCI publication entitled "Design Manual For Structural Stainless Steel" (Third Edition) 18 April 2006. All loads and moments are factored.

Design moment about y-y axis \( \text{MyEd} = 5.04 \text{ kNm} \)
Design shear force in z-direction \( \text{VzEd} = 5.6 \text{ kN} \)
Design axial load (comp.positive) \( \text{NEd} = -400 \text{ kN} \)
Length of member \( L = 3.6 \text{ m} \)
Design moment about z-z axis \( \text{MzEd} = 0 \text{ kNm} \)
168.3 x 5 thick CHS
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)
Estimated net area (tension) \( A_e = 25.6 \text{ cm}^2 \)

CIRCULAR HOLLOW SECTION

Design Summary

Section is satisfactory for bending axial load, local capacity and overall buckling check.
Design axial load \(-400 \text{ kN}\)
Tensile resistance \(512 \text{ kN}\)
Maximum moment y-y axis \(5.04 \text{ kNm}\)
Moment resistance \(26.675 \text{ kNm}\)
Unity factor \(0.97019 \leq 1\)
Location: Ex3 - Biaxial bending

Stainless steel CHS

Calculations in accordance with the SCI publication entitled "Design Manual For Structural Stainless Steel" (Third Edition) 18 April 2006. All loads and moments are factored.

Design moment about y-y axis \( M_{yEd} = 47 \text{ kNm} \)
Design shear force in z-direction \( V_{zEd} = 46 \text{ kN} \)
Design axial load (comp. positive) \( N_{Ed} = 48 \text{ kN} \)
Length of member \( L = 6 \text{ m} \)
Design moment about z-z axis \( M_{zEd} = 2 \text{ kNm} \)
219.1 x 5 thick CHS
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)
Length between restraints yy axis \( L_y = 6000 \text{ mm} \)
Length between restraints zz axis \( L_z = 6000 \text{ mm} \)

WARNING:
As \( D/t > 90e^2 \ (43.789) \), the section is Class 4 slender. With this class section, local buckling will occur before the attainment of yield stress in one or more parts of the cross-section and this is beyond the scope of this proforma.
Location: Ex4 - Axial load only

Stainless steel CHS

Calculations in accordance with the SCI publication entitled "Design Manual For Structural Stainless Steel" (Third Edition) 18 April 2006. All loads and moments are factored.

Design moment about y-y axis \( M_{yEd} = 0 \) kNm
Design shear force in z-direction \( V_{zEd} = 0 \) kN
Design axial load (comp.positive) \( N_{Ed} = 250 \) kN
Length of member \( L = 3.5 \) m
Design moment about z-z axis \( M_{zEd} = 0 \) kNm
159 x 4 thick CHS
Young's Modulus \( E = 200 \) kN/mm²
Length between restraints yy axis \( L_y = 3500 \) mm
Length between restraints zz axis \( L_z = 3500 \) mm

CIRCULAR HOLLOW SECTION 159 mm x 4 mm CHS Grade 1.4401
SECTION
Section is satisfactory for load
Design axial load 250 kN
Compression resistance 320.75 kN
**Location:** Example 6 'Structural Design of Stainless Steel'

### Stainless Steel Section

**Structural Hollow Section Design**

Calculations in accordance with BS5950-1:2000 and SCI publication 'Structural Design of Stainless steel'

All loads and moments are factored.

Factored bending moment axis $zz$ $M_z=2.74 \text{ kNm}$

Factored SF in $y$ direction $F_v=1 \text{ kN}$

Axial load (comp. positive) $F=19.6 \text{ kN}$

Length of member $L=2.7 \text{ m}$

Factored bending moment axis $y-y$ $M_y=0 \text{ kNm}$

100 x 50 x 6 RHS

Length between restraints $z$ axis $L_z=2700 \text{ mm}$

Length between restraints $y$ axis $L_y=2700 \text{ mm}$

Young's Modulus $E=200 \text{ kN/mm}^2$

Far end BM $\beta M=0 \text{ kNm}$

Maximum moment on segment $M_z=2.74 \text{ kNm}$

Far end BM $\beta M_z=0 \text{ kNm}$

Dist. betw. torsional restraints $L_T=2.7 \text{ m}$

**SQUARE HOLLOW SECTION** 100 x 50 x 6 SHS Grade S 1.4401

**SECTION**

Section is satisfactory for axial load, and overall buckling check.

**Axial load** 19.6 kN

**Compression resistance** 109.03 kN

**Maximum moment $z$ axis** 2.74 kNm

**Moment capacity** 9.625 kNm

**Local capacity check** $0.34407 \leq 1$

**Overall buckling checks** $0.40913 \leq 1$

$0.35057 \leq 1$
Location: Slender section bending about major axis only

Stainless Steel Section

Structural Hollow Section Design

Calculations in accordance with BS5950-1:2000 and SCI publication 'Structural Design of Stainless steel'

All loads and moments are factored.

Factored bending moment axis zz $M_z = 77.4\, \text{kNm}$
Factored SF in y direction $F_v = 18\, \text{kN}$
Axial load (comp. positive) $F = 1100\, \text{kN}$
Length of member $L = 4\, \text{m}$
Factored bending moment axis y-y $M_y = 0\, \text{kNm}$
400 x 200 x 6 RHS
Length between restraints z axis $L_z = 4000\, \text{mm}$
Length between restraints y axis $L_y = 4000\, \text{mm}$
Young's Modulus $E = 200\, \text{kN/mm}^2$
Dist. betwn torsional restraints $L_T = 4\, \text{m}$

SQUARE HOLLOW SECTION 400 x 200 x 6 SHS Grade S 1.4462
SECTION
SUMMARY
Section is satisfactory for axial load, and overall buckling check.
Axial load $1100\, \text{kN}$
Compression resistance $1808.9\, \text{kN}$
Maximum moment z axis $77.4\, \text{kNm}$
Moment capacity $293.42\, \text{kNm}$
Local capacity check $0.8324 \leq 1$
Overall buckling checks $0.89674 \leq 1$ $0.89674 \leq 1$
Location: Slender section bending about minor axis only

Stainless Steel Section

Structural Hollow Section Design

Calculations in accordance with BS5950-1:2000 and SCI publication 'Structural Design of Stainless steel'

All loads and moments are factored.

Factored bending moment axis $zz$ $M_z=0$ kNm
Factored SF in $y$ direction $F_v=18$ kN
Axial load (comp. positive) $F=900$ kN
Length of member $L=4$ m
Factored bending moment axis $y-y$ $M_y=24$ kNm
400 x 200 x 6 RHS
Length between restraints $z$ axis $L_z=4000$ mm
Length between restraints $y$ axis $L_y=4000$ mm
Young's Modulus $E=200$ kN/mm²

SECTION

SQUARE HOLLOW SECTION

SUMMARY

Section is satisfactory for axial load, and overall buckling check.

Axial load $900$ kN
Compression resistance $2125.1$ kN
Maximum moment $y$ axis $24$ kNm
Moment capacity $134.75$ kNm
Local capacity check $0.68572 \leq 1$
Location: bi-axial bending - slender z-z axis

Stainless Steel Section

Structural Hollow Section Design

Calculations in accordance with BS5950-1:2000 and SCI publication 'Structural Design of Stainless steel'

All loads and moments are factored.

Factored bending moment axis zz \( M_z = 56 \, \text{kNm} \)
Factored SF in y direction \( F_v = 8 \, \text{kN} \)
Axial load (comp. positive) \( F = 800 \, \text{kN} \)
Length of member \( L = 4 \, \text{m} \)
Factored bending moment axis y-y \( M_y = 20 \, \text{kNm} \)
400 x 200 x 6 RHS
Length between restraints z axis \( L_z = 4000 \, \text{mm} \)
Length between restraints y axis \( L_y = 4000 \, \text{mm} \)
Young's Modulus \( E = 200 \, \text{kN/mm}^2 \)
Dist. betwn torsional restraints \( L_T = 4 \, \text{m} \)

SECTION

400 x 200 x 6 SHS Grade S 1.4462

SUMMARY

Section is satisfactory for axial load, and overall buckling check.

Axial load \( 800 \, \text{kN} \)
Compression resistance \( 1760.5 \, \text{kN} \)
Maximum moment z axis \( 56 \, \text{kNm} \)
Moment capacity \( 273.65 \, \text{kNm} \)
Maximum moment y axis \( 20 \, \text{kNm} \)
Moment capacity \( 129.29 \, \text{kNm} \)
Local capacity check \( 0.79771 \leq 1 \)
Overall buckling checks \( 0.84289 \leq 1 \)
Location: Bi-axial bending - slender section both axes

Stainless Steel Section

Calculations in accordance with BS5950-1:2000 and SCI publication 'Structural Design of Stainless steel'

All loads and moments are factored.

Factored bending moment axis zz \( M_z = 110 \text{ kNm} \)
Factored SF in y direction \( F_v = 21 \text{ kN} \)
Axial load (comp. positive) \( F = 20 \text{ kN} \)
Length of member \( L = 4 \text{ m} \)
Factored bending moment axis y-y \( M_y = 100 \text{ kNm} \)

400 x 200 x 6 RHS
Length between restraints z axis \( L_z = 4000 \text{ mm} \)
Length between restraints y axis \( L_y = 4000 \text{ mm} \)
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)
Far end BM \( \beta M = -55 \text{ kNm} \)
Maximum moment on segment \( M_z = 110 \text{ kNm} \)
Far end BM \( \beta M_z = -55 \text{ kNm} \)
Maximum moment on segment \( M_y = 100 \text{ kNm} \)
Far end BM \( \beta M_y = -50 \text{ kNm} \)
Dist. betwn torsional restraints \( L_T = 4 \text{ m} \)

SQUARE HOLLOW SECTION 400 x 200 x 6 SHS Grade S 1.4462
SECTION  
SUMMARY  
Section is satisfactory for axial load, and overall buckling check.
Axial load 20 kN
Compression resistance 2888.8 kN
Maximum moment z axis 110 kNm
Moment capacity 321.27 kNm
Maximum moment y axis 100 kNm
Moment capacity 220.22 kNm
Local capacity check \( 0.80329 \leq 1 \)
Overall buckling checks \( 0.40529 \leq 1, 0.38474 \leq 1 \)
Stainless Steel Structural

Hollow Section Design to EC3

Calculations in accordance with the SCI publication entitled 'Design Manual For Structural Stainless steel' (Third Edition) dated 18th April 2006.

Loads and moments are factored.

Design moment about y-y axis \( M_{yEd} = 2.74 \text{ kNm} \)
Design shear force in z direction \( V_{zEd} = 1 \text{ kN} \)
Design axial load (comp. positive) \( N_{Ed} = 19.6 \text{ kN} \)
Length of member \( L = 2.7 \text{ m} \)
Design moment about z-z axis \( M_{zEd} = 0 \text{ kNm} \)
100 x 50 x 6 RHS
Length between restraints yy-axis \( L_y = 2700 \text{ mm} \)
Length between restraints zz-axis \( L_z = 2700 \text{ mm} \)
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)

SQUARE HOLLOW SECTION 100 x 50 x 6 SHS Grade S 1.4401
SECTION
SUMMARY
Section is satisfactory for axial load, and overall buckling check.
Design axial load 19.6 kN
Compression resistance 99.12 kN
Design moment y-y axis 2.74 kNm
Moment resistance 8.75 kNm
Overall buckling checks 0.57351 \( \leq 1 \)
0.51088 \( \leq 1 \)
Location: Ex2 - Bending about major axis only

Stainless Steel Structural

Hollow Section Design to EC3

Calculations in accordance with the SCI publication entitled 'Design Manual For Structural Stainless steel' (Third Edition) dated 18th April 2006.

Loads and moments are factored.

Design moment about y-y axis \( My_{Ed} = 77.4 \text{ kNm} \)
Design shear force in z direction \( V_{zEd} = 18 \text{ kN} \)
Design axial load (comp.positive) \( N_{Ed} = 1100 \text{ kN} \)
Length of member \( L = 4 \text{ m} \)
Design moment about z-z axis \( M_{zEd} = 0 \text{ kNm} \)
250 x 150 x 15 RHS
Length between restraints yy-axis \( L_y = 4000 \text{ mm} \)
Length between restraints zz-axis \( L_z = 4000 \text{ mm} \)
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)

SQUARE HOLLOW SECTION

250 x 150 x 15 SHS Grade S 1.4462

SECTION

Section is satisfactory for axial load, and overall buckling check.

SUMMARY

Design axial load \( 1100 \text{ kN} \)
Compression resistance \( 2296.8 \text{ kN} \)
Design moment y-y axis \( 77.4 \text{ kNm} \)
Moment resistance \( 322.71 \text{ kNm} \)
Overall buckling checks \( 0.76674 \leq 1 \)
\( 0.71877 \leq 1 \)
Location: Ex3 - Bending about minor axis only

Stainless Steel Structural

Hollow Section Design to EC3

Calculations in accordance with the SCI publication entitled 'Design Manual For Structural Stainless steel' (Third Edition) dated 18th April 2006.

Loads and moments are factored.

Design moment about y-y axis $MyEd=0$ kNm
Design shear force in z direction $VzEd=18$ kN
Design axial load (comp.positive) $NEd=900$ kN
Length of member $L=4$ m
Design moment about z-z axis $MzEd=24$ kNm
Length between restraints yy-axis $Ly=4000$ mm
Length between restraints zz-axis $Lz=4000$ mm
Young’s Modulus $E=200$ kN/mm²

SQUARE HOLLOW SECTION 350 x 200 x 15 SHS Grade S 1.4462
SECTION
SUMMARY
Section is satisfactory for axial load, and overall buckling check.
Design axial load 900 kN
Compression resistance 4619.1 kN
design moment z-z axis 24 kNm
Moment resistance 456.65 kNm
Location: Ex4 - Bi-axial bending

Stainless Steel Structural

Hollow Section Design to EC3

Calculations in accordance with the SCI publication entitled 'Design Manual For Structural Stainless steel' (Third Edition) dated 18th April 2006.

Loads and moments are factored.

Design moment about y-y axis MyEd=56 kNm
Design shear force in z direction VzEd=8 kN
Length of member L=4 m
Length between restraints yy-axis Ly=4000 mm
Length between restraints zz-axis Lz=4000 mm

SQUARE HOLLOW SECTION

350 x 200 x 15 SHS Grade S 1.4462

Section is satisfactory for axial load, and overall buckling check.

Design axial load 800 kN
Compression resistance 4619.1 kN
Design moment y-y axis 56 kNm
Moment resistance 670.76 kNm
design moment z-z axis 20 kNm
Moment resistance 456.65 kNm

Overall buckling checks 0.32593 ≤ 1
0.30924 ≤ 1
Location: Ex5 - Bi-axial bending

Stainless Steel Structural

Hollow Section Design to EC3

Calculations in accordance with the SCI publication entitled 'Design Manual For Structural Stainless steel' (Third Edition) dated 18th April 2006.

Loads and moments are factored.

Design moment about y-y axis \( M_{yEd} = 110 \ \text{kNm} \)
Design shear force in z direction \( V_{zEd} = 21 \ \text{kN} \)
Design axial load (comp.positive) \( N_{Ed} = 20 \ \text{kN} \)
Length of member \( L = 4 \ \text{m} \)
Design moment about z-z axis \( M_{zEd} = 100 \ \text{kNm} \)
Length between restraints yy-axis \( L_y = 4000 \ \text{mm} \)
Length between restraints zz-axis \( L_z = 4000 \ \text{mm} \)
Young's Modulus \( E = 200 \ \text{kN/mm}^2 \)

SQUARE HOLLOW SECTION

350 x 200 x 15 SHS Grade S 1.4462

Section is satisfactory for axial load, and overall buckling check.

Design axial load \( 20 \ \text{kN} \)
Compression resistance \( 5599.4 \ \text{kN} \)
Design moment y-y axis \( 110 \ \text{kNm} \)
Moment resistance \( 670.76 \ \text{kNm} \)
design moment z-z axis \( 100 \ \text{kNm} \)
Moment resistance \( 456.65 \ \text{kNm} \)
Overall buckling checks \( 0.46314 \leq 1 \)
\( 0.43034 \leq 1 \)
Stainless Steel Structural

Hollow Section Design to EC3

Calculations in accordance with the SCI publication entitled 'Design Manual For Structural Stainless steel' (Third Edition) dated 18th April 2006.

Loads and moments are factored.

Design moment about y-y axis \( M_{yEd} = 2.6 \text{ kNm} \)
Design shear force in z direction \( V_{zEd} = 1 \text{ kN} \)
Design axial load (comp.positive) \( N_{Ed} = 18.6 \text{ kN} \)
Length of member \( L = 2.7 \text{ m} \)
Design moment about z-z axis \( M_{zEd} = 0 \text{ kNm} \)
Length between restraints yy-axis \( L_y = 2700 \text{ mm} \)
Length between restraints zz-axis \( L_z = 2700 \text{ mm} \)
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)

100 x 50 x 6 RHS

Section is satisfactory for axial load, and overall buckling check.

Design axial load \( 18.6 \text{ kN} \)
Compression resistance \( 99.12 \text{ kN} \)
Design moment y-y axis \( 2.6 \text{ kNm} \)
Moment resistance \( 8.75 \text{ kNNm} \)
Overall buckling checks \( 0.54422 \leq 1 \)
\( 0.48479 \leq 1 \)
Simply supported stainless steel beam

Calculations in accordance with BS5950-1: 2000 and SCI publication "Structural Design of Stainless Steel"

Beam span \( L = 6 \text{ m} \)

200 x 75 x 8 Channel

Young's Modulus \( E = 200 \text{ kN/mm}^2 \)

Dead load factor \( \gamma_d = 1.4 \)

Imposed load factor \( \gamma_i = 1.6 \)

Distance from left support \( L_{c(1)} = 2 \text{ m} \)

Unfactored dead load \( G_{kc(1)} = 0.4 \text{ kN} \)

Unfactored imposed load \( Q_{kc(1)} = 0 \text{ kN} \)

Distance from left support \( L_{c(2)} = 4 \text{ m} \)

Unfactored dead load \( G_{kc(2)} = 0.4 \text{ kN} \)

Unfactored imposed load \( Q_{kc(2)} = 0 \text{ kN} \)

Dist. from left support to start \( L_{au(1)} = 0 \text{ m} \)

Distance from left support to end \( L_{bu(1)} = 6 \text{ m} \)

Unfactored dead load \( G_{ku(1)} = 0.196 \text{ kN/m} \)

Unfactored imposed load \( Q_{ku(1)} = 3.85 \text{ kN/m} \)

Length of beam between restraints \( L_T = 2 \text{ m} \)

Maximum end moment on segment \( M_e = 27 \text{ kNm} \)

Start of segment \( m_1 = 27 \text{ kNm} \)

Quarter point \( m_2 = 29 \text{ kNm} \)

Mid-span \( m_3 = 30.435 \text{ kNm} \)

Three quarter point \( m_4 = 29 \text{ kNm} \)

Emissivity \( \varepsilon_r = 0.5 \)

Assumed value of load ratio \( R = 0.5 \)

Since \( M_f > M_b(0) \) \( (15.037 \text{ kNm} > 10.437 \text{ kNm}) \) applied moment exceeds buckling resistance moment. Addition fire resistance is required

DESIGN SUMMARY

Stainless Steel Section

\[ 200 \times 75 \times 8 \text{ Grade 1.4401} \]

Max. shear force \( 19.863 \text{ kN} \)

Shear capacity \( 211.2 \text{ kN} \)

Max. applied moment \( 30.075 \text{ kNm} \)

Moment capacity \( 36.564 \text{ kNm} \)

Buckling resistance \( 31.209 \text{ kNm} \)

Moment factor (mLT) \( 0.99527 \)

Resistance (\( M_b/\text{mLT} \)) \( 31.358 \text{ kNm} \)

Unfactored DL defl \( 2.6408 \text{ mm} \)

Unfactored LL defl \( 26.916 \text{ mm} \)

Limiting deflection \( 30 \text{ mm} \)

Unfactored DL shear at LHE \( 0.988 \text{ kN} \)

DL shear at LHE \( 11.55 \text{ kN} \)

LL shear at LHE \( 11.55 \text{ kN} \)

DL shear at RHE \( 0.988 \text{ kN} \)

LL shear at RHE \( 11.55 \text{ kN} \)
| FIRE RESISTANCE | Heated perimeter | 684 mm² |
| SUMMARY         | Area of section  | 25.3 cm² |
|                 | Ratio (Hp/A)     | 270     |
|                 | Assumed Load Ratio | 0.5   |
|                 | Section does not have an inherent fire resistance of 30 minutes. |
Location: Slender channel

Simply supported stainless steel beam

Calculations in accordance with BS5950-1: 2000 and SCI publication "Structural Design of Stainless Steel"

Beam span \( L = 4 \) m

300 x 100 x 8 Channel

Young's Modulus \( E = 200 \) kN/mm²

Dead load factor \( \gamma_{d} = 1.4 \)

Imposed load factor \( \gamma_{i} = 1.6 \)

Distance from left support to start \( L_{au}(1) = 0 \) m

Distance from left support to end \( L_{bu}(1) = 4 \) m

Unfactored dead load \( G_{ku}(1) = 7 \) kN/m

Unfactored imposed load \( Q_{ku}(1) = 10 \) kN/m

DESIGN SUMMARY

Stainless Steel Section

300 x 100 x 8 Grade 1.4301

Max. shear force 51.6 kN

Shear capacity 302.4 kN

Max. applied moment 51.6 kNm

Moment capacity 64.718 kNm

Buckling resistance 48.632 kNm

Moment factor (mLT) 0.925

Resistance (Mb/mLT) 52.575 kNm

Unfactored DL defln 2.6662 mm

Unfactored LL defln 3.8089 mm

Limiting deflection 20 mm

Unfactored end shears

DL shear at LHE 14 kN

LL shear at LHE 20 kN

DL shear at RHE 14 kN

LL shear at RHE 20 kN
Location: Slender rhs

Simply supported stainless steel beam

Calculations in accordance with BS5950-1: 2000 and SCI publication "Structural Design of Stainless Steel"

Beam span \( L = 6 \text{ m} \)

400 x 200 x 6 RHS

Young's Modulus \( E = 200 \text{ kN/mm}^2 \)

Dead load factor \( \gamma_d = 1.4 \)
Imposed load factor \( \gamma_i = 1.6 \)

Dist. from left support to start \( L_{au(1)} = 0 \text{ m} \)
Distance from left support to end \( L_{bu(1)} = 6 \text{ m} \)
Unfactored dead load \( G_{ku(1)} = 15 \text{ kN/m} \)
Unfactored imposed load \( Q_{ku(1)} = 19 \text{ kN/m} \)

Full lateral restraint provided to compression flange

DESIGN SUMMARY

Stainless Steel Section

<table>
<thead>
<tr>
<th>RECTANGULAR HOLLOW SECTION</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>400 x 200 x 6 RHS</td>
<td>Grade 1.4362</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. shear force</td>
<td>154.2 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear capacity</td>
<td>1104 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. applied moment</td>
<td>231.3 kNm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment capacity</td>
<td>271.35 kNm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unfactored DL defln</td>
<td>9.3926 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unfactored LL defln</td>
<td>11.897 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>30 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DL shear at LHE</td>
<td>45 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LL shear at LHE</td>
<td>57 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DL shear at RHE</td>
<td>45 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LL shear at RHE</td>
<td>57 kN</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**Location:** slender back to back channels

**Simply supported stainless steel beam**

Calculations in accordance with BS5950-1:2000 and SCI publication "Structural Design of Stainless Steel"

Beam span \( L = 6 \) m

400 x 150 x 8 Channel

Young’s Modulus \( E = 200 \) kN/mm²

Dead load factor \( \gamma_d = 1.4 \)

Imposed load factor \( \gamma_i = 1.6 \)

Distance from left support to start \( L_{au}(1) = 0 \) m

Distance from left support to end \( L_{bu}(1) = 6 \) m

Unfactored dead load \( G_{ku}(1) = 8 \) kN/m

Unfactored imposed load \( Q_{ku}(1) = 12 \) kN/m

Full lateral restraint provided to compression flange

Emissivity \( e_r = 0.5 \)

Assumed value of load ratio \( R = 0.5 \)

**DESIGN SUMMARY**

Stainless Steel Section

2No/400 x 150 x 8 Grade 1.4401 Back to back

Max. shear force \( 91.2 \) kN

Shear capacity \( 844.8 \) kN

Max. applied moment \( 136.8 \) kNm

Moment capacity \( 235.04 \) kNm

Unfactored DL defln \( 2.7176 \) mm

Unfactored LL defln \( 4.0763 \) mm

Limiting deflection \( 30 \) mm

Unfactored end shears

<table>
<thead>
<tr>
<th></th>
<th>DL shear at LHE</th>
<th>LL shear at LHE</th>
<th>DL shear at RHE</th>
<th>LL shear at RHE</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL shear</td>
<td>24 kN</td>
<td>36 kN</td>
<td>24 kN</td>
<td>36 kN</td>
</tr>
</tbody>
</table>

**FIRE RESISTANCE SUMMARY**

Heated perimeter \( 1834 \) mm²

Area of section \( 106.6 \) cm²

Ratio \( (H_p/A) \) \( 170 \)

Assumed Load Ratio \( 0.5 \)

Section has an inherent fire resistance of 30 minutes.
Location: slender rhs bending about y-y axis

Simply supported stainless steel beam

Beam span \( L = 6 \) m

150 x 300 x 6 RHS

Young's Modulus \( E = 200 \text{ kN/mm}^2 \)

Dead load factor \( \gamma_d = 1.4 \)

Imposed load factor \( \gamma_i = 1.6 \)

Distance from left support to start \( L_{au}(1) = 0 \) m

Distance from left support to end \( L_{bu}(1) = 6 \) m

Unfactored dead load \( G_{ku}(1) = 5 \text{ kN/m} \)

Unfactored imposed load \( Q_{ku}(1) = 7 \text{ kN/m} \)

Emissivity \( e_r = 0.4 \)

Assumed value of load ratio \( R = 0.4 \)

Since \( M_f > M_{ct} \) \( (32.76 \text{ kNm} > 21.842 \text{ kNm}) \) applied moment exceeds moment capacity. Addition fire resistance is required

**DESIGN SUMMARY**

**RECTANGULAR HOLLOW SECTION**

Stainless Steel Section

- 150 x 300 x 6 RHS Grade 1.4462
- Max. shear force 54.6 kN
- Shear capacity 469.2 kN
- Max. applied moment 81.9 kNm
- Moment capacity 89.517 kNm
- Unfactored DL defln 21.285 mm
- Unfactored LL defln 29.8 mm
- Limiting deflection 30 mm

**Unfactored end shears**

- DL shear at LHE 15 kN
- LL shear at LHE 21 kN
- DL shear at RHE 15 kN
- LL shear at RHE 21 kN

**FIRE RESISTANCE SUMMARY**

- Heated perimeter 600 mm²
- Area of section 51 cm²
- Ratio (Hp/A) 115
- Assumed Load Ratio 0.4

Section does not have an inherent fire resistance of 30 minutes.
Location: Ex1 - Unrestrained channel

Simply supported stainless steel beam

Calculations in accordance with the SCI publication 'Design Manual For Structural Stainless Steel' Third Edition dated 18th April 2006.

Beam span $L = 6 \text{ m}$

200 x 75 x 8 Channel

Young's Modulus $E = 200 \text{ kN/mm}^2$

Permanent load partial factor $\gamma_G = 1.35$

Variable load partial factor $\gamma_Q = 1.5$

Applied point loads on member

Applied permanent and variable point loads are unfactored and distances are measured from LHS support.

Distance from left support $L_c(1) = 2 \text{ m}$

Unfactored permanent load $G_{kc}(1) = 0.4 \text{ kN}$

Unfactored variable load $Q_{kc}(1) = 0 \text{ kN}$

Distance from left support $L_c(2) = 4 \text{ m}$

Unfactored permanent load $G_{kc}(2) = 0.4 \text{ kN}$

Unfactored variable load $Q_{kc}(2) = 0 \text{ kN}$

Applied UDL loading on member

Applied permanent and variable UDL loads are unfactored and distances are measured from LHS support.

Distance from left support to start $L_a(1) = 0 \text{ m}$

Unfactored permanent load $G_{ku}(1) = 0.196 \text{ kN/m}$

Unfactored variable load $Q_{ku}(1) = 3.85 \text{ kN/m}$
Effective length parameter $K=1$
Length for LTB $LT=2$ m
Far end moment $psiM=0$ kNm
Imperfection factor $aLT=0.34$

<table>
<thead>
<tr>
<th>Stainless Steel Section</th>
<th>200 x 75 x 8 Grade 1.4401</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESIGN</td>
<td>Max. shear force 18.659 kN</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Shear resistance 184.75 kN</td>
</tr>
<tr>
<td></td>
<td>Design bending moment 28.258 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment resistance 33.84 kNm</td>
</tr>
<tr>
<td></td>
<td>Buckling resistance 29.949 kNm</td>
</tr>
<tr>
<td></td>
<td>Perm. load deflection 2.6191 mm</td>
</tr>
<tr>
<td></td>
<td>Variable load defln 26.695 mm</td>
</tr>
<tr>
<td></td>
<td>Limiting deflection 30 mm</td>
</tr>
</tbody>
</table>

Unfactored

<table>
<thead>
<tr>
<th>end shears</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perm. load shear @ LHE 0.988 kN</td>
</tr>
<tr>
<td>Var. load shear @ LHE 11.55 kN</td>
</tr>
<tr>
<td>Perm. load shear @ RHE 0.988 kN</td>
</tr>
<tr>
<td>Var. load shear @ RHE 11.55 kN</td>
</tr>
</tbody>
</table>

Fire resistance without applied protection

The critical temperature method given in BS EN 1993-1-2 Clause 4.2.4 will be adopted. For further clarification refer to Thomas Telford publication entitled 'Designer's guide to EN 1991-1-2, EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2'.

Factor for variable actions $\psi_1$ $psi1=0.7$
Max span design bending moment $MyEd=13.81$ kNm

Critical temperature of stainless steel

Design moment (fire limit state) $MfiEd=MyEd=13.81$ kNm
Design buckling resistance $MbRdfi=ChiLTf*McRd=19.239$ kNm
Design moment resistance $MfiRd=ky*(gamM0/gamfi)*MbRdfi/(k1*k2)=27.484$ kNm
Critical temperature $\theta_{acr}$ $taCR=39.19*LOG(comp-1)+482=583.87^\circ$C
Fire resistance period 18 minutes

FIRE RESISTANCE

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Location: Ex2 - Channel section

Simply supported stainless steel beam

Calculations in accordance with the SCI publication 'Design Manual For Structural Stainless Steel' Third Edition dated 18th April 2006.

Beam span L=4 m

300 x 100 x 12 Channel
Young's Modulus E=200 kN/mm²
Permanent load partial factor gamG=1.35
Variable load partial factor gamQ=1.5

Applied UDL loading on member

Applied permanent and variable UDL loads are unfactored and distances are measured from LHS support.

Dist.from left support to start Lau(1)=0 m
Distance from left support to end Lbu(1)=4 m
Unfactored permanent load Gku(1)=7 kN/m
Unfactored variable load Qku(1)=10 kN/m
Effective length parameter K=1
Length for LTB LT=4 m
Factor C1=1.13
Imperfection factor aLT=0.34

Stainless Steel Section
300 x 100 x 12 Grade 1.4301
Max. shear force 48.9 kN
Shear resistance 396.8 kN
Design bending moment 48.9 kNm
Moment resistance 100.8 kNm
Buckling resistance 67.218 kNm
Perm.load deflection 1.8427 mm
Variable load defln 2.6325 mm
Limiting deflection 20 mm
Perm.load shear @ LHE 14 kN
Var.load shear @ LHE 20 kN
Perm.load shear @ RHE 14 kN
Var.load shear @ RHE 20 kN
Location: Ex3 - RHS

Simply supported stainless steel beam

Calculations in accordance with the SCI publication 'Design Manual For Structural Stainless Steel' Third Edition dated 18th April 2006.

Beam span L=6 m

350 x 200 x 12 RHS
Young's Modulus E=200 kN/mm²
Permanent load partial factor gamG=1.35
Variable load partial factor gamQ=1.5

Applied UDL loading on member

Applied permanent and variable UDL loads are unfactored and distances are measured from LHS support.

Dist.from left support to start Lau(1)=0 m
Distance from left support to end Lbu(1)=6 m
Unfactored permanent load Gku(1)=15 kN/m
Unfactored variable load Qku(1)=19 kN/m
Effective length parameter K=1
Length for LTB LT=6 m
Factor C1=1.13

Stainless Steel Section

RECTANGULAR HOLLOW SECTION
350 x 200 x 12 RHS Grade 1.4462
Max. shear force 146.25 kN
Shear resistance 1843.7 kN
Design bending moment 219.38 kNm
Moment resistance 560.78 kNm
Buckling resistance 560.78 kNm
Perm.load deflection 6.7984 mm
Variable load defln 8.6113 mm
Limiting deflection 30 mm

Perm.load shear @ LHE 45 kN
Var.load shear @ LHE 57 kN
Perm.load shear @ RHE 45 kN
Var.load shear @ RHE 57 kN

Unfactored end shears

SCALE 5.48 Office 1007 Proforma 386
**Location:** Ex4 - Back to back channels

**Simply supported stainless steel beam**

Calculations in accordance with the SCI publication 'Design Manual For Structural Stainless Steel' Third Edition dated 18th April 2006.

Beam span \( L = 6 \text{ m} \)

400 x 150 x 15 Channel

Young's Modulus \( E = 200 \text{ kN/mm}^2 \)

Permanent load partial factor \( \gamma_G = 1.35 \)

Variable load partial factor \( \gamma_Q = 1.5 \)

Applied UDL loading on member

Applied permanent and variable UDL loads are unfactored and distances are measured from LHS support.

Dist. from left support to start \( \text{Lau}(1)=0 \text{ m} \)

Distance from left support to end \( \text{Lbu}(1)=6 \text{ m} \)

Unfactored permanent load \( \text{Gku}(1)=8 \text{ kN/m} \)

Unfactored variable load \( \text{Qku}(1)=12 \text{ kN/m} \)

Effective length parameter \( K = 1 \)

Length for LTB \( \text{LT}=6 \text{ m} \)

Factor \( C_1 = 1.13 \)

Imperfection factor \( a_{LT} = 0.34 \)

**Stainless Steel Section**

2No/400 x 150 x 15 Grade 1.4401

**Back to back channels**

Max. shear force \( 86.4 \text{ kN} \)

Shear resistance \( 1385.6 \text{ kN} \)

Design bending moment \( 129.6 \text{ kNm} \)

Moment resistance \( 422 \text{ kNm} \)

Buckling resistance \( 331.69 \text{ kNm} \)

Perm. load deflection \( 1.5997 \text{ mm} \)

Variable load defln \( 2.3995 \text{ mm} \)

Limiting deflection \( 30 \text{ mm} \)

Perm. load shear @ LHE \( 24 \text{ kN} \)

Var. load shear @ LHE \( 36 \text{ kN} \)

Perm. load shear @ RHE \( 24 \text{ kN} \)

Var. load shear @ RHE \( 36 \text{ kN} \)
Fire resistance without applied protection

The critical temperature method given in BS EN 1993-1-2 Clause 4.2.4 will be adopted. For further clarification refer to Thomas Telford publication entitled 'Designer's guide to EN 1991-1-2, EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2'.

Factor for variable actions $\psi_1$ $\psi_1=0.7$
Max span design bending moment $My_{Ed}=73.8$ kNm

Critical temperature of stainless steel

Design moment (fire limit state) $Mfi_{Ed}=My_{Ed}=73.8$ kNm
Design buckling resistance $Mb_{Rdfi}=Mc_{Rd}=464.2$ kNm
Design moment resistance $Mfi_{Rd}=k_y(gamM_0/gamfi)Mb_{Rdfi}/(k_1 *k_2)=663.14$ kNm
Critical temperature $\theta_{acr}$ $ta_{CR}=39.19*LOG(comp-1)+482=813.11$ °C
Fire resistance period 30 minutes

FIRE RESISTANCE Section has an inherent fire resistance of 30 minutes.

SUMMARY
**Simply supported stainless steel beam**

Calculations in accordance with the SCI publication 'Design Manual For Structural Stainless Steel' Third Edition dated 18th April 2006.

Beam span \( L = 6 \, \text{m} \)

150 x 300 x 15 RHS

Young's Modulus \( E = 200 \, \text{kN/mm}^2 \)

Permanent load partial factor \( \text{gamG} = 1.35 \)

Variable load partial factor \( \text{gamQ} = 1.5 \)

Applied UDL loading on member

Applied permanent and variable UDL loads are unfactored and distances are measured from LHS support.

Dist. from left support to start \( \text{Lau}(1) = 0 \, \text{m} \)

Distance from left support to end \( \text{Lbu}(1) = 6 \, \text{m} \)

Unfactored permanent load \( \text{Gku}(1) = 5 \, \text{kN/m} \)

Unfactored variable load \( \text{Qku}(1) = 7 \, \text{kN/m} \)

Stainless Steel Section

**RECTANGULAR HOLLOW SECTION** 150 x 300 x 15 RHS Grade 1.4462

**DESIGN**

Max. shear force \( 51.75 \, \text{kN} \)

Shear resistance \( 933.56 \, \text{kN} \)

Design bending moment \( 77.625 \, \text{kNm} \)

Moment resistance \( 269.23 \, \text{kNm} \)

Perm. load deflection \( 10.503 \, \text{mm} \)

Variable load defln \( 14.704 \, \text{mm} \)

Limiting deflection \( 30 \, \text{mm} \)

Perm. load shear @ LHE 15 kN

Var. load shear @ LHE 21 kN

Perm. load shear @ RHE 15 kN

Var. load shear @ RHE 21 kN

**Fire resistance without applied protection**

The critical temperature method given in BS EN 1993-1-2 Clause 4.2.4 will be adopted. For further clarification refer to Thomas Telford publication entitled 'Designer's guide to EN 1991-1-2, EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2'.
Factor for variable actions $\psi_1$ $\psi_1=0.7$
Max span design bending moment $M_{yEd}=44.55$ kNm

Critical temperature of stainless steel

Design moment (fire limit state) $M_{fEd}=M_{yEd}=44.55$ kNm
Design buckling resistance $M_{bRdfi}=\chi_{LTf}\cdot M_{cRd}=151.68$ kNm
Design moment resistance $M_{fRd}=k_y\cdot \left(\frac{\gamma_M}{\gamma_{fi}}\right)\cdot M_{bRdfi}/(k_1\cdot k_2)=216.69$ kNm
Critical temperature $\theta_{acr}$ $\theta_{acr}=39.19\cdot \text{LOG}(\text{comp}-1)+482=720.83$ °C
Fire resistance period 26 minutes

FIRE RESISTANCE
SUMMARY

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Axially loaded stainless steel column

Calculations are in accordance with BS5950-1:2000 and 'Structural Design of Stainless Steel' published by The Steel Construction Institute.

L = length of member  
F = compressive load

Factored axial compressive load  \( F = 19.6 \text{ kN} \)  
Length between restraints  \( L = 2700 \text{ mm} \)  
Effective length factor (y axis)  \( e_f = 1 \)  
100 x 50 x 6 RHS  
Young's Modulus  \( E = 200 \text{ kN/mm}^2 \)  
Shear Modulus  \( G = 76.9 \text{ kN/mm}^2 \)  

Steel temperature after 30 minutes fire duration

Initial steel temperature is assumed to be 20 degrees C.  
Density at 20 degrees C  \( \rho_{od} = 8000 \text{ kg/m}^3 \)  
Emissivity  \( e_r = 0.4 \)  
Coefficient of heat transfer  \( a_c = 25 \text{ W/m}^2 \text{ C} \)  
The calculation of the temperature rise in the stainless steel is an iterative process, the result of this process is:  
temperature at 30 minutes  \( \theta = 832 \text{ degrees C} \)  

Reduction of mechanical properties at elevated temperature

The following factors are required for calculation of resistance at elevated temperatures, they have been obtained by linear interpolation from Table 6.2 of the design guide for 832 degrees C.  
The value of 2% yield strength at elevated temperature is also required.  
Ultimate strength  \( U_s = 520 \text{ N/mm}^2 \)  
Assumed value of load ratio  \( R = 0.67 \)  
Load at fire limit state  \( F_f = R \times F = 13.132 \text{ kN} \)  
Since \( F_f \leq P_c0 \) ( 13.132 kN \( \leq 39.417 \text{ kN} \) ) applied load within compressive resistance.
### SUMMARY

<table>
<thead>
<tr>
<th>Stainless Steel Section</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>RECTANGULAR HOLLOW SECTION</strong></td>
</tr>
<tr>
<td>100 x 50 x 6 RHS Grade 1.4401</td>
</tr>
<tr>
<td><strong>DESIGN</strong></td>
</tr>
<tr>
<td>Compressive load 19.6 kN</td>
</tr>
<tr>
<td>Compressive resistance 109.03 kN</td>
</tr>
<tr>
<td><strong>FIRE RESISTANCE</strong></td>
</tr>
<tr>
<td>Heated perimeter 300 mm²</td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
</tr>
<tr>
<td>Area of section 15 cm²</td>
</tr>
<tr>
<td>Ratio (Hp/A) 200</td>
</tr>
<tr>
<td>Assumed Load Ratio 0.67</td>
</tr>
<tr>
<td>Section has an inherent fire resistance of 30 minutes.</td>
</tr>
</tbody>
</table>
Axially loaded stainless steel column

Calculations are in accordance with BS5950-1:2000 and 'Structural Design of Stainless Steel' published by The Steel Construction Institute.

L = length of member
F = compressive load

Factored axial compressive load  F=400 kN
x-x axis  Lx=6000 mm
y-y axis  Ly=4000 mm
Effective length factor (y axis)  ef=1
Effective length factor (x axis)  Ef=1
Young's Modulus  E=200 kN/mm²
Shear Modulus  G=76.9 kN/mm²

SUMMARY

Stainless Steel Section
RECTANGULAR HOLLOW SECTION  200 x 125 x 4 RHS  Grade 1.4362
DESIGN  Compressive load  400 kN
SUMMARY  Compressive resistance 440.72 kN
Location: slender channel

Factored axial compressive load  \( F = 350 \text{ kN} \)

x-x axis  \( L_x = 4000 \text{ mm} \)

y-y axis  \( L_y = 2000 \text{ mm} \)

Effective length factor (y axis)  \( e_f = 1 \)

Effective length factor (x axis)  \( E_f = 1 \)

350 x 125 x 8 Channel

Young's Modulus  \( E = 200 \text{ kN/mm}^2 \)

Shear Modulus  \( G = 76.9 \text{ kN/mm}^2 \)

SUMMARY

Stainless Steel Section

CHANNEL  350 x 125 x 8 Grade 1.4462

DESIGN  Compressive load 350 kN

SUMMARY  Compressive resistance 646.48 kN

Unity relationships  

for slender sections 0.68337 ≤ 1

0.97724 ≤ 1
Location: slender double channel

Axially loaded stainless steel column

Calculations are in accordance with BS5950-1:2000 and 'Structural Design of Stainless Steel' published by The Steel Construction Institute.

L = length of member
F = compressive load

Factored axial compressive load \( F = 800 \text{ kN} \)
Length between restraints \( L = 6000 \text{ mm} \)
Effective length factor (y axis) \( \varepsilon_f = 1 \)
350 x 125 x 10 Channel
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)
Shear Modulus \( G = 76.9 \text{ kN/mm}^2 \)

SUMMARY

Stainless Steel Section
DOUBLE CHANNEL
2/350 x 125 x 10 Grade 1.4462
Back to back
DESIGN
Compressive load 800 kN
SUMMARY
Compressive resistance 975.75 kN
Location: Ex1 - Based on example 4 of stainless steel publication

Axially loaded stainless steel column

Calculations are in accordance with the SCI publication entitled 'Design Manual For Structural Stainless steel' (Third Edition) dated 18th April 2006.

NEd = design axial compressive load
L = length of member

Unfactored permanent axial force \( N_p = 6 \, \text{kN} \)
Unfactored variable axial force \( N_v = 7 \, \text{kN} \)
Length of member \( L = 2700 \, \text{mm} \)
Length between restraints y-axis \( L_y = 2700 \, \text{mm} \)
Length between restraints z-axis \( L_z = 2700 \, \text{mm} \)
100 x 50 x 6 RHS
Young's Modulus \( E = 200 \, \text{kN/mm}^2 \)

Stainless Steel Section

RECTANGULAR HOLLOW SECTION 100 x 50 x 6 RHS Grade 1.4401
DESIGN Design compr.load 18.6 kN
SUMMARY Compr.resistance 99.12 kN

Fire resistance without applied protection

The method for calculating the temperature rise in carbon steel can also be applied to stainless steel (refer to SCI publication P403 and BS EN 1993-1-2 Clauses 4.2.4 & 4.2.5).

Determine critical temperature of steel in fire situation

Buckling length in fire situation \( L_{fi} = 1890 \, \text{mm} \)
Critical temperature \( \theta_{cr} \)
\( t_{CR} = \text{TABLE 252 for } m_o = 0.039394, \lambda_{fi} = 1.2292 \)
\( = 776.14 \, ^\circ \text{C} \)
Fire resistance period 30 minutes

Reduction of mechanical properties at elevated temperature

The following reduction factors (as per EN 1993-1-2, Table C.1) are required for calculation of resistance at elevated temperatures.

Young's modulus retention factor \( k_{Et} = 0.64909 \)
0.2% proof strength ret. factor \( k_{0.2p} = 0.42625 \)
Ultimate tensile strength factor \( k_{ult} = 0.39011 \)
Yield strength factor \( k_{2t} = 0.32375 \)
Buckling resistance

Design buckling resistance
\[ Nb_{fRd} = \frac{Chifi*A*k02p*fy}{(gamfi*10)} \]
= 64.173 kN

Design axial compressive force
\[ N_{fEd} = 13 \text{ kN} \]

As \( N_{fEd} \leq Nb_{fRd} \) (13 kN \( \leq 64.173 \text{ kN} \)), the buckling resistance of member is OK.

**FIRE RESISTANCE**

**SUMMARY**

Design axial force 13 kN
Design axial resistance 64.173 kN
Section has an inherent fire resistance of 30 minutes.
Location: Ex2 - RHS section

Axially loaded stainless steel column

Calculations are in accordance with the SCI publication entitled 'Design Manual For Structural Stainless steel' (Third Edition) dated 18th April 2006.

NEd = design axial compressive load
L = length of member

Unfactored permanent axial force \( N_p = 133 \text{ kN} \)
Unfactored variable axial force \( N_v = 133 \text{ kN} \)
Length of member \( L = 6000 \text{ mm} \)
Length between restraints y-axis \( L_y = 6000 \text{ mm} \)
Length between restraints z-axis \( L_z = 4000 \text{ mm} \)
200 x 125 x 10 RHS
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)

Stainless Steel Section

RECTANGULAR HOLLOW SECTION 200 x 125 x 10 RHS Grade 1.4462
DESIGN Design compr.load 379.05 kN
SUMMARY Compr.resistance 970.66 kN

Fire resistance without applied protection

The method for calculating the temperature rise in carbon steel can also be applied to stainless steel (refer to SCI publication P403 and BS EN 1993-1-2 Clauses 4.2.4 & 4.2.5).

Determine critical temperature of steel in fire situation

Buckling length in fire situation \( L_{fi} = 2800 \text{ mm} \)
Critical temperature \( \theta_{cr} \)

\[
t_{CR} = \text{TABLE 256 for } m_0 = 0.10199, \lambda_{fi} = 1.0288 = 670.67 \text{ °C}
\]

Fire resistance period 22 minutes

Reduction of mechanical properties at elevated temperature

The following reduction factors (as per EN 1993-1-2, Table C.1) are required for calculation of resistance at elevated temperatures.

Young's modulus retention factor \( k_E = 0.72467 \)
0.2% proof strength ret. factor \( k_{0.2p} = 0.42574 \)
Ultimate tensile strength factor \( k_{uT} = 0.43574 \)
Yield strength factor \( k_{2t} = 0.37947 \)
Buckling resistance

Design buckling resistance

\[ N_{BfIRd} = \frac{Ch_{\text{fi}}A^*k02p*fy}{(\gamma_{\text{fi}}*10)} \]

= 682.89 kN

Design axial compressive force

\[ N_{fiEd} = 266 \text{ kN} \]

As \( N_{fiEd} \leq N_{BfIRd} \) (266 kN \leq 682.89 kN), the buckling resistance of member is OK.

FIRE RESISTANCE

Design axial force

266 kN

Design axial resistance

682.89 kN

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Location: Ex3 - Channel

Axially loaded stainless steel column

Calculations are in accordance with the SCI publication entitled 'Design Manual For Structural Stainless steel' (Third Edition) dated 18th April 2006.

NEd = design axial compressive load
L = length of member

Unfactored permanent axial force Np=117 kN
Unfactored variable axial force Nv=117 kN
Length of member L=4000 mm
Length between restraints y-axis Ly=4000 mm
Length between restraints z-axis Lz=2000 mm
300 x 100 x 15 Channel
Young's Modulus E=200 kN/mm²

Stainless Steel Section
CHANNEL 300 x 100 x 15 Grade 1.4462
DESIGN Design compr.load 333.45 kN
SUMMARY Compr.resistance 1192.9 kN

Fire resistance without applied protection

The method for calculating the temperature rise in carbon steel can also be applied to stainless steel (refer to SCI publication P403 and BS EN 1993-1-2 Clauses 4.2.4 & 4.2.5).

Determine critical temperature of steel in fire situation

Buckling length in fire situation Lfi=2000 mm
Critical temperature θcr tCR=TABLE 256 for mo=0.077427, lamfi=1.2516
=670.58 °C
Fire resistance period 22 minutes

Reduction of mechanical properties at elevated temperature

The following reduction factors (as per EN 1993-1-2, Table C.1) are required for calculation of resistance at elevated temperatures.
Young's modulus retention factor kEt=0.72471
0.2% proof strength ret. factor k02p=0.42589
Ultimate tensile strength factor kut=0.43589
Yield strength factor k2t=0.37941
Buckling resistance

Design buckling resistance

\[ N_{fiRd} = \frac{Chifi \cdot A \cdot k02p \cdot fy}{(gamfi \cdot 10)} \]

= 920.16 kN

Design axial compressive force

\[ N_{fiEd} = 234 \text{ kN} \]

As \( N_{fiEd} \leq N_{fiRd} \) (234 kN \leq 920.16 kN), the buckling resistance of member is OK.

FIRE RESISTANCE

SUMMARY

Design axial force

\[ 234 \text{ kN} \]

Design axial resistance

\[ 920.16 \text{ kN} \]

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Location: Ex4 - Double channel

Axially loaded stainless steel column

Calculations are in accordance with the SCI publication entitled 'Design Manual For Structural Stainless steel' (Third Edition) dated 18th April 2006.

NEd = design axial compressive load
L = length of member

Unfactored permanent axial force \( N_p = 250 \text{ kN} \)
Unfactored variable axial force \( N_v = 250 \text{ kN} \)
Length of member \( L = 6000 \text{ mm} \)
Length between restraints y-axis \( L_y = 6000 \text{ mm} \)
Length between restraints z-axis \( L_z = 6000 \text{ mm} \)
300 x 100 x 15 Channel
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)
Factor \( \alpha_y = 0.49 \)
Factor \( \alpha_z = 0.49 \)
Factor \( \lambda_m = 0.4 \)

Stainless Steel Section
DOUBLE CHANNEL
2/300 x 100 x 15 Grade 1.4462
Back to back channels

DESIGN
Design compr. load \( 712.5 \text{ kN} \)

SUMMARY
Compr. resistance \( 853.93 \text{ kN} \)

Fire resistance without applied protection

The method for calculating the temperature rise in carbon steel can also be applied to stainless steel (refer to SCI publication P403 and BS EN 1993-1-2 Clauses 4.2.4 & 4.2.5).

Determine critical temperature of steel in fire situation

Buckling length in fire situation \( L_{fi} = 4200 \text{ mm} \)
Critical temperature \( t_{cr} = \text{TABLE 256 for } m_0 = 0.082721, \lambda_{mfi} = 1.9011 \)
= \( 571.23 \degree C \)
Fire resistance period \( 17 \text{ minutes} \)

Reduction of mechanical properties at elevated temperature

The following reduction factors (as per EN 1993-1-2, Table C.1) are required for calculation of resistance at elevated temperatures.
Young's modulus retention factor \( k_E = 0.77151 \)
0.2% proof strength ret. factor \( k_0^{0.2p} = 0.58589 \)
Ultimate tensile strength factor \( k_u = 0.61027 \)
Yield strength factor \( k_2^t = 0.32137 \)
Buckling resistance

Design buckling resistance  
\[ NbfiRd = Chifi * A * k02p * fy / (\gamma_mfi * 10) \]
\[ = 734.27 \text{ kN} \]

Design axial compressive force  
\[ NfiEd = 500 \text{ kN} \]

As \( NfiEd \leq NbfiRd \) (500 kN \leq 734.27 kN), the buckling resistance of member is OK.

FIRE RESISTANCE

SUMMARY

Design axial force  
500 kN

Design axial resistance  
734.27 kN

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
**Location:** Example based on Ex.1 in P291

Factored axial compressive load $F=230$ kN  
Factored axial tensile load $F_t=200$ kN  
100 x 100 x 10 equal angle  
Young's Modulus $E=200$ kN/mm$^2$  
Shear Modulus $G=76.9$ kN/mm$^2$  
Length $L=1500$ mm  
Distance between restraints $L_x=1500$ mm

**SUMMARY**

<table>
<thead>
<tr>
<th>Stainless Steel Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 mm x 100 mm x 10 mm</td>
</tr>
<tr>
<td>Grade 1.4301</td>
</tr>
<tr>
<td>Section is satisfactory for axial load.</td>
</tr>
<tr>
<td>Welding equivalent to at least two bolts at each end.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Compressive load</th>
<th>230 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive resistance</td>
<td>251.3 kN</td>
</tr>
<tr>
<td>Unity relationships</td>
<td>$0.71111 \leq 1$</td>
</tr>
<tr>
<td>for slender sections</td>
<td>$0.99319 \leq 1$</td>
</tr>
<tr>
<td>Tensile load</td>
<td>200 kN</td>
</tr>
<tr>
<td>Tensile capacity</td>
<td>326.13 kN</td>
</tr>
</tbody>
</table>
Location: Example slender back to back equal angles

Factored axial compressive load \( F = 690 \text{kN} \)
Factored axial tensile load \( F_t = 550 \text{kN} \)
400 x 200 x 10 equal angles back to back
Young's Modulus \( E = 200 \text{kN/mm}^2 \)
Shear Modulus \( G = 76.9 \text{kN/mm}^2 \)
Length \( L = 3000 \text{mm} \)
Effective length (x-x axis) \( L_x = 3000 \text{mm} \)
Effective length (y-y axis) \( L_y = 3000 \text{mm} \)
Distance between bolts \( L_c = 1000 \text{mm} \)

SUMMARY

Stainless Steel Section
DOUBLE ANGLE
400 mm x 200 mm x 10 mm  Grade 1.4301
Back to Back
SECTION
Section is satisfactory for axial load.
Welding equivalent to at least two bolts at each end.
Compressive load \( 690 \text{kN} \)
Compressive resistance \( 833.69 \text{kN} \)
Unity relationships \( 0.63583 \leq 1 \)
for slender sections \( 0.96167 \leq 1 \)
Tensile load \( 550 \text{kN} \)
Tensile capacity \( 1480.8 \text{kN} \)
Location: Tension only single angle

Factored axial tensile load: Ft=340 kN
150 x 150 x 12 equal angle
Young's Modulus: E=200 kN/mm²
Shear Modulus: G=76.9 kN/mm²
Diameter of bolts: db=20 mm

SUMMARY

Stainless Steel Section
150 mm x 150 mm x 12 mm Grade 1.4401
Section is satisfactory for axial load.
Tensile load: 340 kN
Tensile capacity: 500.81 kN
**Location:** Tension only double angle

Factored axial tensile load \( \text{Ft}=890 \text{ kN} \)
300 x 150 x 15 equal angles back to back
Young's Modulus \( \text{E}=200 \text{ kN/mm}^2 \)
Shear Modulus \( \text{G}=76.9 \text{ kN/mm}^2 \)

Diameter of bolts \( \text{db}=24 \text{ mm} \)

**SUMMARY**

**Stainless Steel Section**

**DOUBLE ANGLE**

300 mm x 150 mm x 15 mm Grade 1.4301
Back to Back

**SECTION**

Section is satisfactory for axial load.

**SUMMARY**

Tensile load \( 890 \text{ kN} \)
Tensile capacity \( 1312.3 \text{ kN} \)
Location: Ex1 - Single angle checked for compression & tension

Design axial compressive load   \( N_{Ed} = 230 \text{ kN} \)
Design axial tensile load        \( N_{EdT} = 200 \text{ kN} \)

<table>
<thead>
<tr>
<th>Material Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's Modulus</td>
</tr>
<tr>
<td>Length of member</td>
</tr>
<tr>
<td>Distance between restraints</td>
</tr>
</tbody>
</table>

**Stainless Steel Section**

100 mm x 100 mm x 15 mm Grade 1.4301

Section is satisfactory for axial load.

Welding equivalent to at least two bolts at each end has been assumed.

Design compressive load \( 230 \text{ kN} \)
Compressive resistance \( 299.48 \text{ kN} \)
Design tensile load \( 200 \text{ kN} \)
Tensile resistance \( 424.01 \text{ kN} \)
Location: Ex2 - Back to back equal angles

Design axial compressive load \( \text{NEd} = 690 \text{ kN} \)
Design axial tensile load \( \text{NEdT} = 550 \text{ kN} \)

Two stainless steel equal angles back to back
\[ b = h \]
\[ D = 2h \]

300 x 150 x 15 Same size angles back to back
Young's Modulus \( E = 200 \text{ kN/mm}^2 \)
Length of member \( L = 3000 \text{ mm} \)
Effective length (y-y axis) \( L_y = 3000 \text{ mm} \)
Effective length (z-z axis) \( L_z = 3000 \text{ mm} \)
Distance between bolts \( L_c = 1000 \text{ mm} \)

Stainless Steel Section
DOUBLE ANGLE
2No. 150 mm x 150 mm x 15 mm Grade 1.4301 Back to Back angles
DESIGN
Section is satisfactory for axial load.
Welding equivalent to at least two bolts at each end has been assumed.
Design compressive load 690 kN
Compressive resistance 1275.8 kN
Design tensile load 550 kN
Tensile resistance 1438.4 kN
Location: Ex3 - Tension only single angle

Design axial tensile load \( \text{NEdT}=340 \text{ kN} \)

![Diagram of stainless steel equal angle]

150 x 150 x 12 equal angle
Young's Modulus \( E=200 \text{ kN/mm}^2 \)
Diameter of bolts \( d_b=20 \text{ mm} \)

**Stainless Steel Section**
**150 mm x 150 mm x 12 mm Grade 1.4401**

**DESIGN**
Section is satisfactory for axial load.

**SUMMARY**
Design tensile load \( 340 \text{ kN} \)
Tensile resistance \( 455.28 \text{ kN} \)
Location: Ex4 - Tension only double angle

Design axial tensile load $N_{EdT}=890$ kN

![Diagram of double angle connection]

Two stainless steel equal angles back to back
$b = h$
$D = 2h$

300 x 150 x 15 Same size angles back to back

Young's Modulus $E=200$ kN/mm²

Diameter of bolts $d_b=24$ mm

Stainless Steel Section

DOUBLE ANGLE
2No. 150 mm x 150 mm x 15 mm Grade 1.4301 Back to Back angles

DESIGN
Section is satisfactory for axial load.

SUMMARY
Design tensile load 890 kN
Tensile resistance 1193 kN
Location: Ex1 - SHS column under axial load

Concrete filled SHS column

Calculations in accordance with British Steel (Corus) publication TD 296 entitled 'SHS Design Manual for Concrete Filled Columns Part 1 Structural Design'.

Loads and moments are factored.

Factored BM about z-z axis \( M_{dz}=0 \) kNm
Factored SF in y direction \( N_{v}=0 \) kN
Axial compressive load \( N_{d}=1800 \) kN
Length of member \( L=4 \) m
Maximum BM about y-y axis \( M_{dy}=0 \) kNm

200 x 200 x 10 SHS - Hot finished.
Properties (cm): \( A=74.9 \) \( r_{x}=7.72 \) \( Z_{x}=447 \) \( S_{x}=531 \) \( I_{x}=4470 \) \( J=7030 \) \( C=655 \)
(\( r_{z}=r_{x}, \ Z_{z}=Z_{x}, \ S_{z}=S_{x} \) & \( I_{z}=I_{x} \))
Young's Modulus \( E_{s}=210 \) kN/mm\(^2\)

Characteristic concrete strength \( f_{cu}=25 \) N/mm\(^2\)
Length btwn restraints z-z axis \( L_{z}=4 \) m
Length btwn restraints y-y axis \( L_{y}=0 \) m
Effective length factor (z-axis) \( e_{fz}=1 \)

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
SQUARE HOLLOW SECTION
200 x 200 x 10 SHS Grade S 275
WITH CONCRETE INFILL
Cube strength \( 25 \) N/mm\(^2\)
Contribution factor \( 0.16244 \)
Applied axial load \( 1800 \) kN
Squash resistance \( 2237.9 \) kN
Axial resistance \( 1999.2 \) kN
Location: Ex2 - Bending about major axis, other axis restrained

Concrete filled SHS column

Calculations in accordance with British Steel (Corus) publication TD 296 entitled 'SHS Design Manual for Concrete Filled Columns Part 1 Structural Design'.

Loads and moments are factored.

Factored BM about z-z axis: $M_{dz}=150$ kNm
Factored SF in y direction: $N_v=0$ kN
Axial compressive load: $N_d=1500$ kN
Length of member: $L=7.7143$ m
Maximum BM about y-y axis: $M_{dy}=0$ kNm

250 x 150 x 16 RHS - Hot finished.
Properties (cm): $A=115$ $r_x=8.79$ $Z_x=710$ $S_x=906$ $I_x=8880$
$J=8870$ $C=849$ $Z_y=516$ $S_y=625$ $I_y=3870$ $r_y=5.8$
(where $r_z=r_x$, $Z_z=Z_x$, $S_z=S_x$ & $I_z=I_x$)
Far end BM: $M_{dz1}=150$ kNm
Young's Modulus: $E_s=210$ kN/mm$^2$

Characteristic concrete strength: $f_{cu}=40$ N/mm$^2$
Length between restraints z-z axis: $L_z=7.7143$ m
Length between restraints y-y axis: $L_y=0$ m
Effective length factor (z-axis): $\epsilon_{fz}=0.7$

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
RECTANGULAR HOLLOW SECTION
250 x 150 x 16 RHS Grade S 355
WITH CONCRETE INFILL
Cube strength: 40 N/mm$^2$
Contribution factor: 0.10998
Applied axial load: 1500 kN
Squash resistance: 4174.2 kN
Applied moment (z-z): 150 kNm
Far-end BM: 150 kNm
Moment capacity (z-z): 313.53 kNm
**Location: Ex3 - Biaxial Bending**

Concrete filled SHS column

Calculations in accordance with British Steel (Corus) publication TD 296 entitled 'SHS Design Manual for Concrete Filled Columns Part 1 Structural Design'.

Loads and moments are factored.

Factored BM about z-z axis \( M_{dz} = 150 \text{ kNm} \)
Factored SF in y direction \( N_v = 0 \text{ kN} \)
Axial compressive load \( N_d = 1200 \text{ kN} \)
Length of member \( L = 8.5714 \text{ m} \)
Maximum BM about y-y axis \( M_{dy} = 100 \text{ kNm} \)

300 x 200 x 10 RHS - Hot finished.
Properties (cm): \( A = 94.9 \text{ cm}^2, r_x = 11.2 \text{ cm}, Z_x = 788 \text{ cm}, S_x = 956 \text{ cm}, I_x = 11800 \text{ cm}^4 \)
\( J = 12900 \text{ cm}^4, C = 1020 \text{ cm}^4, Z_y = 628 \text{ cm}, S_y = 721 \text{ cm}, I_y = 6280 \text{ cm}^4 \)
\( r_y = 8.13 \text{ cm} \)

Far end BM \( M_{dz1} = 150 \text{ kNm} \)
Far end BM \( M_{dy1} = 100 \text{ kNm} \)
Young's Modulus \( E_s = 210 \text{ kN/mm}^2 \)

Characteristic concrete strength \( f_{cu} = 40 \text{ N/mm}^2 \)
Length btwn restraints z-z axis \( L_z = 8.5714 \text{ m} \)
Length btwn restraints y-y axis \( L_y = 6 \text{ m} \)
Effective length factor (z-axis) \( e_{fz} = 0.7 \)
Effective length factor (y axis) \( e_{fy} = 0.7 \)

**DESIGN SUMMARY**

Section is satisfactory

HOT FINISHED

In accordance with EN 10210

RECTANGULAR HOLLOW SECTION

300 x 200 x 10 RHS  Grade S 355

WITH CONCRETE INFILL

Cube strength \( 40 \text{ N/mm}^2 \)
Contribution factor \( 0.22804 \)
Applied axial load \( 1200 \text{ kN} \)
Squash resistance \( 3971.4 \text{ kN} \)
Axial resistance \( 1377 \text{ kN} \)
Buckling resistance \( 1880.6 \text{ kN} \)
Applied moment (z-z) \( 150 \text{ kNm} \)
Far-end BM \( 150 \text{ kNm} \)
Moment capacity (z-z) \( 336.14 \text{ kNm} \)

Buckling resistance \( 2045.2 \text{ kN} \)
Applied moment (y-y) \( 100 \text{ kNm} \)
Far-end BM \( 100 \text{ kNm} \)
Moment capacity (y-y) \( 249.09 \text{ kNm} \)
Location: Ex4 - Bending about z-z axis, y-y unrestrained

Concrete filled SHS column

Calculations in accordance with British Steel (Corus) publication TD 296 entitled 'SHS Design Manual for Concrete Filled Columns Part 1 Structural Design'.

Loads and moments are factored.

Factored BM about z-z axis \( M_{dz} = 75 \) kNm
Factored SF in y direction \( N_v = 0 \) kN
Axial compressive load \( N_d = 900 \) kN
Length of member \( L = 6 \) m
Maximum BM about y-y axis \( M_{dy} = 0 \) kNm

250 x 150 x 10 RHS - Hot finished.
Properties (cm): \( A = 74.9 \) \( r_x = 9.08 \) \( Z_x = 494 \) \( S_x = 611 \) \( I_x = 6170 \)
\( J = 6090 \) \( C = 605 \) \( Z_y = 367 \) \( S_y = 426 \) \( I_y = 2760 \) \( r_y = 6.06 \)
\( (\text{where } r_z = r_x, \ Z_z = Z_x, \ S_z = S_x \text{ & } I_z = I_x) \)
Far end BM \( M_{dz1} = 0 \) kNm
Young's Modulus \( E_s = 210 \) kN/mm³

Characteristic concrete strength \( f_{cu} = 60 \) N/mm²
Length btwn restraints z-z axis \( L_z = 6 \) m
Length btwn restraints y-y axis \( L_y = 6 \) m
Effective length factor (z-axis) \( e_{fz} = 0.85 \)
Effective length factor (y axis) \( e_{fy} = 1 \)

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
RECTANGULAR HOLLOW SECTION
250 x 150 x 10 RHS Grade S 275
WITH CONCRETE INFILL
Cube strength 60 N/mm²
Contribution factor 0.30044
Applied axial load 900 kN
Squash resistance 2679.4 kN
Buckling resistance 1581.5 kN
Applied moment (z-z) 75 kNm
Far-end BM 0 kNm
Moment capacity (z-z) 171.23 kNm
Location: Ex5 - Varying bending moment 1

Concrete filled SHS column

Calculations in accordance with British Steel (Corus) publication TD 296 entitled 'SHS Design Manual for Concrete Filled Columns Part 1 Structural Design'.

Loads and moments are factored.

Factored BM about z-z axis \( M_{dz} = 56 \text{ kNm} \)
Factored SF in y direction \( N_v = 0 \text{ kN} \)
Axial compressive load \( N_d = 400 \text{ kN} \)
Length of member \( L = 8 \text{ m} \)
Maximum BM about y-y axis \( M_{dy} = 8 \text{ kNm} \)

150 x 150 x 8 SHS - Hot finished.

Properties (cm): \( A = 44.8 \), \( r_x = 5.77 \), \( Z_x = 199 \), \( S_x = 237 \), \( I_x = 1490 \), \( C = 291 \)
(where \( r_z = r_x \), \( Z_z = Z_x \), \( S_z = S_x \) & \( I_z = I_x \))
Far end BM \( M_{dz1} = -28 \text{ kNm} \)
Far end BM \( M_{dy1} = 6 \text{ kNm} \)
Young's Modulus \( E_s = 210 \text{ kN/mm}^2 \)

Characteristic concrete strength \( f_{cu} = 40 \text{ N/mm}^2 \)
Length btwn restraints z-z axis \( L_z = 8 \text{ m} \)
Length btwn restraints y-y axis \( L_y = 4 \text{ m} \)
Effective length factor (z-axis) \( e_{fz} = 0.7 \)
Effective length factor (y axis) \( e_{fy} = 0.7 \)

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
SQUARE HOLLOW SECTION
150 x 150 x 8 SHS Grade S 355
WITH CONCRETE INFILL
Cube strength \( 40 \text{ N/mm}^2 \)
Contribution factor \( 0.1821 \)
Applied axial load \( 400 \text{ kN} \)
Squash resistance \( 1769.5 \text{ kN} \)
Axial resistance \( 493.98 \text{ kN} \)
Buckling resistance \( 410.85 \text{ kN} \)
Applied moment (z-z) \( 56 \text{ kNm} \)
Far-end BM \( -28 \text{ kNm} \)
Moment capacity (z-z) \( 81.981 \text{ kNm} \)
Buckling resistance \( 1414.8 \text{ kN} \)
Applied moment (y-y) \( 8 \text{ kNm} \)
Far-end BM \( 6 \text{ kNm} \)
Moment capacity (y-y) \( 81.981 \text{ kNm} \)
Location: Ex6 - Varying bending moment 2

Concrete filled SHS column

Calculations in accordance with British Steel (Corus) publication TD 296 entitled 'SHS Design Manual for Concrete Filled Columns Part 1 Structural Design'.

Loads and moments are factored.

Factored BM about z-z axis \( M_{dz} = 470 \text{ kNm} \)
Factored SF in y direction \( N_v = 0 \text{ kN} \)
Axial compressive load \( N_d = 300 \text{ kN} \)
Length of member \( L = 9 \text{ m} \)
Maximum BM about y-y axis \( M_{dy} = 0 \text{ kNm} \)

400 x 200 x 12.5 RHS - Hot finished.
Properties (cm): \( A = 142 \text{ cm}^2 \), \( r_x = 14.3 \text{ cm} \), \( Z_x = 1450 \text{ cm}^3 \), \( S_x = 1810 \text{ cm}^3 \), \( I_x = 29100 \text{ cm}^4 \), \( J = 23400 \text{ cm}^4 \), \( r_y = 16.6 \text{ cm} \), \( Z_y = 974 \text{ cm}^3 \), \( S_y = 1110 \text{ cm}^3 \), \( I_y = 9740 \text{ cm}^4 \), \( r_z = 14.3 \text{ cm} \), \( Z_z = Z_x \), \( S_z = S_x \), \( I_z = I_x \)

Far end BM \( M_{dz1} = 47 \text{ kNm} \)
Young's Modulus \( E_s = 210 \text{ kN/mm}^2 \)

Characteristic concrete strength \( f_{cu} = 20 \text{ N/mm}^2 \)
Length btwn restraints z-z axis \( L_z = 9 \text{ m} \)
Length btwn restraints y-y axis \( L_y = 9 \text{ m} \)
Effective length factor (z-axis) \( e_{fz} = 0.85 \)
Effective length factor (y axis) \( e_{fy} = 1 \)

DESIGN SUMMARY
Section is satisfactory
In accordance with EN 10210
RECTANGULAR HOLLOW SECTION 400 x 200 x 12.5 RHS Grade S 275
WITH CONCRETE INFILL
Cube strength 20 N/mm\(^2\)
Contribution factor 0.14227
Applied axial load 300 kN
Squash resistance 4143 kN
Buckling resistance 868.32 kN
Applied moment (z-z) 470 kNm
Far-end BM 47 kNm
Moment capacity (z-z) 483.61 kNm
Location: Ex1 - SHS column under axial load

Concrete filled SHS column


All loads and moments are factored.

Design moment about major axis: \( M_y = 0 \) kNm

Design SF in z direction: \( V_z = 0 \) kN

Design axial compressive load: \( N = 1800 \) kN

Length of member: \( L = 4 \) m

Design moment about minor axis: \( M_z = 0 \) kNm

200 x 200 x 10 SHS - Hot finished.

Properties (cm): \( A = 74.9 \) \( \mathrm{cm}^2 \), \( i_y = 7.72 \) \( \mathrm{cm} \), \( W_{ely} = 447 \) \( \mathrm{cm}^3 \), \( W_{ply} = 531 \) \( \mathrm{cm}^3 \), \( I_y = 4470 \) \( \mathrm{cm}^4 \), \( I_t = 7030 \) \( \mathrm{cm}^4 \), \( W_t = 655 \) \( \mathrm{cm}^3 \)

Permanent design load: \( N_{g} = 1500 \) kN

Steel \( \gamma_a \): \( \gamma_a = 1 \)

Concrete \( \gamma_c \): \( \gamma_c = 1.5 \)

Length btwn restraints y-y axis: \( L_y = 4 \) m

Length btwn restraints z-z axis: \( L_z = 0 \) m

Effective length factor y-y axis: \( K_y = 1 \)

Effective length factor z-z axis: \( K_z = 1 \)

DESIGN SUMMARY

Section is satisfactory

HOT FINISHED

In accordance with EN 10210

SQUARE HOLLOW SECTION

200 x 200 x 10 SHS  Grade S 275

WITH CONCRETE INFILL (NWC)

Cylinder strength: 25 N/mm²

Design axial load: 1800 kN

Column axial resistance: 2220.2 kN
Location: Ex2 - Bending about major axis, other axis restrained

Concrete filled SHS column


All loads and moments are factored.

Design moment about major axis $M_y = 140 \text{ kNm}$
Design SF in z direction $V_z = 20 \text{ kN}$
Design axial compressive load $N = 1500 \text{ kN}$
Length of member $L = 7 \text{ m}$
Design moment about minor axis $M_z = 0 \text{ kNm}$

250 x 150 x 16 RHS - Hot finished.

Properties (cm): $A = 115$ $i_z = 5.8$ $W_{ely} = 710$ $W_{ply} = 906$ $I_y = 8880$ $I_t = 8870$ $W_{elz} = 516$ $W_{plz} = 625$ $I_z = 3870$ $i_y = 8.79$

Permanent design load $N_g = 1200$
Steel $\gamma_a = 1$
Concrete $\gamma_c = 1.5$

Length btw restraints y-y axis $L_y = 7 \text{ m}$
Length btw restraints z-z axis $L_z = 0 \text{ m}$
Effective length factor y-y axis $K_y = 1$
Effective length factor z-z axis $K_z = 1$
Far end BM $N_{yEd} = 0 \text{ kNm}$
Shear plastic resistance $V_{plRd} = A_v \cdot f_y / \sqrt{3} / (\gamma_M \cdot 1000) = 1473.1 \text{ kN}$

Design shear force $V_z = 20 \text{ kN}$
Since $V_z \leq 0.5V_{plRd}$ (20 kN $\leq 736.57$ kN) design shear force is within the shear plastic resistance.

DESIGN SUMMARY

Section is satisfactory
HOT FINISHED In accordance with EN 10210
RECTANGULAR HOLLOW SECTION 250 x 150 x 16 RHS Grade S 355
WITH CONCRETE INFILL (NWC) Cylinder strength 32 N/mm$^2$
Design axial load 1500 kN
Design moment y-y axis 224.02 kNm
Moment capacity y-y axis 256.46 kNm
Location: Ex3 - Biaxial bending

Concrete filled SHS column


All loads and moments are factored.

Design moment about major axis    MyEd=135 kNm
Design SF in z direction          VzEd=16 kN
Design axial compressive load     NEd=1200 kN
Length of member                  L=8.5714 m
Design moment about minor axis    MzEd=100 kNm

300 x 200 x 10 RHS - Hot finished.
Properties (cm): A=94.9 iz=8.13 Wely=788 Wply=956 Iy=11800
                   It=12900 Wt=1020 Welz=628 Wplz=721 Iy=6280 iy=11.2
Permanent design load              NgEd=900
Steel ya                           gama=1
Concrete yc                        gamc=1.5

Length btwn restraints y-y axis    Ly=8.5714 m
Length btwn restraints z-z axis    Lz=6 m
Effective length factor y-y axis  Ky=1
Effective length factor z-z axis  Kz=1
Far end BM                         MyEd1=-135 kNm
Far end BM                         MzEd1=-100 kNm
Shear plastic resistance          VplRd=Av*fy/SQR(3)/(gamM0*1000)
Simplified                     =1167 kN
Design shear force                VzEd=16 kN
Since VzEd ≤ 0.5VplRd (16 kN ≤ 583.52 kN) design shear force is within the shear plastic resistance.

DESIGN SUMMARY                Section is satisfactory
HOT FINISHED                  In accordance with EN 10210
RECTANGULAR HOLLOW SECTION    300 x 200 x 10 RHS  Grade S 355
WITH CONCRETE INFILL (NWC)    Cylinder strength        32 N/mm²
                                Design axial load      1200 kN
                                Design moment y-y axis 135 kNm
                                Moment capacity y-y axis 355.04 kNm
                                Design moment z-z axis 100 kNm
                                Moment capacity z-z axis 262.67 kNm
                                Interaction factors 0.80169 ≤ 1
                                0.83801 ≤ 1
Location: Ex4 - Bending about z-z axis, y-y unrestrained

Concrete filled SHS column


All loads and moments are factored.

Design moment about major axis \( M_{Ed} = 75 \) kNm
Design SF in z direction \( V_{zEd} = 12.5 \) kN
Design axial compressive load \( N_{Ed} = 900 \) kN
Length of member \( L = 6 \) m
Design moment about minor axis \( M_{zEd} = 0 \) kNm

250 x 150 x 10 RHS - Hot finished.
Properties (cm): \( A = 74.9 \) iz = 6.06 \( W_{ely} = 494 \) Wply = 611 Iy = 6170
\( I_t = 6090 \) Wt = 605 Welz = 367 Wplz = 426 Iz = 2760 iyy = 9.08
Permanent design load \( N_{gEd} = 600 \)
Steel \( \gamma_a \) gama = 1
Concrete \( \gamma_c \) gamc = 1.5

Length btwn restraints y-y axis \( L_y = 6 \) m
Length btwn restraints z-z axis \( L_z = 6 \) m
Effective length factor y-y axis \( K_y = 1 \)
Effective length factor z-z axis \( K_z = 1 \)
Far end BM \( M_{Ed1} = 0 \) kNm
Shear plastic resistance \( V_{plRd} = A_v * f_y / \sqrt{3} / (\gamma_{M0} * 1000) \)
= 743.25 kN

Design shear force \( V_{zEd} = 12.5 \) kN
Since \( V_{zEd} \leq 0.5 V_{plRd} \) (12.5 kN \( \leq 371.62 \) kN) design shear force is within the shear plastic resistance.

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
RECTANGULAR HOLLOW SECTION
250 x 150 x 10 RHS Grade S 275
WITH CONCRETE INFILL (NWC)
Cylinder strength 50 N/mm²
Design axial load 900 kN
Design moment y-y axis 91.669 kNm
Moment capacity y-y axis 189.99 kNm
Location: Ex5 - Varying bending moment 1

Concrete filled SHS column


All loads and moments are factored.

Design moment about major axis MyEd=56 kNm
Design SF in z direction VzEd=8.6 kN
Design axial compressive load NEd=400 kN
Length of member L=6.5 m
Design moment about minor axis MzEd=8 kNm

150 x 150 x 12.5 SHS - Hot finished.
Properties (cm): A=67.1 iy=5.57 Wely=277 Wply=342 Iy=2080 It=3370 Wt=402
Permanent design load NgEd=300
Steel γa gama=1
Concrete γc gamc=1.5

Length btwn restraints y-y axis Ly=6.5 m
Length btwn restraints z-z axis Lz=4 m
Effective length factor y-y axis Ky=1
Effective length factor z-z axis Kz=1
Far end BM MyEd1=-28 kNm
Far end BM MzEd1=6 kNm
Shear plastic resistance VplRd=Av*fy/SQR(3)/(gamM0*1000) =687.64 kN
Design shear force VzEd=8.6 kN
Since VzEd ≤ 0.5VplRd ( 8.6 kN ≤ 343.82 kN ) design shear force is within the shear plastic resistance.

DESIGN SUMMARY

Section is satisfactory
In accordance with EN 10210

HOT FINISHED

SQUARE HOLLOW SECTION 150 x 150 x 12.5 SHS Grade S 355
WITH CONCRETE INFILL (NWC)
Cylinder strength 32 N/mm²
Design axial load 400 kN
Design moment y-y axis 56 kNm
Moment capacity y-y axis 122.19 kNm
Design moment z-z axis 8 kNm
Moment capacity z-z axis 122.19 kNm
Interaction factors 0.55102 ≤ 1
0.62006 ≤ 1
Location: Ex6 - Varying bending moment 2

Concrete filled SHS column


All loads and moments are factored.

Design moment about major axis \( M_{yd} = 470 \text{kNm} \)
Design SF in \( z \) direction \( V_{zd} = 52.2 \text{kN} \)
Design axial compressive load \( N_{Ed} = 300 \text{kN} \)
Length of member \( L = 9 \text{m} \)
Design moment about minor axis \( M_{zd} = 0 \text{kNm} \)

400 x 200 x 12.5 RHS - Hot finished.
Properties (cm): \( A = 142 \) iz=8.28 Wely=1450 Wply=1810 Iy=29100
\( It = 23400 \) Wt=1660 Welz=974 Wplz=1110 Iz=9740 iy=14.3

Permanent design load \( N_{Ed} = 200 \text{kN} \)
Steel \( \gamma_a \) \( gama = 1 \)
Concrete \( \gamma_c \) \( gamc = 1.5 \)

Length btwn restraints y-y axis \( L_y = 9 \text{m} \)
Length btwn restraints z-z axis \( L_z = 9 \text{m} \)
Effective length factor y-y axis \( K_y = 1 \)
Effective length factor z-z axis \( K_z = 1 \)
Shear plastic resistance \( V_{plRd} = Av*fy/SQR(3)/(gamM0*1000) \)
\( = 1503 \text{kN} \)

Design shear force \( V_{zd} = 52.2 \text{kN} \)
Since \( V_{zd} \leq 0.5V_{plRd} \) (52.2 kN \leq 751.52 kN) design shear force is within the shear plastic resistance.

**DESIGN SUMMARY**

Section is satisfactory

In accordance with EN 10210

RECTANGULAR HOLLOW SECTION

400 x 200 x 12.5 RHS  Grade S 275

Cylinder strength 25 N/mm²

Design axial load 300 kN
Design moment y-y axis 470 kNm
Moment capacity y-y axis 541.11 kNm
Concrete filled SHS column


All loads and moments are factored.

Design moment about major axis  $M_{Ed}=0 \text{kNm}$
Design SF in z direction  $V_{zEd}=0 \text{kN}$
Design axial compressive load  $N_{Ed}=1200 \text{kN}$
Length of member  $L=4 \text{m}$
Design moment about minor axis  $M_{zEd}=0 \text{kNm}$

150 x 150 x 8 SHS - Hot finished.
Properties (cm): $A=44.8$ $iy=5.77$ $W_{ely}=199$ $W_{ply}=237$ $I_{y}=1490$ $I_{t}=2350$ $W_{t}=291$

Steel $\gamma_a$  $gama=1.05$
Concrete $\gamma_c$  $gmac=1.5$

Length btwn restraints y-y axis  $L_y=4 \text{m}$
Length btwn restraints z-z axis  $L_z=4 \text{m}$
Effective length factor y-y axis  $K_y=1$
Effective length factor z-z axis  $K_z=1$

**DESIGN SUMMARY**

Section is satisfactory

HOT FINISHED  In accordance with EN 10210
SQUARE HOLLOW SECTION  150 x 150 x 8 SHS  Grade S 355
WITH CONCRETE INFILL (NWC)  Cylinder strength  25 N/mm²
Design axial load  1200 kN
Column axial resistance  1265.7 kN
Location: Ex8 - Ex10.5 in Lawrence Martin & John Purkiss book

Concrete filled SHS column

Calculations are in accordance with
with 'The Design Guide for Concrete
Filled SHS Columns' by Y.C. Wang
published in 2009, EC3 Part 1-1 and
EC4 Part 1-1.

All loads and moments are factored.

Design moment about major axis \( M_{yEd} = 18 \text{ kNm} \)
Design SF in \( z \) direction \( V_{zEd} = 4.5 \text{ kN} \)
Design axial compressive load \( N_{Ed} = 400 \text{ kN} \)
Length of member \( L = 4 \text{ m} \)
Design moment about minor axis \( M_{zEd} = 0 \text{ kNm} \)

150 x 100 x 8 RHS - Hot finished.
Properties (cm): \( A = 36.8 \) iz=3.94 Wely=145 Wply=180 Iy=1090
\( It=1200 \) Wt=183 Welz=114 Wplz=135 Iz=569 iy=5.44

Permanent design load \( Ng_{Ed} = 300 \)
Steel \( \gamma_a \) \( gama=1 \)
Concrete \( \gamma_c \) \( gamc=1.5 \)

Length btwn restraints \( y-y \) axis \( L_y = 4 \text{ m} \)
Length btwn restraints \( z-z \) axis \( L_z = 4 \text{ m} \)
Effective length factor \( y-y \) axis \( K_y = 1 \)
Effective length factor \( z-z \) axis \( K_z = 1 \)
Far end BM \( My_{Ed1} = 13.86 \text{ kNm} \)
Far end BM \( Mz_{Ed1} = 0 \text{ kNm} \)
Shear plastic resistance \( V_{plRd} = Av*fy/SQR(3)/(gamM0*1000) \) \( = 452.55 \text{ kN} \)

Design shear force \( V_{zEd} = 4.5 \text{ kN} \)
Since \( V_{zEd} \leq 0.5V_{plRd} \) (4.5 kN \( \leq 226.28 \text{ kN} \)) design shear
force is within the shear plastic resistance.

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
RECTANGULAR HOLLOW SECTION
150 x 100 x 8 RHS Grade S 355
WITH CONCRETE INFILL (NWC)
Cylinder strength 25 N/mm²
Design axial load 400 kN
Design moment \( y-y \) axis 18 kNm
Moment capacity \( y-y \) axis 55.785 kNm
Design moment \( z-z \) axis 0 kNm
Moment capacity \( z-z \) axis 41.464 kNm
Interaction factors 0.60353 \( \leq 1 \)
0.7771 \( \leq 1 \)
Concrete filled SHS column


All loads and moments are factored.

Design moment about major axis  MyEd=15 kNm
Design SF in z direction  VzEd=3.75 kN
Design axial compressive load  NEd=350 kN
Length of member  L=4 m
Design moment about minor axis  MzEd=8 kNm

150 x 100 x 8 RHS - Hot finished.
Properties (cm): A=36.8 iz=3.94 Wely=145 Wply=180 Iy=1090
It=1200 Wt=183 Welz=114 Wpiz=135 Iz=569 iy=5.44
Permanent design load  NgEd=250
Steel γa  gama=1
Concrete γc  gamc=1.5

Length btwn restraints y-y axis  Ly=4 m
Length btwn restraints z-z axis  Lz=4 m
Effective length factor y-y axis  Ky=1
Effective length factor z-z axis  Kz=1
Far end BM  MyEd1=11.55 kNm
Far end BM  MzEd1=6.16 kNm
Shear plastic resistance  VplRd=Av*fy/SQR(3)/(gamM0*1000)
=452.55 kN
Design shear force  VzEd=3.75 kN
Since VzEd ≤ 0.5VplRd ( 3.75 kN ≤ 226.28 kN ) design shear force is within the shear plastic resistance.

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED In accordance with EN 10210
RECTANGULAR HOLLOW SECTION 150 x 100 x 8 RHS Grade S 355
WITH CONCRETE INFILL (NWC) Cylinder strength  25 N/mm²
Design axial load  350 kN
Design moment y-y axis  15 kNmm
Moment capacity y-y axis  58.336 kNm
Design moment z-z axis  8 kNm
Moment capacity z-z axis  43.361 kNm
Interaction factors  0.83612 ≤ 1
0.94799 ≤ 1
Location: Axial load

Concrete filled CHS column

Calculations in accordance with British Steel (Corus) publication TD 296 SHS
Design Manual for Concrete Filled Columns Part 1: Structural Design

All loads and moments are factored

Factored BM about axis zz \( M_dz = 0 \) kNm
Factored SF in y direction \( N_v = 0 \) kN
Axial compressive load \( N_d = 5500 \) kN
Length of member \( L = 3.5 \) m
Maximum BM about y-y axis \( M_{dy} = 0 \) kNm
Modulus of elasticity \( E_s = 210 \) kN/mm²

Characteristic strength \( f_{cu} = 40 \) N/mm²
Length between restraints z-z axis \( L_z = 3.5 \) m
Length between restraints y-y axis \( L_y = 3.5 \) m
Effective length factor (z-axis) \( e_{fz} = 1 \)
Effective length factor (y-axis) \( e_{fy} = 1 \)

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
CIRCULAR HOLLOW
406.4 O.D x 12.5 CHS
SECTION WITH
CONCRETE INFILL
Cube strength \( 40 \) N/mm²
Contribution factor \( 0.29155 \)
Applied axial load \( 5500 \) kN
Squash resistance \( 7053.5 \) kN
Axial resistance \( 6841.9 \) kN
Location: Bending about one axis, other axis restrained

Concrete filled CHS column

Calculations in accordance
British Steel (Corus)
publication TD 296 SHS
Design Manual for Concrete Filled Columns Part 1:
Structural Design

All loads and moments are factored

Factored BM about axis zz \( M_{dz} = 50 \text{ kNm} \)
Factored SF in y direction \( N_{v} = 0 \text{ kN} \)
Axial compressive load \( N_{d} = 3000 \text{ kN} \)
Length of member \( L = 5 \text{ m} \)
Maximum BM about y-y axis \( M_{dy} = 0 \text{ kNm} \)
Modulus of elasticity \( E_{s} = 210 \text{ kN/mm}^2 \)

Characteristic strength \( f_{cu} = 40 \text{ N/mm}^2 \)
Length btwn restraints z-z axis \( L_{z} = 5 \text{ m} \)
Length btwn restraints y-y axis \( L_{y} = 0 \text{ m} \)
Effective length factor (z-axis) \( e_{fz} = 1 \)
Far end BM \( M_{dz1} = 50 \text{ kNm} \)

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
CIRCULAR HOLLOW
244.5 O.D x 20 CHS
SECTION WITH
CONCRETE INFILL
Cube strength \( 40 \text{ N/mm}^2 \)
Contribution factor \( 0.11778 \)
Applied axial load \( 3000 \text{ kN} \)
Squash resistance \( 5019.7 \text{ kN} \)
Axial resistance \( 3096 \text{ kN} \)
Applied moment (z-z) \( 50 \text{ kNm} \)
Moment capacity \( 327.48 \text{ kNm} \)
Location: Bending about both axes

Concrete filled CHS column

Calculations in accordance
British Steel (Corus)
publication TD 296 SHS
Design Manual for Concrete
Filled Columns Part 1:
Structural Design

All loads and moments are factored

Factored BM about axis zz \( M_{dz} = 100\) kNm
Factored SF in y direction \( N_v = 0\) kN
Axial compressive load \( N_d = 4000\) kN
Length of member \( L = 6\) m
Maximum BM about y-y axis \( M_{dy} = 50\) kNm
Modulus of elasticity \( E_s = 210\) kN/mm²

Characteristic strength \( f_{cu} = 40\) N/mm²
Length btwn restraints z-z axis \( L_z = 6\) m
Length btwn restraints y-y axis \( L_y = 6\) m
Effective length factor (z-axis) \( e_{fz} = 0.7\)
Effective length factor (y-axis) \( e_{fy} = 1\)
Far end BM \( M_{dz1} = 100\) kNm
Far end BM \( M_{dy1} = 0\) kNm

DESIGN SUMMARY

Section is satisfactory

HOT FINISHED
In accordance with EN 10210
CIRCULAR HOLLOW 355.6 O.D x 20 CHS
SECTION WITH CONCRETE INFILL
Cube strength 40 N/mm²
Contribution factor 0.1754
Applied axial load 4000 kN
Squash resistance 8028.2 kN
Axial resistance 5688.3 kN
Applied moment (z-z) 100 kNm
Applied moment (y-y) 50 kNm
Moment capacity 743.43 kNm
**Location:** Bending about one axis, other axis unrestrained

Concrete filled CHS column

Calculations in accordance with British Steel (Corus) publication TD 296 SHS Design Manual for Concrete Filled Columns Part 1: Structural Design

All loads and moments are factored

Factored BM about axis zz \( M_dz = 15 \text{ kNm} \)
Factored SF in y direction \( N_v = 0 \text{ kN} \)
Axial compressive load \( N_d = 890 \text{ kN} \)
Length of member \( L = 3 \text{ m} \)
Maximum BM about y-y axis \( M_{dy} = 0 \text{ kNm} \)
Modulus of elasticity \( E_s = 210 \text{ kN/mm}^2 \)

Characteristic strength \( f_{cu} = 40 \text{ N/mm}^2 \)
Length between restraints z-z axis \( L_z = 3 \text{ m} \)
Length between restraints y-y axis \( L_y = 3 \text{ m} \)
Effective length factor (z-axis) \( e_{fz} = 0.85 \)
Effective length factor (y-axis) \( e_{fy} = 1 \)
Far end BM \( M_{dz1} = -15 \text{ kNm} \)

**DESIGN SUMMARY**

Section is satisfactory
In accordance with EN 10210

HOT FINISHED
168.3 O.D x 5 CHS

CIRCULAR HOLLOW

SECTION WITH

CONCRETE INFILL

Cube strength \( 40 \text{ N/mm}^2 \)

Contribution factor \( 0.29948 \)

Applied axial load \( 890 \text{ kN} \)

Squash resistance \( 1182.9 \text{ kN} \)

Axial resistance \( 910.39 \text{ kN} \)

Applied moment (z-z) \( 15 \text{ kNm} \)

Moment capacity \( 46.993 \text{ kNm} \)
Location: Ex1 - CHS concrete filled column with axial load

Concrete filled CHS column


All loads and moments are factored.

Design moment about major axis \( M_{Ed} = 0 \) kNm
Design SF in z direction \( V_{zEd} = 0 \) kN
Design axial compressive load \( N_{Ed} = 5500 \) kN
Length of member \( L = 3.5 \) m
Design moment about minor axis \( M_{zEd} = 0 \) kNm

406.4 dia x 12.5 thick CHS - Hot finished.
Properties (cm): \( A = 154.68 \) iy = 13.933 \( W_{ely} = 1477.9 \) \( W_{ply} = 1940.1 \) \( I_y = 30031 \) \( I_t = 60061 \)
Steel \( \gamma_a \) \( gama = 1 \)
Concrete \( \gamma_c \) \( gamc = 1.5 \)

Length btwn restraints y-y axis \( L_y = 3.5 \) m
Length btwn restraints z-z axis \( L_z = 3.5 \) m
Effective length factor y-y axis \( K_y = 1 \)
Effective length factor z-z axis \( K_z = 1 \)

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
CIRCULAR HOLLOW SECTION
406.4 O.D x 12.5 CHS
WITH CONCRETE INFILL (NWC)
Cylinder strength 32 N/mm²
Design axial load 5500 kN
Column axial resistance 7586.4 kN
Location: Ex2 - Bending about one axis, other axis restrained

Concrete filled CHS column


All loads and moments are factored.

Design moment about major axis $M_{Ed}=50$ kNm
Design SF in z direction $V_{zEd}=10$ kN
Design axial compressive load $N_{Ed}=2500$ kN
Length of member $L=5$ m
Design moment about minor axis $M_{zEd}=0$ kNm

244.5 dia x 16 thick CHS - Hot finished.
Properties (cm): $A=114.86$ $iy=8.0985$ $W_{ely}=616.19$ $W_{ply}=836.76$ $I_y=7532.9$ $I_t=15066$
Permanent design load $N_{gEd}=1800$
Steel $γ_a$ $gama=1$
Concrete $γ_c$ $gammc=1.5$

Length between restraints y-y axis $L_y=5$ m
Length between restraints z-z axis $L_z=0$ m
Effective length factor y-y axis $K_y=1$
Effective length factor z-z axis $K_z=1$
Far end BM $M_{yEd1}=0$ kNm

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
CIRCULAR HOLLOW SECTION
244.5 O.D x 16 CHS
WITH CONCRETE INFILL (NWC)
Cylinder strength $32$ N/mm$^2$
Design axial load $2500$ kN
Design moment y-y axis $130.86$ kNm
Moment capacity y-y axis $178.3$ kNm
Location: Ex3 - Bending about both axes

Concrete filled CHS column


All loads and moments are factored.

Design moment about major axis \( M_y \) = 100 kNm
Design SF in z direction \( V_z \) = 16.67 kN
Design axial compressive load \( N_E \) = 4000 kN
Length of member \( L \) = 6 m
Design moment about minor axis \( M_z \) = 50 kNm

355.6 dia x 16 thick CHS - Hot finished.
Properties (cm): \( A = 170.7 \) \( \sqrt{y} = 12.02 \) \( W_{ely} = 1387.1 \) \( W_{ply} = 1846.6 \) \( I_y = 24663 \) \( I_t = 49326 \)
Permanent design load \( N_g \) = 3000
Steel \( \gamma_a \) = 1
Concrete \( \gamma_c \) = 1.5

Length between restraints y-y axis \( L_y \) = 6 m
Length between restraints z-z axis \( L_z \) = 6 m
Effective length factor y-y axis \( K_y \) = 0.7
Effective length factor z-z axis \( K_z \) = 1
Far end BM \( M_{y_1} \) = 100 kNm
Far end BM \( M_{z_1} \) = 0 kNm

DESIGN SUMMARY

Section is satisfactory
HOT FINISHED
CIRCULAR HOLLOW SECTION
WITH CONCRETE INFILL (NWC)

Cylinder strength \( f_c \) = 32 N/mm²
Design axial load \( N_E \) = 4000 kN
Design moment y-y axis \( M_y \) = 100 kNm
Moment capacity y-y axis \( M_{y_c} \) = 442.38 kNm
Design moment z-z axis \( M_z \) = 50 kNm
Moment capacity z-z axis \( M_{z_c} \) = 442.38 kNm
Interaction factors \( \frac{M_y}{M_{y_c}} \leq 1 \) \( \frac{M_z}{M_{z_c}} \leq 1 \)
Location: Ex4 - Bending about one axis, other axis unrestrained

Concrete filled CHS column


All loads and moments are factored.

Design moment about major axis \( M_{yEd} = 15 \text{ kNm} \)
Design SF in z direction \( V_{zEd} = 5.4 \text{ kN} \)
Design axial compressive load \( N_{Ed} = 890 \text{ kN} \)
Length of member \( L = 2.8 \text{ m} \)
Design moment about minor axis \( M_{zEd} = 0 \text{ kNm} \)

168.3 dia x 5 thick CHS - Hot finished.
Properties (cm): \( A = 25.651 \text{ cm}^2 \)
\( I_y = 5.7762 \text{ cm}^4 \)
\( W_{ely} = 101.7 \text{ cm}^3 \)
\( W_{ply} = 133.38 \text{ cm}^3 \)
\( I_y = 855.85 \text{ cm}^4 \)
\( I_t = 1711.7 \text{ cm}^4 \)

Permanent design load \( N_{GEd} = 600 \text{ kN} \)
Steel \( \gamma_a \) \( = 1 \)
Concrete \( \gamma_c \) \( = 1.5 \)

Length between restraints y-y axis \( L_y = 2.8 \text{ m} \)
Length between restraints z-z axis \( L_z = 2.8 \text{ m} \)
Effective length factor y-y axis \( K_y = 0.85 \)
Effective length factor z-z axis \( K_z = 1 \)
Far end BM \( M_{yEd1} = -15 \text{ kNm} \)

DESIGN SUMMARY
Section is satisfactory
HOT FINISHED
In accordance with EN 10210
CIRCULAR HOLLOW SECTION
168.3 O.D x 5 CHS
WITH CONCRETE INFILL (NWC)
Cylinder strength \( = 32 \text{ N/mm}^2 \)
Design axial load \( = 890 \text{ kN} \)
Design moment y-y axis \( = 20.899 \text{ kNm} \)
Moment capacity y-y axis \( = 25.297 \text{ kNm} \)
Location: Ex1 - 300 x 200 x 10 with bar reinforcement

SHS Concrete Filled Columns

Calculations in accordance with British Steel (Corus) publication TD 296 SHS Design Manual for Concrete Filled Columns Part 2: Fire Resistant Design.

All loads and moments are factored, column assumed to be in simple construction.

No external fire protection is provided, protection based on concrete infill.

Axial compressive load \( F_f = 325 \text{ kN} \)
Factored BM about x-x axis \( M_{fx} = 23 \text{ kNm} \)
Maximum BM about y-y axis \( M_{fy} = 2 \text{ kNm} \)
Length of member \( L = 4.5 \text{ m} \)
300 x 200 x 10 RHS - Hot finished.
Properties (cm): \( A = 94.9 \) \( rx = 11.2 \) \( Zx = 788 \) \( Sx = 956 \) \( Ix = 11800 \)
\( J = 12900 \) \( C = 1020 \) \( Zy = 628 \) \( Sy = 721 \) \( Iy = 6280 \) \( ry = 8.13 \)
Young's Modulus \( E = 210 \text{ kN/mm}^2 \)
Characteristic strength \( f_{cu} = 40 \text{ N/mm}^2 \)
Length btwn restraints x-x axis \( Lx = 4.5 \text{ m} \)
Length btwn restraints y-y axis \( Ly = 4.5 \text{ m} \)
Effective length factor x-x axis \( e_{fx} = 1 \)
Effective length factor y-y axis \( e_{fy} = 1 \)
Reinforcing bar diameter \( d = 25 \text{ mm} \)
Yield strength \( f_{yr} = 425 \text{ N/mm}^2 \)
Modulus of elasticity (bars) \( E_r = 210 \text{ kN/mm}^2 \)

SUMMARY
Section is satisfactory
HOT FINISHED
RECTANGULAR HOLLOW SECTION
300 x 200 x 10 RHS Grade S 355
WITH CONCRETE INFILL
Cube strength \( 40 \text{ N/mm}^2 \)
Period of protection 60 minutes
Axial load 325 kN
Applied moment (x-x) 23 kNm
Applied moment (y-y) 2 kNm
Effective length (x-x) 4.5 m
Effective length (y-y) 4.5 m
Squash resistance 2071.5 kN
Load ratio 0.42309
Limiting load ratio 0.509
Reinforcement:
4/25 mm diameter
Yield strength 425 N/mm²
cover 25 mm
Location: Ex2 - 300 x 300 x 8 SHS with plain/bar reinforcement

SHS Concrete Filled Columns

Calculations in accordance with British Steel (Corus) publication TD 296 SHS Design Manual for Concrete Filled Columns Part 2: Fire Resistant Design.

All loads and moments are factored, column assumed to be in simple construction.

No external fire protection is provided, protection based on concrete infill.

Axial compressive load \( F_f = 455 \) kN
Factored BM about x-x axis \( M_{fx} = 34 \) kNm
Maximum BM about y-y axis \( M_{fy} = 8 \) kNm
Length of member \( L = 4.8 \) m
300 x 300 x 8 SHS - Hot finished.
Properties (cm): \( A = 92.8 \) rz=\( 11.9 \) Zz=\( 875 \) Sz=\( 1010 \) Iz=\( 13100 \) J=\( 20200 \) C=\( 1290 \)
(\( ry = rx, Zy = Zx, Sy = Sx \) and \( Iy = Ix \))
Young's Modulus \( E = 210 \) kN/mm\(^2\)
Characteristic strength \( f_{cu} = 40 \) N/mm\(^2\)
Length btwn restraints x-x axis \( L_x = 4.8 \) m
Length btwn restraints y-y axis \( L_y = 4.8 \) m
Effective length factor x-x axis \( e_{fx} = 0.85 \)
Effective length factor y-y axis \( e_{fy} = 1 \)
Reinforcing bar diameter \( d = 25 \) mm
Yield strength \( f_{yr} = 460 \) N/mm\(^2\)
Modulus of elasticity (bars) \( E_r = 210 \) kN/mm\(^2\)

SUMMARY
Section is satisfactory
In accordance with EN 10210
HOT FINISHED
SQUARE HOLLOW SECTION
300 x 300 x 8 SHS Grade S 355
WITH CONCRETE INFILL
Cube strength \( 40 \) N/mm\(^2\)
Period of protection 60 minutes
Axial load \( 455 \) kN
Applied moment (x-x) \( 34 \) kNm
Applied moment (y-y) \( 8 \) kNm
Effective length (x-x) \( 4.08 \) m
Effective length (y-y) \( 4.8 \) m
Squash resistance \( 2912.9 \) kN
Load ratio 0.34513
Limiting load ratio 0.509
Reinforcement:
4/25 mm diameter
Yield strength \( 460 \) N/mm\(^2\)
cover \( 25 \) mm
**Location: Ex3 - Circular section with plain infill**

**SHS Concrete Filled Columns**

Calculations in accordance with British Steel (Corus) publication TD 296 SHS Design Manual for Concrete Filled Columns Part 2: Fire Resistant Design.

All loads and moments are factored, column assumed to be in simple construction.

No external fire protection is provided, protection based on concrete infill.

Axial compressive load \( F_f = 670 \text{ kN} \)

Factored BM about \( x-x \) axis \( M_{fx} = 14 \text{ kNm} \)

Maximum BM about \( y-y \) axis \( M_{fy} = 14 \text{ kNm} \)

Length of member \( L = 4.75 \text{ m} \)

457 dia x 8 thick CHS - Hot finished.

Properties (cm): \( A = 112.85 \) \( r = 15.877 \) \( Z = 1244.9 \) \( S = 1613 \) \( I = 28446 \)

Young's Modulus \( E = 210 \text{ kN/mm}^2 \)

Characteristic strength \( f_{cu} = 40 \text{ N/mm}^2 \)

Length between restraints \( x-x \) axis \( L_x = 4.75 \text{ m} \)

Length between restraints \( y-y \) axis \( L_y = 4.75 \text{ m} \)

Effective length factor \( x-x \) axis \( e_{fx} = 0.85 \)

Effective length factor \( y-y \) axis \( e_{fy} = 0.85 \)

**SUMMARY**

Section is satisfactory

HOT FINISHED

In accordance with EN 10210

CIRCULAR HOLLOW SECTION

457 O.D. x 8 CHS Grade S 355

WITH CONCRETE INFILL

Cube strength \( 40 \text{ N/mm}^2 \)

Period of protection \( 90 \text{ minutes} \)

Axial load \( 670 \text{ kN} \)

Applied moment \( x-x \) \( 14 \text{ kNm} \)

Applied moment \( y-y \) \( 14 \text{ kNm} \)

Effective length \( x-x \) \( 4.0375 \text{ m} \)

Effective length \( y-y \) \( 4.0375 \text{ m} \)

Squash resistance \( 3900.9 \text{ kN} \)

Load ratio \( 0.28977 \)

Limiting load ratio \( 0.397 \)
Location: Ex1 - SCI P259 Example 1 (355.6 x 8 CHS with r'ment)

Calculations are in accordance with EN 1994-1-2 and SCI publication P259 entitled "Fire Resistance of Concrete Filled Tubes to EC4" or SCI publication P159.

No external fire protection is provided, fire protection is based on concrete infill.

Axial compressive load \( N_f = 2000 \) kN
Maximum BM about y-y axis \( M_{fy} = 60 \) kNm
Maximum BM about z-z axis \( M_{fz} = 10 \) kNm
Length of member \( L = 3 \) m

355.6 dia x 8 thick CHS - Hot finished.
Properties (cm): \( A = 87.361 \) \( iy = 12.293 \) \( W_{ely} = 742.48 \) \( W_{ply} = 966.78 \) \( I_y = 13201 \) \( I_t = 26403 \)

Concrete filled CHS column

All loads and moments are factored, column is assumed to be in simple construction.

\( D = 355.6 \) mm
\( t = 8 \) mm

Cylinder characteristic strength \( f_{ck} = 35 \) N/mm²

Effective lengths

Length btwn restraints y-y axis \( L_y = 3 \) m
Length btwn restraints z-z axis \( L_z = 3 \) m
Effective length factor y-y axis \( e_{fy} = 0.7 \)
Effective length factor z-z axis \( e_{fz} = 0.7 \)
Buckling length of column \( l = 2.1 \) m

Moment resistance, stiffness and squash load - reinforced concrete

Provide four main bars with link reinforcement at least a quarter the diameter of the main longitudinal bars.
Buckling resistance (NfIRd)

Critical Euler buckling load  \( N_{f,icr} = \pi^2 \frac{E I}{l^2} = 34828 \) kN
Non dimensional slenderness \( \lambda^0 \)  \( \lambda^0 = \sqrt{\frac{N_{f,Rd}}{N_{f,icr}}} = 0.3449 \)
Reduction coefficient \( \chi \)  \( \chi = \frac{1}{\phi + (\phi^2 - \lambda^0)^{0.5}} = 0.9261 \)
Buckling resistance  \( N_{f,Rd} = \chi N_{f,ipRd} = 3836.9 \) kN

Slenderness check

Consider y-y axis:
Minimum column moment  \( M_{2y} = -30 \) kNm
Slenderness modification factor  \( k_y = 1 \)
Consider z-z axis:
Minimum column moment  \( M_{2z} = 0 \) kNm
Slenderness modification factor  \( k_z = 1 \)

Interaction of axial loads and moments

The interaction of axial compression and bending is checked using the following expression (SCI P259 Section 3.5):

Unity expression  \( R = \frac{N_f}{N_{f,Rd} + k_y M_{fy} / M_{f,Rd} + k_z M_{fz} / M_{f,Rd}} = 0.90588 \)

As \( R \leq 1 \), section capacity is sufficient.

SUMMARY
Section is satisfactory
HOT FINISHED  In accordance with BS EN 10210
CIRCULAR HOLLOW SECTION 355.6 O.D. x 8 CHS Grade S 355
WITH CONCRETE INFILL
Cylinder strength 35 N/mm²
Period of protection 60 minutes
Axial load 2000 kN
Effective length (y-y) 2.1 m
Effective length (z-z) 2.1 m
Applied moment (y-y) 60 kNm
Applied moment (z-z) 10 kNm
Moment resistance 182 kN
Squash resistance 4143 kN
Buckling resistance 3836.9 kN
Reinforcement:
4/32 mm diameter (use HY bars)
Yield strength 500 N/mm²
Axis distance 50 mm
Minimum link size 8 mm
Maximum link spacing 480 mm
Percentage of r'ment 3.5516 %
Location: Ex2 - SCI P259 Example 2 (350 x 350 x 8 SHS with r'ment)

Calculations are in accordance with EN 1994-1-2 and SCI publication P259 entitled "Fire Resistance of Concrete Filled Tubes to EC4" or SCI publication P159.

No external fire protection is provided, fire protection is based on concrete infill.

Axial compressive load \( N_f = 2000 \text{ kN} \)
Maximum BM about y-y axis \( M_{fy} = 40 \text{ kNm} \)
Maximum BM about z-z axis \( M_{fz} = 10 \text{ kNm} \)
Length of member \( L = 3 \text{ m} \)
350 x 350 x 8 SHS - Hot finished.
Properties (cm): \( A = 109 \) \( i_y = 13.9 \) \( W_{ely} = 1210 \) \( W_{ply} = 1390 \) \( I_y = 21100 \) \( I_t = 32400 \) \( W_t = 1790 \)

Concrete Filled SHS Column

All loads and moments are factored, column is assumed to be in simple construction.

\( h = 350 \text{ mm} \)
\( b = 350 \text{ mm} \)
\( t = 8 \text{ mm} \)

Cylinder characteristic strength \( f_{ck} = 35 \text{ N/mm}^2 \)

Effective lengths

Length btwn restraints y-y axis \( L_y = 3 \text{ m} \)
Length btwn restraints z-z axis \( L_z = 3 \text{ m} \)
Effective length factor y-y axis \( e_{fy} = 1 \)
Effective length factor z-z axis \( e_{fz} = 1 \)
Buckling length of column \( l = 3 \text{ m} \)

Moment resistance, stiffness and squash load - reinforced concrete

Provide four main bars with link reinforcement at least a quarter the diameter of the main longitudinal bars.
Buckling resistance (NfIRD)

Critical Euler buckling load          \( Nficr = \pi^2 \cdot \frac{EI}{l^2} = 24808 \text{ kN} \)
Non dimensional slenderness \( \lambda \)          \( \lambda = \sqrt{\frac{N_{fird}}{N_{ficr}}} = 0.41009 \)
Reduction coefficient \( \chi \)          \( \chi = \frac{1}{\phi + (\phi^2 - \lambda^2)^{0.5}} = 0.89197 \)
Buckling resistance          \( N_{fird} = \chi \cdot N_{fiprd} = 3721.3 \text{ kN} \)

Slenderness check

Consider y-y axis:
Minimum column moment          \( M_{2y} = -20 \text{ kNm} \)
Slenderness modification factor          \( k_y = 1 \)
Consider z-z axis:
Minimum column moment          \( M_{2z} = 0 \text{ kNm} \)
Slenderness modification factor          \( k_z = 1 \)

Interaction of axial loads and moments

The interaction of axial compression and bending is checked using the following expression (SCI P259 Section 3.5):

Unity expression          \( R = \frac{N_f}{N_{fird}} + k_y \cdot \frac{M_{fy}}{M_{fird}} + k_z \cdot \frac{M_{fz}}{M_{fird}} = 0.86149 \)

As \( R \leq 1 \), section capacity is sufficient.

SUMMARY
Section is satisfactory
HOT FINISHED          In accordance with BS EN 10210
SQUARE HOLLOW SECTION          350 x 350 x 8 SHS Grade S 355
WITH CONCRETE INFILL
Cylinder strength          35 N/mm^2
Period of protection          60 minutes
Axial load          2000 kN
Effective length (y-y)          3 m
Effective length (z-z)          3 m
Applied moment (y-y)          40 kNm
Applied moment (z-z)          10 kNm
Moment resistance          154.3 kN
Squash resistance          4172 kN
Buckling resistance          3721.3 kN
Reinforcement:
4/25 mm diameter (use HY bars)
Yield strength          500 N/mm^2
Axis distance          40 mm
Minimum link size          8 mm
Maximum link spacing          375 mm
Percentage of r'ment          2.241 %
Location: Ex3 - CHS 457 x 10 CHS with r'ment and 90 min FR

Calculations are in accordance with EN 1994-1-2 and SCI publication P259 entitled "Fire Resistance of Concrete Filled Tubes to EC4" or SCI publication P159.

No external fire protection is provided, fire protection is based on concrete infill.

Axial compressive load \( N_f = 3500 \text{ kN} \)
Maximum BM about y-y axis \( M_{fy} = 60 \text{ kNm} \)
Maximum BM about z-z axis \( M_{fz} = 10 \text{ kNm} \)
Length of member \( L = 3 \text{ m} \)
457 dia x 10 thick CHS - Hot finished.
Properties (cm): \( A = 140.43 \) \( l_y = 15.808 \) \( W_{el} = 1535.7 \) \( W_{pl} = 1998.4 \) \( I_y = 35091 \) \( I_t = 70183 \)

Cylinder characteristic strength \( f_{ck} = 35 \text{ N/mm}^2 \)

Effective lengths

Length btwn restraints y-y axis \( L_y = 3 \text{ m} \)
Length btwn restraints z-z axis \( L_z = 3 \text{ m} \)
Effective length factor y-y axis \( e_{fy} = 0.7 \)
Effective length factor z-z axis \( e_{fz} = 0.7 \)
Buckling length of column \( l = 2.1 \text{ m} \)

Moment resistance, stiffness and squash load - reinforced concrete

Provide four main bars with link reinforcement at least a quarter the diameter of the main longitudinal bars.
Buckling resistance (NfiRd)

Critical Euler buckling load

Non dimensional slenderness 

Reduction coefficient 

Buckling resistance

Slenderness check

Consider y-y axis:
Minimum column moment
Slenderness modification factor
Consider z-z axis:
Minimum column moment
Slenderness modification factor

Interaction of axial loads and moments

The interaction of axial compression and bending is checked using the following expression (SCI P259 Section 3.5):

Unity expression

As R ≤ 1, section capacity is sufficient.

SUMMARY

HOT FINISHED
CIRCULAR HOLLOW SECTION
WITH CONCRETE INFILL

Section is satisfactory
In accordance with BS EN 10210
457 O.D. x 10 CHS Grade S 355
Cylinder strength
Period of protection
Axial load
Effective length (y-y)
Effective length (z-z)
Applied moment (y-y)
Applied moment (z-z)
Moment resistance
Squash resistance
Buckling resistance
Reinforcement:
4/32 mm diameter (use HY bars)
Yield strength
Axis distance
Minimum link size
Maximum link spacing
Percentage of r'ment
**Location: Ex4 - CHS with plain infill**

Calculations are in accordance with EN 1994-1-2 and SCI publication P259 entitled "Fire Resistance of Concrete Filled Tubes to EC4" or SCI publication P159.

No external fire protection is provided, fire protection is based on concrete infill.

Axial compressive load $N_f = 670 \text{kN}$
Maximum BM about $y$-$y$ axis $M_{fy} = 14 \text{kNm}$
Maximum BM about $z$-$z$ axis $M_{fz} = 14 \text{kNm}$
Length of member $L = 5 \text{m}$
457 dia x 10 thick CHS - Hot finished.
Properties (cm): $A = 140.43$ $i_y = 15.808$ $W_{ely} = 1535.7$ $W_{ply} = 1998.4$ $I_y = 35091$ $I_t = 70183$

Cylinder characteristic strength $f_{ck} = 35 \text{N/mm}^2$

**Effective lengths**

Length btwn restraints $y$-$y$ axis $L_y = 5 \text{m}$
Length btwn restraints $z$-$z$ axis $L_z = 5 \text{m}$
Effective length factor $y$-$y$ axis $e_{fy} = 0.85$
Effective length factor $z$-$z$ axis $e_{fz} = 0.85$
Buckling length of column $l = 4.25 \text{m}$

**Load level $\eta fi$ in fire - plain concrete infill**

Squash load $N_{Rd} = 7846 \text{kN}$

**Non-dimensional slenderness**

Critical Euler buckling load $N_{ficr} = \pi^2 * E * I / l^2 = 25017 \text{kN}$
Non dimensional slenderness $\lambda_0 = \text{SQR}(N_{ficr}/N_{Rd}) = 0.49343$
Slenderness check

Consider y-y axis:
Minimum column moment \( M_{2y} = -7 \text{ kNm} \)
Slenderness modification factor \( k_y = 1 \)

Consider z-z axis:
Minimum column moment \( M_{2z} = 0 \text{ kNm} \)
Slenderness modification factor \( k_z = 1 \)

Interaction of axial loads and moments

The interaction of axial compression and bending is checked using the following expression (SCI P259 Section 3.5):

Unity expression \( R = \frac{N_f}{N_{fi}}R_d + k_y \frac{M_{fy}}{M_{fi}}R_d + k_z \frac{M_{fz}}{M_{fi}}R_d = 0.3964 \)

As \( R \leq 1 \), section capacity is sufficient.

**SUMMARY**

<table>
<thead>
<tr>
<th>HOT FINISHED</th>
<th>In accordance with BS EN 10210</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIRCULAR HOLLOW SECTION</td>
<td>457 O.D. x 10 CHS Grade S 355</td>
</tr>
<tr>
<td>WITH CONCRETE INFILL</td>
<td>Cylinder strength ( 35 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td></td>
<td>Period of protection ( 60 \text{ minutes} )</td>
</tr>
<tr>
<td></td>
<td>Axial load ( 670 \text{ kN} )</td>
</tr>
<tr>
<td></td>
<td>Effective length (y-y) ( 4.25 \text{ m} )</td>
</tr>
<tr>
<td></td>
<td>Effective length (z-z) ( 4.25 \text{ m} )</td>
</tr>
</tbody>
</table>
Location: Ex5 - SHS with plain infill

Calculations are in accordance with EN 1994-1-2 and SCI publication P259 entitled "Fire Resistance of Concrete Filled Tubes to EC4" or SCI publication P159.

No external fire protection is provided, fire protection is based on concrete infill.

Axial compressive load \( N_f = 670 \text{ kN} \)
Maximum BM about y-y axis \( M_{fy} = 14 \text{ kNm} \)
Maximum BM about z-z axis \( M_{fz} = 14 \text{ kNm} \)
Length of member \( L = 5 \text{ m} \)

400 x 400 x 10 SHS - Hot finished.
Properties (cm): \( A = 155 \) \( i_y = 15.9 \) \( W_{ely} = 1960 \) \( W_{ply} = 2260 \) \( I_y = 39100 \) \( I_t = 60100 \) \( W_t = 2900 \)

Concrete Filled SHS Column

All loads and moments are factored, column is assumed to be in simple construction.

\( h = 400 \) mm
\( b = 400 \) mm
\( t = 10 \) mm

Cylinder characteristic strength \( f_{ck} = 35 \text{ N/mm}^2 \)

Effective lengths

Length between restraints y-y axis \( L_y = 5 \text{ m} \)
Length between restraints z-z axis \( L_z = 5 \text{ m} \)
Effective length factor y-y axis \( \varepsilon_{fy} = 0.85 \)
Effective length factor z-z axis \( \varepsilon_{fz} = 0.85 \)
Buckling length of column \( l = 4.25 \text{ m} \)

Load level \( \eta_f \) in fire - plain concrete infill

Squash load \( N_{Rd} = 8095 \text{ kN} \)

Non-dimensional slenderness

Critical Euler buckling load \( N_{fcr} = \frac{\pi^2 E I}{l^2} = 22342 \text{ kN} \)
Non-dimensional slenderness \( \lambda_0 \) \( \lambda_{mt} = \sqrt{\frac{N_{fcr} R_d}{N_{fcr}}} = 0.4871 \)

Slenderness check

Consider y-y axis:
Minimum column moment \( M_{2y} = -7 \text{ kNm} \)
Slenderness modification factor \( k_y = 1 \)
Consider z-z axis:
Minimum column moment $M_{2z} = 0 \text{ kNm}$
Slenderness modification factor $k_z = 1$

**Interaction of axial loads and moments**

The interaction of axial compression and bending is checked using the following expression (SCI P259 Section 3.5):

Unity expression $R = N_f/N_{fiRd} + k_y*M_{fy}/M_{fiRd} + k_z*M_{fz}/M_{fiRd} = 0.43196$

As $R \leq 1$, section capacity is sufficient.

**SUMMARY**

- **Section is satisfactory**
- **In accordance with BS EN 10210**
- **400 x 400 x 10 SHS Grade S 355**
- **Cylinder strength** $35 \text{ N/mm}^2$
- **Period of protection** 60 minutes
- **Axial load** $670 \text{ kN}$
- **Effective length (y-y)** 4.25 m
- **Effective length (z-z)** 4.25 m
Calculations are in accordance with 'Joints in Steel Construction Moment Connections' published by The Steel Construction Institute.

Axial load (+ve compression) \( N = 1300 \) kN
M Moment about X-X axis \( M = 150 \) kNm
Shear on the base in Y direction \( F_y = 80 \) kN

250 x 150 x 16 RHS - Hot finished.
Properties (cm): \( A = 115 \), \( rx = 8.79 \), \( Z_x = 710 \), \( S_x = 906 \), \( I_x = 8880 \),
\( J = 8870 \), \( C = 849 \), \( Z_y = 516 \), \( S_y = 625 \), \( I_y = 3870 \), \( r_y = 5.8 \)

Length of baseplate \( hp = 450 \) mm
Breadth of baseplate \( bp = 350 \) mm
Edge distance to bolt centre line \( k = 50 \) mm
Assumed fillet weld size \( sw = 8 \) mm

Strength of concrete \( f_{cu} = 30 \) N/mm²
Special control must be applied over the placing of the high strength bedding material.
Assumed weld size \( sw = 8 \) mm
Selected baseplate thickness \( tp = 40 \) mm

Number of bolts to be used \( n = 4 \)
Bolt diameter \( bd = 20 \) mm
Selected fillet weld size \( sw = 8 \) mm
<table>
<thead>
<tr>
<th><strong>SUMMARY</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>BASEPLATE</strong></td>
<td>Size 450 mm x 350 mm x 40 mm</td>
</tr>
<tr>
<td><strong>REQUIREMENTS</strong></td>
<td>Grade S 355 steel</td>
</tr>
<tr>
<td>Edge distance</td>
<td>50 mm</td>
</tr>
<tr>
<td>Number of H.D. bolts</td>
<td>4</td>
</tr>
<tr>
<td>Diameter of bolts</td>
<td>M 20</td>
</tr>
<tr>
<td>Grade 8.8</td>
<td></td>
</tr>
<tr>
<td>Concrete/grout (fcu)</td>
<td>30 N/mm²</td>
</tr>
<tr>
<td><strong>WELDS</strong></td>
<td>Fillet weld (all round) 8 mm</td>
</tr>
<tr>
<td>Contact areas on the baseplate and column are machined to give a tight bearing contact.</td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex2 - Large moment SHS

Calculations are in accordance with 'Joints in Steel Construction Moment Connections' published by The Steel Construction Institute.

Axial load (+ve compression)    \( N=700 \text{ kN} \)
Moment about X-X axis           \( M=289 \text{ kNm} \)
Shear on the base in Y direction \( F_y=96 \text{ kN} \)

300 x 300 x 12.5 SHS - Hot finished.
Properties (cm): \( A=142 \), \( r_x=11.7 \), \( Z_x=1300 \), \( S_x=1530 \), \( I_x=19400 \), \( J=30300 \), \( C=1900 \)
Length of baseplate \( h_p=500 \text{ mm} \)
Breadth of baseplate \( b_p=500 \text{ mm} \)
Edge distance to bolt centre line \( k=50 \text{ mm} \)
Assumed fillet weld size \( s_w=15 \text{ mm} \)
Strength of concrete \( f_{cu}=50 \text{ N/mm}^2 \)
Special control must be applied over the placing of the high strength bedding material.
Assumed weld size \( s_w=15 \text{ mm} \)
Selected baseplate thickness \( t_p=45 \text{ mm} \)

Total number of bolts to be used \( n=6 \)
Assumed diameter of bolt \( b_d=24 \text{ mm} \)
Overall embedded depth \( L_o=450 \text{ mm} \)
Assumed cover to reinforcement \( c_v=40 \text{ mm} \)
tension reinforcement \( A_{sp}=0.2 \% \)
Selected fillet weld size \( s_w=15 \text{ mm} \)
SUMMARY

BASEPLATE

Size 500 mm x 500 mm x 45 mm
Grade S 275 steel

REQUIREMENTS

Edge distance 50 mm
Number of H.D. bolts 6
Diameter of bolts M 24
Grade 8.8
Overall embedded depth 450 mm
Anchor plate size: 120 mm x 120 mm x 20 mm
Concrete/grout (fcu) 50 N/mm²

WELDS

Fillet weld (all round) 15 mm
Contact areas on the baseplate and column are machined to give a tight bearing contact.
Location: Ex1 - Loads and section from TD 296

Calculations are in accordance with EC3 Parts 1-1, 1-8 and publication entitled 'Joints in Steel Construction: Simple Joints to EC3' by SCI.

Axial load (+ve compression) \( N_{Ed} = 1300 \text{kN} \)
Moment about y-y axis \( M_{yEd} = 150 \text{kNm} \)
Shear on the base in z direction \( V_{cEd} = 80 \text{kN} \)

250 x 150 x 16 RHS - Hot finished.
Properties (cm): \( A=115 \) iz=5.8 \( W_{ely}=710 \) \( W_{ply}=906 \) \( I_y=8880 \)
\( I_t=8870 \) \( W_{elz}=516 \) \( W_{plz}=625 \) \( I_z=3870 \) iy=8.79
Length of baseplate \( h_p=450 \text{ mm} \)
Breadth of baseplate \( b_p=350 \text{ mm} \)
Edge distance to bolt centre line \( k=50 \text{ mm} \)
Characteristic cylinder strength \( f_{ck}=40 \text{ N/mm}^2 \)
Special control must be applied over the placing of the high strength bedding material.
Selected baseplate thickness \( t_p=40 \text{ mm} \)
Number of bolts to be used \( n=4 \)
Bolt diameter \( b_d=20 \)
Selected fillet weld size \( s_w=8 \text{ mm} \)
## SUMMARY

**BASEPLATE**
Size 450 mm x 350 mm x 40 mm

**REQUIREMENTS**
- Grade S 355 steel
- Edge distance 50 mm
- Number of H.D. bolts 4
- Diameter of bolts 20 mm
- Grade 8.8
- Concrete/grout (fck) 40 N/mm²

**WELDS**
- Fillet weld (all round) 8 mm

Contact areas on the baseplate and column are machined to give a tight bearing contact.

**NOTE:** The surfaces of baseplate and end of column are assumed to be machined square to give bearing contact.
Calculations are in accordance with EC3 Parts 1-1, 1-8 and publication entitled 'Joints in Steel Construction: Simple Joints to EC3' by SCI. 

Axial load (+ve compression) \( \text{NEd}=700 \text{ kN} \)
Moment about y-y axis \( \text{MyEd}=289 \text{ kNm} \)
Shear on the base in z direction \( \text{VcEd}=96 \text{ kN} \)

300 x 300 x 12.5 SHS - Hot finished.
Properties (cm): \( A=142 \) \( iy=11.7 \) \( Wely=1300 \) \( Wply=1530 \) \( Iy=19400 \) \( It=30300 \) \( Wt=1900 \)

Length of baseplate \( hp=500 \text{ mm} \)
Breadth of baseplate \( bp=500 \text{ mm} \)
Edge distance to bolt centre line \( k=50 \text{ mm} \)
Characteristic cylinder strength \( fck=32 \text{ N/mm}^2 \)
Special control must be applied over the placing of the high strength bedding material.
Selected baseplate thickness \( tp=40 \text{ mm} \)
Total number of bolts to be used \( n=6 \)
Assumed diameter of bolt \( bd=24 \text{ mm} \)
Overall embedded depth \( Lo=450 \text{ mm} \)
Assumed cover to reinforcement \( cv=40 \text{ mm} \)
Assumed percentage area of tension reinforcement \( Asp=0.2 \% \)
Punching shear force \( VEd=\text{ABS}(\text{Ft1Ed})=490.73 \text{ kN} \)
Shear stress at control perimeter \( \text{vEd}=\beta \times \text{VEd} 	imes 1000 / (u1 \times \text{dav})=0.222 \text{ N/mm}^2 \)
Design punching shear resistance \( \text{vRdc}=\sqrt{0.18 / \text{gamc}} \times k \times (100 \times \text{p1} \times fck)^{0.333} =0.37817 \text{ N/mm}^2 \)
Shear enhancement \( 2d/\text{av} \) \( \text{senh}=1 \)
The min design punching shear resistance governs and will be adopted.
Design punching shear resistance \( \text{vRdc}=\text{senh} \times 0.035 \times k \times 1.5 \times fck \times 0.5 =0.43824 \text{ N/mm}^2 \)
As \( vEd \leq vRdc \) ( \( 0.222 \text{ N/mm}^2 \leq 0.43824 \text{ N/mm}^2 \) ), the anchorage arrangement is considered suitable.
## SUMMARY

**BASEPLATE**
- Size 500 mm x 500 mm x 40 mm

**REQUIREMENTS**
- Grade S 275 steel
- Edge distance 1.7 mm
- Number of H.D. bolts 6
- Diameter of bolts 24 mm
- Grade 8.8
- Overall embedded depth 450 mm
- Anchor plate size: 120 mm x 120 mm x 20 mm
- Concrete/grout (fck) 32 N/mm²

**WELDS**
- Fillet weld (all round) 15 mm

Contact areas on the baseplate and column are machined to give a tight bearing contact.

NOTE: The surfaces of baseplate and end of column are assumed to be machined square to give bearing contact.
Location: Ex1 - Simply supported beam

Beam sizing

Calculations in accordance with BS5950-1:2000. Loads are factored. Low shear condition applies.

Beam span \( L = 6 \) m
Total factored load \( W = 200 \) kN
Factored bending moment \( M = W \times L / 8 = 150 \) kNm
Minimum elastic modulus \( Z = M \times 10^3 / (\pi y \times 1.2) = 454.55 \) cm³
For a semi-compact section
Minimum elastic modulus \( Z = M \times 10^3 / py = 545.45 \) cm³

Resistance to lateral-torsional buckling - Clause 4.3.6

For sections of uniform cross-section and for simplicity taking \( u \times v = 0.9 \) and \( \beta w = 1.0 \) i.e. equivalent slenderness = 0.9(\lambda)
Values of minimum plastic modulus \( (S) \) based on slenderness \( (\lambda) \) and bending strength \( (p_b) \).

<table>
<thead>
<tr>
<th>Slenderness</th>
<th>Bending strength (N/mm²)</th>
<th>Plastic modulus (cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>38.1</td>
<td>275</td>
<td>545</td>
</tr>
<tr>
<td>50</td>
<td>250</td>
<td>600</td>
</tr>
<tr>
<td>60</td>
<td>228</td>
<td>657</td>
</tr>
<tr>
<td>70</td>
<td>206</td>
<td>729</td>
</tr>
<tr>
<td>80</td>
<td>184</td>
<td>817</td>
</tr>
<tr>
<td>90</td>
<td>163</td>
<td>923</td>
</tr>
<tr>
<td>100</td>
<td>144</td>
<td>1045</td>
</tr>
<tr>
<td>110</td>
<td>127</td>
<td>1184</td>
</tr>
<tr>
<td>120</td>
<td>112</td>
<td>1339</td>
</tr>
<tr>
<td>130</td>
<td>99.4</td>
<td>1509</td>
</tr>
<tr>
<td>140</td>
<td>88.6</td>
<td>1693</td>
</tr>
<tr>
<td>150</td>
<td>79.3</td>
<td>1891</td>
</tr>
<tr>
<td>160</td>
<td>71.3</td>
<td>2103</td>
</tr>
<tr>
<td>170</td>
<td>64.5</td>
<td>2327</td>
</tr>
<tr>
<td>180</td>
<td>58.5</td>
<td>2565</td>
</tr>
</tbody>
</table>
**Location:** Ex2 - Axially loaded section

**Axially loaded section**

Calculations are in accordance with BS5950-1:2000. Load is factored. Length between restraints is for the weakest axis and end restraint factor 1.0. Compressive strengths are based on strut buckling curve (c) values. Section classification is at least semi-compact.

Factored load (comp. positive) $F=1200$ kN
Length between restraints $L=6$ m
Location: Ex3 - Beam check 254 x 146 x 43 UB

Moment capacity of beam subject to lateral load only. Classification of member is assumed to be at least semi-compact.

- Factored bending moment: \( M = 108 \text{ kNm} \)
- Plastic modulus: \( S = 568 \text{ cm}^3 \)
- Elastic modulus: \( Z = 505 \text{ cm}^3 \)
- Length between restraints: \( L_e = 2 \text{ m} \)
- Minimum radius of gyration: \( r = 3.51 \text{ cm} \)
Location: Ex4 - Column check - 254 x 254 x 89UC

Compressive resistance based on strut curve (c) values. The section classification is assumed to be at least semi-compact. The resistance is based on the length between restraints for the weakest axis and for an effective length factor equal to 1.0.

Factored axial load \( F = 1200 \text{ kN} \)
Length between restraints \( L = 6 \text{ m} \)
Radius of gyration \( r = 6.52 \text{ cm} \)
Area of section \( A = 114 \text{ cm}^2 \)
### Location: Ex5 - Check on 254 x 254 x 73 UC

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load (compress.positive)</td>
<td>F=425 kN</td>
</tr>
<tr>
<td>Moment about x-x axis</td>
<td>Mx=190 kNm</td>
</tr>
<tr>
<td>Moment about y-y axis</td>
<td>My=10 kNm</td>
</tr>
<tr>
<td>Gross cross-sectional area</td>
<td>A=92.9 cm²</td>
</tr>
<tr>
<td>Plastic modulus x-x axis</td>
<td>Sx=989 cm³</td>
</tr>
<tr>
<td>Plastic modulus y-y axis</td>
<td>Sy=462 cm³</td>
</tr>
</tbody>
</table>
Location: Ex6 - Safe load table for 457 x 191 x 67 UB

457 x 191 x 67 UB.

Young's Modulus  E=205 kN/mm²
Location: Ex7 - Safe load table for 610 x 229 UB

610 x 229 x 113 UB.
Young's Modulus E=205 kN/mm²
Location: Ex8 - Safe load table for 356 x 171 x 45 UB

356 x 171 x 45 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Location: Ex1 - Simply supported beam

Beam sizing

Calculations in accordance with EC3 Part 1-1.

Loads are factored. Low shear condition applies.

Beam span \( L = 6 \, \text{m} \)
Total factored load \( W = 200 \, \text{kN} \)
Factored bending moment \( M_{yEd} = W \times L / 8 = 150 \, \text{kNm} \)
For a plastic or compact section (Class 1 or Class 2),
Minimum plastic modulus \( W_{pl} = M_{yEd} \times 10^3 / f_y = 545.45 \, \text{cm}^3 \)
For a semi-compact section (Class 3),
Minimum elastic modulus \( W_{el} = M_{yEd} \times 10^3 / f_y = 545.45 \, \text{cm}^3 \)

Resistance to lateral-torsional buckling

The values in table below are for uniform cross-sections. The minimum plastic modulus \( W_{pl} \), is based on the section slenderness \( \lambda \) and bending strength \( s_b \).

<table>
<thead>
<tr>
<th>Slenderness ( \lambda = L_{cr}/i )</th>
<th>Bending strength ( (N/mm^2) )</th>
<th>Minimum plastic modulus ( (cm^3) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>38.6</td>
<td>275</td>
<td>545</td>
</tr>
<tr>
<td>50</td>
<td>251</td>
<td>597</td>
</tr>
<tr>
<td>60</td>
<td>229</td>
<td>654</td>
</tr>
<tr>
<td>70</td>
<td>207</td>
<td>724</td>
</tr>
<tr>
<td>80</td>
<td>185</td>
<td>810</td>
</tr>
<tr>
<td>90</td>
<td>164</td>
<td>913</td>
</tr>
<tr>
<td>100</td>
<td>145</td>
<td>1033</td>
</tr>
<tr>
<td>110</td>
<td>128</td>
<td>1169</td>
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<tr>
<td>120</td>
<td>114</td>
<td>1321</td>
</tr>
<tr>
<td>130</td>
<td>101</td>
<td>1487</td>
</tr>
<tr>
<td>140</td>
<td>90</td>
<td>1667</td>
</tr>
<tr>
<td>150</td>
<td>80.6</td>
<td>1861</td>
</tr>
<tr>
<td>160</td>
<td>72.6</td>
<td>2067</td>
</tr>
<tr>
<td>170</td>
<td>65.6</td>
<td>2287</td>
</tr>
<tr>
<td>180</td>
<td>59.5</td>
<td>2520</td>
</tr>
</tbody>
</table>
Location: Ex2 - Axially loaded column

Axially loaded section

Calculations in accordance with BS EN 1993-1-1:2005. The axial load is factored. Length between restraints is for the weakest axis and end restraint factor 1.0. Compressive strengths are based on strut buckling curve c values. Section classification is at least semi-compact.

Factored load (+ compression ) \( N_{Ed} = 1200 \text{ kN} \)
Length between restraints \( L = 6 \text{ m} \)
Location: Ex3 - Beam check (254 x 146 x 43 UB)

254 x 146 x 43 UKB
Dimensions (mm): h=259.6 b=147.3 tw=7.2 tf=12.7 r=7.6
Properties (cm): Iy=6540 Iz=677 Wply=566 Wplz=141 It=23.9
Moment capacity of beam subject to lateral load only. Classification of member is assumed to be at least semi-compact (i.e. Class 3 or less).
Factored bending moment MyEd=108 kNm
Beam length L=6 m
Length between restraints LT=2 m
Location: Ex4 - Column check (254 x 254 x 89 UC)

254 x 254 x 89 UKC
Dimensions (mm): h=260.3 b=256.3 tw=10.3 tf=17.3 r=12.7
Properties (cm): Iy=14300 Iz=4860 Wply=1220 Wplz=575 It=102
Section classification is assumed to be at least semi-compact. The resistance is based on the length between restraints for the weakest axis and for an effective length factor equal to 1.0.

Factored axial load NEd=1200 kN
Length between restraints L=6 m
Location: Ex5 - Check on 254 x 254 x 73 UC

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load ( + compression )</td>
<td>NEd=425 kN</td>
</tr>
<tr>
<td>Moment about y-y axis</td>
<td>MyEd=190 kNm</td>
</tr>
<tr>
<td>Moment about z-z axis</td>
<td>MzEd=10 kNm</td>
</tr>
<tr>
<td>Gross cross-sectional area</td>
<td>A=92.9 cm²</td>
</tr>
<tr>
<td>Plastic modulus y-y axis</td>
<td>Wply=989 cm³</td>
</tr>
<tr>
<td>Plastic modulus z-z axis</td>
<td>Wplz=462 cm³</td>
</tr>
</tbody>
</table>
Location: Ex6 - Safe load table for 457 x 191 x 67 UB

457 x 191 x 67 UKB
Dimensions (mm): h=453.4 b=189.9 tw=8.5 tf=12.7 r=10.2
Properties (cm): Iy=29400 Iz=1450 Wply=1470 Wplz=237 It=37.1
Location: Ex7 - Safe load table for 610 x 229 x 113 UB

610 x 229 x 113 UKB
Dimensions (mm): h=607.6 b=228.2 tw=11.1 tf=17.3 r=12.7
Properties (cm): Iy=87300 Iz=3430 Wply=3280 Wplz=469 It=111
Location: Ex1 - Based on 'Steelwork Design Guide' example 2

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span L=9 m

533 x 210 x 82 UB.

Young's Modulus E=205 kN/mm²

Dead load factor gamd=1.4

Imposed load factor gami=1.6

Distance from left support Lc(1)=3 m

Dead load (unfactored) Gkc(1)=40 kN

Imposed load (unfactored) Qkc(1)=60 kN

Distance from left support Lc(2)=6 m

Dead load (unfactored) Gkc(2)=20 kN

Imposed load (unfactored) Qkc(2)=30 kN

Dist. from left support to start Lau(1)=0 m

Distance from left support to end Lbu(1)=9 m

Dead load (unfactored) Gku(1)=3 kN/m

Imposed load (unfactored) Qku(1)=0 kN/m

Maximum span bending moment 414.9 kNm

Design shear force Fv=145.57 kN

Length of beam between restraints LT=3 m

Maximum moment on segment Me=414.9 kNm

Far end BM betaM=334 kNm

Force applied through flange Fbw=145.57 kN

Stiff bearing length b1=50 mm

Distance to nearer end be=0 mm

Distance to nearer end of member ae=25 mm

Assumed ratio of Mf/M Rf=0.55

UNIVERSAL BEAM 533 x 210 x 82 UB Grade S 275

DESIGN SUMMARY

Maximum shear force 145.57 kN

Shear capacity 836.83 kN

Max. applied moment 414.9 kNm

Moment capacity 566.5 kNm

Buckling resistance 452.86 kNm

Moment factor (mLT) 0.92201

Resistance (Mb/mLT) 491.17 kNm

Unfactored DL defln 10.604 mm

Unfactored LL defln 11.958 mm

Limiting deflection 25 mm

Unfactored end shears

DL shear at LHE 46.833 kN

LL shear at LHE 50 kN

DL shear at RHE 40.167 kN

LL shear at RHE 40 kN
Applied flange force 145.57 kN
Local web capacity 268.75 kN
Web buckling capacity 157.4 kN

FIRE RESISTANCE
Heated perimeter 1872.6 mm²

SUMMARY
Area of section 105 cm²
Section factor 175
Load Ratio (R) 0.46459

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
**Location: Ex2 - Using parallel flange channel**

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span \( L = 5 \) m

380 x 100 Parallel Flange Channel.

Young's Modulus \( E = 205 \) kN/mm²

Dead load factor \( \gamma_d = 1.4 \)

Imposed load factor \( \gamma_I = 1.6 \)

Distance from left support to start \( L_{au}(1) = 0 \) m

Distance from left support to end \( L_{bu}(1) = 5 \) m

Dead load (unfactored) \( G_{ku}(1) = 9 \) kN/m

Imposed load (unfactored) \( Q_{ku}(1) = 12 \) kN/m

Maximum span bending moment \( 99.375 \) kNm

Design shear force \( F_v = 79.5 \) kN

Assumed ratio of \( M_f/M \) \( R_f = 0.5 \)

**PARALLEL FLANGE CHANNEL**

380 x 100 Grade S 275

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum shear force</td>
<td>79.5 kN</td>
</tr>
<tr>
<td>Shear capacity</td>
<td>573.99 kN</td>
</tr>
<tr>
<td>Max. applied moment</td>
<td>99.375 kNm</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>247.25 kNm</td>
</tr>
<tr>
<td>Buckling resistance</td>
<td>125.3 kNm</td>
</tr>
<tr>
<td>Moment factor (mLT)</td>
<td>0.925</td>
</tr>
<tr>
<td>Resistance (Mb/mLT)</td>
<td>135.45 kNm</td>
</tr>
<tr>
<td>Unfactored DL defln</td>
<td>2.3819 mm</td>
</tr>
<tr>
<td>Unfactored LL defln</td>
<td>3.1758 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>13.889 mm</td>
</tr>
<tr>
<td>DL shear at LHE</td>
<td>22.5 kN</td>
</tr>
<tr>
<td>LL shear at LHE</td>
<td>30 kN</td>
</tr>
<tr>
<td>DL shear at RHE</td>
<td>22.5 kN</td>
</tr>
<tr>
<td>LL shear at RHE</td>
<td>30 kN</td>
</tr>
</tbody>
</table>

**FIRE RESISTANCE SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heated perimeter</td>
<td>1041 mm²</td>
</tr>
<tr>
<td>Area of section</td>
<td>68.7 cm²</td>
</tr>
<tr>
<td>Section factor</td>
<td>150</td>
</tr>
<tr>
<td>Load Ratio (R)</td>
<td>0.36682</td>
</tr>
</tbody>
</table>

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Location: Ex1 - Based on 'Steelwork Design Guide' example 2

Simply supported steel beam

Calculations in accordance with BS EN 1993-1-1:2005.

Beam span

L=9 m

533 x 210 x 82 UKB

Dimensions (mm): h=528.3 b=208.8 tw=9.6 tf=13.2 r=12.7
Properties (cm): Iy=47500 Iz=2010 Wply=2060 Wplz=300 It=51.5

Permanent UDL (including S.W) wd=3 kN/m
Variable UDL wi=0 kN/m

Permanent point load 1 (+ve down) W1d=40 kN
Variable point load 1 W1i=60 kN
Distance from L.H. supp to load 1 x1=3 m

Permanent point load 2 (+ve down) W2d=20 kN
Variable point load 2 W2i=30 kN
Distance from L.H. supp to load 2 x2=6 m

Maximum moment on segment Mec=396.45 kNm
Far end BM psiM=324 kNm
Eff.length between restraints LT=3 m
Stiff bearing length ss=50 mm
Stiff bearing position C=0 mm

UNIVERSAL BEAM

DESIGN SUMMARY

533 x 210 x 82 UB Grade S 275

Maximum shear force 138.22 kN
Shear plastic resist. 865.25 kN
Design moment 396.45 kNm
Moment resistance 566.5 kNm
Buckling resistance 478.91 kNm
Central deflection 11.673 mm
Limiting deflection 25 mm

Fastened end shears

Factored shear at A 138.22 kN
Factored shear at B 114.22 kN
Location: Ex2- Using parallel flange channel

Simply supported steel beam

Calculations in accordance with BS EN 1993-1-1:2005.

Beam span

L=4.9 m

380 x 100 UKPFC
Dimensions (mm): h=380 b=100 tw=9.5 tf=17.5 r=15
Properties (cm): Iy=15000 Iz=643 Wply=933 Wplz=161 It=45.7
Permanent UDL (including S.W) wd=9 kN/m
Variable UDL wi=12 kN/m
Eff.length between restraints LT=6.16 m
Stiff bearing length ss=50 mm
Stiff bearing position C=0 mm

PARALLEL FLANGE CHANNEL 380 x 100 Grade S 275
DESIGN SUMMARY
Maximum shear force 73.868 kN
Shear plastic resist. 581.2 kN
Design moment 90.488 kNm
Moment resistance 247.25 kNm
Buckling resistance 92.123 kNm
Central deflection 2.8595 mm
Limiting deflection 13.611 mm

Factored end shears
Factored shear at A 73.868 kN
Factored shear at B 73.868 kN
Location: Example 2, 'Steelwork Design Guide' with additional loads

Simply supported beam

strengthened by plate

welded to bottom flange

Calculations are in accordance with BS5950 Part 1:2000

Beam span

\[ L = 9 \text{ m} \]

457 x 191 x 89 UB.

Young's Modulus

\[ E = 205 \text{ kN/mm}^2 \]

Thickness of plate

\[ T_p = 15 \text{ mm} \]

Width of plate

\[ B_p = 170 \text{ mm} \]

Dead load factor

\[ \gamma_{d} = 1.4 \]

Imposed load factor

\[ \gamma_{i} = 1.6 \]

Distance from left support

\[ L_c(1) = 3 \text{ m} \]

Dead load (unfactored)

\[ G_{kc}(1) = 60 \text{ kN} \]

Imposed load (unfactored)

\[ Q_{kc}(1) = 80 \text{ kN} \]

Distance from left support

\[ L_c(2) = 6 \text{ m} \]

Dead load (unfactored)

\[ G_{kc}(2) = 40 \text{ kN} \]

Imposed load (unfactored)

\[ Q_{kc}(2) = 50 \text{ kN} \]

Dist. from left support to start

\[ L_{au}(1) = 0 \text{ m} \]

Distance from left support to end

\[ L_{bu}(1) = 9 \text{ m} \]

Dead load (unfactored)

\[ G_{ku}(1) = 3 \text{ kN/m} \]

Imposed load (unfactored)

\[ Q_{ku}(1) = 0 \text{ kN/m} \]

Maximum span bending moment

\[ 594.9 \text{ kNm} \]

Design shear force

\[ F_v = 205.57 \text{ kN} \]

Beam is FULLY RESTRAINED

Weld leg length

\[ s = 6 \text{ mm} \]

UNIVERSAL BEAM

457 x 191 x 89 UB Grade S 275

BOTTOM PLATE

170 mm x 15 mm Grade S 275

connected using 6 mm fillet welds

DESIGN SUMMARY

Shear force

\[ 205.57 \text{ kN} \]

Shear capacity

\[ 773.65 \text{ kN} \]

Applied moment

\[ 594.9 \text{ kNm} \]

Moment capacity

\[ 611.04 \text{ kNm} \]

Unfactored DL defln

\[ 14.287 \text{ mm} \]

Unfactored LL defln

\[ 15.502 \text{ mm} \]

Limiting deflection

\[ 25 \text{ mm} \]

DL shear at LHE

\[ 66.833 \text{ kN} \]

LL shear at LHE

\[ 70 \text{ kN} \]

DL shear at RHE

\[ 60.167 \text{ kN} \]

LL shear at RHE

\[ 60 \text{ kN} \]
Location: Ex1 - From 'Steelwork Design Guide' example 2

Simply supported beam

strengthened by plate

welded to bottom flange

Calculations are in accordance with EC3 Part 1-1:2005.

Beam span: L=9 m

457 x 191 x 89 UKB
Dimensions (mm): h=463.4 b=191.9 tw=10.5 tf=17.7 r=10.2
Properties (cm): Iy=41000 Iz=2090 Wply=2010 Wplz=338 It=90.7
Thickness of plate: Tp=15 mm
Width of plate: Bp=170 mm
Permanent UDL (including S.W) wd=3 kN/m
Variable UDL wi=0 kN/m
Permanent point load 1 (+ve down) W1d=60 kN
Variable point load 1 W1i=80 kN
Distance from L.H. supp to load 1 x1=3 m
Permanent point load 2 (+ve down) W2d=40 kN
Variable point load 2 W2i=50 kN
Distance from L.H. supp to load 2 x2=6 m
Stiff bearing length ss=50 mm
Stiff bearing position C=0 mm
Fillet weld size (leg length) s=6 mm

UNIVERSAL BEAM
457 x 191 x 89 UKB Grade S 275

BOTTOM PLATE
170 mm x 15 mm Grade S 275

DESIGN SUMMARY

Maximum shear force 195.23 kN
Shear plastic resist. 788.5 kN
Design moment 567.45 kNm
Moment capacity 649.22 kNm
Central deflection 15.133 mm
Limiting deflection 25 mm

Factored end shears

Design shear LH support 195.23 kN
Design shear RH support 171.23 kN
- Fillet welds to be used for connecting bottom plate to I-Section
- Weld sizes 6 mm
- Weld design strength to be 222 N/mm²
Location: Ex2 - UDL load only applied to beam

Simply supported beam

strengthened by plate

welded to bottom flange

Calculations are in accordance with EC3 Part 1-1:2005.

Beam span \( L = 6 \text{ m} \)

457 x 191 x 89 UKB

Dimensions (mm): \( h = 463.4 \) \( b = 191.9 \) \( t_w = 10.5 \) \( t_f = 17.7 \) \( r = 10.2 \)

Properties (cm): \( I_y = 41000 \) \( I_z = 2090 \) \( W_{ply} = 2010 \) \( W_{plz} = 338 \) \( I_t = 90.7 \)

Thickness of plate \( T_p = 15 \text{ mm} \)

Width of plate \( B_p = 170 \text{ mm} \)

Permanent UDL (including S.W) \( w_d = 9 \text{ kN/m} \)

Variable UDL \( w_i = 12 \text{ kN/m} \)

Eff.length between restraints \( L_T = 6.4 \text{ m} \)

Fillet weld size (leg length) \( s = 6 \text{ mm} \)

**UNIVERSAL BEAM**

457 x 191 x 89 UKB Grade S 275

**BOTTOM PLATE**

170 mm x 15 mm Grade S 275

**DESIGN SUMMARY**

Maximum shear force 90.45 kN

Shear plastic resist. 788.5 kN

Design moment 135.68 kNm

Moment capacity 649.22 kNm

Buckling resistance 304.31 kNm

Central deflection 1.822 mm

Limiting deflection 16.667 mm

Factored end shears

Design shear LH support 90.45 kN

Design shear RH support 90.45 kN

- Fillet welds to be used for connecting bottom plate to I-Section
- Weld sizes 6 mm
- Weld design strength to be 222 N/mm²
Location: Ex1 - 'Steelwork Design Guide' Example 4

I section beam

Calculations in accordance with BS5950-1:2000.

All loads and moments are factored.

Design moment about z-z axis \( M_z = 282 \text{ kNm} \)
Design shear force \( F_v = 152 \text{ kN} \)
Design axial load (+ve comp) \( F = 0 \text{ kN} \)
Length of member \( L = 9 \text{ m} \)

457 x 152 x 52 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Restraint assumptions

Length between restraints \( L_T = 3 \text{ m} \)
Maximum moment on segment \( M_{LT} = 282 \text{ kNm} \)
Far end BM \( \beta_M = -158 \text{ kNm} \)

UNIVERSAL BEAM 457 x 152 x 52 UB Grade S 275
SECTION
SUMMARY Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.
Factored shear force 152 kN
DESIGN
SUMMARY Shear capacity 564.05 kN
Factored moment 282 kNm
Moment capacity 302.5 kNm
Buckling resistance 244.17 kNm
Moment factor mLT 0.44
Resistance (Mb/mLT) 554.92 kNm
Location: Ex2 - 'Steelwork Design Guide' Example 5

I section beam

Calculations in accordance with BS5950-1:2000.

All loads and moments are factored.

Design moment about z-z axis \( M_z = 282 \text{ kNm} \)
Design shear force \( F_v = 152 \text{ kN} \)
Design axial load (+ve comp) \( F = 0 \text{ kN} \)
Length of member \( L = 9 \text{ m} \)

457 x 191 x 82 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Restraint assumptions

Length between restraints \( L_T = 9 \text{ m} \)
Quarter point \( m_2 = 48 \text{ kNm} \)
Mid-span \( m_3 = 126 \text{ kNm} \)
Three quarter point \( m_4 = 12.75 \text{ kNm} \)
Maximum Moment on segment \( M_{LT} = 282 \text{ kNm} \)

UNIVERSAL BEAM

SECTION

SUMMARY

Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.

Factored shear force \( 152 \text{ kN} \)

DESIGN

SUMMARY

Shear capacity \( 751.41 \text{ kN} \)
Factored moment \( 282 \text{ kNm} \)
Moment capacity \( 503.25 \text{ kNm} \)
Buckling resistance \( 205.85 \text{ kNm} \)
Moment factor \( mL_T \) \( 0.45572 \)
Resistance \( (Mb/mL_T) \) \( 451.71 \text{ kNm} \)
Location: Ex3 - Built-in beam carrying a concrete slab

I section beam
Calculations in accordance with BS5950-1:2000.
All loads and moments are factored.

Design moment about z-z axis \( M_z = 101.03 \) kNm
Design shear force \( F_v = 65.505 \) kN
Design axial load (+ve comp) \( F = 13.743 \) kN
Length of member \( L = 10 \) m

305 x 165 x 46 UB.
Young's Modulus \( E = 205 \) kN/mm²

Buckling about major axis \( L_z = 7150 \) mm
Buckling about minor axis \( L_y = 1860 \) mm

Restraint assumptions
- restraints to both flanges \( L_y = 1860 \) mm
- Distance between ref. axes \( a' = 290 \) mm
- Start end restraint \( R_1 = 101 \) kNm
- Quarter point \( R_2 = 69.8 \) kNm
- Mid-length \( R_3 = 41.5 \) kNm
- Three quarter point \( R_4 = 16.3 \) kNm
- End restraint \( R_5 = 0 \) kNm
- Largest value of moment \( R_S = 101 \) kNm
- Span of beam for deflection \( L_d = 10000 \) mm

DEFLECTION OK.

UNIVERSAL BEAM
305 x 165 x 46 UB Grade S 275
SECTION

SUMMARY
Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.
Factored shear force 65.505 kN
Shear capacity 338.95 kN
Factored moment 101.03 kNm
Moment capacity 198 kNm
Compressive axial load 13.743 kN
Compressive resistance 1414.9 kN
Local capacity check 0.51877 ≤ 1
Overall buckling check 0.51997 ≤ 1
Deflection 5.1326 mm
**Location:** Ex4 - Propped cantilever with point loads (UB Section)

**I section beam**

Calculations in accordance with BS5950-1:2000.

All loads and moments are factored.

Design moment about z-z axis \( M_z = 360 \text{ kNm} \)
Design shear force \( F_v = 160 \text{ kN} \)
Design axial load (+ve comp) \( F = 200 \text{ kN} \)
Length of member \( L = 9 \text{ m} \)

457 x 191 x 74 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Buckling about major axis \( L_z = 9000 \text{ mm} \)
Buckling about minor axis \( L_y = 2250 \text{ mm} \)

**Restraint assumptions**

- restraints to both flanges \( L_y = 2250 \text{ mm} \)
- Distance between ref. axes \( a' = 290 \text{ mm} \)
- Larger end moment \( B_M = 360 \text{ kNm} \)
- Smaller end moment \( M_s = 0 \text{ kNm} \)
- Length between restraints \( L_T = 2.25 \text{ m} \)
- Maximum moment on segment \( M_{LT} = 360 \text{ kNm} \)
- Far end BM \( \beta_M = 120 \text{ kNm} \)
- Quarter point \( m_{z2} = 180 \text{ kNm} \)
- Mid-span \( m_{z3} = 180 \text{ kNm} \)
- Three quarter point \( m_{z4} = 0 \text{ kNm} \)
- Maximum moment on central half \( M_{24} = 240 \text{ kNm} \)

**UNIVERSAL BEAM**

**SECTION**

457 x 191 x 74 UB Grade S 275

**SUMMARY**

Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.

Factored shear force \( 160 \text{ kN} \)

**DESIGN**

Shear capacity \( 678.65 \text{ kN} \)

**SUMMARY**

Factored moment \( 360 \text{ kNm} \)

Moment capacity \( 453.75 \text{ kNm} \)

Compressive axial load \( 200 \text{ kN} \)

Compressive resistance \( 2204.5 \text{ kN} \)

Local capacity check \( 0.87027 \leq 1 \)

Overall buckling check \( 0.54672 \leq 1 \)

Overall buckling check \( 0.53812 \leq 1 \)

\( 0.73487 \leq 1 \)
Location: Ex5 - Channel section (tensile load)

Channel Section Beam

Calculations in accordance with BS5950-1:2000.

All loads and moments are factored.

Design moment about z-z axis \( M_z = 187.5 \text{ kNm} \)
Design shear force \( F_v = 150 \text{ kN} \)
Design axial load (+ve comp) \( F = -200 \text{ kN} \)
Length of member \( L = 5 \text{ m} \)
380 x 100 Parallel Flange Channel.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Estimated net area (tension) \( A_e = 68.7 \text{ cm}^2 \)

PARALLEL FLANGE CHANNEL

380 x 100 Grade S 275

SECTION

Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.

Factored shear force 150 kN

SUMMARY

DESIGN

Shear capacity 573.99 kN

Factored moment 187.5 kNm

Moment capacity 247.25 kNm

Tensile axial load 200 kN

Tensile capacity 1820.6 kN

Local capacity check 0.86821 \( \leq 1 \)
Location: Ex6 - 'Steelwork Design Guide' Example 4 (fire check)

I section beam

Calculations in accordance with BS5950-1:2000.

All loads and moments are factored.

Design moment about z-z axis \( M_z = 282 \text{ kNm} \)
Design shear force \( F_v = 152 \text{ kN} \)
Design axial load (+ve comp) \( F = 0 \text{ kN} \)
Length of member \( L = 9 \text{ m} \)

457 x 152 x 52 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Restraint assumptions

Length between restraints \( L_T = 3 \text{ m} \)
Maximum moment on segment \( M_{LT} = 282 \text{ kNm} \)
Far end BM \( \beta M = -158 \text{ kNm} \)
values to applied factored values \( R_f = 0.6 \)

UNIVERSAL BEAM

SECTION

SUMMARY

Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.
Factored shear force 152 kN

DESIGN

SUMMARY

Shear capacity 564.05 kN
Factored moment 282 kNm
Moment capacity 302.5 kNm
Buckling resistance 244.17 kNm
Moment factor \( m_{LT} \) 0.44
Resistance \( (M_b/m_{LT}) \) 554.92 kNm

FIRE RESISTANCE

SUMMARY

Heated perimeter 1494 mm²
Area of section 66.6 cm²
Section factor 220 /m
Load Ratio \( (R) \) 0.55934

Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided.
Location: Example 12 Steel Building Design

Open Section Beam

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis \( MyEd = 146 \) kNm
Design shear force \( VzEd = 46 \) kN
Design axial load (+ve comp) \( NEd = 1100 \) kN
Length of member \( L = 6 \) m

457 x 191 x 67 UKB
Dimensions (mm): \( h = 453.4 \) b = 189.9 \( tw = 8.5 \) tf = 12.7 \( r = 10.2 \)
Properties (cm): \( Iy = 29400 \) Iz = 1450 \( Wply = 1470 \) Wplz = 237 \( It = 37.1 \)
Effective length parameter \( K = 1 \)
Length for LTB \( LT = 3 \) m
Maximum moment on section \( M = 146 \) kNm
Far end moment \( psiM = 0 \) kNm
Maximum end moment \( Mh(1) = 0 \) kNm
Maximum span moment \( Ms(1) = 146 \) kNm
End moment \( pMh(1) = 0 \) kNm
Maximum end moment \( Mh(2) = 146 \) kNm
Maximum span moment \( Ms(2) = 79 \) kNm
End moment \( pMh(2) = 0 \) kNm

SECTION 457 x 191 x 67 UKB Grade S 355
DESIGN Design compression load 1100 kN
SUMMARY Axial resistance 3035.3 kN
Buckling resistance 1902.5 kN
Design BM y-y axis 146 kNm
Moment resistance (y-y) 460.39 kNm
Interaction factors \( 0.69607 \leq 1 \)
\( 0.87247 \leq 1 \)
Location: Class 1 section design

Open Section Beam

Calculations in accordance with BS EN 1993-1-1:2005.
Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis $M_{Ed}=646 \text{ kNm}$
Design shear force $V_{zEd}=226 \text{ kN}$
Design axial load (+ve comp) $N_{Ed}=900 \text{ kN}$
Length of member $L=8 \text{ m}$

533 x 312 x 151 UKB
Dimensions (mm): $h=542.5 \text{ b}=312 \text{ tw}=12.7 \text{ tf}=20.3 \text{ r}=12.7$
Properties (cm): $I_{y}=101000 \text{ I}_{z}=10300 \text{ W}_{ply}=4150 \text{ W}_{plz}=1010 \text{ I}_{t}=216$
Effective length parameter $K=0.85$
Length for LTB $L_{T}=8 \text{ m}$
Correction factor $k_{c}=0.9$
Maximum end moment $M_{h}(1)=646 \text{ kNm}$
Maximum span moment $M_{s}(1)=-323 \text{ kNm}$
End moment $pM_{h}(1)=646 \text{ kNm}$
Maximum end moment $M_{h}(2)=646 \text{ kNm}$
Maximum span moment $M_{s}(2)=-323 \text{ kNm}$
End moment $pM_{h}(2)=646 \text{ kNm}$

SECTION
533 x 312 x 151 UKB Grade S 275
DESIGN
Design compression load 900 kN
Axial resistance 5088 kN
Buckling resistance 2337.8 kN
Design BM y-y axis 646 kNm
Moment resistance (y-y) 1099.8 kNm
Interaction factors $0.53217 \leq 1$
$0.96614 \leq 1$

SUMMARY
Axial resistance 5088 kN
Buckling resistance 2337.8 kN
Design BM y-y axis 646 kNm
Moment resistance (y-y) 1099.8 kNm
Interaction factors $0.53217 \leq 1$
$0.96614 \leq 1$
Location: High axial load and moment

Open Section Beam

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis \( M_{Ed} = 323 \text{ kNm} \)
Design shear force \( V_{zEd} = 126 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 2000 \text{ kN} \)
Length of member \( L = 6 \text{ m} \)

305 x 305 x 158 UKC
Dimensions (mm): \( h = 327.1 \text{ b = 311.2} \text{ tw = 15.8} \text{ tf = 25} \text{ r = 15.2} \)
Properties (cm): \( I_y = 38800 \text{ Iz = 12600} \text{ W_{ply} = 2680} \text{ W_{plz = 1230} It = 378} \)
Effective length parameter \( K = 0.85 \)
Length for LTB \( L_T = 6 \text{ m} \)
Maximum moment on section \( M = 323 \text{ kNm} \)
Far end moment \( pM(1) = -323 \text{ kNm} \)
Far end bending moment \( pM(2) = -323 \text{ kNm} \)

SECTION 305 x 305 x 158 UKC Grade S 275
DESIGN Design compression load 2000 kN
SUMMARY Axial resistance 5326.5 kN
Buckling resistance 3337.2 kN
Design BM y-y axis 323 kNm
Moment resistance (y-y) 710.2 kNm
Interaction factors \( 0.62626 \leq 1 \)
\( 0.8984 \leq 1 \)
Location: Tensile load and moment

Open Section Beam

Calculations in accordance with BS EN 1993-1-1:2005.
Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis       MyEd=124 kNm
Design shear force                    VzEd=200 kN
Design axial load (+ve comp)         NEd=-480 kN
Length of member                     L=7 m

406 x 178 x 67 UKB
Dimensions (mm): h=409.4 b=178.8 tw=8.8 tf=14.3 r=10.2
Properties (cm): Iy=24300 Iz=1370 Wply=1350 Wplz=237 It=46.1
Effective length parameter           K=1
Length for LTB                        LT=7 m
Correction factor                     kc=0.9

SECTION 406 x 178 x 67 UKB Grade S 275
DESIGN Design tensile load 480 kN
SUMMARY Axial resistance 2351.3 kN
             Design BM y-y axis 124 kNm
             Moment resistance (y-y) 371.25 kNm
             Interaction factors 0.8684 ≤ 1
Location: Example 13, 'Steelwork Design Guide'

Rafter stability

Calculations in accordance with BS5950-1:2000 Clause 5.3 and Annex G - Members with one flange laterally restrained.

Loads and moments are factored.

UNIFORM SECTION OF RAFTER

Design moment about z-z axis \(M_z = 283.8\) kNm
Design shear force \(F_v = 150\) kN
Design axial load (+ve comp) \(F = 92.615\) kN
Length of member \(L = 12.145\) m

406 x 178 x 60 UB.
Young's Modulus \(E = 205\) kN/mm²

Axial load at hinge position \(F' = 89.537\) kN
Far end bending moment value \(M_f = 279.1\) kNm
Lateral restraints \(L_y = 1500\) mm
Torsional restraints \(L_{yt} = 4500\) mm
Design moment \(M_1 = 279.1\) kNm
Design axial load \(F_1 = 92.615\) kN

Far end BM \(\beta_M = 247.83\) kNm
Quarter point \(m_2 = -162.85\) kNm
Mid-span \(m_3 = 131.77\) kNm
Three quarter point \(m_4 = 273.69\) kNm
Maximum moment on segment \(M'_z = 573.68\) kNm
Maximum moment on central half \(M_24 = 273.69\) kNm

HOGGING MOMENT REGION

533 x 210 x 101 UB.
Vertical overall depth of section \(D_o = 810\) mm
Steel design to BS5950-1:2000 and Eurocode 3
Portal frame rafter design
Copyright 1986-2019 Fitzroy Systems Ltd.
Made by: IFB
Date: 02/12/19
Ref No: SC411 BS

Eaves to 1st. torsional restraint \(L_{ly1}=1226.6\) mm

- Maximum applied moment: \(BM_1=573.68\) kNm
- Axial load: \(F_{c1}=126.2\) kN
- Pitch of haunch: \(\gamma=17.14^\circ\)
- Distance between ref. axes: \(a'=245\) mm
- Larger end moment: \(BM=573.68\) kNm
- Smaller end moment: \(Ms=419.19\) kNm
- Resistance moment: \(M_{bi}=M_b*(1-\frac{F_c}{P_c})=943.16\) kNm
- Design moment: \(M_{xi}=BM=573.68\) kNm

The relationship is satisfied, thus the stability is ensured between effective torsional restraints for the section chosen.

1st to 2nd torsional restraint \(L_{ly2}=1500\) mm

- Maximum applied moment: \(BM_2=419.19\) kNm
- Axial load: \(F_{c2}=122.77\) kN
- Distance between ref. axes: \(a'=245\) mm
- Larger end moment: \(BM=419.19\) kNm
- Smaller end moment: \(Ms=251\) kNm
- Resistance moment: \(M_{bi}=M_b*(1-\frac{F_c}{P_c})=653.96\) kNm
- Design moment: \(M_{xi}=BM=419.19\) kNm

The relationship is satisfied, thus the stability is ensured between effective torsional restraints for the section chosen.

### HOGGING ZONE B

**Check segment from sharp end of haunch to point of contraflexure**

- Distance between lateral restraints: \(L_{yl}=2776\) mm
- Start end restraint: \(N_1=251\) kNm
- Quarter point: \(N_2=181\) kNm
- Mid-length: \(N_3=116\) kNm
- Three quarter point: \(N_4=55\) kNm
- End restraint: \(N_5=0\) kNm
- Largest value of moment: \(RS=251\) kNm

**Overall out-of-plane buckling resistance - Annex G Section G.2.1**

\[
\frac{F}{Pc} + \frac{mt.Mz}{Mb} \leq 1
\]

Unity factor: \(u_{fy}=\frac{F}{Pc+mt*ABS(MB)}/Mb=0.86029\)
DESIGN SUMMARY

\[1500 \quad X \quad X \quad X\]

\[810\]

\[\text{Zone A} \leftarrow \text{Zone B}\]

\[1226.6 \text{ mm} \quad \text{Lateral restraint}\]

\[1226.6 \text{ mm} \quad \text{Torsional restraint}\]

\[X = \text{lateral restraint}\]

\[* = \text{torsional restraint}\]

**UNIVERSAL BEAM**

406 x 178 x 60 UB Grade S 275

**HAUNCH SECTION**

533 x 210 x 101 UB Grade S 275

Section is satisfactory for bending, buckling resistance between intermediate restraints and overall buckling between torsional restraints.

Restraint spacing:

Rafter plastic hinge is assumed to form just below the portal apex at the point of maximum sagging moment. Provide restraint to both flanges at the plastic hinge location. If this is not practicable, restraint to both flanges should be provided within a distance of half the depth of the member along the flanges of the member from the location of the plastic hinge.

- **Adjacent to apex hinge**: Lateral restraint 1100 mm
- **Along the rafter**: Lateral restraints 1500 mm, Torsional restraints 4500 mm
- **At eaves position**: Lateral restraint 1226.6 mm, Torsional restraint 1226.6 mm, Torsional restraint 2726.6 mm
Location: Example - small span

Rafter stability

Calculations in accordance with BS5950-1: 2000 Clause 5.3 and Annex G - Members with one flange laterally restrained.

Loads and moments are factored.

UNIFORM SECTION OF RAFTER

Design moment about z-z axis \( M_z = 47 \text{ kNm} \)
Design shear force \( F_v = 40 \text{ kN} \)
Design axial load (+ve comp) \( F = 14.064 \text{ kN} \)
Length of member \( L = 4.5 \text{ m} \)

178 x 102 x 19 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Axial load at hinge position \( F' = 12.64 \text{ kN} \)
Far end bending moment value \( M_f = 37.694 \text{ kNm} \)
Lateral restraints \( L_y = 1100 \text{ mm} \)
Torsional restraints \( L_{yt} = 2200 \text{ mm} \)
Design moment \( M_1 = 37.694 \text{ kNm} \)
Design axial load \( F_1 = 14.064 \text{ kN} \)

Far end BM \( \beta M = 17.835 \text{ kNm} \)
Quarter point \( m_2 = -5.4082 \text{ kNm} \)
Mid-span \( m_3 = 26.149 \text{ kNm} \)
Three quarter point \( m_4 = 43.957 \text{ kNm} \)
Maximum moment on segment \( M_z' = 48.015 \text{ kNm} \)
Maximum moment on central half \( M_{24} = 43.957 \text{ kNm} \)

HOGGING MOMENT REGION

178 x 102 x 19 UB.
Vertical overall depth of section \( D_o = 320 \text{ mm} \)
eaves to 1st. torsional restraint Lty1=2109.2 mm
Maximum applied moment BM1=47.034 kNm
Axial load Fc1=19.68 kN
Pitch of haunch Gamma=17.14°
Distance between ref. axes a'=110 mm
Start end restraint R1=47.034 kNm
Quarter point R2=27.031 kNm
Mid-length R3=9.3794 kNm
Three quarter point R4=-5.9191 kNm
End restraint R5=-18.865 kNm
Largest value of moment RS=47.034 kNm
Resistance moment Mbi=Mb*(1-Fc/Pc)=85.455 kNm
Design moment Mxi=BM=47.034 kNm

The relationship is satisfied, thus the stability is ensured between effective torsional restraints for the section chosen.

**DESIGN SUMMARY**

```
  1100 X X X
  1100 C.L. of member
  1100

  Zone A <-> Zone B

  1009.2 mm Lateral restraint
  2109.2 mm to next torsional restraint

  leg C.L X = lateral restraint
  * = torsional restraint
```

**UNIVERSAL BEAM**
178 x 102 x 19 UB Grade S 275

**HAUNCH SECTION**
178 x 102 x 19 UB Grade S 275
Section is satisfactory for bending, buckling resistance between intermediate restraints and overall buckling between torsional restraints.

Restraint spacing:
Rafter plastic hinge is assumed to form just below the portal apex at the point of maximum sagging moment. Provide restraint to both flanges at the plastic hinge location. If this is not practicable, restraint to both flanges should be provided within a distance of half the depth of the member along the flanges of the member from the location of the plastic hinge.
<table>
<thead>
<tr>
<th>Location</th>
<th>Lateral restraint</th>
<th>Torsional restraint</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adjacent to apex hinge</td>
<td>825 mm</td>
<td></td>
</tr>
<tr>
<td>Along the rafter</td>
<td>1100 mm</td>
<td>2200 mm</td>
</tr>
<tr>
<td>At eaves position</td>
<td>1009.2 mm</td>
<td>2109.2 mm</td>
</tr>
</tbody>
</table>
Location: Uniform section only based on Example 13

```
 y
 _____________
 |          |
 |          |
 |          |
 |          |
 |___________|
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**Rafter stability**

Calculations in accordance with BS5950-1:2000 Clause 5.3 and Annex G - Members with one flange laterally restrained.

Loads and moments are factored.

---

**UNIFORM SECTION OF RAFTER**

- Design moment about z-z axis: $M_z = 283.8 \text{ kNm}$
- Design shear force: $F_v = 150 \text{ kN}$
- Design axial load (+ve comp): $F = 92 \text{ kN}$
- Length of member: $L = 12.145 \text{ m}$

- 406 x 178 x 60 UB.
- Young's Modulus: $E = 205 \text{ kN/mm}^2$
- Lateral restraints: $L_y = 1500 \text{ mm}$
- Torsional restraints: $L_{yt} = 4500 \text{ mm}$
- Design moment: $M_1 = 279 \text{ kNm}$
- Design axial load: $F_1 = 92 \text{ kN}$

- Length of rafter: $L_z = 12145 \text{ mm}$
- Far end BM: $\beta M = 248 \text{ kNm}$
- Quarter point: $m_2 = 176 \text{ kNm}$
- Mid-span: $m_3 = 223 \text{ kNm}$
- Three quarter point: $m_4 = 258 \text{ kNm}$
- Maximum moment on segment: $M_z' = 279 \text{ kNm}$
- Maximum moment on central half: $M_{24} = 258 \text{ kNm}$

---

**HOGGING MOMENT REGION**

- Axial force in rafter at eaves: $F_c = 126.2 \text{ kN}$
- Tension flange: $L_p = 1500 \text{ mm}$
- Restraints to both flanges: $L_{ty} = 279 \text{ mm}$
- Distance between ref. axes: $a' = 245 \text{ mm}$
- Start end restraint: $R_1 = 251 \text{ kNm}$
- Quarter point: $R_2 = 181 \text{ kNm}$
- Mid-length: $R_3 = 116 \text{ kNm}$
- Three quarter point: $R_4 = 55 \text{ kNm}$
- End restraint: $R_5 = 0 \text{ kNm}$
Largest value of moment \( RS = 251 \, kNm \)

### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>1500</th>
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<th>1500</th>
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<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

C.L. of member

---

**UNIVERSAL BEAM**

406 x 178 x 60 UB Grade S 275

Section is satisfactory for bending, buckling resistance between intermediate restraints and overall buckling between torsional restraints.

**Restraint spacing:**

Rafter plastic hinge is assumed to form just below the portal apex at the point of maximum sagging moment. Provide restraint to both flanges at the plastic hinge location. If this is not practicable, restraint to both flanges should be provided within a distance of half the depth of the member along the flanges of the member from the location of the plastic hinge.

**Along the rafter**

- Lateral restraints 1500 mm
- Torsional restraints 4500 mm

**At eaves position**

- Lateral restraint 1500 mm
- Torsional restraint 2776 mm
**Location:** Ex1 - Based on example 13, 'Steelwork Design Guide'

**Rafter stability**

Calculations in accordance with BS EN 1993-1-1:2005 Clause 6.3.5 and Annex BB.3.

The proforma assumes a plastic hinge forms just below the portal apex (at the point of max sagging moment).

Loads and moments are factored.

**UNIFORM SECTION OF RAFTER**

- Design moment about major axis $M_{Ed}=283.8$ kNm
- Design shear force $V_{zEd}=150$ kN
- Design axial load (+ve comp) $N_{Ed}=92.615$ kN
- Length of member $L=12.145$ m
- 406 x 178 x 60 UKB
- Axial load at hinge position $N_{Ed}'=89.537$ kN
- Correction factor $k_{c}=1$
- Dist. between lateral restraints $L_{z}=1500$ mm
- Dist. between torsional restraints $L_{zt}=4500$ mm
- Design span moment near the apex $M_{Ed1}=279.1$ kNm
- Design axial load $N_{Ed1}=92.615$ kN
- Length of the whole rafter $L_{y}=15524$ mm

- Maximum moment on segment $M=279$ kNm
- Far end moment $p_{si_{M}}=247$ kNm
- Maximum end moment $M_{h(1)}=573$ kNm
- Maximum span moment $M_{s(1)}=-279$ kNm
- End moment $p_{M_{h(1)}}=-247$ kNm
**HOGGING MOMENT REGION**

533 x 210 x 101 UKB  
Vertical overall depth of section ho=810 mm  
Pitch of rafter  \( \theta = 14.6^\circ \)  
Pitch of haunch  \( \gamma = 17.14^\circ \)

<table>
<thead>
<tr>
<th>2045 mm</th>
<th>955 mm</th>
<th>975 mm</th>
<th>1500 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>(L_p)</td>
<td>(L_mh)</td>
<td>(L_m)</td>
<td>(L_z)</td>
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<tr>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>C.L. of member</td>
</tr>
</tbody>
</table>

810  
Assume a plastic hinge at point B. Zone B terminates at point of contraflexure.

**ELASTIC STABILITY CHECKS - Zones A & B**

**Zone A - segment adjacent to column face**

The haunched rafter segment adjacent to column face (2045 mm long) in hogging moment zone A will be checked.

- Dist. between eff. torsional restr. \( L_{tz3} = 2045 \) mm
- Max design moment on segment  \( BM_1 = MA = 573 \) kNm
- Min design moment on segment  \( BM_3 = MAB = 375.38 \) kNm
- Design axial load  \( N_{Ed3} = 92 \) kN
- Length for LTB  \( LT = Lz/1000 = 2.045 \) m
- Critical moment  \( Mcr = C_1 * E_t * S_r / 10^6 = 2800.1 \) kNm
- Design buckling resistance moment  \( Mb_{Rd} = \chi L_T * W_{ply} * f_y / (\gamma M_1 * 10^3) \)  
  \( = 792 \) kNm

As \( My_{Ed}' \leq Mb_{Rd} \) (573 kNm \( \leq 792 \) kNm), design moment is within the buckling resistance moment. Hence OK.

- Resistance moment  \( M_{bi} = Mb_{Rd} * (1-N_{Ed}' / Nb_{Rd}) = 736.35 \) kNm
- Design moment  \( M_{xi} = BM = 573 \) kNm
- As \( M_{xi} \leq M_{bi} \) stability is ensured between effective torsional restraints for the section chosen.

**Zone B - segment adjacent to sharp end of haunch**

The uniform rafter segment adjacent to the sharp end of the haunch in hogging moment zone B (2776 mm long) will be checked.

- Dist. between eff. torsional restr. \( L_{tz4} = 2776 \) mm
- Maximum design moment  \( BM_1 = MB = 283.8 \) kNm
- Minimum design moment  \( BM_4 = 0 \) kNm
- Design axial load  \( N_{Ed4} = 92 \) kN
Length for LTB

Critical moment

Design buckling resistance moment

As \( \text{MEd'} \leq \text{MbRd} \) (283.8 kNm ≤ 330 kNm), design moment is within the buckling resistance moment. Hence OK.

Design moment

As \( \text{Mxi} \leq \text{Mbi} \) stability is ensured between effective torsional restraints for the section chosen.

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>UNIVERSAL BEAM</th>
<th>406 x 178 x 60 UKB Grade S 275</th>
</tr>
</thead>
<tbody>
<tr>
<td>HAUNCH SECTION</td>
<td>533 x 210 x 101 UKB Grade S 275</td>
</tr>
<tr>
<td>Section is satisfactory for bending, buckling resistance between intermediate restraints and overall buckling between torsional restraints.</td>
<td></td>
</tr>
</tbody>
</table>

**Restraint spacing:**

Rafter plastic hinge is assumed to form just below the portal apex at the point of maximum sagging moment and at the sharp end of the haunch. Provide restraint to both flanges at the plastic hinge locations. If this is not practicable, restraint to both flanges should be provided within a distance of half the depth of the member.

<table>
<thead>
<tr>
<th>Adjacent to apex hinge</th>
<th>First lateral restraint</th>
<th>975 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Along the rafter</td>
<td>Lateral restraints</td>
<td>1500 mm</td>
</tr>
<tr>
<td></td>
<td>Torsional restraints</td>
<td>4500 mm</td>
</tr>
</tbody>
</table>

**Restraint spacing (measured from column face):**

<table>
<thead>
<tr>
<th>At eaves position</th>
<th>Lateral restraint</th>
<th>2045 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Torsional restraint</td>
<td>2045 mm</td>
</tr>
<tr>
<td></td>
<td>Torsional restraint</td>
<td>3000 mm</td>
</tr>
</tbody>
</table>

For spacing of lateral restraints adjacent to the sharp end of haunch see diagram in "Haunch stability" section.
Location: Ex2 - Small span rafter

Rafter stability

Calculations in accordance with BS EN 1993-1-1:2005 Clause 6.3.5 and Annex BB.3.

The proforma assumes a plastic hinge forms just below the portal apex (at the point of max sagging moment).

Loads and moments are factored.

UNIFORM SECTION OF RAFTER

Design moment about major axis $M_{Ed}=45$ kNm
Design shear force $V_{zEd}=40$ kN
Design axial load (+ve comp) $N_{Ed}=14$ kN
Length of member $L=4.5$ m
203 x 102 x 23 UKB
Axial load at hinge position $N_{Ed}'=14$ kN
Correction factor $k_c=1$
Dist. between lateral restraints $L_z=1100$ mm
Dist. between torsional restraints $L_{zt}=2200$ mm
Design span moment near the apex $M_{Ed1}=38$ kNm
Design axial load $N_{Ed1}=15$ kN

Length of the whole rafter $L_y=5100$ mm
Correction factor $k_c=1$
Maximum end moment $M_{h}(1)=48$ kNm
Maximum span moment $M_{s}(1)=-38$ kNm
End moment $pM_{h}(1)=-35$ kNm

HOGGING MOMENT REGION

203 x 102 x 23 UKB
Vertical overall depth of section $h_0=375$ mm
Pitch of rafter $\theta=11.3^\circ$
Pitch of haunch $\gamma=17.14^\circ$
ELASTIC STABILITY CHECKS - Zones A & B

Zone A - segment adjacent to column face

The haunched rafter segment adjacent to column face (10 mm long) in hogging moment zone A will be checked.

Dist. between eff. torsional restr. $\text{LTz}_3$ = 10 mm

Max design moment on segment $BM_1$ = $MA$ = 48 kNm

Min design moment on segment $BM_3$ = $MAB$ = 47.947 kNm

Design axial load $N_{Ed3}$ = 14 kN

Length for $LTB$ $LT$ = $L_{z}/1000$ = 0.01 m

Critical moment $Mc_r$ = $C_1*E_t*S_r/10^6$ = 5.9676E6 kNm

Design buckling resistance moment $Mb_{Rd}$ = $Ch_l LT*W_{ply}*f_y/(\gamma M_{1}*10^3)$

As $My_{Ed'} \leq Mb_{Rd}$ (48 kNm $\leq$ 136.95 kNm), design moment is within the buckling resistance moment. Hence OK.

Resistence moment $Mb_i$ = $Mb_{Rd}*(1-N_{Ed'}/N_{Rd})$ = 134.55 kNm

Design moment $Mx_i$ = $BM$ = 48 kNm

As $Mx_i \leq Mb_i$ stability is ensured between effective torsional restraints for the section chosen.

Zone B - segment adjacent to sharp end of haunch

The uniform rafter segment adjacent to the sharp end of the haunch in hogging moment zone B (1800 mm long) will be checked.

Dist. between eff. torsional restr. $\text{LTz}_4$ = 1800 mm

Maximum design moment $BM_1$ = $MB$ = 45 kNm

Minimum design moment $BM_4$ = 0 kNm

Design axial load $N_{Ed4}$ = 14 kN

Length for $LTB$ $LT$ = $L_{z}/1000$ = 1.8 m

Critical moment $Mc_r$ = $C_1*E_t*S_r/10^6$ = 225.93 kNm

Design buckling resistance moment $Mb_{Rd}$ = $Ch_l LT*W_{ply}*f_y/(\gamma M_{1}*10^3)$

As $My_{Ed'} \leq Mb_{Rd}$ (45 kNm $\leq$ 64.35 kNm), design moment is within the buckling resistance moment. Hence OK.
Resistance moment $M_{bi} = M_{bRd} \times (1 - N_{Ed}' / N_{bRd}) = 63.22 \text{ kNm}$

Design moment $M_{xi} = BM = 45 \text{ kNm}$

As $M_{xi} \leq M_{bi}$ stability is ensured between effective torsional restraints for the section chosen.

## DESIGN SUMMARY

### UNIVERSAL BEAM

203 x 102 x 23 UKB Grade S 275

### HAUNCH SECTION

203 x 102 x 23 UKB Grade S 275

Section is satisfactory for bending, buckling resistance between intermediate restraints and overall buckling between torsional restraints.

Restraint spacing:

Rafter plastic hinge is assumed to form just below the portal apex at the point of maximum sagging moment and at the sharp end of the haunch. Provide restraint to both flanges at the plastic hinge locations. If this is not practicable, restraint to both flanges should be provided within a distance of half the depth of the member.

Adjacent to apex hinge  First lateral restraint  650 mm
Along the rafter  Lateral restraints  1100 mm
Torsional restraints  2200 mm

Restraint spacing (measured from column face):

At eaves position  Lateral restraint  10 mm
Torsional restraint  570 mm

For spacing of lateral restraints adjacent to the sharp end of haunch see diagram in "Haunch stability" section.
Location: Ex3 - Uniform section only based on Example 13

Rafter stability

Calculations in accordance with BS EN 1993-1-1:2005 Clause 6.3.5 and Annex BB.3.

The proforma assumes a plastic hinge forms just below the portal apex (at the point of max sagging moment).

Loads and moments are factored.

UNIFORM SECTION OF RAFTER

Design moment about major axis \( M_{yEd} = 283.8 \text{ kNm} \)
Design shear force \( V_{zEd} = 150 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 92 \text{ kN} \)
Length of member \( L = 12.145 \text{ m} \)
406 x 178 x 60 UKB
Axial load at hinge position \( N_{Ed'} = 92 \text{ kN} \)
Correction factor \( k_c = 1 \)
Dist. between lateral restraints \( L_z = 1500 \text{ mm} \)
Dist. between torsional restraints \( L_{zt} = 4500 \text{ mm} \)
Design span moment near the apex \( M_{yEd1} = 279 \text{ kNm} \)
Design axial load \( N_{Ed1} = 92 \text{ kN} \)

Length of the whole rafter \( L_y = 12145 \text{ mm} \)

Correction factor \( k_c = 1 \)
Maximum end moment \( M_{h(1)} = 284 \text{ kNm} \)
Maximum span moment \( M_{s(1)} = -279 \text{ kNm} \)
End moment \( pM_{h(1)} = -247 \text{ kNm} \)

HOGGING MOMENT REGION

Eaves to 1st. torsional restraint \( L_{zt1} = 1226 \text{ mm} \)
Maximum design moment \( B_{M1} = 284 \text{ kNm} \)
Axial load \( N_{Ed1} = 92 \text{ kN} \)
Correction factor \( k_c = 1 \)
Resistance moment \( M_{bi} = M_{bRd} \times (1 - N_{Ed'}/N_{bRd}) = 315.57 \text{ kNm} \)
Design moment \( M_{xi} = BM = 284 \text{ kNm} \)
As \( M_{xi} \leq M_{bi} \) stability is ensured between effective torsional restraints for the section chosen.
1st to 2nd torsional restraint \( L_{tz2} = 1500 \) mm
Max moment @ 1st tors.restraint \( B M_2 = 251 \) kNm
Axial load @ 1st tors.restraint \( N_{Ed2} = 90 \) kN
Correction factor \( k_c = 1 \)
Resistance moment \( M_{bi} = M_{bRd} \times (1 - N_{Ed}'/N_{bRd}) = 315.88 \) kNm
Design moment \( M_{xi} = BM = 251 \) kNm
As \( M_{xi} \leq M_{bi} \) stability is ensured between effective torsional restraints for the section chosen.

**DESIGN SUMMARY**

\[ \begin{array}{c|c|c}
1226 \text{ mm} & 1500 \\
\hline
\text{X} & \text{X} & \text{X} \\
\hline
\text{C.L. of member} & \text{C.L. of member} & \text{C.L. of member} \\
\hline
\text{leg} & \text{leg} & \text{leg} \\
\end{array} \]

\( X = \) lateral restraint
\( * = \) torsional restraint

**UNIVERSAL BEAM**
406 x 178 x 60 UKB Grade S 275
Section is satisfactory for bending, buckling resistance between intermediate restraints and overall buckling between torsional restraints.

Restraint spacing:
Rafter plastic hinge is assumed to form just below the portal apex at the point of maximum sagging moment. Provide restraint to both flanges at the plastic hinge location.

Adjacent to apex hinge
- First lateral restraint 975 mm
Along the rafter
- Lateral restraints 1500 mm
- Torsional restraints 4500 mm

Restraint spacing (measured from column face):

- At eaves position
  - Lateral restraint 1226 mm
  - Torsional restraint 1226 mm

- Torsional restraint 2726 mm
Sample output for SCALE Proforma 411. (ans=3)           Page:  3
Steel design to BS5950-1:2000 and Eurocode 3         Made by:  IFB
Portal frame rafter design                              Date:  02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.              Ref No:  SC411 EC

284 kNm

Hogging BM

Point of contraflexure

Apex

279 kNm

leg

C.L

12145 mm
Location: Ex4 - Based on SCI P399 with hinge at sharp end of haunch

Rafter stability

Calculations in accordance with BS EN 1993-1-1:2005 Clause 6.3.5 and Annex BB.3.

The proforma assumes a plastic hinge forms just below the portal apex (at the point of max sagging moment).

Loads and moments are factored.

UNIFORM SECTION OF RAFTER

Design moment about major axis $My_{Ed}=915 \text{ kNm}$
Design shear force $Vz_{Ed}=100 \text{ kN}$
Design axial load (+ve comp) $N_{Ed}=164 \text{ kN}$
Length of member $L=16 \text{ m}$
610 x 229 x 101 UKB
Axial load at hinge position $N_{Ed}'=164 \text{ kN}$
Correction factor $kc=1$
Dist. between lateral restraints $L_z=1800 \text{ mm}$
Dist. between torsional restraints $L_{zt}=5400 \text{ mm}$
Design span moment near the apex $My_{Ed1}=915 \text{ kNm}$
Design axial load $N_{Ed1}=164 \text{ kN}$
Length of the whole rafter $L_y=20000 \text{ mm}$

Maximum moment on segment $M=915 \text{ kNm}$
Far end moment $psiM=904 \text{ kNm}$
Maximum end moment $M_{h(1)}=1711 \text{ kNm}$
Maximum span moment $M_{s(1)}=-915 \text{ kNm}$
End moment $pM_{h(1)}=-904 \text{ kNm}$
HOGGING MOMENT REGION

610 x 229 x 101 UKB
Vertical overall depth of section ho=1167 mm
Pitch of rafter \( \theta = 8 \)°
Pitch of haunch \( \gamma = 17.14 \)°

\[
\begin{array}{cccccc}
2723 \text{ mm} & 935 \text{ mm} & 950 \text{ mm} & 1800 \text{ mm} \\
\text{Lp} & \text{Lmh} & \text{Lm} & \text{Lz}
\end{array}
\]

\[\begin{array}{cccccc}
\text{X} & \text{X} & \text{X} & \text{X} & \text{X}
\end{array}\]

C.L. of member

Assume a plastic hinge at point B. Zone B terminates at point of contraflexure.

\[\text{Design axial load} \quad N_{Ed3} = 164 \text{ kN}
\]

\[\text{Length for } LTB \quad LT = Lz/1000 = 2.723 \text{ m}
\]

\[\text{Critical moment} \quad M_{cr} = C_1 \cdot E_t \cdot S_r / 10^6 = 5751.9 \text{ kNm}
\]

\[\text{Design buckling resistance moment} \quad M_{brd} = C_1 \cdot L_T \cdot W_{pl} \cdot f_y / (\gamma \cdot M_{1} \cdot 10^3)
\]

\[= 2488.6 \text{ kNm}
\]

As \( M_{yEd'} \leq M_{brd} \) (1711 kNm \( \leq 2488.6 \text{ kNm} \), design moment is within the buckling resistance moment. Hence OK.

\[\text{Resistance moment} \quad M_{bi} = M_{brd} \cdot (1 - N_{Ed}' / N_{brd}) = 2342.4 \text{ kNm}
\]

\[\text{Design moment} \quad M_{xi} = M_{brd} = 1711 \text{ kNm}
\]

As \( M_{xi} \leq M_{bi} \) stability is ensured between effective torsional restraints for the section chosen.

Zone B - segment adjacent to sharp end of haunch

The uniform rafter segment adjacent to the sharp end of the haunch in hogging moment zone B (4000 mm long) will be checked.

\[\text{Dist. between eff. torsional restr.} \quad L_{tz4} = 3305 \text{ mm}
\]

\[\text{Maximum design moment} \quad B_{M1} = M_B = 781 \text{ kNm}
\]

\[\text{Minimum design moment} \quad B_{M4} = 135.7 \text{ kNm}
\]

\[\text{Design axial load} \quad N_{Ed4} = 164 \text{ kN}
\]
Length for LTB \( LT = \frac{Lz}{1000} = 3.305 \text{ m} \)

Critical moment \( Mcr = C1*\frac{Et*Sr}{10^6} = 2763.1 \text{ kNm} \)

Design buckling resistance moment \( MbRd = \frac{\chi LT*Wply*fy}{\gamma M1*10^3} \)

\( = 999.96 \text{ kNm} \)

As \( MyEd' \leq MbRd \ (781 \text{ kNm} \leq 999.96 \text{ kNm}) \), design moment is within the buckling resistance moment. Hence OK.

Resistance moment \( Mbi = MbRd*(1-NEd'/NbRd) = 941.25 \text{ kNm} \)

Design moment \( Mxi = BM = 781 \text{ kNm} \)

As \( Mxi \leq Mbi \) stability is ensured between effective torsional restraints for the section chosen.

**DESIGN SUMMARY**

**UNIVERSAL BEAM** 610 x 229 x 101 UKB Grade S 355

**HAUNCH SECTION** 610 x 229 x 101 UKB Grade S 355

Section is satisfactory for bending, buckling resistance between intermediate restraints and overall buckling between torsional restraints.

Restraint spacing:

Rafter plastic hinge is assumed to form just below the portal apex at the point of maximum sagging moment and at the sharp end of the haunch. Provide restraint to both flanges at the plastic hinge locations. If this is not practicable, restraint to both flanges should be provided within a distance of half the depth of the member.

Adjacent to apex hinge  First lateral restraint 950 mm

Along the rafter  Lateral restraints 1800 mm

Torsional restraints 5400 mm

Restraint spacing (measured from column face):

At eaves position  Lateral restraint 2723 mm

Torsional restraint 2723 mm

Torsional restraint 3658 mm

Provide an additional bottom flange torsional restraint at a distance \( \leq 3305 \text{ mm} \) from the sharp end of the haunch in the uniform section and before the point of contraflexure.

For spacing of lateral restraints adjacent to the sharp end of haunch see diagram in "Haunch stability" section.
Steel design to BS5950-1:2000 and Eurocode 3
Portal frame rafter design
Copyright 1986-2019 Fitzroy Systems Ltd.

1711 kNm
781 kNm

Point of contraflexure
Apex

Zone A
Zone B
Zone C

915 kNm

leg
3658 mm
4000 mm
12000 mm

C.L
Location: Ex5 - As Ex4 but without a hinge at sharp end of haunch

Rafter stability

Calculations in accordance with BS EN 1993-1-1:2005 Clause 6.3.5 and Annex BB.3.

The proforma assumes a plastic hinge forms just below the portal apex (at the point of max sagging moment).

Loads and moments are factored.

UNIFORM SECTION OF RAFTER

- Design moment about major axis: $My_{Ed} = 915 \text{ kNm}$
- Design shear force: $Vz_{Ed} = 100 \text{ kN}$
- Design axial load (+ve comp): $N_{Ed} = 164 \text{ kN}$
- Length of member: $L = 16 \text{ m}$
- Axial load at hinge position: $N_{Ed}' = 164 \text{ kN}$
- Correction factor: $k_c = 1$
- Dist. between lateral restraints: $L_z = 1800 \text{ mm}$
- Dist. between torsional restraints: $L_{zt} = 5400 \text{ mm}$
- Design span moment near the apex: $My_{Ed1} = 915 \text{ kNm}$
- Design axial load: $N_{Ed1} = 164 \text{ kN}$

- Length of the whole rafter: $L_y = 20000 \text{ mm}$
- Maximum moment on segment: $M = 915 \text{ kNm}$
- Far end moment: $p_{siM} = 904 \text{ kNm}$
- Maximum end moment: $M_{h(1)} = 1711 \text{ kNm}$
- Maximum span moment: $M_{s(1)} = -915 \text{ kNm}$
- End moment: $pM_{h(1)} = -904 \text{ kNm}$
HOGGING MOMENT REGION

610 x 229 x 101 UKB
Vertical overall depth of section ho=1167 mm
Pitch of rafter \( \Theta = 8 \) °
Pitch of haunch \( \Gamma = 17.14 \) °

\[
\begin{align*}
&\text{1829 mm} & \text{1829 mm} & \text{1800 mm} \\
&L_p & L_p & L_z \\
&X & X & X & X & X
\end{align*}
\]

1167 \( \text{C.L. of member} \) \( \text{Hogging BM Zone B} \)
\( L_z \) \( \text{Ls} \) \( \text{Lk} \) \( \text{Hogging BM Zone B} \)
\( \text{terminates at point of contraflexure} \)
\( \text{X} = \text{lateral restraint} \)
\( \text{* = torsional restraint} \)

ELASTIC STABILITY CHECKS - Zones A & B

Zone A - segment adjacent to column face

The haunched rafter segment adjacent to column face (1829 mm long) in hogging moment zone A will be checked.
Dist. between eff. torsional restr. \( L_{t3} = 1829 \) mm
Max design moment on segment \( BM_1 = MA = 1711 \) kNm
Min design moment on segment \( BM_3 = MAB = 1246 \) kNm
Design axial load \( N_{Ed3} = 164 \) kN
Length for LTB \( LT = L_z / 1000 = 1.829 \) m
Critical moment \( Mcr = C1 * E_t * S_r / 10^6 = 11723 \) kNm
Design buckling resistance moment \( Mb_{Rd} = C1 * L_T * W_{ply} * f_y / (\gamma M_1 * 10^3) = 2488.6 \) kNm

As \( My_{Ed} \leq Mb_{Rd} \) (1711 kNm \( \leq 2488.6 \) kNm), design moment is within the buckling resistance moment. Hence OK.
Resistance moment \( M_{bi} = Mb_{Rd} * (1 - N_{Ed} / Nb_{Rd}) = 2342.4 \) kNm
Design moment \( M_{xi} = BM = 1711 \) kNm
As \( M_{xi} \leq M_{bi} \) stability is ensured between effective torsional restraints for the section chosen.

Zone B - segment adjacent to sharp end of haunch

The uniform rafter segment adjacent to the sharp end of the haunch in hogging moment zone B (4000 mm long) will be checked.
Dist. between eff. torsional restr. \( L_{t4} = 3305 \) mm
Maximum design moment \( BM_1 = MB = 781 \) kNm
Minimum design moment \( BM_4 = 135.7 \) kNm
Design axial load \( N_{Ed4} = 164 \) kN
Length for LTB  \( LT = \frac{Lz}{1000} = 3.305 \text{ m} \)

Critical moment  \( Mcr = C1*Et*Sr/10^6 = 2763.1 \text{ kNm} \)

Design buckling resistance moment  \( MbRd = \text{ChiLT}*Wply*fy/(\text{gamM1}*10^3) = 999.96 \text{ kNm} \)

As  \( MyEd' \leq MbRd \) (781 kNm \( \leq 999.96 \text{ kNm} \)), design moment is within the buckling resistance moment. Hence OK.

Resistance moment  \( Mbi = MbRd*(1-NEd'/NbRd) = 941.25 \text{ kNm} \)

Design moment  \( Mxi = BM = 781 \text{ kNm} \)

As  \( Mxi \leq Mbi \) stability is ensured between effective torsional restraints for the section chosen.

**DESIGN SUMMARY**

**UNIVERSAL BEAM**  
610 x 229 x 101 UKB Grade S 355

**HAUNCH SECTION**  
610 x 229 x 101 UKB Grade S 355

Section is satisfactory for bending, buckling resistance between intermediate restraints and overall buckling between torsional restraints.

Restraint spacing:
Rafter plastic hinge is assumed to form just below the portal apex at the point of maximum sagging moment. Provide restraint to both flanges at the plastic hinge location. If this is not practicable, restraint to both flanges should be provided within a distance of half the depth of the member.

Adjacent to apex hinge  
First lateral restraint 950 mm

Along the rafter  
Lateral restraints 1800 mm
Torsional restraints 5400 mm

Restraint spacing (measured from column face):

At eaves position  
Lateral restraint 1829 mm
Torsional restraint 1829 mm
Torsional restraint 3658 mm

Provide an additional bottom flange torsional restraint at a distance \( \leq 3305 \text{ mm} \) from the sharp end of the haunch on the higher side beyond point B.
• 1711 kNm
  • 781 kNm
Hogging BM
  • Point of contraflexure
  • Apex

<table>
<thead>
<tr>
<th>Zone A</th>
<th>Zone B</th>
<th>Zone C</th>
</tr>
</thead>
<tbody>
<tr>
<td>3658 mm</td>
<td>4000 mm</td>
<td>12000 mm</td>
</tr>
<tr>
<td>Sagging BM</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Leg 915 kNm
Location: Ex6 - Uses UC section for rafter and haunch

Rafter stability

Calculations in accordance with BS EN 1993-1-1:2005 Clause 6.3.5 and Annex BB.3.

The proforma assumes a plastic hinge forms just below the portal apex (at the point of max sagging moment).

Loads and moments are factored.

UNIFORM SECTION OF RAFTER

Design moment about major axis \( M_{Ed}=600 \text{ kNm} \)
Design shear force \( V_{zEd}=200 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed}=400 \text{ kN} \)
Length of member \( L=6 \text{ m} \)
356 x 368 x 129 UKC
Axial load at hinge position \( N_{Ed}'=400 \text{ kN} \)
Correction factor \( k_{c}=1 \)
Dist. between lateral restraints \( L_{z}=1500 \text{ mm} \)
Dist. between torsional restraints \( L_{zt}=4500 \text{ mm} \)
Design span moment near the apex \( M_{Ed1}=400 \text{ kNm} \)
Design axial load \( N_{Ed1}=200 \text{ kN} \)

Length of the whole rafter \( L_{y}=9000 \text{ mm} \)

Maximum moment on segment \( M=400 \text{ kNm} \)
Far end moment \( psiM=390 \text{ kNm} \)
Maximum end moment \( M_{h(1)}=600 \text{ kNm} \)
Maximum span moment \( M_{s(1)}=-400 \text{ kNm} \)
End moment \( pM_{h(1)}=-380 \text{ kNm} \)
HOGGING MOMENT REGION

356 x 368 x 129 UKC
Vertical overall depth of section ho=650 mm
Pitch of rafter Theta=14°
Pitch of haunch Gamma=17°

<table>
<thead>
<tr>
<th>610 mm</th>
<th>2390 mm</th>
<th>1000 mm</th>
<th>1500 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lp</td>
<td>Lmh</td>
<td>Lm</td>
<td>Lz</td>
</tr>
</tbody>
</table>

X X X X X X

C.L. of member

Assume a plastic hinge at point B. Zone B terminates at point of contraflexure.

ELASTIC STABILITY CHECKS - Zones A & B

Zone A - segment adjacent to column face

The haunched rafter segment adjacent to column face (610 mm long) in hogging moment zone A will be checked.
Dist. between eff. torsional restr. Ltz3=610 mm
Max design moment on segment BM1=MA=600 kNm
Min design moment on segment BM3=MAB=559.33 kNm
Design axial load NEd3=92 kN
Length for LTB LT=Lz/1000=0.61 m
Critical moment Mcr=C1*Et*Sr/10^6=255535 kNm
Design buckling resistance moment MbRd=ChiLT*Wply*fy/(gamM1*10^3) =1287.9 kNm
As MyEd' ≤ MbRd (600 kNm ≤ 1287.9 kNm), design moment is within the buckling resistance moment. Hence OK.
Resistance moment Mbi=MbRd*(1-NEd'/NbRd)=1260.6 kNm
Design moment Mxi=BM=600 kNm
As Mxi ≤ Mbi stability is ensured between effective torsional restraints for the section chosen.

Zone B - segment adjacent to sharp end of haunch

The uniform rafter segment adjacent to the sharp end of the haunch in hogging moment zone B (2776 mm long) will be checked.
Dist. between eff. torsional restr. Ltz4=2776 mm
Maximum design moment BM1=MB=400 kNm
Minimum design moment BM4=0 kNm
Design axial load NEd4=92 kN
Length for LTB

LT=Lz/1000=2.776 m

Critical moment

Mcr=C1*Et*Sr/10^6=12380 kNm

Design buckling resistance moment

MbRd=ChiLT*Wply*fy/(gamM1*10^3)=657.2 kNm

As MyEd' ≤ MbRd (400 kNm ≤ 657.2 kNm), design moment is within the buckling resistance moment. Hence OK.

Resistance moment

Mbi=MbRd*(1-NEd'/NbRd)=643.29 kNm

Design moment

Mxi=BM=400 kNm

As Mxi ≤ Mbi stability is ensured between effective torsional restraints for the section chosen.

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>UNIVERSAL COLUMN</th>
<th>356 x 368 x 129 UKC Grade S 275</th>
</tr>
</thead>
<tbody>
<tr>
<td>HAUNCH SECTION</td>
<td>356 x 368 x 129 UKB Grade S 275</td>
</tr>
</tbody>
</table>

Section is satisfactory for bending, buckling resistance between intermediate restraints and overall buckling between torsional restraints.

Restraint spacing:

Rafter plastic hinge is assumed to form just below the portal apex at the point of maximum sagging moment and at the sharp end of the haunch. Provide restraint to both flanges at the plastic hinge locations. If this is not practicable, restraint to both flanges should be provided within a distance of half the depth of the member.

<table>
<thead>
<tr>
<th>Adjacent to apex hinge</th>
<th>First lateral restraint</th>
<th>1000 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Along the rafter</td>
<td>Lateral restraints</td>
<td>1500 mm</td>
</tr>
<tr>
<td></td>
<td>Torsional restraints</td>
<td>4500 mm</td>
</tr>
</tbody>
</table>

Restraint spacing (measured from column face):

| At eaves position      | Lateral restraint       | 610 mm  |
|                        | Torsional restraint     | 3000 mm |

For spacing of lateral restraints adjacent to the sharp end of haunch see diagram in 'Haunch stability' section.
Sample output for SCALE Proforma 411. (ans=6) Page: 4
Steel design to BS5950-1:2000 and Eurocode 3 Made by: IFB
Portal frame rafter design Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd. Ref No: SC411 EC

- 600 kNm
- Hogging BM
- Point of contraflexure
- 400 kNm
- Sagging BM
- Apex

Zone A: 3000 mm
Zone B: 2776 mm
Zone C: 3224 mm

SCALE 5.48 Office 1007 Proforma 411
Location: Ex7 - Uses UC section for rafter and haunch

Rafter stability

Calculations in accordance with BS EN 1993-1-1:2005 Clause 6.3.5 and Annex BB.3.

The proforma assumes a plastic hinge forms just below the portal apex (at the point of max sagging moment).

Loads and moments are factored.

UNIFORM SECTION OF RAFTER

Design moment about major axis  MyEd=600 kNm
Design shear force  VzEd=200 kN
Design axial load (+ve comp)  NEd=200 kN
Length of member  L=6 m
356 x 368 x 129 UC
Dist. between lateral restraints  Lz=1500 mm
Dist. between torsional restraints  Lzt=4500 mm
Design span moment near the apex  MyEd1=400 kNm
Design axial load  NEd1=200 kN
Length of the whole rafter  Ly=9000 mm

Maximum moment on segment  M=600 kNm
Far end moment  psiM=390 kNm
Maximum end moment  Mh(1)=600 kNm
Maximum span moment  Ms(1)=-400 kNm
End moment  pMh(1)=-380 kNm

HOGGING MOMENT REGION

eaves to 1st. torsional restraint  Ltz1=4500 mm
Maximum design moment  BM1=600 kNm
Axial load  NEd1=200 kN
Maximum moment on member  M=600 kNm
Far end moment  psiM=390 kNm
Resistance moment  Mbi=MbRd*(1-NEd'/NbRd)=626.96 kNm
Design moment  Mxi=BM=600 kNm

As Mxi ≤ Mbi stability is ensured between effective torsional restraints for the section chosen.
**DESIGN SUMMARY**

```
1500 mm             1500
³ X                  X                   X ³
— — — — — — — — — — — — — — — — — — — — — — — — C.L. of member
³ *                  leg
C.L          4500 mm      to next torsional restraint
³ X                  X                   X ³
X = lateral restraint
* = torsional restraint
```

**UNIVERSAL COLUMN**

356 x 368 x 129 UKC Grade S 275
Section is satisfactory for bending, buckling resistance between intermediate restraints and overall buckling between torsional restraints.

Restraint spacing:
Rafter plastic hinge is assumed to form just below the portal apex at the point of maximum sagging moment. Provide restraint to both flanges at the plastic hinge location.

Along the rafter
- Lateral restraints 1500 mm
- Torsional restraints 4500 mm

Restraint spacing (measured from column face):

- At eaves position
  - Lateral restraint 1500 mm
  - Torsional restraint 4500 mm

- 600 kNm
  - Hogging BM
  - Point of contraflexure
  - Apex

- Sagging BM
  - 400 kNm
Location: Example 3 'SHS Design Examples'

### Structural Hollow Section Design

Calculations are in accordance with BS5950-1:2000 Sections 4.7 and 4.8.

All loads and moments are factored.

Factored bending moment axis \( zz \) \( M_z = 5.04 \text{ kNm} \)
Factored SF in \( y \) direction \( F_v = 5.6 \text{ kN} \)
Axial load (+ve compression) \( F = 598 \text{ kN} \)
Length of member \( L = 3.6 \text{ m} \)

150 x 150 x 6.3 SHS - Hot finished.

Properties (cm): \( A = 35.8 \), \( r_x = 5.85 \), \( Z_x = 163 \), \( S_x = 192 \), \( I_x = 1220 \), \( J = 1910 \), \( C = 240 \)
Length between restraints \( z \) axis \( L_z = 3600 \text{ mm} \)
Length between restraints \( y \) axis \( L_y = 1800 \text{ mm} \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

| Quarter point | \( m_2 = 0 \text{ kNm} \) |
| Mid-span      | \( m_3 = 5.04 \text{ kNm} \) |
| Three quarter point | \( m_4 = 0 \text{ kNm} \) |
| Quarter point | \( m_{z2} = 0 \text{ kNm} \) |
| Mid-span      | \( m_{z3} = 5.04 \text{ kNm} \) |
| Three quarter point | \( m_{z4} = 0 \text{ kNm} \) |
| Maximum moment on central half | \( M_{24} = 5.04 \text{ kNm} \) |
| Quarter point | \( m_2 = 0 \text{ kNm} \) |
| Mid-span      | \( m_3 = 5.04 \text{ kNm} \) |
| Three quarter point | \( m_4 = 0 \text{ kNm} \) |

**HOT FINISHED**

**SQUARE HOLLOW SECTION**

**SECTION**

**SUMMARY**

In accordance with EN 10210

| Axial load | 598 kN |
| Compresson resistance | 890.21 kN |
| Maximum moment \( z \) axis | 5.04 kNm |
| Moment capacity | 52.8 kNm |
| Local capacity check | \( 0.70287 \leq 1 \) |
| Overall buckling checks | \( 0.77377 \leq 1 \), \( 0.65972 \leq 1 \) |
**Location: Example 2 - Bending and axial load**

**Structural Hollow Section Design**

Calculations are in accordance with BS5950-1:2000 Sections 4.7 and 4.8.

All loads and moments are factored.

Factored bending moment axis \( zz \) \( M_z = 350 \) kNm
Factored SF in \( y \) direction \( F_v = 129 \) kN
Axial load (+ve compression) \( F = -400 \) kN
Length of member \( L = 7.6 \) m

400 x 200 x 8 RHS - Cold formed.

Properties (cm): \( A = 91.2 \) \( r_x = 14.4 \) \( Z_x = 949 \) \( S_x = 1170 \) \( I_x = 19000 \)
\( J = 15800 \) \( C = 1130 \) \( Z_y = 652 \) \( S_y = 728 \) \( I_y = 6520 \)
\( r_y = 8.45 \)

TENSILE LOAD APPLIED

Young's Modulus \( E = 205 \) kN/mm\(^2\)

Estimated net area (tension) \( A_e = 91.2 \) cm\(^2\)

**COLD FORMED**

**RECTANGULAR HOLLOW SECTION** 400 x 200 x 8 RHS Grade S 355

**SECTION** Section is satisfactory for axial load, and overall buckling check.

**SUMMARY**

Axial load \(-400\) kN
Axial capacity 3237.6 kN
Maximum moment \( z \) axis 350 kNm
Moment capacity 415.35 kNm
Reduced moment cap. 408.31 kNm
Location: Example 3 - minor axis - compressive load

Structural Hollow Section Design

Calculations are in accordance with BS5950-1:2000 Sections 4.7 and 4.8.

All loads and moments are factored.

Factored bending moment axis \( zz \) \( M_z = 208 \text{ kNm} \)
Factored SF in y direction \( F_v = 194 \text{ kN} \)
Axial load (+ve compression) \( F = 300 \text{ kN} \)
Length of member \( L = 8 \text{ m} \)

250 x 450 x 8 RHS - Hot finished.

Properties (cm): \( A = 109 \) \( r_x = 10.6 \) \( Z_x = 971 \) \( S_x = 1080 \) \( I_x = 12100 \)
\( J = 27100 \) \( C = 1630 \) \( Z_y = 1340 \) \( S_y = 1620 \) \( I_y = 30100 \) \( r_y = 16.6 \)
Length between restraints \( z \) axis \( L_z = 8000 \text{ mm} \)
Length between restraints \( y \) axis \( L_y = 8000 \text{ mm} \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Far end BM \( \beta_m = 100 \text{ kNm} \)
Force applied through flange \( F_{bw} = 194 \text{ kN} \)
Stiff bearing length \( b_1 = 300 \text{ mm} \)
Distance to nearer end of member \( a_e = 150 \text{ mm} \)

HOT FINISHED

RECTANGULAR HOLLOW SECTION

Summary

Section is satisfactory for axial load, and overall buckling check.
Axial load \( 300 \text{ kN} \)
Compression resistance \( 2620.4 \text{ kN} \)
Maximum moment \( z \text{ axis} \) \( 208 \text{ kNm} \)
Moment capacity \( 254.43 \text{ kNm} \)
Local capacity check \( 0.9159 \leq 1 \)
Overall buckling checks \( 0.97881 \leq 1 \)
\( 0.4276 \leq 1 \)
Location: Example 3 'SHS Design Examples' - fire check

Structural Hollow Section Design

Calculations are in accordance with BS5950-1:2000 Sections 4.7 and 4.8.

All loads and moments are factored.

Factored bending moment axis zz $M_z = 5.04 \, \text{kNm}$
Factored SF in y direction $F_v = 5.6 \, \text{kN}$
Axial load (+ve compression) $F = 598 \, \text{kN}$
Length of member $L = 3.6 \, \text{m}$

150 x 150 x 6.3 SHS - Hot finished.
Properties (cm): $A = 35.8 \, \text{cm}^2 \quad r_x = 5.85 \, \text{cm} \quad Z_x = 163 \, \text{cm}^3 \quad S_x = 192 \, \text{cm}^2 \quad I_x = 1220 \, \text{cm}^4 \quad J = 1910 \, \text{cm}^4 \quad C = 240 \, \text{cm}$
Length between restraints z axis $L_z = 3600 \, \text{mm}$
Length between restraints y axis $L_y = 1800 \, \text{mm}$
Young's Modulus $E = 205 \, \text{kN/mm}^2$

Quarter point $m_2 = 0 \, \text{kNm}$
Mid-span $m_3 = 5.04 \, \text{kNm}$
Three quarter point $m_4 = 0 \, \text{kNm}$
Quarter point $m_{z2} = 0 \, \text{kNm}$
Mid-span $m_{z3} = 5.04 \, \text{kNm}$
Three quarter point $m_{z4} = 0 \, \text{kNm}$
Maximum moment on central half $M_{24} = 5.04 \, \text{kNm}$
Quarter point $m_2 = 0 \, \text{kNm}$
Mid-span $m_3 = 5.04 \, \text{kNm}$
Three quarter point $m_4 = 0 \, \text{kNm}$

HOT FINISHED
SQUARE HOLLOW SECTION
150 x 150 x 6.3 SHS Grade S 275
SECTION
Section is satisfactory for axial load, and overall buckling check.
SUMMARY
Axial load 598 kN
Compression resistance 890.21 kN
Maximum moment z axis 5.04 kNm
Moment capacity 52.8 kNm
Local capacity check $0.70287 \leq 1$
Overall buckling checks $0.77377 \leq 1$
$0.65972 \leq 1$
FIRE RESISTANCE
SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heated perimeter</td>
<td>450 mm²</td>
</tr>
<tr>
<td>Area of section</td>
<td>35.8 cm²</td>
</tr>
<tr>
<td>Section factor</td>
<td>125 /m</td>
</tr>
<tr>
<td>Load Ratio (R)</td>
<td>0.67</td>
</tr>
</tbody>
</table>

Section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Hollow section beam design

Calculations are in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Moment about y-y axis \( M_{yEd} = 10.1 \text{kNm} \)
Shear force \( V_{zEd} = 11.2 \text{kN} \)
Axial load (+ve compression) \( N_{Ed} = 525 \text{kN} \)
Length of member \( L = 3.6 \text{m} \)
150 x 150 x 5 SHS - Hot finished.
Properties (cm): \( A = 28.7 \text{cm}^2 \), \( i_y = 5.9 \text{cm} \), \( W_{ely} = 134 \text{cm}^3 \), \( W_{ply} = 156 \text{cm}^3 \), \( I_{y} = 1000 \text{cm}^4 \), \( I_{t} = 1550 \text{cm}^4 \), \( W_t = 197 \text{cm}^2 \)
Shear force at maximum BM \( V = 11.2 \text{kN} \)
Length between restraints (y-y) \( L_y = 3.6 \text{m} \)
Length between restraints (z-z) \( L_z = 1.8 \text{m} \)
Maximum end moment \( M_h(1) = 10.1 \text{kNm} \)
Maximum span moment \( M_s(1) = -10.1 \text{kNm} \)
End moment \( pM_h(1) = 10.1 \text{kNm} \)
Effective length parameter \( K = 1 \)
Length for LTB \( L_T = 1.8 \text{m} \)
Maximum moment on section \( M = 10.1 \text{kNm} \)
Far end moment \( psiM = -10.1 \text{kNm} \)

**HOT FINISHED**

**SQUARE HOLLOW SECTION** 150 x 150 x 5 SHS Grade S 355

**DESIGN**

Design BM y-y axis 10.1 kNm
Moment resistance (y-y) 55.38 kNm
Design compression load 525 kN
Buckling resistance 811.76 kN
Interaction factors \( 0.84909 \leq 1 \), \( 0.66209 \leq 1 \)
Location: Bending only

Hollow section beam design

Calculations are in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Moment about y-y axis \( M_{yEd} = 160 \text{ kNm} \)
Shear force \( V_{zEd} = 120 \text{ kN} \)
Axial load (+ve compression) \( N_{Ed} = 0 \text{ kN} \)
Length of member \( L = 8 \text{ m} \)
260 x 140 x 8 RHS - Hot finished.
Properties (cm): \( A = 60.8 \text{ cm}^2 \) \( iz = 5.78 \) \( W_{ely} = 413 \) \( W_{ply} = 511 \) \( I_y = 5370 \) \( W_t = 4700 \) \( W_{elz} = 290 \) \( W_{plz} = 331 \) \( I_z = 2030 \) \( i_y = 9.4 \)
Shear force at maximum BM \( V = 0 \text{ kN} \)
Effective length parameter \( K = 0.85 \)
Length for LTB \( L_T = 8 \text{ m} \)
Span of beam for deflection \( L_d = 8000 \text{ mm} \)

HOT FINISHED In accordance with EN 10210
RECTANGULAR HOLLOW SECTION 260 x 140 x 8 RHS Grade S 355
DESIGN Design BM y-y axis 160 kNm
SUMMARY Moment resistance (y-y) 181.41 kNm
Location: Default example

Circular Hollow Section (CHS)

Calculations in accordance with BS 5950:Part 1:2000
Sections 4.7 and 4.8.

All loads and moments are factored.

Design moment axis zz  \( M_z = 5.04 \text{ kNm} \)
Design SF in y direction  \( F_v = 5.6 \text{ kN} \)
Axial load (+ve compression)  \( F = 598 \text{ kN} \)
Length of member  \( L = 3.6 \text{ m} \)
Factored moment about axis yy  \( M_y = 0 \text{ kNm} \)
193.7 dia x 5 thick CHS - Hot finished.
Properties (cm): A=29.641  \( r = 6.6739 \)  \( Z = 136.32 \)  \( S = 178.08 \)  \( I = 1320.2 \)
Young's Modulus  \( E = 205 \text{ kN/mm}^2 \)
Length between restraints z axis  \( L_z = 3600 \text{ mm} \)
Length between restraints y axis  \( L_y = 1800 \text{ mm} \)
Quarter point  \( m_2 = 0 \text{ kNm} \)
Mid-span  \( m_3 = 5.04 \text{ kNm} \)
Three quarter point  \( m_4 = 0 \text{ kNm} \)
Quarter point  \( m_{z2} = 0 \text{ kNm} \)
Mid-span  \( m_{z3} = 5.04 \text{ kNm} \)
Three quarter point  \( m_{z4} = 0 \text{ kNm} \)
Maximum moment on central half  \( M_{24} = 5.04 \text{ kNm} \)

HOT FINISHED  
CIRCULAR HOLLOW  
SELECTION & DESIGN  
SUMMARY

In accordance with EN 10210
193.7 mm O.D. x 5 mm CHS Grade S 275
Section is satisfactory for bending axial load, local capacity and overall buckling check.
Axial load  \( 598 \text{ kN} \)
Compression resistance  \( 756.09 \text{ kN} \)
Maximum moment z zaxis  \( 5.04 \text{ kNm} \)
Moment capacity  \( 48.972 \text{ kNm} \)
Local capacity check  \( 0.83655 \leq 1 \)
Overall buckling check  \( 0.90581 \leq 1 \)
Location: Tensile load and moment about one axis

Circular Hollow Section (CHS)


All loads and moments are factored.

Design moment axis zz \( M_z = 125 \text{ kNm} \)
Design SF in y direction \( F_y = 98 \text{ kN} \)
Axial load (+ve compression) \( F = -300 \text{ kN} \)
Length of member \( L = 9 \text{ m} \)
Factored moment about axis yy \( M_y = 0 \text{ kNm} \)

TENSILE LOAD APPLIED
323.9 dia x 5 thick CHS - Hot finished.

Properties (cm): \( A = 50.093 \) \( r = 11.276 \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Estimated net area (tension) \( A_e = 50 \text{ cm}^2 \)

HOT FINISHED
CIRCULAR HOLLOW
SECTION & DESIGN
SUMMARY

In accordance with EN 10210
323.9 mm O.D. x 5 mm CHS Grade S 355
Section is satisfactory for bending axial load, local capacity and overall buckling check.
Axial load \(-300 \text{ kN}\)
Tensile capacity \(1775 \text{ kN}\)
Maximum moment z zaxis \(125 \text{ kNm}\)
Moment capacity \(157.47 \text{ kNm}\)
Local capacity check \(0.9628 \leq 1\)
Location: Biaxial bending

![Circular Hollow Section (CHS)]


All loads and moments are factored.

Design moment axis zz \( M_z = 47 \text{ kNm} \)
Design SF in y direction \( F_y = 46 \text{ kN} \)
Axial load (+ve compression) \( F = 48 \text{ kN} \)
Length of member \( L = 6 \text{ m} \)
Factored moment about axis yy \( M_y = 24 \text{ kNm} \)
219.1 dia x 5 thick CHS - Cold formed.
Properties (cm): \( A = 33.631 \) \( r = 7.5716 \)
\( Z = Z_x = 176 \) \( S = S_x = 229.24 \) \( I = I_x = 1928 \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Length between restraints z axis \( L_z = 6000 \text{ mm} \)
Length between restraints y axis \( L_y = 6000 \text{ mm} \)
Far end BM \( \beta_M = 0 \text{ kNm} \)
Maximum moment on segment \( M_z = 47 \text{ kNm} \)
Far end BM \( \beta_M = 0 \text{ kNm} \)
Far end BM \( \beta_M = 0 \text{ kNm} \)
Maximum moment on segment \( M_{max_y} = 24 \text{ kNm} \)
Far end BM \( \beta_M = 0 \text{ kNm} \)

COLD FORMED
CIRCULAR HOLLOW
SECTION & DESIGN
SUMMARY

In accordance with EN 10219
219.1 mm O.D. x 5 mm CHS Grade S 355
Section is satisfactory for bending axial load, local capacity and overall buckling check.
Axial load 48 kN
Compression resistance 755.88 kN
Maximum moment z axis 47 kNm
Moment capacity 74.975 kNm
Maximum moment y axis 24 kNm
Moment capacity 74.975 kNm
Local capacity check 0.98719 \( \leq \) 1
Overall buckling checks 0.5476 \( \leq \) 1
0.62598 \( \leq \) 1
Location: Default example + fire check

Circular Hollow Section (CHS)


All loads and moments are factored.

Design moment axis zz \( M_z = 5.04 \) kNm
Design SF in y direction \( F_y = 5.6 \) kN
Axial load (+ve compression) \( F = 598 \) kN
Length of member \( L = 3.6 \) m
Factored moment about axis yy \( M_y = 0 \) kNm
193.7 dia x 5 thick CHS - Hot finished.
Properties (cm): \( A = 29.641 \) \( r = 6.6739 \) \( Z = 136.32 \) \( S = 178.08 \) \( I = 1320.2 \)
Young's Modulus \( E = 205 \) kN/mm²

Length between restraints z axis \( L_z = 3600 \) mm
Length between restraints y axis \( L_y = 1800 \) mm
Quarter point \( m_2 = 0 \) kNm
Mid-span \( m_3 = 5.04 \) kNm
Three quarter point \( m_4 = 0 \) kNm
Quarter point \( m_{z2} = 0 \) kNm
Mid-span \( m_{z3} = 5.04 \) kNm
Three quarter point \( m_{z4} = 0 \) kNm
Maximum moment on central half \( M_{24} = 5.04 \) kNm
values to applied factored values \( R_f = 0.55 \)

HOT FINISHED
CIRCULAR HOLLOW SECTION & DESIGN
SUMMARY

In accordance with EN 10210
193.7 mm O.D. x 5 mm CHS Grade S 275
Section is satisfactory for bending axial load, local capacity and overall buckling check.
Axial load 598 kN
Compression resistance 756.09 kN
Maximum moment z zaxis 5.04 kNm
Moment capacity 48.972 kNm
Local capacity check 0.83655 ≤ 1
Overall buckling check 0.90581 ≤ 1

FIRE RESISTANCE SUMMARY

Heated perimeter 608.53 mm²
Area of section 29.641 cm²
Section factor 205 /m
Load Ratio (R) 0.47463
Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided.
Location: Default example - intermediate column length

Circular Hollow Section (CHS)

Calculations are in accordance with BS EN 1993-1-1:2005.
Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Moment about y-y axis $My_{Ed}=50 \text{ kNm}$
Shear force $V_{zEd}=22 \text{ kN}$
Axial load (+ve compression) $N_{Ed}=760 \text{ kN}$
Length of member $L=7 \text{ m}$
Moment about z-z axis $M_{zEd}=50 \text{ kNm}$
Accompanying shear force $V_{yEd}=22 \text{ kNm}$
244.5 dia x 8 thick CHS - Hot finished.
Properties (cm): $A=59.439 \quad i_y=8.3663 \quad W_{ely}=340.32 \quad W_{ply}=447.63 \quad I_y=4160.4 \quad I_t=8320.9$
Shear force at maximum BM $V=22 \text{ kN}$
Length between restraints (y-y) $L_y=7 \text{ m}$
Length between restraints (z-z) $L_z=7 \text{ m}$
Far end bending moment $pM(1)=-50 \text{ kNm}$
Far end bending moment $pM(2)=-50 \text{ kNm}$
Length for LTB $L_T=7 \text{ m}$
Effective length parameter $K=0.85$
Maximum moment on section $M=50 \text{ kNm}$
Far end moment $p_{siM}=-50 \text{ kNm}$

HOT FINISHED In accordance with EN 10210
CIRCULAR HOLLOW 244.5 mm O.D. x 8 mm CHS Grade S 355
DESIGN Design BM y-y axis 50 kNm
SUMMARY Moment resistance (y-y) 158.91 kNm
Design BM z-z axis 50 kNm
Moment resistance (z-z) 158.91 kNm
Design compression load 760 kN
Axial resistance 2110.1 kN
Interaction factors $0.89904 \leq 1$
$0.89904 \leq 1$
Location: Bending only

Circular Hollow Section (CHS)

Calculations are in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Moment about y-y axis $M_{yEd}=155 \text{ kNm}$
Shear force $V_{zEd}=77.5 \text{ kN}$
Axial load (+ve compression) $N_{Ed}=0 \text{ kN}$
Length of member $L=8 \text{ m}$
Moment about z-z axis $M_{zEd}=70 \text{ kNm}$
Accompanying shear force $V_{yEd}=30 \text{ kNm}$

244.5 dia x 8 thick CHS - Cold formed.
Properties (cm): $A=59.439$ $i_y=8.3663$ $W_{ely}=340.32$ $W_{ply}=447.63$ $I_y=4160.4$ $I_t=8320.9$

Shear force at maximum BM $V=0 \text{ kN}$
Length for LTB $L_T=8 \text{ m}$
Effective length parameter $K=0.85$
Span of member for deflection $L_{d}=8000 \text{ mm}$

COLD FORMED

CIRCULAR HOLLOW 244.5 mm O.D. x 8 mm CHS Grade S 355
DESIGN
Moment resistance (y-y) 155 kNm
SUMMARY
Moment resistance (y-y) 158.91 kNm
Design BM $y$-$y$ axis 155 kNm
Design BM $z$-$z$ axis 70 kNm
Moment resistance (z-z) 158.91 kNm
Location: Tension and bending

Circular Hollow Section (CHS)

Calculations are in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Moment about y-y axis \( M_{yEd} = 45 \text{ kNm} \)
Shear force \( V_{zEd} = 78 \text{ kN} \)
Axial load (+ve compression) \( N_{Ed} = -500 \text{ kN} \)
Length of member \( L = 8 \text{ m} \)
Moment about z-z axis \( M_{zEd} = 0 \text{ kNm} \)
244.5 dia x 5 thick CHS - Cold formed.
Properties (cm): \( A = 37.621 \text{ cm}^2 \)
\( i_y = 8.4694 \text{ cm} \)
\( W_{ely} = 220.74 \text{ cm}^3 \)
\( W_{ply} = 286.84 \text{ cm}^3 \)
\( I_y = 2698.6 \text{ cm}^4 \)
\( I_t = 5397.2 \text{ cm}^4 \)
Shear force at maximum BM \( V = 78 \text{ kN} \)
Length for LTB \( L_{LT} = 8 \text{ m} \)
Effective length parameter \( K = 1 \)
Maximum moment on section \( M = 45 \text{ kNm} \)
Far end moment \( \psi M = 0 \text{ kNm} \)

COLD FORMED
CIRCULAR HOLLOW 244.5 mm O.D. x 5 mm CHS Grade S 355
DESIGN
Design BM y-y axis 45 kNm
Moment resistance (y-y) 78.364 kNm
Design tensile load 500 kN
Axial resistance 1335.5 kN
Interaction factors 0.94863 ≤ 1
Location: Compression and bending about one axis

Circular Hollow Section (CHS)

Calculations are in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Moment about y-y axis \( M_{yEd} = 45 \text{ kNm} \)
Shear force \( V_{zEd} = 78 \text{ kN} \)
Axial load (+ve compression) \( N_{Ed} = 350 \text{ kN} \)
Length of member \( L = 8 \text{ m} \)
Moment about z-z axis \( M_{zEd} = 0 \text{ kNm} \)
273 dia x 5 thick CHS - Cold formed.

Properties (cm): \( A = 42.097 \text{ cm}^2 \) \( i_y = 9.4769 \text{ cm} \) \( W_{ely} = 276.98 \text{ cm}^3 \) \( W_{ply} = 359.16 \text{ cm}^3 \) \( I_y = 3780.8 \text{ cm}^4 \) \( I_t = 7561.6 \text{ cm}^4 \)

Shear force at maximum BM \( V = 78 \text{ kN} \)
Length between restraints (y-y) \( L_y = 8 \text{ m} \)
Length between restraints (z-z) \( L_z = 8 \text{ m} \)
Far end bending moment \( pM(1) = 0 \text{ kNm} \)
Length for LTB \( L_T = 8 \text{ m} \)
Effective length parameter \( K = 1 \)
Maximum moment on section \( M = 45 \text{ kNm} \)
Far end moment \( psiM = 0 \text{ kNm} \)

COLD FORMED
CIRCULAR HOLLOW
DESIGN
SUMMARY

In accordance with EN 10219

273 mm O.D. x 5 mm CHS Grade S 355
Design BM y-y axis \( 45 \text{ kNm} \)

Moment resistance (y-y) \( 98.329 \text{ kNm} \)
Design compression load \( 350 \text{ kN} \)
Axial resistance \( 1494.5 \text{ kN} \)
Interaction factors \( 0.84088 \leq 1 \)
\( 0.76994 \leq 1 \)
Location: Example 'Design of RHS Slimflor Floor Edge Beams'

Simply supported edge beam

Slim floor construction non-composite beam. Calculations are in accordance with BS5950 and the SCI publication entitled 'Design of RHS Slimflor Edge beams'.

A plate is welded to the bottom of the hollow section to support the precast floor units.

Beam span
L1=6 m

Beam cross-centres
L2=6 m

Precast floor units
wu=2.44 kN/m²

Construction load
wc=0.5 kN/m²

Floor (decking) & minor services
wf=0.2 kN/m²

Ceiling & services
wci=0.5 kN/m²

Self weight of steel
ws=1.2 kN/m

Cladding (dead)
Wcl=8 kN/m

Occumncy
wo=3.5 kN/m²

Partitions
wp=1 kN/m²

Loading summary

1. Cladding
Wc=1.4*Wcl*L1=67.2 kN

2. Self weight
Wsw=1.4*ws*L1=10.08 kN

3. Dead + Imposed
Wdes=(1.4*(wf+wu+wci)+1.6*wi)*L1*L2
 /2=208.73 kN

Wc=67.2 kN

Wdes=208.73 kN

Wsw=10.08 kN

250 x 150 x 16 RHS - Hot finished.

Properties (cm):
A=115 rx=8.79 Zx=710 Sx=906 Ix=8880
J=8870 C=849 Zy=516 Sy=625 Iy=3870 ry=5.8

Young's Modulus
E=205 kN/mm²

Thickness of plate
Tp=15 mm

Width of plate
Bp=270 mm
Weld leg length: $s = 6 \text{ mm}$  
Thickness of plate: $t_p = 10 \text{ mm}$  
Diameter of bolts: $d_b = 20 \text{ mm}$

---

Depth of end plate: $D_{ep} = 400 \text{ mm}$  
Width of end plate: $B_{ep} = 200 \text{ mm}$  
Bolt cross-centres: $g = 140 \text{ mm}$  
Bolt pitch vertically: $a_1 = 340 \text{ mm}$

---

**HOT FINISHED**  
**RECTANGULAR HOLLOW SECTION**  
$250 \times 150 \times 16 \text{ RHS}$  
Grade S 355  
Design strength: $355 \text{ N/mm}^2$  

**BOTTOM PLATE**  
$270 \text{ mm} \times 15 \text{ mm}$  
Grade S 355  
Design strength: $355 \text{ N/mm}^2$  
Design shear force: $143 \text{ kN}$

---

**DESIGN SUMMARY**  
Shear capacity: $1530.9 \text{ kN}$  
Design moment: $214.51 \text{ kNm}$  
Buckling resistance: $463 \text{ kNm}$  

(A) Ultimate limit state  
Buckling check: $0.43257 \leq 1$  
Local capacity check: $243.18 \text{ N/mm}^2$

(B) Serviceability state  
Imposed load deflection: $8.7333 \text{ mm}$  
Limiting deflection: $12 \text{ mm}$
Location: Ex1 - From 'Design of RHS Slimflor Floor Edge Beams'

Simply supported edge beam

Slim floor construction non-composite beam. Calculations are in accordance with EC3 Part 1-1 and SCI publication entitled 'Design of RHS Slimflor Edge beams'.

A plate is welded to the bottom of the hollow section to support the precast floor units.

Beam span
L1=6 m

Beam cross-centres
L2=6 m

Precast floor units
wu=2.44 kN/m²

Construction load
wc=0.5 kN/m²

Floor (decking) & minor services
wf=0.2 kN/m²

Ceiling & services
wci=0.5 kN/m²

Self weight of steel
ws=1.2 kN/m

Cladding (permanent)
Wc1=8 kN/m

Occupancy
wo=3.5 kN/m²

Partitions
wp=1 kN/m²

\[ Wc=64.8 \text{ kN} \]

\[ Wdes=197.8 \text{ kN} \]

\[ Wsw=9.72 \text{ kN} \]

250 x 150 x 16 RHS - Hot finished.

Properties (cm): A=115 iz=5.8 Wely=710 Wply=906 Iy=8880
It=8870 Wt=849 Welz=516 Wplz=625 Iz=3870 iy=8.79

Dimensions (mm): h=250 b=150 t=16

Thickness of plate
Tp=15 mm

Width of plate
Bp=270 mm
Weld size (leg length) \( s = 6 \text{ mm} \)
Thickness of plate \( \text{tep} = 10 \text{ mm} \)
Diameter of bolts \( \text{db} = 20 \text{ mm} \)

Depth of end plate \( \text{hep} = 400 \text{ mm} \)
Width of end plate \( \text{bep} = 200 \text{ mm} \)
Bolt cross-centres \( p_3 = 140 \text{ mm} \)
Bolt pitch vertically \( p_1 = 340 \text{ mm} \)
Weld size (leg length) \( \text{sw} = 12 \text{ mm} \)

HOT FINISHED
RECTANGULAR HOLLOW SECTION
250 x 150 x 16 RHS Grade S355
Design strength 355 N/mm²

BOTTOM PLATE
270 mm x 15 mm Grade S355
Design strength 355 N/mm²
Design shear force 136.16 kN

DESIGN
Shear plastic resistance 1473.1 kN

SUMMARY
Design moment 204.24 kNm
Buckling resistance 463.2 kNm
Variable load deflection 8.5254 mm
Limiting deflection 12 mm
Interaction factor 0.47081 ≤ 1
Simply supported beam

Slim floor construction
Non-composite beam.
Calculations are in accordance with BS5950 and 'Slim Floor Design and Construction' published by SCI.

Beam span
L1=7.5 m
Beam cross-centres
L2=7.5 m

Precast units
wu=2.67 kN/m²
Self weight of steel
ws=0.24 kN/m²
Construction load
wc=0.5 kN/m²
Floor & minor services
wf=0.25 kN/m²
Ceiling
wci=0 kN/m²
Grout to beam (to half depth)
wg=0.15 kN/m²
Occupancy
wo=2.5 kN/m²
Partitions
wp=1 kN/m²
305 x 305 x 118 UC.
Young's Modulus
E=205 kN/mm²
Thickness of plate
Tp=15 mm
Width of plate
Bp=510 mm
Weld leg length
s=8 mm
Thickness of plate
tp=10 mm
Grade of steel
pgrade=275
Diameter of bolts
db=24 mm
Total number of bolts
bn=6
CONNECTION SUMMARY

10                               310
ô                         ÃÄÄÄÄÄÄÄÄÄÄÄÄÄ´
Â     Ú¿         Â          Â    ÚÄÄÄÄÄÄÄÄÄÄÄÄÄ¿  Â
90     ³ÆÍÍÍÍÍÍÍ  Á 20      90    ³ÍÍÍÍÍÍÊÍÍÍÍÍͳ  ³ Dp=
Å     Ƶ                    Å    ³ o    º    o ³  ³ 340 mm
80     ³³                   80    ³      º      ³  ³
Å     Ƶ                    Å    ³ o    º    o ³  ³
80     ³³                   80    ³      º      ³  ³
Å     Ƶ                    Å    ³ o    º    o ³  ³
90     ³ÆÍÍÍÍÍÍÍ            90    ³ÍÍÍÍÍÍÊÍÍÍÍÍͳ  ³
Á     ÀÙ                    Á    ÀÄÄÄÄÄÄÄÄÄÄÄÄÄÙ  Á
ÃÄÅÄÄÄÄÄÄÄÄÄÅÄ´

Number of bolts 6
Bolt diameter 24 mm
Bolt grade 8.8
Plate grade S 275

UNIVERSAL COLUMN
305 x 305 x 118 UC Grade S 355
Design strength 345 N/mm²

BOTTOM PLATE
510 mm x 15 mm Grade S 355
Design strength 355 N/mm²
Design shear force 278.8 kN
Shear capacity 781.2 kN

DESIGN SUMMARY
Design moment 522.8 kNm
Buckling resistance 536.5 kNm
Buckling check 0.6404 ≤ 1
Local capacity check 195 N/mm²
In-service stage
Local capacity check 357.1 N/mm²
Serviceability stress 179 N/mm²
Imposed load deflection 12 mm
Limiting deflection 20.83 mm
Simply supported beam

Slim floor construction non-composite beam. Calculations are in accordance with EC3 Part 1-1 & SCI P385 entitled 'Design of Steel Beams in Torsion'.

A plate is welded to the bottom flange of the UC section extending beyond the bottom flange by 100 mm either side to support the precast floor units.

Beam span L1=7.5 m
Beam cross-centres L2=7.5 m

Precast concrete units wu=2.67 kN/m²
Self weight of steel ws=0.24 kN/m²
Construction load wc=0.5 kN/m²
Floor & minor services wf=0.25 kN/m²
Ceiling wc=0 kN/m²
Grout to beam (to half depth) wg=0.15 kN/m²
Occupancy wo=2.5 kN/m²
Partitions wp=1 kN/m²
305 x 305 x 118 UKC
Thickness of plate Tp=15 mm
Width of plate Bp=510 mm
Weld size (leg length) s=8 mm
Thickness of end plate tep=10 mm
Diameter of bolts db=24 mm
Total number of bolts to be used bn=6
Weld size (leg length) sw=10 mm
**CONNECTION SUMMARY**

<table>
<thead>
<tr>
<th>tep=10 mm</th>
<th>bep=300 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>e1</td>
<td>e4=10 mm 100 mm</td>
</tr>
<tr>
<td>p1</td>
<td>o</td>
</tr>
<tr>
<td>p1</td>
<td>o</td>
</tr>
<tr>
<td>e3</td>
<td>e4=10 mm 110 mm</td>
</tr>
<tr>
<td>p3=140 mm</td>
<td>e2=80 mm e2=80 mm</td>
</tr>
</tbody>
</table>

- **Number of bolts**: 6
- **Bolt diameter**: 24 mm
- **Bolt grade**: 8.8
- **Plate grade**: S 275

**UNIVERSAL COLUMN**
- 305 x 305 x 118 UKC Grade S 355
- Design strength: 345 N/mm²

**BOTTOM PLATE**
- 510 mm x 15 mm Grade S 355
- Design strength: 355 N/mm²
- Design shear force: 264.9 kN

**DESIGN SUMMARY**
- Shear plastic resistance: 855.7 kN
- Design moment: 496.7 kNm
- Buckling resistance: 615.5 kNm
- Variable load deflection: 11.71 mm
- Limiting deflection: 20.83 mm
- Interaction factors: 0.5594 ≤ 1
  - 0.4678 ≤ 1
  - 0.9428 ≤ 1

**NOTE:** Use normal weight concrete (NWC) and provide reinforcement through holes in the beam web to tie the floor on each side of the beam together in order to meet robustness requirements. The use of T12 x 1200 long tie bars @ 600 c/c would normally suffice.
**Location: Ex2 - Semi-composite**

**Simply supported beam**

Slim floor construction non-composite beam. Calculations are in accordance with EC3 Part 1-1 & SCI P385 entitled 'Design of Steel Beams in Torsion'.

A plate is welded to the bottom flange of the UC section extending beyond the bottom flange by 100 mm either side to support the precast floor units.

Beam span  \( L_1 = 7.5 \text{ m} \)
Beam cross-centres  \( L_2 = 7.5 \text{ m} \)

Precast concrete units  \( w_u = 2.67 \text{ kN/m}^2 \)
Self weight of steel  \( w_s = 0.24 \text{ kN/m}^2 \)
Construction load  \( w_c = 0.5 \text{ kN/m}^2 \)
Floor & minor services  \( w_f = 0.25 \text{ kN/m}^2 \)
Ceiling  \( w_{ci} = 0 \text{ kN/m}^2 \)
Beam encasement  \( w_g = 0.15 \text{ kN/m}^2 \)
Occupancy  \( w_o = 2.5 \text{ kN/m}^2 \)
Partitions  \( w_p = 1 \text{ kN/m}^2 \)

305 x 305 x 118 UKC
Thickness of plate  \( T_p = 15 \text{ mm} \)
Width of plate  \( B_p = 510 \text{ mm} \)
Weld size (leg length)  \( s = 8 \text{ mm} \)
Thickness of end plate  \( t_{ep} = 10 \text{ mm} \)
Diameter of bolts  \( d_b = 24 \text{ mm} \)
Total number of bolts to be used  \( b_n = 4 \)
Weld size (leg length)  \( s_{w} = 10 \text{ mm} \)
CONNECTION SUMMARY

Number of bolts 4
Bolt diameter 24 mm
Bolt grade 8.8
Plate grade S 275

UNIVERSAL COLUMN
305 x 305 x 118 UKC Grade S 355
Design strength 345 N/mm²

BOTTOM PLATE
510 mm x 15 mm Grade S 355
Design strength 355 N/mm²
Design shear force 264.9 kN

DESIGN
Shear plastic resistance 855.7 kN

SUMMARY
Design moment 496.7 kNm
Buckling resistance 615.5 kNm
Variable load deflection 9.946 mm
Limiting deflection 20.83 mm
Interaction factors 0.5594 ≤ 1
0.4678 ≤ 1

NOTE: Use normal weight concrete (NWC) and provide reinforcement through holes in the beam web to tie the floor on each side of the beam together in order to meet robustness requirements. The use of T12 x 1200 long tie bars @ 600 c/c would normally suffice.
Simply supported beam

Slim floor construction
non-composite beam.
Calculations are in
accordance with BS5950
and SCI publication
titled 'Slim Floor
Construction Using Deep
Decking'.

Beam span      L1=7.5 m
Beam cross-centres    L2=7.5 m

254 x 254 x 107 UC.
Young's modulus E=205 kN/mm²
Thickness of plate Tp=15 mm
Width of plate Bp=460 mm
Concrete Grade fcu=30 N/mm²
Air dry density for concrete Dc=18 kN/m³
Wet density for concrete D'c=19 kN/m³
Slab depth Ds=300 mm
Depth of profiling Dp=285 mm
Centres of profiles Ct=600 mm
Width at inside of profile b=548 mm
Width at outside of profile p=424 mm
Deck and end closures wg=0.2 kN/m²
Construction load wc=0.5 kN/m²
Floor & minor services wf=0.25 kN/m²
Ceiling wci=0 kN/m²

Occupancy wo=3.5 kN/m²
Partitions wp=1 kN/m²
Weld leg length s=8 mm
Thickness of end plate tp=10 mm
Grade of steel pgrade=355
Diameter of bolts db=20 mm
Total number of bolts to be used bn=6
Distance to first row of bolts a1=80 mm
Edge distance a2=15 mm
Pitch of bolts a3=70 mm
Bolt cross-centres g=140 mm
Width of end plate Bep=250 mm
CONNECTION SUMMARY

Number of bolts  6
Bolt diameter    20 mm
Bolt grade       8.8
Plate grade      S 355

UNIVERSAL COLUMN 254 x 254 x 107 UC Grade S 355
    Design strength  345 N/mm²

BOTTOM PLATE 460 mm x 15 mm Grade S 355
    Design strength  355 N/mm²
    Design shear force 268 kN

DESIGN SUMMARY
    Shear capacity  706.6 kN
    Moment capacity 625.8 kNm
    Buckling check  0.2103 ≤ 1
    Imposed load deflection 20.88 mm
    Deflection to span ratio 1: 359
    Total deflection  31.55 mm
    Deflection to span ratio 1: 237

NOTE: Use lightweight aggregate concrete (LWC) and provide top reinforcement to tie the floor on each side of the beam together in order to meet robustness requirements.
**Simply supported beam**

Slim floor construction non-composite beam. Calculations are in accordance with EC3 Part 1-1 & SCI P385 entitled 'Design of Steel Beams in Torsion'.

A plate is welded to the bottom flange of the UC section extending beyond the bottom flange by 100 mm either side to support the metal decking.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam span</td>
<td>L1=7.5 m</td>
</tr>
<tr>
<td>Beam cross-centres</td>
<td>L2=7.5 m</td>
</tr>
<tr>
<td>254 x 254 x 107 UKC</td>
<td></td>
</tr>
<tr>
<td>Thickness of plate</td>
<td>Tp=15 mm</td>
</tr>
<tr>
<td>Width of plate</td>
<td>Bp=460 mm</td>
</tr>
<tr>
<td>Concrete density (const.stage)</td>
<td>D'c=19 kN/m³</td>
</tr>
<tr>
<td>Concrete density (final stage)</td>
<td>Dc=18 kN/m³</td>
</tr>
<tr>
<td>Slab depth</td>
<td>Ds=300 mm</td>
</tr>
<tr>
<td>Depth of profiling</td>
<td>Dp=285 mm</td>
</tr>
<tr>
<td>Centres of profiles</td>
<td>Ct=600 mm</td>
</tr>
<tr>
<td>Width at inside of profile</td>
<td>wip=548 mm</td>
</tr>
<tr>
<td>Width at outside of profile</td>
<td>wor=424 mm</td>
</tr>
<tr>
<td>Deck and end closures</td>
<td>wg=0.2 kN/m²</td>
</tr>
<tr>
<td>Construction load</td>
<td>wc=0.5 kN/m²</td>
</tr>
<tr>
<td>Floor &amp; minor services</td>
<td>wf=0.25 kN/m²</td>
</tr>
<tr>
<td>Ceiling</td>
<td>wci=0 kN/m²</td>
</tr>
<tr>
<td>Occupancy</td>
<td>wo=3.5 kN/m²</td>
</tr>
<tr>
<td>Partitions</td>
<td>wp=1 kN/m²</td>
</tr>
<tr>
<td>Weld size (leg length)</td>
<td>s=8 mm</td>
</tr>
<tr>
<td>Thickness of end plate</td>
<td>tep=10 mm</td>
</tr>
<tr>
<td>Diameter of bolts</td>
<td>db=20 mm</td>
</tr>
<tr>
<td>Total number of bolts to be used</td>
<td>bn=4</td>
</tr>
<tr>
<td>Distance from top edge of plate</td>
<td>e4=10 mm</td>
</tr>
<tr>
<td>Distance to first row of bolts</td>
<td>e1=100 mm</td>
</tr>
<tr>
<td>Pitch of bolts vertically</td>
<td>p1=80 mm</td>
</tr>
<tr>
<td>Bolt cross-centres</td>
<td>p3=140 mm</td>
</tr>
<tr>
<td>Width of end plate</td>
<td>bep=250 mm</td>
</tr>
</tbody>
</table>
Weld strength between beam and end plate

Load on weld  \( F_{wEdw} = \frac{W_33}{4 \times (h_{ep} - 2 \times e_4)} = 0.46914 \text{ kN/mm} \)

Weld size (leg length)  \( s_{w} = 10 \text{ mm} \)

Since  \( F_{wEdw} \leq F_{wRdw} \) (0.46914 kN/mm ≤ 1.6884 kN/mm), the 10 mm fillet weld is satisfactory.

**CONNECTION SUMMARY**

\[ \begin{array}{c}
\text{tep}=10 \\
\text{bep}=250 \text{ mm} \\
\text{hep}=290 \text{ mm} \\
p_3=140 \text{ mm} \\
e_2=55 \text{ mm} \\
e_2=55 \text{ mm} \\
\end{array} \]

Number of bolts 4
Bolt diameter 20 mm
Bolt grade 8.8
Plate grade S 355

**UNIVERSAL COLUMN**
- Design strength 345 N/mm²

**BOTTOM PLATE**
- Design strength of plate 355 N/mm²
- Design shear force 253.3 kN

**DESIGN SUMMARY**
- Design moment 475 kNm
- Moment resistance 625.8 kNm
- Buckling resistance 470.2 kNm
- Variable load deflection 20.38 mm
- Deflection to span ratio 1: 367
- Total deflection 30.8 mm
- Deflection to span ratio 1: 243
- Interaction factors 0.2062 ≤ 1

**NOTE:** Use lightweight aggregate concrete (LWAC) and provide top reinforcement to tie the floor on each side of the beam together in order to meet robustness requirements.
Location: User defined section

Simply supported beam

Slim floor construction non-composite beam. Calculations are in accordance with EC3 Part 1-1 & SCI P385 entitled 'Design of Steel Beams in Torsion'.

A plate is welded to the bottom flange of the UC section extending beyond the bottom flange by 100 mm either side to support the metal decking.

Beam span                            L1=7.5 m 
Beam cross-centres                    L2=7.5 m

Thickness of plate                   Tp=15 mm
Width of plate                       Bp=460 mm
Concrete density (const.stage)       D'c=19 kN/m³
Concrete density (final stage)       Dc=18 kN/m³
Slab depth                           Ds=300 mm
Depth of profiling                   Dp=285 mm
Centres of profiles                  Ct=600 mm
Width at inside of profile           wip=548 mm
Width at outside of profile          wop=424 mm
Deck and end closures                wg=0.2 kN/m²
Construction load                    wc=0.5 kN/m²
Floor & minor services               wf=0.25 kN/m²
Ceiling                              wci=0 kN/m²

Occupancy                            wo=3.5 kN/m²
Partitions                           wp=1 kN/m²
Weld size (leg length)               s=8 mm
Thickness of end plate               tep=10 mm
Diameter of bolts                    db=20 mm
Total number of bolts to be used     bn=4
Distance from top edge of plate      e4=10 mm
Distance to first row of bolts       e1=100 mm
Pitch of bolts vertically            pl=100 mm
Bolt cross-centres                   p3=140 mm
Width of end plate                   bep=250 mm

SCALE 5.48  Office 1007  Proforma 419
Weld strength between beam and end plate

Load on weld: $F_{W_{Edw}} = \frac{W_33}{(4*(hep-2*e4))} = 0.46914 \text{ kN/mm}$

Weld size (leg length): $s_w = 10 \text{ mm}$

Since $F_{W_{Edw}} \leq F_{W_{Rdw}}$ (0.46914 kN/mm ≤ 1.6884 kN/mm), the 10 mm fillet weld is satisfactory.

**CONNECTION SUMMARY**

**tep=10**

<table>
<thead>
<tr>
<th>e1</th>
<th>e4=10</th>
</tr>
</thead>
<tbody>
<tr>
<td>p1</td>
<td>100 mm</td>
</tr>
<tr>
<td>e3</td>
<td>100 mm</td>
</tr>
<tr>
<td></td>
<td>hep= 290 mm</td>
</tr>
</tbody>
</table>

**bep=250 mm**

<table>
<thead>
<tr>
<th>e4=10</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 mm</td>
</tr>
<tr>
<td>hep= 290 mm</td>
</tr>
<tr>
<td>p3=140 mm</td>
</tr>
</tbody>
</table>

Number of bolts: 4

Bolt diameter: 20 mm

Bolt grade: 8.8

Plate grade: S 355

**UNIVERSAL COLUMN**

266.7 x 258.8 x 107 UC Grade S 355

Design strength: 345 N/mm²

**BOTTOM PLATE**

460 mm x 15 mm Grade S 355

Design strength of plate: 355 N/mm²

Design shear force: 253.3 kN

**DESIGN SUMMARY**

Shear plastic resistance: 759 kN

Design moment: 475 kNm

Moment resistance: 625.8 kNm

Buckling resistance: 469.9 kNm

Variable load deflection: 20.37 mm

Deflection to span ratio: 1: 368

Total deflection: 30.78 mm

Deflection to span ratio: 1: 243

Interaction factors: 0.2086 ≤ 1

**NOTE:** Use lightweight aggregate concrete (LWAC) and provide top reinforcement to tie the floor on each side of the beam together in order to meet robustness requirements.
Location: Example in Moment Connections publication

Unstiffened column slab base

Calculations are in accordance with 'Joints in Steel Construction Moment Connections' published by The Steel Construction Institute.

ELEVATION PLAN

Axial load (+ve compression) \( N = 300 \) kN
Moment about X-X axis \( M = 345 \) kNm
Shear on the base in Y direction \( F_y = 75 \) kN

305 x 305 x 118 UC.
Length of baseplate \( h_p = 600 \) mm
Breadth of baseplate \( b_p = 600 \) mm
Edge distance to bolt centre line \( k = 75 \) mm
Strength of concrete \( f_{cu} = 30 \) N/mm²
Assumed weld size \( s_w = 12 \) mm
Selected baseplate thickness \( t_p = 50 \) mm

Total number of bolts to be used \( n = 8 \)
Assumed diameter of bolt \( b_d = 24 \) mm
Overall embedded depth \( L_o = 450 \) mm
Assumed cover to reinforcement \( c_v = 50 \) mm
tension reinforcement \( A_{sp} = 0.15 \% \)
Selected fillet weld size \( s_w = 12 \) mm

Web welds

Length of web weld \( L_{wv} = 2 \times d_c = 493.4 \) mm
Weld force per mm \( F_{yw} = F_y \times 10^{-3} / L_{wv} = 152.01 \) N/mm
Weld size required \( s_{ww} = F_{yw} / (0.7 \times p_w) = 0.98706 \) mm
Selected fillet weld size \( s_{fw} = 8 \) mm
### SUMMARY

**BASEPLATE**
Size 600 mm x 600 mm x 50 mm

**REQUIREMENTS**
- Grade S 275 steel
- Edge distance 75 mm
- Number of H.D. bolts 8
- Diameter of bolts M 24
- Grade 8.8
- Overall embedded depth 450 mm
- Anchor plate size: 120 mm x 120 mm x 20 mm
- Concrete/grout (fcu) 30 N/mm²

### WELDS

**TENSION**
- Fillet weld 12 mm
  - Contact areas on the baseplate and column are machined to give a tight bearing contact.
  - The specified tension weld will be used for both flanges.

**WEB**
- Fillet weld 8 mm
Location: Example based on proforma 462 arrangement

Unstiffened column slab base

Calculations are in accordance with 'Joints in Steel Construction Moment Connections' published by The Steel Construction Institute.

Axial load (+ve compression) \( N = 38.34 \text{ kN} \)
Moment about X-X axis \( M = 204 \text{ kNm} \)
Shear on the base in Y direction \( F_y = 12.15 \text{ kN} \)

533 x 210 x 101 UB.
Length of baseplate \( h_p = 700 \text{ mm} \)
Breadth of baseplate \( b_p = 400 \text{ mm} \)
Edge distance to bolt centre line \( k = 50 \text{ mm} \)
Strength of concrete \( f_{cu} = 25 \text{ N/mm}^2 \)
Assumed weld size \( s_w = 8 \text{ mm} \)
Selected baseplate thickness \( t_p = 25 \text{ mm} \)

Total number of bolts to be used \( n = 6 \)
Assumed diameter of bolt \( b_d = 24 \text{ mm} \)
Overall embedded depth \( L_o = 450 \text{ mm} \)
Assumed cover to reinforcement \( c_v = 50 \text{ mm} \)
tension reinforcement \( A_{sp} = 0.2 \% \)
Selected fillet weld size \( s_w = 8 \text{ mm} \)

Web welds

Length of web weld \( L_{wv} = 2 \times d_c = 953 \text{ mm} \)
Weld force per mm \( F_{yw} = F_y \times 10^3 / L_{wv} = 12.749 \text{ N/mm} \)
Weld size required \( s_{ww} = F_{yw} / (0.7 \times p) = 0.082787 \text{ mm} \)
Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)
### SUMMARY

<table>
<thead>
<tr>
<th>BASEPLATE</th>
<th>Size 700 mm x 400 mm x 25 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>REQUIREMENTS</td>
<td>Grade S 275 steel</td>
</tr>
<tr>
<td></td>
<td>Edge distance 50 mm</td>
</tr>
<tr>
<td></td>
<td>Number of H.D. bolts 6</td>
</tr>
<tr>
<td></td>
<td>Diameter of bolts M 24</td>
</tr>
<tr>
<td></td>
<td>Grade 8.8</td>
</tr>
<tr>
<td></td>
<td>Overall embedded depth 450 mm</td>
</tr>
<tr>
<td></td>
<td>Anchor plate size: 120 mm x 120 mm x 20 mm</td>
</tr>
<tr>
<td></td>
<td>Concrete/grout (fcu) 25 N/mm²</td>
</tr>
</tbody>
</table>

### WELDS

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Fillet weld 8 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contact areas on the baseplate and column are machined to give a tight bearing contact. The specified tension weld will be used for both flanges.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>WEB</th>
<th>Fillet weld 8 mm</th>
</tr>
</thead>
</table>
Location: Odd numbers arrangement

Unstiffened column slab base

Calculations are in accordance with 'Joints in Steel Construction: Moment Connections' published by The Steel Construction Institute.

Axial load (+ve compression) \( N = 200 \text{ kN} \)
Moment about X-X axis \( M = 350 \text{ kNm} \)
Shear on the base in Y direction \( F_y = 50 \text{ kN} \)

Length of baseplate \( h_p = 600 \text{ mm} \)
Breadth of baseplate \( b_p = 400 \text{ mm} \)
Edge distance to bolt centre line \( k = 50 \text{ mm} \)
Strength of concrete \( f_{cu} = 20 \text{ N/mm}^2 \)
Assumed weld size \( s_w = 12 \text{ mm} \)
Selected baseplate thickness \( t_p = 20 \text{ mm} \)

Total number of bolts to be used \( n = 6 \)
Assumed diameter of bolt \( b_d = 30 \text{ mm} \)
Overall embedded depth \( L_o = 450 \text{ mm} \)
Assumed cover to reinforcement \( c_v = 50 \text{ mm} \)
tension reinforcement \( A_{sp} = 0.32 \% \)
Selected fillet weld size \( s_w = 12 \text{ mm} \)
Selected fillet weld size \( s_w_c = 12 \text{ mm} \)

Web welds

Length of web weld \( L_{w_w} = 2 \times d_c = 772 \text{ mm} \)
Weld force per mm \( F_{yw} = F_y \times 10^3 / L_{w_w} = 64.767 \text{ N/mm} \)
Weld size required \( s_{ww} = F_{yw} / (0.7 \times p_w) = 0.73072 \text{ mm} \)
Selected fillet weld size \( s_{fw} = 12 \text{ mm} \)
### SUMMARY

<table>
<thead>
<tr>
<th>BASEPLATE</th>
<th>Size 600 mm x 400 mm x 20 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>REQUIREMENTS</td>
<td>Grade S 275 steel</td>
</tr>
<tr>
<td></td>
<td>Edge distance 50 mm</td>
</tr>
<tr>
<td></td>
<td>Number of H.D. bolts 6</td>
</tr>
<tr>
<td></td>
<td>Diameter of bolts M 30</td>
</tr>
<tr>
<td></td>
<td>Grade 8.8</td>
</tr>
<tr>
<td></td>
<td>Overall embedded depth 450 mm</td>
</tr>
<tr>
<td></td>
<td>Anchor plate size: 150 mm x 150 mm x 25 mm</td>
</tr>
<tr>
<td></td>
<td>Base plate set in a shallow pocket</td>
</tr>
<tr>
<td></td>
<td>Concrete/grout (fcu) 20 N/mm²</td>
</tr>
</tbody>
</table>

### WELDS

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Fillet weld 12 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>COMPRESSION</td>
<td>Fillet weld 12 mm</td>
</tr>
<tr>
<td>WEB</td>
<td>Fillet weld 12 mm</td>
</tr>
</tbody>
</table>

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Location: Ex1 - Example in Moment Connections publication

Unstiffened column slab base

Calculations are in accordance with EC3 Parts 1-1, 1-8 and publication entitled 'Joints in Steel Construction: Simple Joints to EC3' by SCI.

Axial load (compression) \( N_{Ed} = 300 \text{ kN} \)
Moment about y-y axis \( M_{yEd} = 345 \text{ kNm} \)
Shear on the base in z direction \( V_{cEd} = 75 \text{ kN} \)

305 x 305 x 118 UKC
Length of baseplate \( h_p = 600 \text{ mm} \)
Breadth of baseplate \( b_p = 600 \text{ mm} \)
Edge distance to bolt centre line \( k = 75 \text{ mm} \)
Characteristic cylinder strength \( f_{ck} = 25 \text{ N/mm}^2 \)
Selected baseplate thickness \( t_p = 50 \text{ mm} \)
Total number of bolts to be used \( n = 8 \)
Assumed diameter of bolt \( b_d = 24 \text{ mm} \)
Overall embedded depth \( L_o = 450 \text{ mm} \)
Assumed cover to reinforcement \( c_v = 50 \text{ mm} \)
Assumed percentage area of tension reinforcement \( A_{sp} = 0.15 \% \)
Punching shear force \( V_{Ed} = \text{ABS}(F_{tlEd}) = 775.18 \text{ kN} \)
Shear stress at control perimeter \( \nu_{Ed} = \beta \cdot \nu_{Ed} \cdot 1000/(u_1 \cdot d_{av}) \)
\[= 0.36062 \text{ N/mm}^2 \]
Design punching shear resistance \( \nu_{Rdc} = (0.18/g_{mc}) \cdot k^* (100 \cdot P_1 \cdot f_{ck})^0.333 = 0.31812 \text{ N/mm}^2 \)
Shear enhancement 2d/av \( \text{senh} = 1 \)
The min design punching shear resistance governs and will be adopted.
Design punching shear resistance \( \nu_{Rdc} = \text{senh} \cdot 0.035 \cdot k^{1.5} \cdot f_{ck}^{0.5} = 0.39033 \text{ N/mm}^2 \)

As \( \nu_{Ed} \leq \nu_{Rdc} \) (0.36062 N/mm² ≤ 0.39033 N/mm²), the anchorage arrangement is considered suitable.
Column tension flange welds

Weld design shear strength
\[ fvwd = \frac{fu}{\sqrt{3} \cdot \beta \cdot \gamma} \]
\[ = 222.79 \text{ N/mm}^2 \]

Force in the tension flange
\[ T_t = \frac{M_y \cdot E}{D} \cdot 10^3 / (h_c - t_f + c) - T_a = 1051.4 \text{ kN} \]

Weld force per mm
\[ F_{wRd} = \frac{F_{wF}}{2 \cdot b_c - t_w} = 1.7442 \text{ kN/mm} \]

Weld size required
\[ s_{wf} = \frac{F_{wRd} \cdot 10^3}{0.7 \cdot fvwd} = 11.184 \text{ mm} \]

Selected fillet weld size
\[ s_{w} = 12 \text{ mm} \]

Column compression flange welds

The specified tension weld will be used for both flanges.

Column web welds

Weld force per mm
\[ F_{wRd1} = \frac{V_c \cdot E}{L_w} = 152.01 \text{ N/mm} \]

Weld size required
\[ s_{ww} = \frac{F_{wRd1}}{0.7 \cdot fvwd} = 0.9747 \text{ mm} \]

Selected fillet weld size
\[ s_{fw} = 8 \text{ mm} \]

SUMMARY

BASEPLATE
Size 600 mm x 600 mm x 50 mm

REQUIREMENTS
Grade S 275 steel
Edge distance 75 mm
Number of H.D. bolts 8
Diameter of bolts M 24
Grade 8.8
Overall embedded depth 450 mm
Anchor plate size:
120 mm x 120 mm x 20 mm
Concrete/grout (fck) 25 N/mm²

WELD SUMMARY
TENSION
Fillet weld 12 mm
Contact areas on the baseplate and column are machined to give a tight bearing contact.
The specified tension weld will be used for both flanges.

WEB
Fillet weld 8 mm
Location: Ex2 - Example based on proforma 462 arrangement

Unstiffened column slab base

Calculations are in accordance with EC3 Parts 1-1, 1-8 and publication entitled 'Joints in Steel Construction: Simple Joints to EC3' by SCI.

Axial load (compression) \( N_{Ed} = 38.34 \text{ kN} \)

Moment about y-y axis \( M_{yEd} = 204 \text{ kNm} \)

Shear on the base in z direction \( V_{cEd} = 12.15 \text{ kN} \)

533 x 210 x 101 UKB

Length of baseplate \( h_p = 700 \text{ mm} \)

Breadth of baseplate \( b_p = 400 \text{ mm} \)

Edge distance to bolt centre line \( k = 50 \text{ mm} \)

Characteristic cylinder strength \( f_{ck} = 20 \text{ N/mm}^2 \)

Selected baseplate thickness \( t_p = 25 \text{ mm} \)

Total number of bolts to be used \( n = 6 \)

Assumed diameter of bolt \( b_d = 24 \text{ mm} \)

Overall embedded depth \( L_o = 450 \text{ mm} \)

Assumed cover to reinforcement \( c_v = 50 \text{ mm} \)

Assumed percentage area of tension reinforcement \( A_{sp} = 0.2 \% \)

Punching shear force \( V_{Ed} = \text{ABS}(F_{tlEd}) = 345.34 \text{ kN} \)

Shear stress at control perimeter \( \sigma_{Ed} = \beta_{VEd} * V_{Ed} * 1000 / (u_1 * d_{av}) \)

\( = 0.16885 \text{ N/mm}^2 \)

Design punching shear resistance \( \sigma_{Rdc} = (0.18 / \gamma_{mc}) * k^* (100 * p_1 * fck)^0.333 = 0.32503 \text{ N/mm}^2 \)

Shear enhancement \( 2d_{av} / d_{av} \)

\( \text{senh}=1 \)

The min design punching shear resistance governs and will be adopted.

Design punching shear resistance \( \sigma_{Rdc} = \text{senh} * 0.035 * k^* 1.5 * fck^0.5 \)

\( = 0.34912 \text{ N/mm}^2 \)

As \( \sigma_{Ed} \leq \sigma_{Rdc} \) (\( 0.16885 \text{ N/mm}^2 \leq 0.34912 \text{ N/mm}^2 \)), the anchorage arrangement is considered suitable.
**Column tension flange welds**

Weld design shear strength: \( fvwd = \frac{fu}{\sqrt{3}/(\beta_{aw} \cdot \gamma_M^2)} \)

- \( = 222.79 \text{ N/mm}^2 \)

Force in the tension flange: \( T_t = \frac{MyE_d}{10^3/(hc-tfc)} - Ta = 381.98 \text{ kN} \)

Weld force per mm: \( FwRd = \frac{T_{ff}}{2*bc-tw} = 0.93348 \text{ kN/mm} \)

Weld size required: \( sw_f = \frac{FwRd*10^3/(0.7*fvwd)}{0.7*fvwd} = 5.9857 \text{ mm} \)

Selected fillet weld size: \( sw = 8 \text{ mm} \)

**Column compression flange welds**

The specified tension weld will be used for both flanges.

**Column web welds**

Weld force per mm: \( FwRd_1 = \frac{VcE_d}{10^3/Lw} = 12.749 \text{ N/mm} \)

Weld size required: \( sw_w = FwRd_1/(0.7*fvwd) = 0.081751 \text{ mm} \)

Selected fillet weld size: \( sw = 8 \text{ mm} \)

**SUMMARY**

**BASEPLATE**

| Size 700 mm x 400 mm x 25 mm |

**REQUIREMENTS**

- Grade S 275 steel
- Edge distance 50 mm
- Number of H.D. bolts 6
- Diameter of bolts M 24
- Grade 8.8
- Overall embedded depth 450 mm
- Anchor plate size: 120 mm x 120 mm x 20 mm
- Concrete/grout (fck) 20 N/mm²

**WELD SUMMARY**

**TENSION**

Fillet weld 8 mm

Contact areas on the baseplate and column are machined to give a tight bearing contact.

The specified tension weld will be used for both flanges.

**WEB**

Fillet weld 8 mm
Location: Ex3 - Odd numbers arrangement

Unstiffened column slab base

Calculations are in accordance with EC3 Parts 1-1, 1-8 and publication entitled 'Joints in Steel Construction: Simple Joints to EC3' by SCI.

Axial load (compression) \( \text{NE}_\text{Ed}=200 \text{ kN} \)
Moment about y-y axis \( \text{M}_\text{y}_\text{Ed}=350 \text{ kNm} \)
Shear on the base in z direction \( \text{V}_\text{c}_\text{Ed}=50 \text{ kN} \)

Length of baseplate \( \text{hp}=600 \text{ mm} \)
Breadth of baseplate \( \text{bp}=400 \text{ mm} \)
Edge distance to bolt centre line \( \text{k}=50 \text{ mm} \)
Characteristic cylinder strength \( \text{f}_\text{ck}=25 \text{ N/mm}^2 \)
Selected baseplate thickness \( \text{tp}=35 \text{ mm} \)
Total number of bolts to be used \( \text{n}=6 \)
Assumed diameter of bolt \( \text{bd}=30 \text{ mm} \)
Overall embedded depth \( \text{Lo}=450 \text{ mm} \)
Assumed cover to reinforcement \( \text{cv}=50 \text{ mm} \)
Assumed percentage area of tension reinforcement \( \text{As}_\text{p}=0.42 \% \)
Punching shear force \( \text{V}_{\text{Ed}}=\text{ABS}(\text{Ft}_\text{Ed})=652.69 \text{ kN} \)
Shear stress at control perimeter \( \text{v}_\text{Ed}=\text{beta}_\text{Ed}^\times\text{VE}_\text{Ed}^\times1000/(\text{u}_1^\times\text{dav}) \)
\( =0.43137 \text{ N/mm}^2 \)
Design punching shear resistance \( \text{v}_{\text{Rdc}}=(0.18/\text{gam}_\text{c})^\times(100^\times\text{p}_1^\times\text{fck})^\times0.333=0.44823 \text{ N/mm}^2 \)
As \( \text{v}_\text{Ed} \leq \text{v}_{\text{Rdc}} (0.43137 \text{ N/mm}^2 \leq 0.44823 \text{ N/mm}^2) \), the anchorage arrangement is considered suitable.

Column tension flange welds

Weld design shear strength \( \text{fv}_{\text{wd}}=\text{fu}/\text{SQR}(3)/(\text{betaw}_\text{x}\text{gamM}_2) \)
\( =222.79 \text{ N/mm}^2 \)
Force in the tension flange \( \text{T}_\text{t}=\text{My}_\text{Ed}_\times10^3/(\text{hc}-\text{tfc})-\text{T}_\text{a}=752.6 \text{ kN} \)
Weld force per mm \( \text{Fw}_{\text{Rd}}=\text{Tff}/(2\times\text{bc}-\text{twc})=1.5518 \text{ kN/mm} \)
Weld size required \( \text{sw}_{\text{f}}=\text{Fw}_{\text{Rd}}\times10^3/(0.7\times\text{fv}_{\text{wd}})=9.9502 \text{ mm} \)
Selected fillet weld size \( \text{sw}=12 \text{ mm} \)
Column compression flange welds

The specified tension weld will be used for both flanges.

Column web welds

Weld force per mm  \[ F_{wRd1} = \frac{V_c E_d \times 10^3}{L_{wv}} = 64.767 \text{ N/mm} \]
Weld size required  \[ s_{ww} = \frac{F_{wRd1}}{0.7 \times f_{wvd}} = 0.4153 \text{ mm} \]
Selected fillet weld size  \[ s_{fw} = 12 \text{ mm} \]

Summary

**BASEPLATE**  
Size 600 mm x 400 mm x 35 mm

**REQUIREMENTS**  
Grade S 275 steel
Edge distance 50 mm
Number of H.D. bolts 6
Diameter of bolts M 30
Grade 8.8
Overall embedded depth 450 mm
Anchor plate size: 150 mm x 150 mm x 25 mm
Concrete/grout (fck) 25 N/mm²

Weld Summary

**TENSION**  
Fillet weld 12 mm
Contact areas on the baseplate and column are machined to give a tight bearing contact.
The specified tension weld will be used for both flanges.

**WEB**  
Fillet weld 12 mm
Location: Ex4 - SCI Example 20

Unstiffened column slab base

Calculations are in accordance with EC3 Parts 1-1, 1-8 and publication entitled 'Joints in Steel Construction: Simple Joints to EC3' by SCI.

Axial load (compression) \( N_{Ed}=1380 \text{ kN} \)
Moment about y-y axis \( M_{yEd}=185 \text{ kNm} \)
Shear on the base in z direction \( V_{cEd}=75 \text{ kN} \)

305 x 305 x 137 UKC

Length of baseplate \( h_p=600 \text{ mm} \)
Breadth of baseplate \( b_p=600 \text{ mm} \)
Edge distance to bolt centre line \( k=75 \text{ mm} \)
Characteristic cylinder strength \( f_{ck}=25 \text{ N/mm}^2 \)
Selected baseplate thickness \( t_p=40 \text{ mm} \)
Number of bolts to be used \( n=4 \)
Bolt diameter \( b_d=24 \)

Column tension flange welds

Weld design shear strength \( f_{vwd}=f_u/SQR(3)\) \( \times \) (betaw \( \times \) gamM2) 
\( =222.79 \text{ N/mm}^2 \)

As tension is not developed in the foundation, a nominal weld size needs to be specified.
Selected fillet weld size \( s_w=8 \text{ mm} \)

Column compression flange welds

The specified tension weld will be used for both flanges.
Column web welds

Weld force per mm \[ F_{Wd1} = \frac{VcEd \times 10^3}{Lwv} = 152.01 \text{ N/mm} \]
Weld size required \[ s_{WW} = \frac{F_{Wd1}}{0.7 \times f_{Wd}} = 0.9747 \text{ mm} \]
Selected fillet weld size \[ s_{FW} = 8 \text{ mm} \]

SUMMARY

BASEPLATE
Size 600 mm x 600 mm x 40 mm

REQUIREMENTS
Grade S 275 steel
Edge distance 75 mm
Number of H.D. bolts 4
Diameter of bolts M 24
Grade 8.8
Concrete/grout (fck) 25 N/mm²

WELD SUMMARY

TENSION
Fillet weld 8 mm
Contact areas on the baseplate and column are machined to give a tight bearing contact.
The specified tension weld will be used for both flanges.

WEB
Fillet weld 8 mm
Location: Ex1 - Based on joints publication

Bolted extended end plate connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( M_a = 405 \text{ kNm} \)
Factored shear force \( V = 60 \text{ kN} \)
Axial force (positive compression) \( N = 0 \text{ kN} \)

254 x 254 x 107 UC.
533 x 210 x 92 UB.
Diameter of bolts \( b_d = 24 \text{ mm} \)
Thickness of plate \( t_p = 25 \text{ mm} \)
Total number of bolts \( b_n = 8 \)
Number of shear bolts \( n_s = 2 \)
Distance to first row of bolts \( e_x = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( p_1 = 100 \text{ mm} \)
Pitch of tension bolts \( p_2 = 90 \text{ mm} \)
Distance tension to shear bolts \( p_3 = 320 \text{ mm} \)
Edge distance for shear bolts \( p_n = 100 \text{ mm} \)
Bolt cross-centres \( g = 100 \text{ mm} \)
Width of end plate \( b_p = 250 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts  Pr1=370.98 kN
Row 2  Pr2=314.76 kN
Row 3  Pr3=286.25 kN

Penetration butt weld size  sbt=8 mm
Superimposed fillet weld size  swt=10 mm
Selected fillet weld size  sfw=8 mm

Connection is considered to be suitable.

CONNECTION SUMMARY

Connection forms part of a two sided connection.

Number of bolts  8
Bolt diameter  24 mm
Bolt grade  8.8
Tension  Partial penetration butt welds  8 mm
Flange Welds  Superimposed fillet welds  10 mm
Web fillet weld  8 mm
Compression flange fillet weld  8 mm

Supporting column section  254 x 254 x 107 UC
Grade S 275
Beam / Rafter section  533 x 210 x 92 UB
Grade S 275

Column stiffener details

No enhancement of the column required.
Location: Ex2 - Using 6 rows of tension bolts

Bolted extended end plate connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

\[
\begin{align*}
\text{Factored moment} & \quad M_a = 678 \text{ kNm} \\
\text{Factored shear force} & \quad V = 230 \text{ kN} \\
\text{Axial force (+ve compression)} & \quad N = 100 \text{ kN}
\end{align*}
\]

254 x 254 x 107 UC.
686 x 254 x 125 UB.
Diameter of bolts \( b_d = 24 \text{ mm} \)
Thickness of plate \( t_p = 25 \text{ mm} \)
Total number of bolts \( b_n = 14 \)
Number of shear bolts \( n_s = 2 \)
Distance to first row of bolts \( e_x = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( p_1 = 100 \text{ mm} \)
Pitch of tension bolts \( p_2 = 90 \text{ mm} \)
Edge distance for shear bolts \( p_n = 100 \text{ mm} \)
Over-all length of end plate \( L_c = 805 \text{ mm} \)
Bolt cross-centres \( g = 100 \text{ mm} \)
Width of end plate \( b_p = 250 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Top row of bolts</th>
<th>Pr1=370.98 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Row 2</td>
<td>Pr2=314.76 kN</td>
</tr>
<tr>
<td>Row 3</td>
<td>Pr3=286.25 kN</td>
</tr>
<tr>
<td>Row 4</td>
<td>Pr4=286.25 kN</td>
</tr>
<tr>
<td>Row 5</td>
<td>Pr5=286.25 kN</td>
</tr>
<tr>
<td>Row 6</td>
<td>Pr6=286.25 kN</td>
</tr>
</tbody>
</table>

Weld size \( ss = 10 \text{ mm} \)
Penetration butt weld size \( sbt = 8 \text{ mm} \)
Superimposed fillet weld size \( swt = 10 \text{ mm} \)
Selected fillet weld size \( sfw = 8 \text{ mm} \)

Connection is considered to be suitable.

CONNECTION SUMMARY

Connection forms part of a two sided connection.

<table>
<thead>
<tr>
<th>ex</th>
<th>25</th>
</tr>
</thead>
<tbody>
<tr>
<td>p1</td>
<td>100</td>
</tr>
<tr>
<td>p2</td>
<td>90</td>
</tr>
<tr>
<td>p2</td>
<td>90</td>
</tr>
<tr>
<td>p2</td>
<td>90</td>
</tr>
<tr>
<td>p2</td>
<td>90</td>
</tr>
<tr>
<td>p3</td>
<td>195</td>
</tr>
<tr>
<td>pn</td>
<td>37.1</td>
</tr>
</tbody>
</table>

Number of bolts 14
Bolt diameter 24 mm
Bolt grade 8.8
Tension Partial penetration butt welds 8 mm
Flange Welds Superimposed fillet welds 10 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm
Supporting column section 254 x 254 x 107 UC
Beam / Rafter section 686 x 254 x 125 UB
Column stiffener details

Compression stiffeners

2/120 mm x 25 mm Grade S 275
connected by 4/10 mm fillet welds.
**Location: Ex3 - Supplementary web plates and high shear**

**Bolted extended end plate connection - beam to column**

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( M_a = 680 \) kNm
Factored shear force \( V = 647 \) kN
Axial force (+ve compression) \( N = -100 \) kN

305 x 305 x 118 UC.
762 x 267 x 134 UB.

Diameter of bolts \( b_d = 24 \) mm
Thickness of plate \( t_p = 25 \) mm
Total number of bolts \( b_n = 14 \)
Number of shear bolts \( n_s = 4 \)
Distance to first row of bolts \( e_x = 50 \) mm
Distance to beam flange \( x = 40 \) mm
Pitch to second row of bolts \( p_1 = 100 \) mm
Pitch of tension bolts \( p_2 = 90 \) mm
Edge distance for shear bolts \( p_n = 100 \) mm
Over-all length of end plate \( L_c = 880 \) mm
Bolt cross-centres \( g = 140 \) mm
Width of end plate \( b_p = 300 \) mm
Assumed web fillet weld size \( s_{ww} = 8 \) mm
Assumed flange fillet weld size \( s_{wf} = 12 \) mm
SUMMARY OF BOLT POTENTIAL RESISTANCES

- Top row of bolts: $P_{r1} = 343.58 \, kN$
- Row 2: $P_{r2} = 259.37 \, kN$
- Row 3: $P_{r3} = 255.4 \, kN$
- Row 4: $P_{r4} = 199.88 \, kN$
- Row 5: $P_{r5} = 160.88 \, kN$

Breadth: $b_{sw} = 245 \, mm$
Length: $L_s = 1180 \, mm$
Thickness: $t_{sw} = 12 \, mm$

Selected fillet weld size: $s_{wt} = 12 \, mm$
Selected fillet weld size: $s_{fw} = 10 \, mm$
Penetration butt weld size: $s_{btc} = 9 \, mm$
Superimposed fillet weld size: $s_{wtc} = 10 \, mm$

Connection is considered to be suitable.

CONNECTION SUMMARY

Connection is one sided beam to column arrangement.

Number of bolts: 14
Bolt diameter: 24 mm
Bolt grade: 8.8
Welds
- Tension flange fillet weld: 12 mm
- Web fillet weld: 10 mm
Compression
- Partial penetration butt welds: 9 mm
Flange Welds
- Superimposed fillet welds: 10 mm
Supporting column section: 305 x 305 x 118 UC
Grade S 275
Beam / Rafter section: 762 x 267 x 134 UB
Grade S 275
Column stiffener details

Supplementary web plate(s)  1 plate 12 mm thick
                        245 mm x 1180 mm
                        12 mm fillet weld all round.
Location: Ex4 - Diagonal stiffeners also used for compression

Bolted extended end plate connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( Ma = 370 \text{ kNm} \)
Factored shear force \( V = 128 \text{ kN} \)
Axial force (+ve compression) \( N = 120 \text{ kN} \)

305 x 305 x 97 UC.
610 x 229 x 101 UB.

Diameter of bolts \( bd = 24 \text{ mm} \)
Thickness of plate \( tp = 25 \text{ mm} \)
Total number of bolts \( bn = 8 \)
Number of shear bolts \( ns = 2 \)
Distance to first row of bolts \( ex = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( p1 = 100 \text{ mm} \)
Pitch of tension bolts \( p2 = 90 \text{ mm} \)
Edge distance for shear bolts \( pn = 100 \text{ mm} \)
Over-all length of end plate \( Lc = 730 \text{ mm} \)
Bolt cross-centres \( g = 140 \text{ mm} \)
Width of end plate \( bp = 300 \text{ mm} \)
Assumed web fillet weld size \( sww = 8 \text{ mm} \)
Assumed flange fillet weld size \( swf = 10 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts \( P_{r1} = 317.69 \text{ kN} \)
Row 2 \( P_{r2} = 193.89 \text{ kN} \)
Row 3 \( P_{r3} = 110.98 \text{ kN} \)
Weld size \( sc = 6 \text{ mm} \)

Slip factor for preloaded bolts \( \mu = 0.5 \)
Coefficient for type of hole \( K_s = 1 \)
Selected fillet weld size \( swt = 10 \text{ mm} \)
Selected fillet weld size \( sfw = 10 \text{ mm} \)

Connection is considered to be suitable.

CONNECTION SUMMARY

Connection is one sided beam to column arrangement.

Number of bolts 8
Bolt diameter 24 mm
Bolt grade HSFG of general grade (8.8) to BS4395 Part 1. Nuts grade 10.
Welds Tension flange fillet weld 10 mm
Web fillet weld 10 mm
Compression flange fillet weld 10 mm
Supporting column section 305 x 305 x 97 UC
Grade S 275
Beam / Rafter section 610 x 229 x 101 UB
Grade S 275

Column stiffener details

Diagonal stiffeners 2/145 mm x 20 mm Grade S 275
on line between the beam tension and compression flanges connected by 4/6 mm fillet welds.
Diagonal stiffeners also act as compression stiffeners.
Location: Ex5 - Backing plates example

Bolted extended end plate connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( Ma = 630 \text{ kNm} \)
Factored shear force \( V = 267 \text{ kN} \)
Axial force (+ve compression) \( N = 200 \text{ kN} \)

305 x 305 x 97 UC.
686 x 254 x 140 UB.
Diameter of bolts \( bd = 24 \text{ mm} \)
Thickness of plate \( tp = 25 \text{ mm} \)
Total number of bolts \( bn = 14 \)
Number of shear bolts \( ns = 2 \)
Distance to first row of bolts \( ex = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( p1 = 100 \text{ mm} \)
Pitch of tension bolts \( p2 = 90 \text{ mm} \)
Edge distance for shear bolts \( pn = 100 \text{ mm} \)
Over-all length of end plate \( Lc = 815 \text{ mm} \)
Bolt cross-centres \( g = 140 \text{ mm} \)
Width of end plate \( bp = 280 \text{ mm} \)
Assumed web fillet weld size \( sww = 8 \text{ mm} \)
Assumed flange fillet weld size \( swf = 12 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Row</th>
<th>Pr</th>
<th>kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top row of bolts</td>
<td>Pr1</td>
<td>305.93</td>
</tr>
<tr>
<td>Row 2</td>
<td>Pr2</td>
<td>205.65</td>
</tr>
<tr>
<td>Row 3</td>
<td>Pr3</td>
<td>110.98</td>
</tr>
<tr>
<td>Row 4</td>
<td>Pr4</td>
<td>110.98</td>
</tr>
<tr>
<td>Row 5</td>
<td>Pr5</td>
<td>110.98</td>
</tr>
<tr>
<td>Row 6</td>
<td>Pr6</td>
<td>110.98</td>
</tr>
</tbody>
</table>

- Weld size: ss = 8 mm
- Thickness: tbp = 15 mm
- Width: bbp = 145 mm
- Length: Lbp = 780 mm
- Selected fillet weld size: swt = 12 mm

Selected fillet weld size: sfw = 10 mm
Connection is considered to be suitable.

CONNECTION SUMMARY

Connection forms part of a two-sided connection.

Number of bolts: 14
Bolt diameter: 24 mm
Bolt grade: 8.8
Welds:
- Tension flange fillet weld: 12 mm
- Web fillet weld: 10 mm
- Compression flange fillet weld: 8 mm

Supporting column section: 305 x 305 x 97 UC
Grade: S 275

Beam / Rafter section: 686 x 254 x 140 UB
Grade: S 275
### Column stiffener details

<table>
<thead>
<tr>
<th>Component</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression stiffeners</td>
<td>2/140 mm x 20 mm Grade S 275 connected by 4/8 mm fillet welds.</td>
</tr>
<tr>
<td>Backing plates</td>
<td>2/15 mm x 145 mm x 780 mm Grade S 275</td>
</tr>
</tbody>
</table>
Location: Ex6 - Tension web stiffeners similar to example 1

Bolted extended end plate connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( M_a = 350 \text{ kNm} \)
Factored shear force \( V = 60 \text{ kN} \)
Axial force (+ve compression) \( N = 0 \text{ kN} \)

254 x 254 x 73 UC.
533 x 210 x 92 UB.
Diameter of bolts \( b_d = 24 \text{ mm} \)
Thickness of plate \( t_p = 25 \text{ mm} \)
Total number of bolts \( b_n = 8 \)
Number of shear bolts \( n_s = 2 \)
Distance to first row of bolts \( e_x = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( p_1 = 100 \text{ mm} \)
Pitch of tension bolts \( p_2 = 90 \text{ mm} \)
Distance tension to shear bolts \( p_3 = 320 \text{ mm} \)
Edge distance for shear bolts \( p_n = 100 \text{ mm} \)
Bolt cross-centres \( g = 100 \text{ mm} \)
Width of end plate \( b_p = 250 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts                  Pr1=297.07 kN
Row 2                             Pr2=231.52 kN
Row 3                             Pr3=140.42 kN
Weld size                         ss=8 mm
Stiffener width                   bsgt=100 mm
Stiffener depth                   Ls=180 mm
Proposed rib thickness            tsr=10 mm
Proposed weld size                ssr=10 mm
Penetration butt weld size        sbt=8 mm
Superimposed fillet weld size     swt=10 mm
Selected fillet weld size         sfw=8 mm

Connection is considered to be suitable.

CONNECTION SUMMARY

Connection forms part of a two sided connection.

Number of bolts 8
Bolt diameter 24 mm
Bolt grade 8.8
Tension Partial penetration butt welds 8 mm
Flange Welds Superimposed fillet welds 10 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm

Supporting column section 254 x 254 x 73 UC
                          Grade S 275
Beam / Rafter section   533 x 210 x 92 UB
                          Grade S 275
**Column stiffener details**

<table>
<thead>
<tr>
<th>Type</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression stiffeners</td>
<td>2/120 mm x 15 mm Grade S 275 connected by 4/8 mm fillet welds.</td>
</tr>
<tr>
<td>Tension rib stiffeners</td>
<td>Partial depth 10 mm thick For curtailment length refer to Clause 4.5.10.</td>
</tr>
<tr>
<td></td>
<td>2/100 mm wide placed midway between the following bolt rows:</td>
</tr>
<tr>
<td></td>
<td>1 and 2</td>
</tr>
</tbody>
</table>
Location: Ex7 - Compression & tension web stiffeners

Bolted extended end plate connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( Ma = 350 \, \text{kNm} \)
Factored shear force \( V = 80 \, \text{kN} \)
Axial force (+ve compression) \( N = 0 \, \text{kN} \)

254 x 254 x 73 UC.
533 x 210 x 92 UB.
Diameter of bolts \( bd = 24 \, \text{mm} \)
Thickness of plate \( tp = 25 \, \text{mm} \)
Total number of bolts \( bn = 8 \)
Number of shear bolts \( ns = 2 \)
Distance to first row of bolts \( ex = 50 \, \text{mm} \)
Distance to beam flange \( x = 40 \, \text{mm} \)
Pitch to second row of bolts \( p1 = 100 \, \text{mm} \)
Pitch of tension bolts \( p2 = 90 \, \text{mm} \)
Distance tension to shear bolts \( p3 = 320 \, \text{mm} \)
Edge distance for shear bolts \( pn = 100 \, \text{mm} \)
Bolt cross-centres \( g = 100 \, \text{mm} \)
Width of end plate \( bp = 250 \, \text{mm} \)
Assumed web fillet weld size \( sww = 8 \, \text{mm} \)
Assumed flange fillet weld size \( swf = 12 \, \text{mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts \( Pr_1 = 297.07 \) kN
Row 2 \( Pr_2 = 231.52 \) kN
Row 3 \( Pr_3 = 140.42 \) kN

Weld size \( ss = 8 \) mm
Stiffener width \( bsgt = 100 \) mm
Stiffener depth \( L_s = 180 \) mm
Proposed rib thickness \( tsr = 10 \) mm
Proposed weld size \( ssr = 10 \) mm

Selected fillet weld size \( swt = 10 \) mm
Selected fillet weld size \( sfw = 8 \) mm

Connection is considered to be suitable.

CONNECTION SUMMARY

Connection forms part of a two sided connection.

Number of bolts 8
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 10 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm

Supporting column section 254 x 254 x 73 UC
Grade S 275

Beam / Rafter section 533 x 210 x 92 UB
Grade S 275

Column stiffener details

Compression stiffeners 2/120 mm x 15 mm Grade S 275
connected by 4/8 mm fillet welds.

Tension rib stiffeners Partial depth 10 mm thick
For curtailment length refer to Clause 4.5.10.
2/100 mm wide placed midway between the following bolt rows:
1 and 2
2 and 3
Location: Ex8 - tension web stiffeners similar to example 1,6 & 7

Bolted extended end plate connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( Ma = 350 \text{ kNm} \)
Factored shear force \( V = 60 \text{ kN} \)
Axial force (+ve compression) \( N = 0 \text{ kN} \)

254 x 254 x 73 UC.
533 x 210 x 92 UB.
Diameter of bolts \( bd = 24 \text{ mm} \)
Thickness of plate \( tp = 25 \text{ mm} \)
Total number of bolts \( bn = 8 \)
Number of shear bolts \( ns = 2 \)
Distance to first row of bolts \( ex = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( pl = 100 \text{ mm} \)
Pitch of tension bolts \( p2 = 90 \text{ mm} \)
Distance tension to shear bolts \( p3 = 320 \text{ mm} \)
Edge distance for shear bolts \( pn = 100 \text{ mm} \)
Bolt cross-centres \( g = 100 \text{ mm} \)
Width of end plate \( bp = 250 \text{ mm} \)
Assumed web fillet weld size \( sww = 8 \text{ mm} \)
Assumed flange fillet weld size \( swf = 12 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts  $P_r1=297.07$ kN
Row 2  $P_r2=231.52$ kN
Row 3  $P_r3=140.42$ kN

Weld size  $ss=8$ mm
Stiffener width  $bsgt=100$ mm
Stiffener depth  $L_s=180$ mm
Proposed rib thickness  $tsr=10$ mm
Proposed weld size  $ssr=10$ mm
Penetration butt weld size  $sbt=8$ mm
Superimposed fillet weld size  $swt=12$ mm
Selected fillet weld size  $sfw=8$ mm

Connection is considered to be suitable.

CONNECTION SUMMARY

Connection forms part of a two sided connection.

Number of bolts  8
Bolt diameter  24 mm
Bolt grade  8.8
Tension  Partial penetration butt welds  8 mm
Flange Welds  Superimposed fillet welds  12 mm
Web fillet weld  8 mm
Compression flange fillet weld  8 mm

Supporting column section  254 x 254 x 73 UC
  Grade S 275
Beam / Rafter section  533 x 210 x 92 UB
  Grade S 275
Column stiffener details

Compression stiffeners

2/120 mm x 15 mm Grade S 275
connected by 4/8 mm fillet welds.

Tension rib stiffeners

Partial depth 10 mm thick
For curtailment length refer to Clause 4.5.10.
2/100 mm wide placed midway between the following bolt rows:
2 and 3
Location: Ex9 - 6 rows of bolts with tension stiffeners

Bolted extended end plate connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( Ma = 1004 \ \text{kNm} \)
Factored shear force \( V = 129 \ \text{kN} \)
Axial force (+ve compression) \( N = -59 \ \text{kN} \)

610 x 305 x 149 UB.
762 x 267 x 134 UB.

Diameter of bolts \( bd = 24 \ \text{mm} \)
Thickness of plate \( tp = 25 \ \text{mm} \)
Total number of bolts \( bn = 14 \)
Number of shear bolts \( ns = 2 \)
Distance to first row of bolts \( ex = 50 \ \text{mm} \)
Distance to beam flange \( x = 40 \ \text{mm} \)
Pitch to second row of bolts \( p1 = 100 \ \text{mm} \)
Pitch of tension bolts \( p2 = 90 \ \text{mm} \)
Edge distance for shear bolts \( pn = 100 \ \text{mm} \)
Over-all length of end plate \( Lc = 880 \ \text{mm} \)
Bolt cross-centres \( g = 140 \ \text{mm} \)
Width of end plate \( bp = 280 \ \text{mm} \)
Assumed web fillet weld size \( sww = 8 \ \text{mm} \)
Assumed flange fillet weld size \( swf = 12 \ \text{mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts $Pr_1 = 357.31$ kN
Row 2 $Pr_2 = 264.54$ kN
Row 3 $Pr_3 = 260.05$ kN
Row 4 $Pr_4 = 260.05$ kN
Row 5 $Pr_5 = 225.11$ kN
Row 6 $Pr_6 = 181.85$ kN
Weld size $sc = 6$ mm
Weld size $ss = 8$ mm
Stiffener width $bsgt = 110$ mm
Proposed rib thickness $tsr = 15$ mm
Proposed weld size $ssr = 10$ mm

Penetration butt weld size $sbt = 8$ mm
Superimposed fillet weld size $swt = 12$ mm
Selected fillet weld size $sfw = 8$ mm
Connection is considered to be suitable.

CONNECTION SUMMARY

Connection is one sided beam to column arrangement.

Number of bolts 14
Bolt diameter 24 mm
Bolt grade 8.8
Tension Partial penetration butt welds 8 mm
Flange Welds Superimposed fillet welds 12 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm

Supporting column section 610 x 305 x 149 UB
Grade S 275
Beam / Rafter section 762 x 267 x 134 UB
Grade S 275
## Column stiffener details

<table>
<thead>
<tr>
<th>Type</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diagonal stiffeners</td>
<td>2/145 mm x 10 mm Grade S 275  on line between the beam tension and compression flanges, connected by 4/6 mm fillet welds.</td>
</tr>
<tr>
<td>Compression stiffeners</td>
<td>2/140 mm x 15 mm Grade S 275  connected by 4/8 mm fillet welds.</td>
</tr>
</tbody>
</table>
| Tension rib stiffeners| Provide full depth stiffeners 15 mm thick midway between rows 1 and 2. Any remaining ribs are partial depth. For curtailment length refer to Clause 4.5.10 of the code. 2/110 mm wide placed midway between the following bolt rows: 1 and 2, 2 and 3, 3 and 4, 4 and 5, 5 and 6.
Location: Ex10 - as small as you can get

Bolted extended end plate connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment $Ma=20.579 \text{ kNm}$
Factored shear force $V=12 \text{ kN}$
Axial force (+ve compression) $N=-8 \text{ kN}$

152 x 152 x 23 UC.
152 x 89 x 16 UB.
Diameter of bolts $bd=20 \text{ mm}$
Thickness of plate $tp=20 \text{ mm}$
Total number of bolts $bn=6$
Number of shear bolts $ns=2$
Distance to first row of bolts $ex=40 \text{ mm}$
Distance to beam flange $x=30 \text{ mm}$
Pitch to second row of bolts $p1=70 \text{ mm}$
Edge distance for shear bolts $pn=50 \text{ mm}$
Over-all length of end plate $Lc=250 \text{ mm}$
Bolt cross-centres $g=70 \text{ mm}$
Width of end plate $bp=130 \text{ mm}$
Assumed web fillet weld size $sww=6 \text{ mm}$
Assumed flange fillet weld size $swf=6 \text{ mm}$
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts
Row 2
Weld size
Stiffener width
Proposed rib thickness
Proposed weld size
Selected fillet weld size
Selected fillet weld size
Connection is considered to be suitable.

CONNECTION SUMMARY

Connection is one sided beam to column arrangement.

Number of bolts 6
Bolt diameter 20 mm
Bolt grade 8.8
Welds Tension flange fillet weld 8 mm
Web fillet weld 8 mm
Compression flange fillet weld 6 mm
Supporting column section 152 x 152 x 23 UC Grade S 275
Beam / Rafter section 152 x 89 x 16 UB Grade S 275

Column stiffener details

Diagonal stiffeners 2/60 mm x 10 mm Grade S 275
on line between the beam tension and compression flanges
connected by 4/6 mm fillet welds.
Tension rib stiffeners Provide full depth stiffeners
10 mm thick midway between rows 1 and 2. Any remaining ribs are partial depth. For curtailment length refer to Clause 4.5.10 of the code.
2/55 mm wide placed midway between the following bolt rows:
1 and 2
Location: Ex1 - Compression stiffener to supporting column

Bolted extended end plate connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment
MyEd=405 kNm
Factored shear force
VEd=60 kN
Axial force (+ve compression)
NEd=0 kN

Supporting column details
254 x 254 x 107 UKC

Supported beam details
533 x 210 x 92 UKB

Bolt details
Diameter of bolts
db=24 mm
Tensile resistance (EC3 Part 1-8)
FtRd=k2*fub*At/(gamM2*10^3)=203.33 kN

End plate details
Thickness of plate
tp=25 mm
Total number of bolts
bn=8
Number of shear bolts
ns=2
Distance to first row of bolts
ex=50 mm
Distance to beam flange
x=40 mm
Pitch to second row of bolts
p2=100 mm
Pitch of tension bolts
p1=90 mm
Distance tension to shear bolts \( p_1' = 320 \text{ mm} \)
Edge distance for shear bolts \( e_1' = 100 \text{ mm} \)
Bolt cross-centres \( p_3 = 100 \text{ mm} \)
Width of end plate \( b_p = 250 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

<table>
<thead>
<tr>
<th>Row</th>
<th>Bolt Potential Resistances (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>( P_{r1} = 377.26 )</td>
</tr>
<tr>
<td>Row 2</td>
<td>( P_{r2} = 321.03 )</td>
</tr>
<tr>
<td>Row 3</td>
<td>( P_{r3} = 292.53 )</td>
</tr>
</tbody>
</table>

Design axial load on column \( N_{Edc} = 500 \text{ kN} \)
Weld size (leg length) \( s_s = 8 \text{ mm} \)
Penetration butt weld size \( s_{bt} = 8 \text{ mm} \)
Superimposed fillet weld size \( s_{wt} = 12 \text{ mm} \)
Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)

**CONNECTION SUMMARY** - all dimensions are in mm

```
  25
 ex    40
  +  50
    100
  p2   90
    100
  320
p1                  36.9
    100
    100
p1'                660
    100
  e1'
```

Number of bolts \( 8 \)
Bolt diameter \( 24 \text{ mm} \)
Bolt grade \( 8.8 \)
Tension Partial penetration butt welds \( 8 \text{ mm} \)
Flange Welds Superimposed fillet welds \( 12 \text{ mm} \)
Web fillet weld \( 8 \text{ mm} \)
Compression flange fillet weld \( 8 \text{ mm} \)

Supporting column section \( 254 \times 254 \times 107 \text{ UKC} \)
Grade S 275
Beam / Rafter section \( 533 \times 210 \times 92 \text{ UKB} \)
Grade S 275

Column stiffener details

Compression stiffeners \( 2/120 \text{ mm} \times 15 \text{ mm} \) Grade S 275
connected by \( 4/8 \text{ mm} \) fillet welds.
Location: Ex2 - Compression & tension stiffeners

Bolted extended end plate connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment MyEd=778 kNm
Factored shear force VEd=215 kN
Axial force (+ve compression) NEd=100 kN

Supporting column details
254 x 254 x 107 UKC

Supported beam details
686 x 254 x 125 UKB

Bolt details
Diameter of bolts db=24 mm
Tensile resistance (EC3 Part 1-8) FtRd=k2*fub*At/(gamM2*10^3)=203.33 kN

End plate details
Thickness of plate tp=25 mm
Total number of bolts bn=14
Number of shear bolts ns=2
Distance to first row of bolts ex=50 mm
Distance to beam flange x=40 mm
Pitch to second row of bolts p2=100 mm
Pitch of tension bolts p1=90 mm
Edge distance for shear bolts  e1'=100 mm
Over-all length of end plate  hp=805 mm
Bolt cross-centres  p3=100 mm
Width of end plate  bp=250 mm
Assumed web fillet weld size  sww=8 mm
Assumed flange fillet weld size  swf=12 mm

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

- Top row of bolts  Pr1=377.26 kN
- Row 2  Pr2=321.03 kN
- Row 3  Pr3=292.53 kN
- Row 4  Pr4=292.53 kN
- Row 5  Pr5=292.53 kN
- Row 6  Pr6=292.53 kN

Design axial load on column  NEdc=500 kN
Weld size (leg length)  ss=15 mm
Stiffener width  bsgt=100 mm
Proposed rib thickness  tsr=15 mm
Proposed weld size  ssr=12 mm

Selected fillet weld size  swt=15 mm
Selected fillet weld size  sfw=8 mm

**CONNECTION SUMMARY** - all dimensions in diagram are in mm

Number of bolts  14
Bolt diameter  24 mm
Bolt grade  8.8
Welds  
- Tension flange fillet weld  15 mm
- Web fillet weld  8 mm
- Compression flange fillet weld  8 mm

Supporting column section  254 x 254 x 107 UKC
Grade S 275
Beam / Rafter section  686 x 254 x 125 UKB
Grade S 275

Column stiffener details

Compression stiffeners 2/120 mm x 30 mm Grade S 275 connected by 4/15 mm fillet welds.

Tension stiffeners Full depth 15 mm thick 2/100 mm wide placed midway between the following bolt rows:
1 and 2
2 and 3
Location: Ex3 - Using 'K' Stiffeners

Bolted extended end plate connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( My_{Ed} = 680 \text{ kNm} \)
Factored shear force \( V_{Ed} = 430 \text{ kN} \)
Axial force (+ve compression) \( N_{Ed} = -100 \text{ kN} \)

Supporting column details
305 x 305 x 118 UKC

Supported beam details
762 x 267 x 134 UKB

Bolt details
Diameter of bolts \( db = 24 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( Ft_{Rd} = k^2 f_{ub} A_t / (\gamma_m 2 \times 10^3) = 203.33 \text{ kN} \)

End plate details
Thickness of plate \( tp = 25 \text{ mm} \)
Total number of bolts \( bn = 14 \)
Number of shear bolts \( ns = 4 \)
Distance to first row of bolts \( ex = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( p_2 = 100 \text{ mm} \)
Pitch of tension bolts \( p_1 = 90 \text{ mm} \)
Edge distance for shear bolts \( e_1' = 100 \text{ mm} \)
Over-all length of end plate  hp=880 mm  
Bolt cross-centres  p3=140 mm  
Width of end plate  bp=300 mm  
Assumed web fillet weld size  sww=8 mm  
Assumed flange fillet weld size  swf=12 mm

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

<table>
<thead>
<tr>
<th>Row</th>
<th>Potential Resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top row</td>
<td>Pr1=349.85</td>
</tr>
<tr>
<td>Row 2</td>
<td>Pr2=265.64</td>
</tr>
<tr>
<td>Row 3</td>
<td>Pr3=261.67</td>
</tr>
<tr>
<td>Row 4</td>
<td>Pr4=181.05</td>
</tr>
<tr>
<td>Row 5</td>
<td>Pr5=160.88</td>
</tr>
</tbody>
</table>

Design axial load on column  NEdc=500 kN
Weld size  sc=8 mm
Weld size (leg length)  ss=8 mm
Selected fillet weld size  swt=12 mm
Selected fillet weld size  sfw=10 mm
Penetration butt weld size  sbtc=9 mm
Superimposed fillet weld size  swtc=10 mm

**CONNECTION SUMMARY**

Number of bolts  14
Bolt diameter  24 mm
Bolt grade  8.8
Welds  
  Tension flange fillet weld  12 mm
  Web fillet weld  10 mm
Compression  
  Partial penetration butt welds  9 mm
Flange Welds  
  Superimposed fillet welds  10 mm
Supporting column section  305 x 305 x 118 UKC  
  Grade S 275
Beam / Rafter section  762 x 267 x 134 UKB  
  Grade S 275
Column stiffener details

'K' stiffeners

2/145 mm x 15 mm Grade S 275
on line between the beam tension and compression flanges
connected by 4/8 mm fillet welds.

Compression stiffeners

2/145 mm x 15 mm Grade S 275
connected by 4/8 mm fillet welds.
Bolted extended end plate connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( My_{Ed} = 370 \text{ kNm} \)
Factored shear force \( V_{Ed} = 128 \text{ kN} \)
Axial force (+ve compression) \( N_{Ed} = 120 \text{ kN} \)

Supporting column details
305 x 305 x 97 UKC

Supported beam details
610 x 229 x 101 UKB

Bolt details
Diameter of bolts \( d_b = 24 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( F_{Rd} = k_2 f_{ub} A_t / (\gamma_M^2 10^3) = 203.33 \text{ kN} \)

End plate details
Thickness of plate \( t_p = 25 \text{ mm} \)
Total number of bolts \( b_n = 8 \)
Number of shear bolts \( n_s = 2 \)
Distance to first row of bolts \( e_x = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( p_2 = 100 \text{ mm} \)
Pitch of tension bolts \( p_1 = 90 \text{ mm} \)
Edge distance for shear bolts \( e_1' = 100 \text{ mm} \)
Over-all length of end plate hp=730 mm
Bolt cross-centres p3=140 mm
Width of end plate bp=300 mm
Assumed web fillet weld size sww=8 mm
Assumed flange fillet weld size swf=10 mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts Pr1=312.2 kN
Row 2 Pr2=199.38 kN
Row 3 Pr3=110.98 kN
Design axial load on column NEdc=500 kN
Weld size sc=6 mm
Selected fillet weld size swt=10 mm
Selected fillet weld size sfw=10 mm

CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 8
Bolt diameter 24 mm
Bolt grade Preloaded HSFG bolts
General grade (8.8) with grade 10 nuts.
Welds Tension flange fillet weld 10 mm
Web fillet weld 10 mm
Compression flange fillet weld 10 mm
Supporting column section 305 x 305 x 97 UKC
Grade S 275
Beam / Rafter section 610 x 229 x 101 UKB
Grade S 275

Column stiffener details

Diagonal stiffeners 2/145 mm x 20 mm Grade S 275
on line between the beam tension and compression flanges connected by 4/6 mm fillet welds.
Diagonal stiffeners also act as compression stiffeners.
Location: Ex5 - Backing plates example

Bolted extended end plate connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment: MyEd=630 kNm
Factored shear force: VEd=215 kN
Axial force (+ve compression): NEd=200 kN

Supporting column details
305 x 305 x 97 UKC

Supported beam details
686 x 254 x 140 UKB

Bolt details
Diameter of bolts: db=24 mm
Tensile resistance (EC3 Part 1-8): FtRd=k2*fub*At/(gamM2*10^3)=203.33 kN

End plate details
Thickness of plate: tp=25 mm
Total number of bolts: bn=14
Number of shear bolts: ns=2
Distance to first row of bolts: ex=50 mm
Distance to beam flange: x=40 mm
Pitch to second row of bolts: p2=100 mm
Pitch of tension bolts: p1=90 mm
Edge distance for shear bolts        e1' = 100 mm
Over-all length of end plate        hp = 815 mm
Bolt cross-centres                  p3 = 140 mm
Width of end plate                  bp = 280 mm
Assumed web fillet weld size        sww = 8 mm
Assumed flange fillet weld size     swf = 12 mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts                  Pr1 = 312.2 kN
Row 2                             Pr2 = 199.38 kN
Row 3                             Pr3 = 110.98 kN
Row 4                             Pr4 = 110.98 kN
Row 5                             Pr5 = 110.98 kN
Row 6                             Pr6 = 110.98 kN
Design axial load on column       Nedc = 500 kN
Weld size (leg length)            ss = 10 mm
Thickness of backing plate        tbp = 15 mm
Width of backing plate            bbp = 145 mm
Length                            Lbp = 780 mm
Selected fillet weld size         swt = 12 mm
Selected fillet weld size         sfw = 10 mm

CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 14
Bolt diameter 24 mm
Bolt grade 8.8
Welds
  Tension flange fillet weld 12 mm
  Web fillet weld 10 mm
  Compression flange fillet weld 8 mm
Supporting column section 305 x 305 x 97 UKC
  Grade S 275
Beam / Rafter section 686 x 254 x 140 UKB
  Grade S 275
Column stiffener details

<table>
<thead>
<tr>
<th>Column stiffener details</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression stiffeners</td>
<td>2/140 mm x 20 mm Grade S 275 connected by 4/10 mm fillet welds.</td>
</tr>
<tr>
<td>Backing plates</td>
<td>2/15 mm x 145 mm x 780 mm Grade S 275</td>
</tr>
</tbody>
</table>
Location: Ex6 - Compression and tension stiffeners

Bolted extended end plate connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment

Factored shear force

Axial force (+ve compression)

Supporting column details

254 x 254 x 73 UKC

Supported beam details

533 x 210 x 92 UKB

Bolt details

Diameter of bolts

Tensile resistance (EC3 Part 1-8)

End plate details

Thickness of plate

Total number of bolts

Number of shear bolts

Distance to first row of bolts

Distance to beam flange

Pitch to second row of bolts

Pitch of tension bolts
Distance tension to shear bolts $p_1' = 320$ mm
Edge distance for shear bolts $e_1' = 100$ mm
Bolt cross-centres $p_3 = 100$ mm
Width of end plate $b_p = 250$ mm
Assumed web fillet weld size $s_{ww} = 8$ mm
Assumed flange fillet weld size $s_{wf} = 12$ mm

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

- Top row of bolts $P_{r1} = 303.34$ kN
- Row 2 $P_{r2} = 225.24$ kN
- Row 3 $P_{r3} = 140.42$ kN
- Design axial load on column $N_{Edc} = 500$ kN
- Weld size (leg length) $s_s = 8$ mm
- Stiffener width $b_{sgt} = 100$ mm
- Stiffener depth $L_s = 180$ mm
- Proposed rib thickness $t_{sr} = 10$ mm
- Proposed weld size $s_{sr} = 10$ mm
- Penetration butt weld size $s_{bt} = 8$ mm
- Superimposed fillet weld size $s_{wt} = 10$ mm
- Selected fillet weld size $s_{fw} = 8$ mm

**CONNECTION SUMMARY**

```
25                          250
ex    ô                       50    ÅÄÄÄÄÄÄÄÄÄÄÄÄÄ´
Å     Ú¿         40             Å    ÚÄÄÄÄÄÄÄÄÄÄÄÄÄ¿    Å
Å     Ƶ         Å              Å    ³ o         o ³    ³
p2    ³ÆÍÍÍÍÍÍÍ  Á             100   ³   ÍÍÍËÍÍÍ   ³    ³
Å     Ƶ                        Å    ³ o    º    o ³    ³
p1    ³³                       90    ³      º      ³    ³
Å     Ƶ                        Å    ³ o    º    o ³    ³
³     ³³                        ³    ³      º      ³    ³
p1'   ³³                       320   ³      º      ³    ³ 660
³     ³³                        ³    ³      º      ³    ³
Å     Ƶ                        Å    ³ o    º    o ³    ³
e1'   ³ÆÍÍÍÍÍÍÍ   ¹            100   ³   ÍÍÍÊÍÍÍ   ³    ³
Á     ÀÙ         Á 36.9         Á    ÀÄÄÄÄÄÄÄÄÄÄÄÄÄÙ    Á
100
```

- Number of bolts 8
- Bolt diameter 24 mm
- Bolt grade 8.8
- Tension Partial penetration butt welds 8 mm
- Flange Welds Superimposed fillet welds 10 mm
  - Web fillet weld 8 mm
  - Compression flange fillet weld 8 mm
- Supporting column section 254 x 254 x 73 UKC
  - Grade S 275
- Beam / Rafter section 533 x 210 x 92 UKB
  - Grade S 275
Column stiffener details

Compression stiffeners
2/120 mm x 15 mm Grade S 275 connected by 4/8 mm fillet welds.

Tension stiffeners
Partial depth 10 mm thick. For curtailment length see code. 2/100 mm wide placed midway between the following bolt rows: 1 and 2
Location: Ex7 - Tension web stiffeners

Bolted extended end plate connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( \text{MyEd}=360 \) kNm
Factored shear force \( \text{VED}=80 \) kN
Axial force (+ve compression) \( \text{NEd}=0 \) kN

Supporting column details
254 x 254 x 73 UKC

Supported beam details
533 x 210 x 92 UKB

Bolt details
Diameter of bolts \( \text{db}=24 \) mm
Tensile resistance (EC3 Part 1-8) \( \text{FtRd}=k2*fub*At/(\text{gamM2}*10^3)=203.33 \) kN

End plate details
Thickness of plate \( \text{tp}=25 \) mm
Total number of bolts \( \text{bn}=8 \)
Number of shear bolts \( \text{ns}=2 \)
Distance to first row of bolts \( \text{ex}=50 \) mm
Distance to beam flange \( \text{x}=40 \) mm
Pitch to second row of bolts \( \text{p2}=100 \) mm
Pitch of tension bolts \( \text{p1}=90 \) mm
Distance tension to shear bolts  \( p'_1 = 320 \text{ mm} \)
Edge distance for shear bolts  \( e'_1 = 100 \text{ mm} \)
Bolt cross-centres  \( p_3 = 100 \text{ mm} \)
Width of end plate  \( b_p = 250 \text{ mm} \)
Assumed web fillet weld size  \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size  \( s_{wf} = 12 \text{ mm} \)

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

Top row of bolts  \( P_{r1} = 303.34 \text{ kN} \)
Row 2  \( P_{r2} = 225.24 \text{ kN} \)
Row 3  \( P_{r3} = 140.42 \text{ kN} \)
Design axial load on column  \( N_{Edc} = 500 \text{ kN} \)
Weld size (leg length)  \( s_s = 8 \text{ mm} \)
Stiffener width  \( b_{sgt} = 100 \text{ mm} \)
Stiffener depth  \( L_s = 180 \text{ mm} \)
Proposed rib thickness  \( t_{sr} = 10 \text{ mm} \)
Proposed weld size  \( s_{sr} = 10 \text{ mm} \)
Selected fillet weld size  \( s_{wt} = 12 \text{ mm} \)
Selected fillet weld size  \( s_{fw} = 8 \text{ mm} \)

**CONNECTION SUMMARY** - all dimensions in diagram are in mm

![Diagram showing connection dimensions and details](image-url)

Number of bolts 8
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 12 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Supporting column section 254 x 254 x 73 UKC
Grade S 275
Beam / Rafter section 533 x 210 x 92 UKB
Grade S 275
### Column stiffener details

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Compression stiffeners</strong></td>
<td>2/120 mm x 15 mm Grade S 275 connected by 4/8 mm fillet welds.</td>
</tr>
<tr>
<td><strong>Tension stiffeners</strong></td>
<td>Partial depth 10 mm thick. For curtailment length see code. 2/100 mm wide placed midway between the following bolt rows: 1 and 2 2 and 3</td>
</tr>
</tbody>
</table>
Location: Ex8 - Compression and tension stiffeners

Bolted extended end plate connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment

Factored shear force

Axial force (+ve compression)

Supporting column details

254 x 254 x 73 UKC

Supported beam details

533 x 210 x 92 UKB

Bolt details

Diameter of bolts

Tensile resistance (EC3 Part 1-8)

End plate details

Thickness of plate

Total number of bolts

Number of shear bolts

Distance to first row of bolts

Distance to beam flange

Pitch to second row of bolts

Pitch of tension bolts

SCALE 5.48 Office 1007 Proforma 422
Distance tension to shear bolts \( p_1' = 320 \text{ mm} \)
Edge distance for shear bolts \( e_1' = 100 \text{ mm} \)
Bolt cross-centres \( p_3 = 100 \text{ mm} \)
Width of end plate \( b_p = 250 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

<table>
<thead>
<tr>
<th>Top row of bolts</th>
<th>( P_{r1} = 303.34 \text{ kN} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Row 2</td>
<td>( P_{r2} = 225.24 \text{ kN} )</td>
</tr>
<tr>
<td>Row 3</td>
<td>( P_{r3} = 140.42 \text{ kN} )</td>
</tr>
<tr>
<td>Design axial load on column</td>
<td>( N_{Edc} = 500 \text{ kN} )</td>
</tr>
<tr>
<td>Weld size (leg length)</td>
<td>( s_s = 8 \text{ mm} )</td>
</tr>
<tr>
<td>Stiffener width</td>
<td>( b_{sgt} = 100 \text{ mm} )</td>
</tr>
<tr>
<td>Stiffener depth</td>
<td>( L_s = 180 \text{ mm} )</td>
</tr>
<tr>
<td>Proposed rib thickness</td>
<td>( t_{sr} = 10 \text{ mm} )</td>
</tr>
<tr>
<td>Proposed weld size</td>
<td>( s_{sr} = 10 \text{ mm} )</td>
</tr>
<tr>
<td>Penetration butt weld size</td>
<td>( s_{bt} = 8 \text{ mm} )</td>
</tr>
<tr>
<td>Superimposed fillet weld size</td>
<td>( s_{wt} = 12 \text{ mm} )</td>
</tr>
<tr>
<td>Selected fillet weld size</td>
<td>( s_{fw} = 8 \text{ mm} )</td>
</tr>
</tbody>
</table>

**CONNECTION SUMMARY** - all dimensions in diagram are in mm

Number of bolts 8
Bolt diameter 24 mm
Bolt grade 8.8
Tension Partial penetration butt welds 8 mm
Flange Welds Superimposed fillet welds 12 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm
Supporting column section 254 x 254 x 73 UKC
Grade S 275
Beam / Rafter section 533 x 210 x 92 UKB
Grade S 275
## Column stiffener details

<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression stiffeners</td>
<td>2/120 mm x 15 mm Grade S 275 connected by 4/8 mm fillet welds.</td>
</tr>
<tr>
<td>Tension stiffeners</td>
<td>Partial depth 10 mm thick. For curtailment length see code. 2/100 mm wide placed midway between the following bolt rows: 2 and 3</td>
</tr>
</tbody>
</table>
Location: Ex9 - Shear, Compression & Tension stiffeners provided

Bolted extended end plate connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment $My_{Ed}=1004$ kNm
Factored shear force $V_{Ed}=129$ kN
Axial force (+ve compression) $N_{Ed}=-59$ kN

Supporting column details
610 x 305 x 149 UKB

Supported beam details
762 x 267 x 134 UKB

Bolt details
Diameter of bolts $d_b=24$ mm
Tensile resistance (EC3 Part 1-8) $F_{Rd}=k_2*f_{ub}*A_t/(\gamma_M2*10^3)=203.33$ kN

End plate details
Thickness of plate $t_p=25$ mm
Total number of bolts $b_n=14$
Number of shear bolts $n_s=2$
Distance to first row of bolts $e_1=50$ mm
Distance to beam flange $x=40$ mm
Pitch to second row of bolts $p_2=100$ mm
Pitch of tension bolts $p_1=90$ mm
Edge distance for shear bolts $e_1'=100$ mm
Over-all length of end plate \( hp = 880 \text{ mm} \)
Bolt cross-centres \( p_3 = 140 \text{ mm} \)
Width of end plate \( bp = 280 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

- Top row of bolts \( P_{r1} = 363.58 \text{ kN} \)
- Row 2 \( P_{r2} = 270.82 \text{ kN} \)
- Row 3 \( P_{r3} = 266.33 \text{ kN} \)
- Row 4 \( P_{r4} = 266.33 \text{ kN} \)
- Row 5 \( P_{r5} = 200.01 \text{ kN} \)
- Row 6 \( P_{r6} = 181.85 \text{ kN} \)

Design axial load on column \( N_{Edc} = 500 \text{ kN} \)
Weld size \( sc = 6 \text{ mm} \)
Weld size (leg length) \( ss = 8 \text{ mm} \)
Stiffener width \( b_{sgt} = 110 \text{ mm} \)
Proposed rib thickness \( ts_{r} = 15 \text{ mm} \)
Proposed weld size \( ss_{r} = 12 \text{ mm} \)
Penetration butt weld size \( sb_{t} = 8 \text{ mm} \)
Superimposed fillet weld size \( sw_{t} = 12 \text{ mm} \)
Selected fillet weld size \( sf_{w} = 8 \text{ mm} \)

**CONNECTION SUMMARY - all dimensions in diagram are in mm**

<table>
<thead>
<tr>
<th>ex</th>
<th>25</th>
</tr>
</thead>
<tbody>
<tr>
<td>p2</td>
<td>40</td>
</tr>
<tr>
<td>p1</td>
<td>100</td>
</tr>
<tr>
<td>p1</td>
<td>90</td>
</tr>
<tr>
<td>p1</td>
<td>90</td>
</tr>
<tr>
<td>p1</td>
<td>90</td>
</tr>
<tr>
<td>p1</td>
<td>90</td>
</tr>
<tr>
<td>p1</td>
<td>90</td>
</tr>
<tr>
<td>e1'</td>
<td>270</td>
</tr>
</tbody>
</table>

Number of bolts \( 14 \)
Bolt diameter \( 24 \text{ mm} \)
Bolt grade \( 8.8 \)
Tension Partial penetration butt welds \( 8 \text{ mm} \)
Flange Welds Superimposed fillet welds \( 12 \text{ mm} \)
Web fillet weld \( 8 \text{ mm} \)
Compression flange fillet weld \( 8 \text{ mm} \)

Supporting column section \( 610 \times 305 \times 149 \text{ UKB} \)
Grade S 275

Beam / Rafter section

762 x 267 x 134 UKB
Grade S 275

Column stiffener details

Diagonal stiffeners

2/145 mm x 10 mm Grade S 275 on line between the beam tension and compression flanges connected by 4/6 mm fillet welds.

Compression stiffeners

2/145 mm x 20 mm Grade S 275 connected by 4/8 mm fillet welds.

Tension stiffeners

Provide full depth stiffeners 15 mm thick midway between rows 1 and 2. Any remaining ribs are partial depth. For curtailment length see code. 2/110 mm wide placed midway between the following bolt rows:

1 and 2
2 and 3
3 and 4
4 and 5
5 and 6
Location: Ex10 - Diagonal shear stiffeners & tension stiffeners

Bolted extended end plate connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment $\text{MyEd}=20.579 \text{ kNm}$
Factored shear force $\text{VEd}=12 \text{ kN}$
Axial force (+ve compression) $\text{NEd}=-8 \text{ kN}$

Supporting column details
152 x 152 x 23 UKC

Supported beam details
152 x 89 x 16 UKB

Bolt details
- Diameter of bolts $\text{db}=20 \text{ mm}$
- Tensile resistance (EC3 Part 1-8) $\text{FtRd}=k2*\text{fub*At}/(\gamma_\text{M2}*10^3)=141.12 \text{ kN}$

End plate details
- Thickness of plate $\text{tp}=20 \text{ mm}$
- Total number of bolts $\text{bn}=6$
- Number of shear bolts $\text{ns}=2$
- Distance to first row of bolts $\text{ex}=40 \text{ mm}$
- Distance to beam flange $\text{x}=30 \text{ mm}$
- Pitch to second row of bolts $\text{p2}=70 \text{ mm}$
- Edge distance for shear bolts $\text{e1}^'=50 \text{ mm}$
- Over-all length of end plate $\text{hp}=250 \text{ mm}$
Bolt cross-centres \[ p_3 = 70 \text{ mm} \]
Width of end plate \[ b_p = 130 \text{ mm} \]
Assumed web fillet weld size \[ s_{ww} = 6 \text{ mm} \]
Assumed flange fillet weld size \[ s_{wf} = 6 \text{ mm} \]

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

- Top row of bolts \[ P_{r1} = 75.971 \text{ kN} \]
- Row 2 \[ P_{r2} = 34.209 \text{ kN} \]
- Design axial load on column \[ N_{Edc} = 500 \text{ kN} \]
- Weld size \[ s_c = 6 \text{ mm} \]
- Stiffener width \[ b_{sgt} = 55 \text{ mm} \]
- Proposed rib thickness \[ t_{sr} = 10 \text{ mm} \]
- Proposed weld size \[ s_{sr} = 8 \text{ mm} \]

Selected fillet weld size
- Selected fillet weld size \[ s_{wt} = 8 \text{ mm} \]
- Selected fillet weld size \[ s_{fw} = 8 \text{ mm} \]

**CONNECTION SUMMARY - all dimensions in diagram are in mm**

```
\[ \begin{array}{c}
\text{Number of bolts} & 6 \\
\text{Bolt diameter} & 20 \text{ mm} \\
\text{Bolt grade} & 8.8 \\
\text{Welds} & \begin{array}{l}
\text{Tension flange fillet weld} \\
8 \text{ mm}
\end{array} \\
\begin{array}{l}
\text{Web fillet weld} \\
8 \text{ mm}
\end{array} \\
\begin{array}{l}
\text{Compression flange fillet weld} \\
6 \text{ mm}
\end{array} \\
\text{Supporting column section} & 152 \times 152 \times 23 \text{ UKC} \\
& \text{Grade S 275} \\
\text{Beam / Rafter section} & 152 \times 89 \times 16 \text{ UKB} \\
& \text{Grade S 275} \\
\text{Column stiffener details} & \\
\text{Diagonal stiffeners} & 2/60 \text{ mm} \times 10 \text{ mm} \text{ Grade S 275} \\
& \text{on line between the beam tension} \\
& \text{and compression flanges} \\
& \text{connected by 4/6 mm fillet welds.} \\
\text{Tension stiffeners} & \text{Provide full depth stiffeners} \\
& 10 \text{ mm} \text{ thick midway between} \\
& \text{rows 1 and 2. Any remaining} \\
& \text{ribs are partial depth. For} \\
& \text{curtailment length see code.} \\
& 2/55 \text{ mm wide placed midway}
\end{array} \]
```
between the following bolt rows:
1 and 2
Location: Ex1 - Three rows of tension bolts

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment $Ma=332$ kNm
Factored shear force $V=128$ kN
Axial force (+ve compression) $N=90$ kN

254 x 254 x 107 UC.
533 x 210 x 92 UB.
Diameter of bolts $bd=24$ mm
Thickness of plate $tp=25$ mm
Total number of bolts $bn=8$
Number of shear bolts $ns=2$
Distance to first row of bolts $ex=75$ mm
Distance to beam flange $x=15$ mm
Pitch of tension bolts $p2=90$ mm
Distance tension to shear bolts $p3=230$ mm
Edge distance for shear bolts $pes=100$ mm
Bolt cross-centres $g=100$ mm
Width of end plate $bp=200$ mm
Assumed web fillet weld size $sww=8$ mm
Assumed flange fillet weld size $swf=12$ mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Row</th>
<th>Potential Resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top row</td>
<td>$Pr1=375.14$</td>
</tr>
<tr>
<td>Row 2</td>
<td>$Pr2=303.2$</td>
</tr>
<tr>
<td>Row 3</td>
<td>$Pr3=286.25$</td>
</tr>
</tbody>
</table>
Connection of stiffeners to column web

Effective length of weld \( L_w = 4(L_s - 2s_{ww}) = 718.8 \text{ mm} \)
Assuming the load on the welds to be equal to the crushing resistance of stiffener.
Load on welds \( f_{ws} = \frac{P_{cs}10^3}{L_w} = 1161.3 \text{ N/mm} \)
For electrode grade 35
Design strength \( p_w = \text{TABLE 37 for } s_{grade}=275, Es=35 = 220 \text{ N/mm}^2 \)
Weld size \( s_{s} = 8 \text{ mm} \)
Capacity of weld \( p_{ws} = 0.7s_{s}p_w = 1232 \text{ N/mm} \)
ADOPT 2/120 mm x 15 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/8 mm fillet welds.

WELD DESIGN
Tension flange welds

For electrode grade 35
Design strength \( p_w = \text{TABLE 37 for } grad_m=275, Es=35 = 220 \text{ N/mm}^2 \)
Weld size required \( s_{wfr} = \frac{F_w10^3}{(0.7p_w)} = 11.297 \text{ mm} \)
Selected fillet weld size \( s_{wt} = 12 \text{ mm} \)

Web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, \( t_b \).
Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)
Shear zone - assuming a fillet weld of 8 mm continued for the full depth, the capacity of the beam web weld for vertical shear is:
Length of weld for shear \( L_{ws} = D_b - 2(T_b + r_b) - L_{wt} = 178.3 \text{ mm} \)
Capacity of weld for shear \( P_{sw} = 2*0.7s_{fw}p_wL_w/10^3 = 439.33 \text{ kN} \)
Vertical shear \( V = 128 \text{ kN} \)
Web weld size 8 mm is considered suitable.
CONNECTION SUMMARY

Number of bolts  8
Bolt diameter    24 mm
Bolt grade       8.8
Welds            Tension flange fillet weld  12 mm
                 Web fillet weld           8 mm
                 Compression flange fillet weld 8 mm
Supporting column section  254 x 254 x 107 UC
Beam / Rafter section     533 x 210 x 92 UB

Column stiffener details

Compression stiffeners  2/120 mm x 15 mm Grade S 275
connected by 4/8 mm fillet welds.

The connection is two sided.
Location: Ex2 - Four rows of tension bolts and mini haunch

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( Ma = 589 \) kNm
Factored shear force \( V = 137 \) kN
Axial force (+ve compression) \( N = 100 \) kN

254 x 254 x 107 UC.
533 x 210 x 92 UB.

Flange thickness \( Th = 15.6 \) mm
Flange width \( Bh = 209.3 \) mm
Web thickness \( th = 10.1 \) mm
Haunch depth \( Dh = 180 \) mm
Haunch angle to the beam flange \( \phi' = 30^\circ \)
Diameter of bolts \( bd = 24 \) mm
Thickness of plate \( tp = 25 \) mm
Total number of bolts \( bn = 10 \)
Number of shear bolts \( ns = 2 \)
Distance to first row of bolts \( ex = 75 \) mm
Distance to beam flange \( x = 15 \) mm
Pitch of tension bolts \( p2 = 90 \) mm
Edge distance for shear bolts \( pes = 100 \) mm
Over-all length of end plate \( Lc = 765 \) mm
Bolt cross-centres \( g = 120 \) mm
Width of end plate \( bp = 220 \) mm
Assumed web fillet weld size \( sww = 8 \) mm
Assumed flange fillet weld size \( swf = 12 \) mm
SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Row</th>
<th>Potential Resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top row</td>
<td>Pr1=366.8</td>
</tr>
<tr>
<td>Row 2</td>
<td>Pr2=265.19</td>
</tr>
<tr>
<td>Row 3</td>
<td>Pr3=265.19</td>
</tr>
<tr>
<td>Row 4</td>
<td>Pr4=265.19</td>
</tr>
</tbody>
</table>

Connection of stiffeners to column web

Effective length of weld \( L_w = 4 \times (L_{sn} - 2 \times s_{ww}) = 718.8 \text{ mm} \)

Assuming the load on the welds to be equal to the crushing resistance of stiffener.

Load on welds \( f_{ws} = \frac{P_{cs} \times 10^3}{L_w} = 1050.7 \text{ N/mm} \)

For electrode grade 35

Design strength \( \frac{p_w}{E_s} = \text{TABLE 37 for } s_{grade}=275, \frac{E_s}{Es}=35, p_w = 220 \text{ N/mm}^2 \)

Weld size \( s_s = 8 \text{ mm} \)

Capacity of weld \( p_{ws} = 0.7 \times s_s \times p_w = 1232 \text{ N/mm} \)

ADOPT 2/110 mm x 15 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/8 mm fillet welds.

WELD DESIGN

Tension flange welds

For electrode grade 35

Design strength \( \frac{p_w}{E_s} = \text{TABLE 37 for } gradm=275, \frac{E_s}{Es}=35, p_w = 220 \text{ N/mm}^2 \)

Weld size required \( s_{fr} = \frac{F_w \times 10^3}{0.7 \times p_w} = 10.046 \text{ mm} \)

Selected fillet weld size \( s_{wt} = 12 \text{ mm} \)

Web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, tb.

Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)

Shear zone - assuming a fillet weld of 8 mm continued for the full depth, the capacity of the beam web weld for vertical shear is:

Length of weld for shear \( L_{ws} = D_b - 2 \times (T_b + r_b) - L_{wt} = 251 \text{ mm} \)

Capacity of weld for shear \( P_{sw} = 2 \times 0.7 \times s_{fw} \times p_w \times L_{ws} / 10^3 = 618.46 \text{ kN} \)

Vertical shear \( V = 137 \text{ kN} \)

Web weld size 8 mm is considered suitable.
Number of bolts 10
Bolt diameter 24 mm
Bolt grade 8.8
Welds
Tension flange fillet weld 12 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm
Supporting column section 254 x 254 x 107 UC
Beam / Rafter section 533 x 210 x 92 UB
Haunch section 15.6 mm x 209.3 mm flange, 10.1 mm web
180 mm overall depth

Column stiffener details

Compression stiffeners 2/110 mm x 15 mm Grade S 275
connected by 4/8 mm fillet welds.

The connection is two sided.
**Location: Ex3 - Six rows of tension bolts and mini haunch**

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( Ma = 1500 \text{ kNm} \)
Factored shear force \( V = 237 \text{ kN} \)
Axial force (+ve compression) \( N = 100 \text{ kN} \)

305 x 305 x 158 UC.
686 x 254 x 140 UB.

Flange thickness \( Th = 19 \text{ mm} \)
Flange width \( Bh = 253.7 \text{ mm} \)
Web thickness \( th = 12.4 \text{ mm} \)
Haunch depth \( Dh = 450 \text{ mm} \)
Haunch angle to the beam flange \( \phi ' = 30^\circ \)
Diameter of bolts \( bd = 24 \text{ mm} \)
Thickness of plate \( tp = 25 \text{ mm} \)
Total number of bolts \( bn = 14 \)
Number of shear bolts \( ns = 2 \)
Distance to first row of bolts \( ex = 75 \text{ mm} \)
Distance to beam flange \( x = 15 \text{ mm} \)
Pitch of tension bolts \( p2 = 90 \text{ mm} \)
Edge distance for shear bolts \( pes = 100 \text{ mm} \)
Over-all length of end plate \( Lc = 1200 \text{ mm} \)
Bolt cross-centres \( g = 100 \text{ mm} \)
Width of end plate \( bp = 250 \text{ mm} \)
Assumed web fillet weld size \( sww = 10 \text{ mm} \)
Assumed flange fillet weld size \( swf = 12 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Row</th>
<th>Potential Resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>Pr1 = 395.36</td>
</tr>
<tr>
<td>Row 2</td>
<td>Pr2 = 361.92</td>
</tr>
<tr>
<td>Row 3</td>
<td>Pr3 = 328.48</td>
</tr>
<tr>
<td>Row 4</td>
<td>Pr4 = 295.03</td>
</tr>
<tr>
<td>Row 5</td>
<td>Pr5 = 261.59</td>
</tr>
<tr>
<td>Row 6</td>
<td>Pr6 = 228.15</td>
</tr>
</tbody>
</table>

Connection of stiffeners to column web

Effective length of weld: \( L_w = 4 \times (L_{sn} - 2 \times s_{ww}) = 884.4 \, \text{mm} \)

Assuming the load on the welds to be equal to the crushing resistance of stiffener.

Load on welds: \( f_{ws} = P_{cs} \times 10^3 / L_w = 1169.8 \, \text{N/mm} \)

For electrode grade 35

Design strength: \( p_w = \text{TABLE 37 for sgrade}=275, \, E_s=35 \) = 220 N/mm²

Weld size: \( s_{s} = 10 \, \text{mm} \)

Capacity of weld: \( p_{ws} = 0.7 \times s_{s} \times p_w = 1540 \, \text{N/mm} \)

ADOPT 2/140 mm x 16 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/10 mm fillet welds.

WELD DESIGN

Tension flange welds

For electrode grade 35

Design strength: \( p_w = \text{TABLE 37 for gradm}=275, \, E_s=35 \) = 220 N/mm²

Weld size required: \( s_{fr} = F_w \times 10^3 / (0.7 \times p_w) = 10.085 \, \text{mm} \)

Selected fillet weld size: \( s_{wt} = 12 \, \text{mm} \)

Web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, \( t_b \).

Selected fillet weld size: \( s_{fw} = 10 \, \text{mm} \)

Shear zone - assuming a fillet weld of 10 mm continued for the full depth, the capacity of the beam web weld for vertical shear is:

Length of weld for shear: \( L_{ws} = D_b - 2 \times (T_b + r_b) - L_{wt} = 502.8 \, \text{mm} \)

Capacity of weld for shear: \( P_{sw} = 2 \times 0.7 \times s_{fw} \times p_w \times L_{ws} / 10^3 = 1548.6 \, \text{kN} \)

Vertical shear: \( V = 237 \, \text{kN} \)

Web weld size 10 mm is considered suitable.
CONNECTION SUMMARY

Number of bolts 14
Bolt diameter 24 mm
Bolt grade 8.8
Welds  
  Tension flange fillet weld 12 mm
  Web fillet weld 10 mm
  Compression flange fillet weld 8 mm
Supporting column section 305 x 305 x 158 UC
Beam / Rafter section 686 x 254 x 140 UB
Haunch section 19 mm x 253.7 mm flange, 12.4 mm web
  450 mm overall depth

Column stiffener details

Compression stiffeners 2/140 mm x 16 mm Grade S 275
  connected by 4/10 mm fillet welds.

The connection is two sided.
Location: Ex4 - UB to UB connection

The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( Ma=312 \text{ kNm} \)
Factored shear force \( V=345 \text{ kN} \)
Axial force (+ve compression) \( N=200 \text{ kN} \)

457 x 191 x 98 UB.
406 x 178 x 67 UB.
Flange thickness \( Th=15 \text{ mm} \)
Flange width \( Bh=180 \text{ mm} \)
Web thickness \( th=10 \text{ mm} \)
Haunch depth \( Dh=300 \text{ mm} \)
Haunch angle to the beam flange \( \phi'=30^\circ \)
Diameter of bolts \( bd=24 \text{ mm} \)
Thickness of plate \( tp=24 \text{ mm} \)
Total number of bolts \( bn=10 \)
Number of shear bolts \( ns=4 \)
Distance to first row of bolts \( ex=90 \text{ mm} \)
Distance to beam flange \( x=25 \text{ mm} \)
Pitch of tension bolts \( p2=70 \text{ mm} \)
Edge distance for shear bolts \( pes=100 \text{ mm} \)
Over-all length of end plate \( Lc=770 \text{ mm} \)
Bolt cross-centres \( g=100 \text{ mm} \)
Width of end plate \( bp=180 \text{ mm} \)
Assumed web fillet weld size \( sww=8 \text{ mm} \)
Assumed flange fillet weld size \( swf=10 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts \( P_{r1} = 354.24 \) kN  
Row 2 \( P_{r2} = 265.62 \) kN  
Row 3 \( P_{r3} = 265.62 \) kN

Connection of stiffeners to column web

Effective length of weld \( L_w = 4 \times (L_{sn} - 2 \times s_{ww}) = 1544 \) mm  
Load on welds \( f_{ws} = (P_{RT} - P_{bw}) \times 10^3 / L_w = 263.2 \) N/mm  
For electrode grade 35 \( P_w = \text{TABLE 37 for } s_{grade} = 275, \ Es = 35 \)  
Design strength \( = 220 \) N/mm\(^2\)  
Weld size \( s_{s} = 6 \) mm  
Capacity of weld \( P_{ws} = 0.7 \times s_{s} \times P_w = 924 \) N/mm  
ADOPT 2/90 mm x 20 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/6 mm fillet welds.  
Slip factor for preloaded bolts \( \mu = 0.5 \)  
Coefficient for type of hole \( K_s = 1 \)

WELD DESIGN

Tension flange welds

For electrode grade 35 \( P_w = \text{TABLE 37 for } gradm = 275, \ Es = 35 \)  
Design strength \( = 220 \) N/mm\(^2\)  
Weld size required \( s_{wfr} = F_w \times 10^3 / (0.7 \times P_w) = 11.54 \) mm  
Selected fillet weld size \( s_{wt} = 12 \) mm

Web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses \( \geq \) the web thickness, \( t_b \).  
Selected fillet weld size \( s_{fw} = 8 \) mm  
Shear zone - assuming a fillet weld of 8 mm continued for the full depth, the capacity of the beam web weld for vertical shear is:  
Length of weld for shear \( L_{ws} = D_b - 2 \times (T_b + r_b) - L_{wt} = 393.4 \) mm  
Capacity of weld for shear \( P_{sw} = 2 \times 0.7 \times s_{fw} \times P_w \times L_{ws} / 10^3 = 969.34 \) kN  
Vertical shear \( V = 345 \) kN  
Web weld size 8 mm is considered suitable.
Connection Summary

Number of bolts: 10
Bolt diameter: 24 mm
Bolt grade: HSFG
General grade (8.8) to BS4395 Part 1. Nuts grade 10.
Welds:
- Tension flange fillet weld: 12 mm
- Web fillet weld: 8 mm
- Compression flange fillet weld: 8 mm
Supporting column section: 457 x 191 x 98 UB
Beam / Rafter section: 406 x 178 x 67 UB
Haunch section: 15 mm x 180 mm flange, 10 mm web 300 mm overall depth

Column stiffener details

Compression stiffeners: 2/90 mm x 20 mm Grade S 275
connected by 4/6 mm fillet welds.

The connection is one sided.
Location: Ex1 - Three rows of tension bolts

Bolted flush end plate connection - beam to column

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments. Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is attached to the supporting column flange.

Factored moment \( My_{Ed} = 332 \text{ kNm} \)
Factored shear force \( V_{Ed} = 128 \text{ kN} \)
Axial force (+ve compression) \( N_{Ed} = 90 \text{ kN} \)

Supporting column details
254 x 254 x 107 UKC

Supported beam details
533 x 210 x 92 UKB

Bolt details
Diameter of bolts \( db = 24 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( F_{Rd} = k_2 f_{ub} A_t / (\gamma M^2 \times 10^3) = 203.33 \text{ kN} \)

End plate details
Thickness of plate \( tp = 25 \text{ mm} \)
Number of bolts in end plate \( bn = 8 \)
Number of shear bolts \( ns = 2 \)
Distance to beam flange \( x_1 = 15 \text{ mm} \)
Distance to first row of bolts \( e_1 = 75 \text{ mm} \)
Pitch of tension bolts \( p_1 = 90 \text{ mm} \)
Distance tension to shear bolts  \( p_1' = 230 \text{ mm} \)
Edge distance for shear bolts  \( e_1' = 100 \text{ mm} \)
Cross-centres of bolts  \( p_3 = 100 \text{ mm} \)
Width of end plate  \( b_p = 200 \text{ mm} \)
Assumed web fillet weld size  \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size  \( s_{wf} = 12 \text{ mm} \)

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top row of bolts</td>
<td>( P_{r1} = 381.42 \text{ kN} )</td>
</tr>
<tr>
<td>Row 2</td>
<td>( P_{r2} = 309.47 \text{ kN} )</td>
</tr>
<tr>
<td>Row 3</td>
<td>( P_{r3} = 292.53 \text{ kN} )</td>
</tr>
<tr>
<td>Design axial load on column</td>
<td>( N_{Edc} = 500 \text{ kN} )</td>
</tr>
</tbody>
</table>

**Connection of stiffeners to column web**

- Effective length of weld  \( L_w = 4 \times (L_{sn} - 2 \times s_{ww}) = 718.8 \text{ mm} \)
- Load on welds  \( F_{wEds} = F_{Crds} / L_w = 1.1613 \text{ kN/mm} \)
- Weld design strength  \( f_{wEds} = f_u / \sqrt{3} / (\beta_{aw} \cdot \gamma_{M2}) = 222.79 \text{ N/mm}^2 \)
- Weld size (leg length)  \( s_s = 8 \text{ mm} \)
- Resistance of weld  \( F_{wRds} = 0.7 \times s_s \times f_{wEds} / 1000 = 1.2476 \text{ kN/mm} \)

Adopt 2/120 mm x 15 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/8 mm fillet welds.

**Beam tension flange welds**

- Weld design strength  \( f_{vwd} = f_u / \sqrt{3} / (\beta_{aw} \cdot \gamma_{M2}) = 222.79 \text{ N/mm}^2 \)
- Tension resistance of the flange  \( T_f = b_p \times t_{fb} \times f_{yb} / 10^3 = 858 \text{ kN} \)
- Tension force in top 2 bolt rows  \( T_{fc} = P_{r1} + P_{r2} = 690.89 \text{ kN} \)
- Weld force per mm  \( F_{wEdf} = T_{fb} / (2 \times b_p - t_{wb}) = 1.772 \text{ kN/mm} \)
- Selected fillet weld size  \( s_{wt} = 12 \text{ mm} \)

**Beam web welds**

- Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses \( \geq t_{wb} \) \( s_{fw} = 8 \text{ mm} \)
- Shear zone - assume an 8 mm FW continued for the full depth.
- Length of weld for shear  \( L_{ws} = h_b - 2 \times (t_{fb} + r_b) - L_{wt} = 178.3 \text{ mm} \)
- Resistance of weld for shear  \( F_{wRds} = 2 \times 0.7 \times s_{fw} \times f_{vwd} \times L_{ws} / 10^3 = 444.9 \text{ kN} \)
- Vertical shear  \( V_{Ed} = 128 \text{ kN} \)

An 8 mm weld size is therefore considered suitable.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts  8
Bolt diameter  24 mm
Bolt grade  8.8
Welds  Tension flange fillet weld  12 mm
Web fillet weld  8 mm
Compression flange fillet weld  8 mm
Supporting column section  254 x 254 x 107 UKC
Beam / Rafter section  533 x 210 x 92 UKB

Column stiffener details

Compression stiffeners  2/120 mm x 15 mm Grade S 275
connected by 4/8 mm fillet welds.

The connection under consideration is two sided.
Location: Ex2 - Four rows of tension bolts and mini haunch

Bolted flush end plate connection - beam to column

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments. Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is attached to the supporting column flange.

Factored moment MyEd=589 kNm
Factored shear force VEd=137 kN
Axial force (+ve compression) NEd=100 kN

Supporting column details
254 x 254 x 132 UKC

Supported beam details
533 x 210 x 92 UKB

Haunch details
Flange thickness tfh=15.6 mm
Flange width bh=209.3 mm
Web thickness twh=10.1 mm
Haunch depth hh=180 mm
Haunch angle to the beam flange phi'=30°

Bolt details
Diameter of bolts db=24 mm
Tensile resistance (EC3 Part 1-8) FtRd=k2*fub*At/(gamM2*10^3)=203.33 kN
End plate details

- Thickness of plate: \( t_p = 25 \text{ mm} \)
- Number of bolts in end plate: \( b_n = 10 \)
- Number of shear bolts: \( n_s = 2 \)
- Distance to beam flange: \( x_1 = 15 \text{ mm} \)
- Distance to first row of bolts: \( e_1 = 75 \text{ mm} \)
- Pitch of tension bolts: \( p_1 = 90 \text{ mm} \)
- Edge distance for shear bolts: \( e_{1}' = 100 \text{ mm} \)
- Over-all length of end plate: \( h_p = 765 \text{ mm} \)
- Cross-centres of bolts: \( p_3 = 120 \text{ mm} \)
- Width of end plate: \( b_p = 220 \text{ mm} \)
- Assumed web fillet weld size: \( s_{ww} = 8 \text{ mm} \)
- Assumed flange fillet weld size: \( s_{wf} = 12 \text{ mm} \)

SUMMARY OF BOLT POTENTIAL RESISTANCES

- Top row of bolts: \( P_{r1} = 406.66 \text{ kN} \)
- Row 2: \( P_{r2} = 316.37 \text{ kN} \)
- Row 3: \( P_{r3} = 281.95 \text{ kN} \)
- Row 4: \( P_{r4} = 236.51 \text{ kN} \)
- Design axial load on column: \( N_{Edc} = 500 \text{ kN} \)

Connection of stiffeners to column web

- Effective length of weld: \( L_w = 4 \times (L_{sn} - 2 \times s_{ww}) = 718.8 \text{ mm} \)
- Load on welds: \( F_{wEds} = F_{crds} / L_w = 1.1613 \text{ kN/mm} \)
- Weld design strength: \( f_{wds1} = f_u / \sqrt{3} / (\beta_{aw} \cdot \gamma_{M2}) = 222.79 \text{ N/mm}^2 \)
- Weld size (leg length): \( s_s = 10 \text{ mm} \)
- Resistance of weld: \( F_{wRds} = 0.7 \times s_s \times f_{wds1} / 1000 = 1.5595 \text{ kN/mm} \)
- Adopt 2/120 mm x 15 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/10 mm fillet welds.
- Thickness of backing plate: \( t_{bp} = 25 \text{ mm} \)
- Width of backing plate: \( b_{bp} = 125 \text{ mm} \)
- Length of backing plate: \( L_{bp} = 550 \text{ mm} \)

Beam tension flange welds

- Weld design strength: \( f_{wds} = f_u / \sqrt{3} / (\beta_{aw} \cdot \gamma_{M2}) = 222.79 \text{ N/mm}^2 \)
- Tension resistance of the flange: \( T_f = b_b \times t_{fb} \times f_y / 10^3 = 897.9 \text{ kN} \)
- Tension force in top 2 bolt rows: \( T_{fc} = P_{r1} + P_{r2} = 723.02 \text{ kN} \)
- Weld force per mm: \( F_{wEdf} = T_f / (2 \times b_b - t_{wb}) = 1.7699 \text{ kN/mm} \)
- Selected fillet weld size: \( s_{wt} = 12 \text{ mm} \)

Beam web welds

- Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses \( \geq \) the web thickness, \( t_{wb} \).
- Selected fillet weld size: \( s_{fw} = 8 \text{ mm} \)

- Shear zone - assume an 8 mm FW continued for the full depth.

Length of weld for shear: \( L_{ws} = h_b - 2 \times (t_{fb} + r_b) - L_{wt} = 251 \text{ mm} \)
Resistance of weld for shear  \[ F_{\text{wRds}} = 2 \times 0.7 \times sfw \times fvwd \times Lws / 10^3 \]  
\[ = 626.31 \text{ kN} \]

Vertical shear  \[ V_{\text{Ed}} = 137 \text{ kN} \]
An 8 mm weld size is therefore considered suitable.

**CONNECTION SUMMARY - all dimensions in diagram are in mm**

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<tr>
<td>e1'</td>
<td></td>
<td>100</td>
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</table>

36.9  50  120  50

Number of bolts 10
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 12 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm

Supporting column section 254 x 254 x 132 UKC
Beam / Rafter section 533 x 210 x 92 UKB
Haunch section 15.6 mm x 209.3 mm flange, 10.1 mm web 180 mm overall depth.

Column stiffener details

- Compression stiffeners 2/120 mm x 15 mm Grade S 275 connected by 4/10 mm fillet welds.
- Backing plates 2/25 mm x 125 mm x 550 mm Grade S 275

The connection under consideration is two sided.
Bolted flush end plate connection - beam to column

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments. Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is attached to the supporting column flange.

Factored moment: $\text{MyEd}=1500\ \text{kNm}$
Factored shear force: $\text{VED}=215\ \text{kN}$
Axial force (+ve compression): $\text{NEd}=100\ \text{kN}$

Supporting column details

305 x 305 x 158 UKC

Supported beam details

686 x 254 x 140 UKB

Haunch details

Flange thickness: $\text{tfh}=19\ \text{mm}$
Flange width: $\text{bh}=253.7\ \text{mm}$
Web thickness: $\text{twh}=12.4\ \text{mm}$
Haunch depth: $\text{hh}=550\ \text{mm}$
Haunch angle to the beam flange: $\phi'=30^\circ$

Bolt details

Diameter of bolts: $\text{db}=24\ \text{mm}$
Tensile resistance (EC3 Part 1-8): $\text{FtRd}=k_2*f_{ub}*A_t/(\gamma_M^2*10^3)=203.33\ \text{kN}$
End plate details

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of plate</td>
<td>tp=25 mm</td>
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<tr>
<td>Number of bolts in end plate</td>
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</tr>
<tr>
<td>Number of shear bolts</td>
<td>ns=2</td>
</tr>
<tr>
<td>Distance to beam flange</td>
<td>x1=15 mm</td>
</tr>
<tr>
<td>Distance to first row of bolts</td>
<td>e1=75 mm</td>
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<tr>
<td>Pitch of tension bolts</td>
<td>p1=90 mm</td>
</tr>
<tr>
<td>Edge distance for shear bolts</td>
<td>e1'=100 mm</td>
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<tr>
<td>Over-all length of end plate</td>
<td>hp=1300 mm</td>
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<td>Cross-centres of bolts</td>
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<tr>
<td>Width of end plate</td>
<td>bp=250 mm</td>
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<td>Assumed web fillet weld size</td>
<td>sww=10 mm</td>
</tr>
<tr>
<td>Assumed flange fillet weld size</td>
<td>swf=12 mm</td>
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SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Row</th>
<th>Potential Resistance (kN)</th>
</tr>
</thead>
<tbody>
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<td>Top</td>
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<tr>
<td>Row 2</td>
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<td>Row 3</td>
<td>Pr3=324.55</td>
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<tr>
<td>Row 4</td>
<td>Pr4=312.33</td>
</tr>
<tr>
<td>Row 5</td>
<td>Pr5=280.89</td>
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<tr>
<td>Row 6</td>
<td>Pr6=249.44</td>
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<tr>
<td>Total</td>
<td>Pr=2015.81</td>
</tr>
</tbody>
</table>

Connection of stiffeners to column web

Effective length of weld: Lw=4*(Lsn-2*sww)=884.4 mm
Load on welds: FwEds=FcRds/Lw=1.4622 kN/mm
Weld design strength: fvwd1=fu/SQR(3)/(betaw*gamM2) =222.79 N/mm²
Weld size (leg length): ss=10 mm
Resistance of weld: FwRds=0.7*ss*fvwd1/1000=1.5595 kN/mm
Adopt 2/140 mm x 20 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/10 mm fillet welds.

Beam tension flange welds

Weld design strength: fvwd=fu/SQR(3)/(betaw*gamM2) =222.79 N/mm²
Tension resistance of the flange: Tf=bp*tfb*fyb/10^3=1258.8 kN
Tension force in top 2 bolt rows: Tfc=Pr1+Pr2=781.87 kN
Weld force per mm: FwEdf=Tfb/(2*bp-twb)=1.6035 kN/mm
Selected fillet weld size: swt=12 mm

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, twb.
Selected fillet weld size: sfw=10 mm

Shear zone - assume an 10 mm FW continued for the full depth.
Length of weld for shear: Lws=hb-2*(tfb+rb)-Lwt=602.8 mm
Resistance of weld for shear: FwRds=2*0.7*sfw*fvwd*Lws/10^3 =1880.2 kN
Vertical shear: VEd=215 kN
An 10 mm weld size is therefore considered suitable.

**CONNECTION SUMMARY** - all dimensions in diagram are in mm

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
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</tbody>
</table>

Number of bolts 14  
Bolt diameter 24 mm  
Bolt grade 8.8  
Welds  
- Tension flange fillet weld 12 mm  
- Web fillet weld 10 mm  
- Compression flange fillet weld 8 mm  

Supporting column section 305 x 305 x 158 UKC  
Beam / Rafter section 686 x 254 x 140 UKB  
Haunch section 19 mm x 253.7 mm flange, 12.4 mm web  
550 mm overall depth.

Column stiffener details

Compression stiffeners 2/140 mm x 20 mm Grade S 275  
connected by 4/10 mm fillet welds.

The connection under consideration is two sided.
Location: Ex4 - UKB to UKB connection

Bolted flush end plate connection - beam to column

The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force. Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is attached to the supporting column flange.

Factored moment \( MyEd = 312 \text{ kNm} \)
Factored shear force \( VEd = 345 \text{ kN} \)
Axial force (+ve compression) \( NEd = 200 \text{ kN} \)

Supporting column details
457 x 191 x 98 UKB

Supported beam details
406 x 178 x 67 UKB

Haunch details
Flange thickness \( tfh = 15 \text{ mm} \)
Flange width \( bh = 180 \text{ mm} \)
Web thickness \( twh = 10 \text{ mm} \)
Haunch depth \( hh = 300 \text{ mm} \)
Haunch angle to the beam flange \( \phi' = 30^\circ \)

Bolt details
Diameter of bolts \( db = 24 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( FtRd = k2*fub*At/(\gamma M2*10^3) = 203.33 \text{ kN} \)
End plate details

- Thickness of plate: \( tp = 24 \text{ mm} \)
- Number of bolts in end plate: \( bn = 10 \)
- Number of shear bolts: \( ns = 4 \)
- Distance to beam flange: \( x_1 = 25 \text{ mm} \)
- Distance to first row of bolts: \( e_1 = 90 \text{ mm} \)
- Pitch of tension bolts: \( p_1 = 70 \text{ mm} \)
- Edge distance for shear bolts: \( e'_1 = 100 \text{ mm} \)
- Over-all length of end plate: \( hp = 770 \text{ mm} \)
- Cross-centres of bolts: \( p_3 = 100 \text{ mm} \)
- Width of end plate: \( bp = 180 \text{ mm} \)
- Assumed web fillet weld size: \( s_{ww} = 8 \text{ mm} \)
- Assumed flange fillet weld size: \( swf = 10 \text{ mm} \)

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

- Top row of bolts: \( P_{r1} = 349.05 \text{ kN} \)
- Row 2: \( P_{r2} = 260.43 \text{ kN} \)
- Row 3: \( P_{r3} = 260.43 \text{ kN} \)
- Design axial load on column: \( N_{Edc} = 500 \text{ kN} \)

**Connection of stiffeners to column web**

- Effective length of weld: \( L_w = 4 \times (L_{sn} - 2 \times s_{ww}) = 1544 \text{ mm} \)
- Load on welds: \( F_{wEds} = \frac{(PRT - F_{cRdcw})}{L_w} = 0.3919 \text{ kN/mm} \)
- Weld design strength: \( f_{vwd} = \frac{f_u}{\sqrt{3}} / (\beta_w \cdot \gamma_{M2}) = 222.79 \text{ N/mm}^2 \)
- Weld size (leg length): \( s_s = 6 \text{ mm} \)
- Resistance of weld: \( F_{wRds} = 0.7 \times s_s \times f_{vwd} / 1000 = 0.93571 \text{ kN/mm} \)
- Adopt 2/90 mm x 20 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/6 mm fillet welds.

**Beam tension flange welds**

- Weld design strength: \( f_{vwd} = \frac{f_u}{\sqrt{3}} / (\beta_w \cdot \gamma_{M2}) = 222.79 \text{ N/mm}^2 \)
- Tension resistance of the flange: \( T_f = b_b \times t_{fb} \times f_{yb} / 10^3 = 703.13 \text{ kN} \)
- Tension force in top 2 bolt rows: \( T_{fc} = P_{r1} + P_{r2} = 566.36 \text{ kN} \)
- Weld force per mm: \( F_{wEdf} = T_{fb} / (2 \times b_b - t_{wb}) = 1.6237 \text{ kN/mm} \)
- Selected fillet weld size: \( s_{wt} = 12 \text{ mm} \)

**Beam web welds**

- Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses \( t_{wb} \) the web thickness.
  - Selected fillet weld size: \( s_{fw} = 8 \text{ mm} \)
- Shear zone - assume an 8 mm FW continued for the full depth.
  - Length of weld for shear: \( L_{ws} = h_b - 2 \times (t_{fb} + r_b) - L_{wt} = 393.4 \text{ mm} \)
  - Resistance of weld for shear: \( F_{wRds} = 2 \times 0.7 \times s_{fw} \times f_{vwd} \times L_{ws} / 10^3 = 981.63 \text{ kN} \)
- Vertical shear: \( V_{Ed} = 345 \text{ kN} \)
  - An 8 mm weld size is therefore considered suitable.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 10
Bolt diameter 24 mm
Bolt grade Preloaded HSFG bolts
   General grade (8.8) with grade 10 nuts.
Welds Tension flange fillet weld 12 mm
   Web fillet weld 8 mm
   Compression flange fillet weld 8 mm
Supporting column section 457 x 191 x 98 UKB
Beam / Rafter section 406 x 178 x 67 UKB
Haunch section 15 mm x 180 mm flange, 10 mm web
   300 mm overall depth.

Column stiffener details

Compression stiffeners 2/90 mm x 20 mm Grade S 275
   connected by 4/6 mm fillet welds.

The connection under consideration is one sided.
Location: Ex5 - Compression & tension rib stiffeners plus haunch

Bolted flush end plate connection - beam to column

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments. Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is attached to the supporting column flange.

Factored moment MyEd=589 kNm
Factored shear force VEd=137 kN
Axial force (+ve compression) NEd=100 kN

Supporting column details

254 x 254 x 132 UKC

Supported beam details

533 x 210 x 92 UKB

Haunch details

Flange thickness tfh=15.6 mm
Flange width bh=209.3 mm
Web thickness twh=10.1 mm
Haunch depth hh=180 mm
Haunch angle to the beam flange phi'=30°

Bolt details

Diameter of bolts db=24 mm
Tensile resistance (EC3 Part 1-8) FtRd=k2*fub*At/(gamM2*10^3)=203.33 kN
End plate details

- Thickness of plate: $t_p = 25$ mm
- Number of bolts in end plate: $b_n = 10$
- Number of shear bolts: $n_s = 2$
- Distance to beam flange: $x_1 = 15$ mm
- Distance to first row of bolts: $e_1 = 85$ mm
- Pitch of tension bolts: $p_1 = 70$ mm
- Edge distance for shear bolts: $e_{1'} = 85$ mm
- Over-all length of end plate: $h_p = 765$ mm
- Cross-centres of bolts: $p_3 = 120$ mm
- Width of end plate: $b_p = 220$ mm
- Assumed web fillet weld size: $s_{ww} = 8$ mm
- Assumed flange fillet weld size: $s_{wf} = 12$ mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

- Top row of bolts: $P_{r1} = 406.66$ kN
- Row 2: $P_{r2} = 292.79$ kN
- Row 3: $P_{r3} = 265.14$ kN
- Row 4: $P_{r4} = 265.14$ kN
- Design axial load on column: $N_{Edc} = 500$ kN

Connection of stiffeners to column web

- Effective length of weld: $L_w = 4 \times (L_{sn} - 2 \times s_{ww}) = 718.8$ mm
- Load on welds: $F_{WEds} = F_{C Rds} / L_w = 1.1613$ kN/mm
- Weld design strength: $f_{wd1} = f_u / \sqrt{3} / (\beta_{w} \cdot \gamma_{M2})$
  $= 222.79$ N/mm²
- Weld size (leg length): $s_s = 10$ mm
- Resistance of weld: $F_{W Rds} = 0.7 \times s_s \times f_{wd1} / 1000 = 1.5595$ kN/mm
- Adopt 2/120 mm x 15 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/10 mm fillet welds.
- Stiffener width: $b_{sgt} = 100$ mm
- Proposed rib stiffener thickness: $t_{sr} = 20$ mm
- Proposed weld size: $s_{sr} = 15$ mm

Beam tension flange welds

- Weld design strength: $f_{wd} = f_u / \sqrt{3} / (\beta_{w} \cdot \gamma_{M2})$
  $= 222.79$ N/mm²
- Tension resistance of the flange: $T_f = b_b \times t_{fb} \times f_{yb} / 10^3 = 897.9$ kN
- Tension force in top 2 bolt rows: $T_{fc} = P_{r1} + P_{r2} = 699.45$ kN
- Weld force per mm: $F_{WEdf} = T_f / (2 \times b_b - t_{wb}) = 1.7122$ kN/mm
- Selected fillet weld size: $s_{wt} = 12$ mm

Beam web welds

- Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, $t_{wb}$.
- Selected fillet weld size: $s_{fw} = 8$ mm

- Shear zone - assume an 8 mm FW continued for the full depth.
Length of weld for shear \( L_{ws} = h_b - 2*(t_{fb}+r_b) - L_{wt} = 301 \text{ mm} \)

Resistance of weld for shear \( F_{wRds} = 2*0.7*sfw*fvwd*L_{ws}/10^3 \)
\( = 751.07 \text{ kN} \)

Vertical shear \( V_{Ed} = 137 \text{ kN} \)

An 8 mm weld size is therefore considered suitable.

**CONNECTION SUMMARY - all dimensions in diagram are in mm**

![Diagram](image)

- Number of bolts: 10
- Bolt diameter: 24 mm
- Bolt grade: 8.8
- Welds:
  - Tension flange fillet weld: 12 mm
  - Web fillet weld: 8 mm
  - Compression flange fillet weld: 8 mm
- Supporting column section: 254 x 254 x 132 UKC
- Beam / Rafter section: 533 x 210 x 92 UKB
- Haunch section: 15.6 mm x 209.3 mm flange, 10.1 mm web 180 mm overall depth.

**Column stiffener details**

- Compression stiffeners: 2/120 mm x 15 mm Grade S 275 connected by 4/10 mm fillet welds.
- Tension stiffeners: Full depth 20 mm thick. 2/100 mm wide placed midway between the following bolt rows: 1 and 2

The connection under consideration is two sided.
Location: Ex6 - Two rows of tension bolts

Bolted flush end plate connection - beam to column

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments. Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is attached to the supporting column flange.

Factored moment MyEd=100 kNm
Factored shear force VEd=128 kN
Axial force (+ve compression) NEd=90 kN

Supporting column details
254 x 254 x 107 UKC

Supported beam details
533 x 210 x 92 UKB

Bolt details
Diameter of bolts db=20 mm
Tensile resistance (EC3 Part 1-8) FtRd=k2*fub*At/(gamM2*10^3)=141.12 kN

End plate details
Thickness of plate tp=25 mm
Number of bolts in end plate bn=6
Number of shear bolts ns=2
Distance to beam flange xl=15 mm
Distance to first row of bolts e1=100 mm
Pitch of tension bolts p1=75 mm
Edge distance for shear bolts \( e_1' = 100 \text{ mm} \)
Over-all length of end plate \( h_p = 600 \text{ mm} \)
Cross-centres of bolts \( p_3 = 100 \text{ mm} \)
Width of end plate \( b_p = 200 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)

### SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Component</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top row of bolts</td>
<td>( P_{r1} = 282.24 \text{ kN} )</td>
</tr>
<tr>
<td>Row 2</td>
<td>( P_{r2} = 234.16 \text{ kN} )</td>
</tr>
<tr>
<td>Design axial load on column</td>
<td>( N_{Edc} = 500 \text{ kN} )</td>
</tr>
</tbody>
</table>

**Connection of stiffeners to column web**

Effective length of weld \( L_w = 4 \times (L_{sn} - 2 \times s_{ww}) = 718.8 \text{ mm} \)
Load on welds \( F_{wEds} = F_{cRds}/L_w = 1.1613 \text{ kN/mm} \)
Weld design strength \( f_{wd1} = f_u / \sqrt{3} / (\beta_{w} \gamma_{M2}) = 222.79 \text{ N/mm}^2 \)
Weld size (leg length) \( s_s = 8 \text{ mm} \)
Resistance of weld \( F_{wRds} = 0.7 \times s_s \times f_{wd1}/1000 = 1.2476 \text{ kN/mm} \)
Adopt 2/120 mm x 15 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/8 mm fillet welds.

**Beam tension flange welds**

Weld design strength \( f_{vd} = f_u / \sqrt{3} / (\beta_{w} \gamma_{M2}) = 222.79 \text{ N/mm}^2 \)
Tension resistance of the flange \( T_f = b_p t_f b \cdot f_y b / 10^3 = 858 \text{ kN} \)
Tension force in top 2 bolt rows \( T_{fc} = P_{r1} + P_{r2} = 516.4 \text{ kN} \)
Weld force per mm \( F_{wEdf} = T_{fb} / (2 \times b_p - t_w b) = 1.3245 \text{ kN/mm} \)
Selected fillet weld size \( s_{wt} = 10 \text{ mm} \)

**Beam web welds**

A full strength weld will be used.
As the beam web ≤ 11.3 mm, no calculations are needed for tension or shear provided that full strength welds are used.
Full strength weld \( s_{wf'} = \text{INT}(s_{wf'}) = 7 \text{ mm} \)
Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 6
Bolt diameter 20 mm
Bolt grade 8.8
Welds  
  Tension flange fillet weld 10 mm
  Web fillet weld 8 mm
  Compression flange fillet weld 8 mm
Supporting column section 254 x 254 x 107 UKC
Beam / Rafter section 533 x 210 x 92 UKB

Column stiffener details

Compression stiffeners 2/120 mm x 15 mm Grade S 275
  connected by 4/8 mm fillet welds.

The connection under consideration is two sided.
Location: Ex1 - Two rows of tension bolts

Bolted flush end plate connection - beam to column

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments. Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is attached to the supporting column flange.

Factored moment  MyEd=136 kNm
Factored shear force  VEd=164.6 kN
Axial force (+ve compression)  NEd=4.43 kN

Supporting column details
203 x 203 x 60 UKC

Supported beam details
356 x 171 x 57 UKB

Bolt details
Diameter of bolts  db=24 mm
Tensile resistance (EC3 Part 1-8)  FtRd=k2*fub*At/(gamM2*10^3)=203.33 kN

End plate details
Thickness of plate  tp=15 mm
Number of bolts in end plate  bn=6
Number of shear bolts  ns=2
Distance to beam flange  xl=30 mm
Distance to first row of bolts  e1=90 mm
Pitch of tension bolts  p1=120 mm
Distance tension to shear bolts \( p_1' = 230 \text{ mm} \)
Edge distance for shear bolts \( e_1' = 93 \text{ mm} \)
Cross-centres of bolts \( p_3 = 90 \text{ mm} \)
Width of end plate \( b_p = 190 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 8 \text{ mm} \)

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

| Top row of bolts | \( P_{r1} = 302.96 \text{ kN} \) |
| Row 2            | \( P_{r2} = 250.75 \text{ kN} \) |
| Design axial load on column | \( N_{Edc} = 521.8 \text{ kN} \) |

**Connection of stiffeners to column web**

| Effective length of weld | \( L_w = 4 \times (L_{sn} - 2 \times s_{ww}) = 556.8 \text{ mm} \) |
| Load on welds            | \( F_{wEds} = F_{cRds} / L_w = 1.215 \text{ kN/mm} \) |
| Weld design strength     | \( f_{vwd1} = f_u / \sqrt{3} / (\beta_{w} \cdot \gamma_{M2}) \) = 222.79 N/mm² |
| Weld size (leg length)   | \( s_s = 8 \text{ mm} \) |
| Resistance of weld       | \( F_{wRds} = 0.7 \times s_s \times f_{vwd1} / 1000 = 1.2476 \text{ kN/mm} \) |
| Adopt 2/95 mm x 15 mm plate stiffeners Grade S 275 adjacent to the haunch compression flange connected by 4/8 mm fillet welds. |
| Stiffener width           | \( b_{sgt} = 75 \text{ mm} \) |
| Proposed rib stiffener thickness | \( t_{sr} = 10 \text{ mm} \) |
| Proposed weld size       | \( s_{sr} = 8 \text{ mm} \) |

**Beam tension flange welds**

| Weld design strength     | \( f_{vwd} = f_u / \sqrt{3} / (\beta_{w} \cdot \gamma_{M2}) \) = 222.79 N/mm² |
| Tension resistance of the flange | \( T_f = b_b \times t_{fb} \times f_{yb} / 10^3 = 615.62 \text{ kN} \) |
| Tension force in top 2 bolt rows | \( T_{fc} = P_{r1} + P_{r2} = 586.31 \text{ kN} \) |
| Weld force per mm        | \( F_{wEdf} = T_{fb} / (2 \times b_b - t_{wb}) = 1.7434 \text{ kN/mm} \) |
| Selected fillet weld size | \( s_{wt} = 12 \text{ mm} \) |

**Beam web welds**

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, \( t_{wb} \).

| Selected fillet weld size | \( s_{fw} = 8 \text{ mm} \) |

Shear zone - assume an 8 mm FW continued for the full depth.

| Length of weld for shear  | \( L_{ws} = h_b - 2 \times (t_{fb} + r_b) - L_{wt} = 76.95 \text{ mm} \) |
| Resistance of weld for shear | \( F_{wRds} = 2 \times 0.7 \times s_{fw} \times f_{vwd} \times L_{ws} / 10^3 = 192.01 \text{ kN} \) |
| Vertical shear            | \( V_{Ed} = 164.6 \text{ kN} \) |
An 8 mm weld size is therefore considered suitable.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 6
Bolt diameter 24 mm
Bolt grade 8.8
Welds
Tension flange fillet weld 12 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm
Supporting column section 203 x 203 x 60 UKC
Beam / Rafter section 356 x 171 x 57 UKB

Column stiffener details

Compression stiffeners 2/95 mm x 15 mm Grade S 275
connected by 4/8 mm fillet welds.
Tension stiffeners Full depth 10 mm thick.
2/75 mm wide placed midway
between the following bolt rows:
1 and 2

The connection under consideration is two sided.
Location: Ex1 - Based on Joints publication

The calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice one-way extended

Design assumptions:
- all bolts are subject to tension
- shear is evenly distributed between the tension bolts
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible.
- pes is the edge distance to shear bolts if provided.

Factored moment \( M = 410 \text{ kNm} \)
Factored shear force \( V = 60 \text{ kN} \)
Axial force (+ve compression) \( N = 0 \text{ kN} \)

533 x 210 x 92 UB.
Diameter of bolts \( b_d = 24 \text{ mm} \)
Thickness of plate \( t_p = 25 \text{ mm} \)
Total number of tension bolts \( b_n = 6 \)
Distance to first row of bolts \( e_x = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( p_1 = 100 \text{ mm} \)
Pitch of tension bolts \( p_2 = 90 \text{ mm} \)
Edge distance to bottom of flange \( p_{ed} = 36.9 \text{ mm} \)
Edge distance to shear bolts \( p_{es} = 100 \text{ mm} \)
Bolt cross-centres \( g = 100 \text{ mm} \)
Width of end plate \( b_p = 250 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

Top row of bolts \( P_{r1} = 370.98 \text{ kN} \)
Row 2 \( P_{r2} = 395.36 \text{ kN} \)
Row 3 \( P_{r3} = 318.89 \text{ kN} \)
Beam tension flange welds

As an alternative to a single fillet weld of minimum size 15 mm a partial penetration butt weld with a superimposed fillet weld will probably be a more economical solution.

Penetration butt weld size   \( sbt = 8 \text{ mm} \)
Superimposed fillet weld size \( swt = 10 \text{ mm} \)

Tension stress \( \leq pyb \ (275 \text{ N/mm}^2) \), hence OK.
Shear stress \( \leq pyb \ (275 \text{ N/mm}^2) \), hence OK.

Tension stress \( \leq \text{allowable shear stress, hence OK.} \)
Provide partial penetration butt welds (8 mm preparation), with 10 mm superimposed fillet welds.

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses \( \geq \) the web thickness, \( tb \).

Selected fillet weld size   \( sfw = 8 \text{ mm} \)
Length of web weld   \( Lwt = (p1-x-Tb-rb)+(nt/2-2)*p2+1.73*g/2 = 208.2 \text{ mm} \)

Shear zone - assuming a fillet weld of 8 mm continued for the full depth, the capacity of the beam web weld for vertical shear:

Length of weld for shear \( Lws = Db-2*(Tb+rb)-Lwt = 268.3 \text{ mm} \)
Capacity of weld for shear \( Psw = 2*0.7*sfw*pw*Lws/10^3 = 661.09 \text{ kN} \)
Vertical shear \( V = 60 \text{ kN} \)
Web weld size 8 mm is considered to be suitable.

Compression welds

Assuming a bearing fit between the flange and end plate, the use of an 8 mm fillet weld will suffice for the beam compression flange.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 6
Bolt diameter 24 mm
Bolt grade 8.8
Tension Partial penetration butt welds 8 mm
Flange Welds Superimposed fillet welds 10 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm
Beam / Rafter section 533 x 210 x 92 UB
Location: Ex2 - Reversible moment 12 bolts

The calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice two-way extended

Factored moment \( M = 510 \text{ kNm} \)
Factored shear force \( V = 129 \text{ kN} \)
Axial force (+ve compression) \( N = 0 \text{ kN} \)

533 x 210 x 92 UB.
Diameter of bolts \( b_d = 24 \text{ mm} \)
Thickness of plate \( t_p = 25 \text{ mm} \)
Total number of tension bolts \( b_n = 12 \)
Distance to first row of bolts \( e_x = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( p_1 = 100 \text{ mm} \)
Pitch of tension bolts \( p_2 = 90 \text{ mm} \)
Bolt cross-centres \( g = 100 \text{ mm} \)
Width of end plate \( b_p = 250 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts \( P_{r1} = 370.98 \text{ kN} \)
Row 2 \( P_{r2} = 395.36 \text{ kN} \)
Row 3 \( P_{r3} = 318.89 \text{ kN} \)
Beam tension flange welds

As an alternative to a single fillet weld of minimum size 15 mm a partial penetration butt weld with a superimposed fillet weld will probably be a more economical solution.

Penetration butt weld size \( sbt = 8 \, \text{mm} \)

Superimposed fillet weld size \( swt = 10 \, \text{mm} \)

Tension stress \( \leq \) \( pyb \ (275 \, \text{N/mm}^2) \), hence OK.

Shear stress \( \leq \) allowable shear stress, hence OK.

Provide partial penetration butt welds (8 mm preparation), with 10 mm superimposed fillet welds.

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses \( \geq \) the web thickness, \( tb \).

Selected fillet weld size \( sfw = 8 \, \text{mm} \)

Length of web weld

\[
L_{wt} = (p_1 - x - T_b - r_b) + (n/2 - 2) \times p_2 + 1.73 \times g / 2 = 208.2 \, \text{mm}
\]

Shear zone - assuming a fillet weld of 8 mm continued for the full depth, the capacity of the beam web weld for vertical shear:

Length of weld for shear \( L_{ws} = D_b - 2 \times (T_b + r_b) - L_{wt} = 268.3 \, \text{mm} \)

Capacity of weld for shear \( P_{sw} = 2 \times 0.7 \times sfw \times pw \times L_{ws} / 10^3 = 661.09 \, \text{kN} \)

Vertical shear \( V = 129 \, \text{kN} \)

Web weld size 8 mm is considered to be suitable.

Compression welds

As moment is reversible tension and compression surfaces are interchangeable, welding must be the same for both flanges.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 12
Bolt diameter 24 mm
Bolt grade 8.8
Tension Partial penetration butt welds 8 mm
Flange Welds Superimposed fillet welds 10 mm
Web fillet weld 8 mm
Compression Partial penetration butt welds 8 mm
Flange Welds Superimposed fillet welds 10 mm
Beam / Rafter section 533 x 210 x 92 UB
Location: Ex3 - Reversible moment 8 bolts

The calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice two-way extended

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (x-x axis) of beam
- moment is reversible
- moment is carried by half the bolts
- shear is carried by the other half of the bolts.
- the connection consists of two identical beam & end plate arrangements.

Factored moment M=46 kNm
Factored shear force V=60 kN
Axial force (+ve compression) N=0 kN

178 x 102 x 19 UB.
Diameter of bolts bd=20 mm
Thickness of plate tp=20 mm
Total number of tension bolts bn=8
Distance to first row of bolts ex=40 mm
Distance to beam flange x=35 mm
Pitch to second row of bolts pl=80 mm
Bolt cross-centres g=60 mm
Width of end plate bp=120 mm
Assumed web fillet weld size sww=6 mm
Assumed flange fillet weld size swf=6 mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts Pr1=207.41 kN
Row 2 Pr2=289.1 kN
Slip factor for preloaded bolts $\mu = 0.5$
Coefficient for type of hole $K_s = 1$

**Beam tension flange welds**

Selected fillet weld size $\text{swt} = 8$ mm

**Beam web welds**

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses $\geq$ the web thickness, $t_b$.
Selected fillet weld size $s_{fw} = 8$ mm
Length of web weld $L_{wt} = (p_1 - x - T_b - r_b) + 1.73^*g/2 = 81.4$ mm

Shear zone - assuming a fillet weld of 8 mm continued for the full depth, the capacity of the beam web weld for vertical shear:
Length of weld for shear $L_{ws} = D_b - 2*(T_b + r_b) - L_{wt} = 65.4$ mm
Capacity of weld for shear $P_{sw} = 2*0.7*s_{fw}*p_w*L_{ws}/10^3 = 161.15$ kN
Vertical shear $V = 60$ kN
Web weld size 8 mm is considered to be suitable.

**Compression welds**

As moment is reversible tension and compression surfaces are interchangeable, welding must be the same for both flanges.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 8
Bolt diameter 20 mm
Bolt grade HSFG
General grade (8.8) to BS4395 Part 1. Nuts grade 10.
Welds Tension flange fillet weld 8 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm
Beam / Rafter section 178 x 102 x 19 UB
Location: Ex4 - Larger beam no shear bolts

The calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice one-way extended

Design assumptions:
- all bolts are subject to tension
- shear is evenly distributed between the tension bolts
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible.
- pes is the edge distance to shear bolts if provided.

Factored moment \( M = 772 \text{ kNm} \)
Factored shear force \( V = 238 \text{ kN} \)
Axial force (+ve compression) \( N = 0 \text{ kN} \)

686 x 254 x 152 UB.
Diameter of bolts \( bd = 24 \text{ mm} \)
Thickness of plate \( tp = 25 \text{ mm} \)
Total number of tension bolts \( bn = 12 \)
Distance to first row of bolts \( ex = 50 \text{ mm} \)
Distance to beam flange \( x = 40 \text{ mm} \)
Pitch to second row of bolts \( p1 = 100 \text{ mm} \)
Pitch of tension bolts \( p2 = 90 \text{ mm} \)
Edge distance to bottom of flange \( ped = 42.5 \text{ mm} \)
Bolt cross-centres \( g = 140 \text{ mm} \)
Width of end plate \( bp = 230 \text{ mm} \)
Assumed web fillet weld size \( sww = 10 \text{ mm} \)
Assumed flange fillet weld size \( swf = 15 \text{ mm} \)

SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Row</th>
<th>Potential Resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>370.81</td>
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<tr>
<td>Row 2</td>
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<td>222.35</td>
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<tr>
<td>Row 6</td>
<td>164.68</td>
</tr>
</tbody>
</table>
Beam tension flange welds

As an alternative to a single fillet weld of minimum size 16 mm a partial penetration butt weld with a superimposed fillet weld will probably be a more economical solution.

Penetration butt weld size \( sbt = 9 \text{ mm} \)
Superimposed fillet weld size \( swt = 11 \text{ mm} \)
Tension stress \( \leq pyb \ (265 \text{ N/mm}^2) \), hence OK.
Tension stress \( \leq pyb \ (265 \text{ N/mm}^2) \), hence OK.
Shear stress \( \leq \text{allowable shear stress} \), hence OK.

Provide partial penetration butt welds (9 mm preparation), with 11 mm superimposed fillet welds.

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses \( \geq \) the web thickness, \( tb \).
Selected fillet weld size \( sfw = 10 \text{ mm} \)
Length of web weld \( Lwt = (p1-x-Tb-rb)+(nt/2-2)*p2+1.73*g/2 = 504.9 \text{ mm} \)

Shear zone - assuming a fillet weld of 10 mm continued for the full depth, the capacity of the beam web weld for vertical shear:
Length of weld for shear \( Lws = Db-2*(Tb+rb)-Lwt = 110.2 \text{ mm} \)
Capacity of weld for shear \( Psw = 2*0.7*sfw*pw*Lws/10^3 = 339.42 \text{ kN} \)
Vertical shear \( V = 238 \text{ kN} \)
Web weld size 10 mm is considered to be suitable.

Compression welds

Assuming a bearing fit between the flange and end plate, the use of an 8 mm fillet weld will suffice for the beam compression flange.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 12
Bolt diameter 24 mm
Bolt grade 8.8
Tension Partial penetration butt welds 9 mm
Flange Welds Superimposed fillet welds 11 mm
Web fillet weld 10 mm
Compression flange fillet weld 8 mm
Beam / Rafter section 686 x 254 x 152 UB
**Bolted end plate beam splice one-way extended**

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

The bolted end plate beam splice is one-way extended.

Design assumptions:
- all bolts are in tension except of the bottom two which will resist all the applied shear load
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible
- p1' is the distance between tension and shear bolts.

Factored moment \( M_{Ed} = 410 \) kNm
Factored shear force \( V_{Ed} = 60 \) kN
Axial force (+ve compression) \( N_{Ed} = 0 \) kN

**Supported beam details**

533 x 210 x 92 UKB

**Bolt details**

Diameter of bolts \( d_b = 24 \) mm
Tensile resistance (EC3 Part 1-8) \( F_{Rd} = k_2 \cdot f_{ub} \cdot A_t / (\gamma \cdot M_2 \cdot 10^3) = 203.33 \) kN

**End plate details**

Thickness of plate \( t_p = 25 \) mm
Distance to first row of bolts \( e_{x} = 50 \) mm
Distance to beam flange \( x = 40 \) mm
Pitch to second row of bolts \( p_2 = 100 \) mm
Pitch of tension bolts \( p_1 = 90 \) mm
Edge distance to bottom of flange \( x_2 = 36.9 \) mm
Edge distance to shear bolts \( e_{1'} = 100 \) mm
Bolt cross-centres \( p_3 = 100 \) mm
Width of end plate \( b_p = 250 \) mm
Assumed web fillet weld size \( s_{ww} = 8 \) mm
Assumed flange fillet weld size \( s_{wf} = 12 \) mm
SUMMARY OF BOLT POTENTIAL RESISTANCES

| Top row of bolts | Pr1=377.26 kN |
| Row 2            | Pr2=406.66 kN |
| Row 3            | Pr3=328 kN    |

Beam tension flange welds

Weld design strength \( f_{vwd} = \frac{f_u}{\sqrt{3}} \frac{1}{\beta_w \gamma_{M2}} \) = 222.79 N/mm²

The force in the tension flange welds is the lesser of:

- Tension resistance of the flange \( T_f = b_b \cdot t_f b \cdot f_y b / 10^3 = 897.9 \text{ kN} \)
- Tension force in top 3 bolt rows \( T_{fc} = P_{r1} + P_{r2} + P_{r3} = 1111.9 \text{ kN} \)

Weld size required \( s_{wr} = F_{wEdf} \cdot 10^3 / (0.7 \cdot f_{vwd}) = 14.094 \text{ mm} \)

Penetration butt weld size \( s_{bt} = 8 \text{ mm} \)

Superimposed fillet weld size \( s_{wt} = 12 \text{ mm} \)

Tension stress \( \leq f_y b \) (275 N/mm²), hence OK.

Shear stress \( \leq f_y b \) (275 N/mm²), hence OK.

Provide partial penetration butt welds (8 mm preparation), with 12 mm superimposed fillet welds.

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, \( t_{wb} \).

Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)

Shear zone - assume an 8 mm FW continued for the full depth.

Length of weld for shear \( L_{ws} = h_b - 2 \cdot (t_f b + r_b) - L_w t = 268.3 \text{ mm} \)

Resistance of weld for shear \( F_{wRds} = 2 \cdot 0.7 \cdot s_{fw} \cdot f_{vwd} \cdot L_w s / 10^3 = 669.47 \text{ kN} \)

Vertical shear \( V_{Ed} = 60 \text{ kN} \)

An 8 mm fillet weld size is therefore considered suitable.

Compression welds

Assuming a bearing fit between the flange and end plate, the use of an 8 mm fillet weld will suffice for the beam compression flange.
CONNECTION SUMMARY - all dimensions in diagram are in mm

<table>
<thead>
<tr>
<th>ex</th>
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</thead>
<tbody>
<tr>
<td>p2</td>
<td>40</td>
<td>50</td>
</tr>
<tr>
<td>p1</td>
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<td>100</td>
</tr>
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<td>p1'</td>
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<td>320</td>
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<tr>
<td>e1'</td>
<td></td>
<td>100</td>
</tr>
</tbody>
</table>

Number of bolts 8
Bolt diameter 24 mm
Bolt grade 8.8
Tension Partial penetration butt welds 8 mm
Flange Welds Superimposed fillet welds 12 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm
Beam / Rafter section 533 x 210 x 92 UKB
Location: Ex2 - Reversible moment 12 bolts

Bolted end plate beam splice one-way extended

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The bolted end plate beam splice is two-way extended.

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (y-y axis) of beam
- moment is reversible
- moment is carried by half the bolts
- shear is carried by the other half of the bolts
- the connection consists of two identical beam and end plate arrangements
- p1' is the distance between tension and shear bolts.

Factored moment MyEd=510 kNm
Factored shear force VEd=129 kN
Axial force (+ve compression) NEd=0 kN

Supported beam details
533 x 210 x 92 UKB

Bolt details
Diameter of bolts db=24 mm
Tensile resistance (EC3 Part 1-8) FtRd=k2*fub*At/(gamM2*10^3)=203.33 kN

End plate details
Thickness of plate tp=25 mm
Distance to first row of bolts ex=50 mm
Distance to beam flange x=40 mm
Pitch to second row of bolts p2=100 mm
Pitch of tension bolts p1=90 mm
Bolt cross-centres p3=100 mm
Width of end plate bp=250 mm
Assumed web fillet weld size sww=8 mm
Assumed flange fillet weld size swf=12 mm
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts \( Pr_1 = 377.26 \text{kN} \)
Row 2 \( Pr_2 = 406.66 \text{kN} \)
Row 3 \( Pr_3 = 328 \text{kN} \)

Beam tension flange welds

Weld design strength \( fvwd = \frac{fu}{\sqrt{3}}(\beta_{aw} \cdot \gamma_{M2}) \)
= \( 222.79 \text{N/mm}^2 \)

The force in the tension flange welds is the lesser of:
Tension resistance of the flange \( Tf = bb \cdot tfb \cdot fyb / 10^3 = 897.9 \text{kN} \)
Tension force in top 3 bolt rows \( Tfc = Pr_1 + Pr_2 + Pr_3 = 1111.9 \text{kN} \)
Weld size required \( swfr = FwEdf \cdot 10^3 / (0.7 \cdot fvwd) = 14.094 \text{mm} \)
Penetration butt weld size \( sbt = 8 \text{mm} \)
Superimposed fillet weld size \( swt = 12 \text{mm} \)

Penetration butt weld size \( sbt = 8 \text{mm} \)
Shear stress \( \leq f_yb \ (275 \text{N/mm}^2) \), hence OK.
Shear stress \( \leq f_yb \ (275 \text{N/mm}^2) \), hence OK.

Provide partial penetration butt welds (8 mm preparation), with
12 mm superimposed fillet welds.

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses \( \geq \) the web thickness, \( twb \).
Selected fillet weld size \( sfw = 8 \text{mm} \)

Shear zone - assume an 8 mm FW continued for the full depth.
Length of weld for shear \( Lws = hb - 2 \cdot (tfb + rb) - Lwt = 268.3 \text{mm} \)
Resistance of weld for shear \( FwRds = 2 \cdot 0.7 \cdot sfw \cdot fvwd \cdot Lws / 10^3 \)
= \( 669.47 \text{kN} \)

Vertical shear \( VEd = 129 \text{kN} \)
An 8 mm fillet weld size is therefore considered suitable.

Compression welds

As moment is reversible tension and compression surfaces are interchangeable, welding must be the same for both flanges.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 12
Bolt diameter 24 mm
Bolt grade 8.8
Tension Partial penetration butt welds 8 mm
Flange Welds Superimposed fillet welds 12 mm
Web fillet weld 8 mm
Compression Partial penetration butt welds 8 mm
Flange Welds Superimposed fillet welds 12 mm
Beam / Rafter section 533 x 210 x 92 UKB
Location: Ex3 - Reversible moment 8 bolts

Bolted end plate beam splice one-way extended

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

The bolted end plate beam splice is two-way extended.

Design assumptions:
• the bolt arrangement is symmetrical about the centre line (y-y axis) of beam
• moment is reversible
• moment is carried by half the bolts
• shear is carried by the other half of the bolts
• the connection consists of two identical beam and end plate arrangements
• p1' is the distance between tension and shear bolts.

Factored moment MyEd=46 kNm
Factored shear force VEd=60 kN
Axial force (+ve compression) NEd=0 kN

Supported beam details
178 x 102 x 19 UKB

Bolt details
Diameter of bolts db=20 mm
Tensile resistance (EC3 Part 1-8) FtRd=k2*fub*At/(gamM2*10^3)=141.12 kN

End plate details
Thickness of plate tp=20 mm
Distance to first row of bolts ex=40 mm
Distance to beam flange x=35 mm
Pitch to second row of bolts p2=80 mm
Bolt cross-centres p3=60 mm
Width of end plate bp=120 mm
Assumed web fillet weld size sww=6 mm
Assumed flange fillet weld size swf=6 mm
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts  Pr1=203.6 kN
Row 2  Pr2=282.24 kN

Beam tension flange welds

Weld design strength  \( f_{\text{vwd}} = \frac{f_u}{\sqrt{3}} \times \left( \frac{\gamma_{M2}}{\beta_{\omega}} \right) \)
=222.79 N/mm²

The force in the tension flange welds is the lesser of:
Tension resistance of the flange  \( T_f = b_b \times t_{fb} \times f_{yb} / 10^3 = 219.86 \text{ kN} \)
Tension force in top 3 bolt rows  \( T_{fc} = Pr1 + Pr2 + Pr3 = 274.82 \text{ kN} \)
Weld size required  \( s_{wfr} = F_{wEd} \times 10^3 / (0.7 \times f_{vwd}) = 7.1345 \text{ mm} \)
Selected fillet weld size  \( s_{wt} = 8 \text{ mm} \)

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, \( t_{wb} \).
Selected fillet weld size  \( s_{fw} = 8 \text{ mm} \)

Shear zone - assume an 8 mm FW continued for the full depth.
Length of weld for shear  \( L_{ws} = h_b - 2 \times (t_{fb} + r_b) - L_{wt} = 65.4 \text{ mm} \)
Resistance of weld for shear  \( F_{wRds} = 2 \times 0.7 \times s_{fw} \times f_{vwd} \times L_{ws} / 10^3 = 163.19 \text{ kN} \)
Vertical shear  \( V_{Ed} = 60 \text{ kN} \)
An 8 mm fillet weld size is therefore considered suitable.

Compression welds

As moment is reversible tension and compression surfaces are interchangeable, welding must be the same for both flanges.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts: 8
Bolt diameter: 20 mm
Bolt grade: Preloaded HSFG bolts. General grade (8.8) with grade 10 nuts.
Welds: Tension flange fillet weld 8 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm
Beam / Rafter section: 178 x 102 x 19 UKB
Location: Ex4 - Larger beam with shear bolts

Bolted end plate beam splice one-way extended

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

The bolted end plate beam splice is one-way extended.

Design assumptions:
- all bolts are in tension except of the bottom two which will resist all the applied shear load
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible
- p1' is the distance between tension and shear bolts.

Factored moment
Factored shear force
Axial force (+ve compression)

Supported beam details

686 x 254 x 152 UKB

Bolt details

Diameter of bolts
Tensile resistance (EC3 Part 1-8)

End plate details

Thickness of plate
Distance to first row of bolts
Distance to beam flange
Pitch to second row of bolts
Pitch of tension bolts
Edge distance to bottom of flange
Edge distance to shear bolts
Bolt cross-centres
Width of end plate
Assumed web fillet weld size
Assumed flange fillet weld size

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SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts  Pr1=377.09 kN
Row 2        Pr2=406.66 kN
Row 3        Pr3=314.29 kN
Row 4        Pr4=256.5 kN
Row 5        Pr5=228.7 kN
Row 6        Pr6=169.39 kN

Beam tension flange welds

Weld design strength  \( f_vwd = \frac{fu}{\sqrt{3}/(\beta_{aw} \cdot \gamma_{M2})} \)
=222.79 N/mm²

The force in the tension flange welds is the lesser of:
Tension resistance of the flange  \( T_f = b_p \cdot t_{fb} \cdot f_{yb}/10^3 = 1280 \text{ kN} \)
Tension force in top 3 bolt rows  \( T_{fc} = P_{r1} + P_{r2} + P_{r3} = 1098 \text{ kN} \)
Weld size required  \( s_{fr} = F_{wEdf} \cdot 10^3/(0.7 \cdot f_{vwd}) = 15.758 \text{ mm} \)
Penetration butt weld size  \( s_{bt} = 9 \text{ mm} \)
Superimposed fillet weld size  \( s_{wt} = 12 \text{ mm} \)
Tension stress \( \leq f_{yb} \ (265 \text{ N/mm}^2) \), hence OK.
Tension stress \( \leq f_{yb} \ (265 \text{ N/mm}^2) \), hence OK.
Shear stress \( \leq \) allowable shear stress, hence OK.

Provide partial penetration butt welds (9 mm preparation), with
12 mm superimposed fillet welds.

Beam web welds

Tension zone - a full strength weld is recommended in the web tension
zone with the sum of the throat thicknesses \( \geq \) the web thickness, \( t_{wb} \).
Selected fillet weld size  \( s_{fw} = 10 \text{ mm} \)

Shear zone - assume an 10 mm FW continued for the full depth.
Length of weld for shear  \( L_{ws} = h_b - 2 \cdot (t_{fb} + r_b) - L_{wt} = 110.2 \text{ mm} \)
Resistance of weld for shear  \( F_{wRds} = 2 \cdot 0.7 \cdot s_{fw} \cdot f_{vwd} \cdot L_{ws}/10^3 \)
=343.72 kN
Vertical shear  \( V_{Ed} = 215 \text{ kN} \)
An 10 mm fillet weld size is therefore considered suitable.

Compression welds

Assuming a bearing fit between the flange and end plate, the use of
an 8 mm fillet weld will suffice for the beam compression flange.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 14
Bolt diameter 24 mm
Bolt grade 8.8
Tension Partial penetration butt welds 9 mm
Flange Welds Superimposed fillet welds 12 mm
Web fillet weld 10 mm
Compression flange fillet weld 8 mm
Beam / Rafter section 686 x 254 x 152 UKB
Flush beam splice

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice one-way

Design assumptions:
- All bolts are subject to tension
- Shear is evenly distributed between the bolts end plate arrangements
- The connection consists of two identical beam and end plate arrangements
- Moment is not reversible
- Pes is the edge distance to shear bolts if provided.

Factored moment $ M = 405 \text{ kNm} $
Factored shear force $ V = 60 \text{ kN} $
Axial force (+ve compression) $ N = 0 \text{ kN} $

Supported beam details

533 x 210 x 92 UB.

Bolt details

Diameter of bolts $ b_d = 24 \text{ mm} $

End plate details

Thickness of plate $ t_p = 25 \text{ mm} $
Total number of bolts $ b_n = 8 $
Distance to first row of bolts $ e_x = 75 \text{ mm} $
Distance to beam top flange $ x = 15 \text{ mm} $
Pitch of tension bolts $ p_2 = 90 \text{ mm} $
Edge distance to bottom of flange $ p_{ed} = 36.9 \text{ mm} $
Bolt cross-centres $ g = 100 \text{ mm} $
Width of end plate $ b_p = 250 \text{ mm} $
Assumed web fillet weld size $ s_{ww} = 8 \text{ mm} $
Assumed flange fillet weld size $ s_{wf} = 12 \text{ mm} $
SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Top row of bolts</th>
<th>Pr1=395.36 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Row 2</td>
<td>Pr2=318.89 kN</td>
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<tr>
<td>Row 3</td>
<td>Pr3=242.42 kN</td>
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<tr>
<td>Row 4</td>
<td>Pr4=165.94 kN</td>
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WELD DESIGN

Beam tension flange welds

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<thead>
<tr>
<th>Weld design strength</th>
<th>pw=220 N/mm²</th>
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<tbody>
<tr>
<td>Weld size required</td>
<td>swfr=Fw<em>10^3/(0.7</em>pw)=11.354 mm</td>
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<tr>
<td>Selected fillet weld size</td>
<td>swt=12 mm</td>
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</table>

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, tb.

Selected fillet weld size sfw=8 mm

Shear zone - assuming a fillet weld of 8 mm continued for the full depth, the capacity of the beam web weld for vertical shear:

Length of weld for shear Lws=Db-2*(Tb+rb)-Lwt=88.3 mm

Capacity of weld for shear Psw=2*0.7*sfw*pw*Lws/10^3=217.57 kN

Vertical shear V=60 kN

Web weld size 8 mm is considered to be suitable.

Compression welds

Assuming a bearing fit between the flange and end plate, the use of an 8 mm fillet weld will suffice for the beam compression flange.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts: 8
Bolt diameter: 24 mm
Bolt grade: 8.8
Welds:
- Tension flange fillet weld: 12 mm
- Web fillet weld: 8 mm
- Compression flange fillet weld: 8 mm
Beam / Rafter section: 533 x 210 x 92 UB
Flush beam splice

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

**Bolted end plate beam splice one-way**

Design assumptions:
- all bolts are subject to tension
- shear is evenly distributed between the bolts end plate arrangements
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible
- pes is the edge distance to shear bolts if provided.

Factored moment \( M = 610 \text{ kNm} \)
Factored shear force \( V = 60 \text{ kN} \)
Axial force (+ve compression) \( N = 0 \text{ kN} \)

**Supported beam details**

686 x 254 x 125 UB.

**Bolt details**

Diameter of bolts \( bd = 24 \text{ mm} \)

**End plate details**

Thickness of plate \( tp = 25 \text{ mm} \)
Total number of bolts \( bn = 12 \)
Distance to first row of bolts \( ex = 95 \text{ mm} \)
Distance to beam top flange \( x = 40 \text{ mm} \)
Pitch of tension bolts \( p2 = 90 \text{ mm} \)
Edge distance to bottom of flange \( ped = 42.9 \text{ mm} \)
Edge distance to shear bolts \( pes = 100 \text{ mm} \)
Bolt cross-centres \( g = 100 \text{ mm} \)
Width of end plate \( bp = 260 \text{ mm} \)
Assumed web fillet weld size \( sww = 8 \text{ mm} \)
Assumed flange fillet weld size \( swf = 12 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Row</th>
<th>Pr</th>
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<tr>
<td>Top row of bolts</td>
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<tr>
<td>Row 2</td>
<td>Pr2=337.48 kN</td>
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<td>Row 3</td>
<td>Pr3=279.61 kN</td>
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<td>Row 4</td>
<td>Pr4=221.73 kN</td>
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<td>Pr5=163.85 kN</td>
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<td>Row 6</td>
<td>Pr6=105.98 kN</td>
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</tbody>
</table>

WELD DESIGN

Beam tension flange welds

Weld design strength \( pw=220 \text{ N/mm}^2 \)
Weld size required \( swfr=Fw*10^3/(0.7*pw)=9.6272 \text{ mm} \)
Selected fillet weld size \( swt=10 \text{ mm} \)

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, \( tb \).
Selected fillet weld size \( sfw=9 \text{ mm} \)

Shear zone - assuming a fillet weld of 9 mm continued for the full depth, the capacity of the beam web weld for vertical shear:
Length of weld for shear \( Lws=Db-2*(Tb+rb)-Lwt=55 \text{ mm} \)
Capacity of weld for shear \( Psw=2*0.7*sfw*pw*Lws/10^3=152.46 \text{ kN} \)
Vertical shear \( V=60 \text{ kN} \)
Web weld size 9 mm is considered to be suitable.

Compression welds

Assuming a bearing fit between the flange and end plate, the use of an 8 mm fillet weld will suffice for the beam compression flange.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 12
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 10 mm
- Web fillet weld 9 mm
- Compression flange fillet weld 8 mm
Beam / Rafter section 686 x 254 x 125 UB
Flush beam splice

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice two-way

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (x-x axis) of the beam
- moment is reversible
- moment is carried by half of the bolts
- shear is carried by the other half of the bolts
- the connection consists of two identical beam and end plate arrangements.

Factored moment $M=280$ kNm
Factored shear force $V=238$ kN
Axial force (+ve compression) $N=0$ kN

Supported beam details

533 x 210 x 92 UB.

Bolt details

Diameter of bolts $bd=24$ mm

End plate details

Thickness of plate $tp=25$ mm
Total number of bolts $bn=8$
Distance to first row of bolts $ex=90$ mm
Edge distance $x=36$ mm
Pitch of tension bolts $p2=90$ mm
Bolt cross-centres $g=140$ mm
Width of end plate $bp=220$ mm
Assumed web fillet weld size $sww=8$ mm
Assumed flange fillet weld size $swf=12$ mm
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts                  Pr1=395.36 kN
Row 2                             Pr2=267.79 kN

WELD DESIGN

Beam tension flange welds

Weld design strength              pw=220 N/mm²
Weld size required                swfr=Fw*10^3/(0.7*pw)=10.541 mm
Selected fillet weld size         swt=12 mm

Beam web welds

Tension zone - a full strength weld is recommended in the web tension
zone with the sum of the throat thicknesses ≥ the web thickness, tb.
Selected fillet weld size          sfw=8 mm

Shear zone - assuming a fillet weld of 8 mm continued for the full
depth, the capacity of the beam web weld for vertical shear:
Length of weld for shear           Lws=Db-2*(Tb+rb)-Lwt=239.7 mm
Capacity of weld for shear         Psw=2*0.7*sfw*pw*Lws/10^3=590.62 kN
Vertical shear                     V=238 kN
Web weld size 8 mm is considered to be suitable.

Compression welds

As moment is reversible tension and compression surfaces are
interchangeable, welding must be the same for both flanges.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 8
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 12 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 12 mm
Beam / Rafter section 533 x 210 x 92 UB
Flush beam splice

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice two-way

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (x-x axis) of the beam
- moment is reversible
- moment is carried by half of the bolts
- shear is carried by the other half of the bolts
- the connection consists of two identical beam and end plate arrangements.

Factored moment \( M = 35 \text{ kNm} \)
Factored shear force \( V = 129 \text{ kN} \)
Axial force (+ve compression) \( N = 0 \text{ kN} \)

Supported beam details

178 x 102 x 19 UB.

Bolt details

Diameter of bolts \( \text{bd} = 24 \text{ mm} \)

End plate details

Thickness of plate \( \text{tp} = 25 \text{ mm} \)
Total number of bolts \( \text{bn} = 4 \)
Distance to first row of bolts \( \text{ex} = 80 \text{ mm} \)
Edge distance \( \text{x} = 30 \text{ mm} \)
Bolt cross-centres \( \text{g} = 60 \text{ mm} \)
Width of end plate \( \text{bp} = 120 \text{ mm} \)
Assumed web fillet weld size \( \text{sww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( \text{swf} = 8 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts  Pr1=395.36 kN

WELD DESIGN
Beam tension flange welds

Weld design strength  pw=220 N/mm²
Weld size required  swfr=Fw*10^3/(0.7*pw)=7.2249 mm
Selected fillet weld size  swt=8 mm

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, tb.
Selected fillet weld size  sfw=8 mm

Shear zone - assuming a fillet weld of 8 mm continued for the full depth, the capacity of the beam web weld for vertical shear:
Length of weld for shear  Lws=Db-2*(Tb+rb)-Lwt=60.4 mm
Capacity of weld for shear  Psw=2*0.7*sfw*pw*Lws/10^3=148.83 kN
Vertical shear  V=129 kN
Web weld size 8 mm is considered to be suitable.

Compression welds

As moment is reversible tension and compression surfaces are interchangeable, welding must be the same for both flanges.
connection summary - all dimensions in diagram are in mm

Number of bolts 4
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 8 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Beam / Rafter section 178 x 102 x 19 UB
Flush beam splice

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice one-way (non-reversible moment)

Design assumptions:
- all bolts are in tension except of the bottom two which will resist all the applied shear force
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible.

Factored moment \( \text{MyEd}=405 \text{ kNm} \)
Factored shear force \( \text{VEd}=60 \text{ kN} \)
Axial force (+ve compression) \( \text{NEd}=0 \text{ kN} \)

Supported beam details

533 x 210 x 92 UKB

Bolt details

Diameter of bolts \( \text{db}=24 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( \text{FtRd}=k2*fub*At/(\gammaM2*10^3)=203.33 \text{ kN} \)

End plate details

Thickness of plate \( \text{tp}=25 \text{ mm} \)
Distance to first row of bolts \( \text{ex}=75 \text{ mm} \)
Distance to beam top flange \( \text{x}=15 \text{ mm} \)
Pitch of tension bolts \( \text{pl}=90 \text{ mm} \)
Edge distance to bottom of flange \( \text{x2}=36.9 \text{ mm} \)
Edge distance to shear bolts \( \text{el}'=100 \text{ mm} \)
Bolt cross-centres \( \text{p3}=100 \text{ mm} \)
Width of end plate \( \text{bp}=250 \text{ mm} \)
Assumed web fillet weld size \( \text{sww}=8 \text{ mm} \)
Assumed flange fillet weld size \( \text{swf}=12 \text{ mm} \)
## SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Row</th>
<th>Potential Resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top row of bolts</td>
<td>Pr1=406.66</td>
</tr>
<tr>
<td>Row 2</td>
<td>Pr2=328</td>
</tr>
<tr>
<td>Row 3</td>
<td>Pr3=249.34</td>
</tr>
<tr>
<td>Row 4</td>
<td>Pr4=170.69</td>
</tr>
</tbody>
</table>

### Beam tension flange welds

- **Weld design strength**
  \[ fv_{wd} = \frac{f_u}{\sqrt{3}} \left( \frac{1}{\beta_{aw} \cdot \gamma_{M2}} \right) \]
  \[ = 222.79 \text{ N/mm}^2 \]

- **Weld size required**
  \[ sw_{fr} = \frac{F_{wEdf} \cdot 10^3}{0.7 \cdot fv_{wd}} = 11.532 \text{ mm} \]

- **Selected fillet weld size**
  \[ sw_{t} = 12 \text{ mm} \]

### Beam web welds

- **Tension zone** - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, \( t_{wb} \).
- **Selected fillet weld size**
  \[ sw_{f} = 8 \text{ mm} \]
- **Length of web weld**
  \[ L_{wt} = (ex - x - t_{fb} - rb) + (t_{row} - 1) \cdot p_{1} + \frac{1.73 \cdot p_{3}}{2} = 388.2 \text{ mm} \]

- **Shear zone** - assume an 8 mm FW continued for the full depth.
- **Length of weld for shear**
  \[ L_{ws} = hb - 2 \cdot (t_{fb} + rb) - L_{wt} = 88.3 \text{ mm} \]
- **Resistance of weld for shear**
  \[ F_{wRds} = 2 \cdot 0.7 \cdot sw_{f} \cdot fv_{wd} \cdot L_{ws} / 10^3 = 220.33 \text{ kN} \]

### Compression welds

Assuming a bearing fit between the flange and end plate, the use of an 8 mm fillet weld will suffice for the beam compression flange.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 10
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 12 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Beam / Rafter section 533 x 210 x 92 UKB
Flush beam splice

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice one-way (non-reversible moment)

Design assumptions:
- all bolts are in tension except of the bottom two which will resist all the applied shear force
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible.

Factored moment \( M_{Ed} = 610 \text{ kNm} \)
Factored shear force \( V_{Ed} = 60 \text{ kN} \)
Axial force (+ve compression) \( N_{Ed} = 0 \text{ kN} \)

Supported beam details

686 x 254 x 125 UKB

Bolt details

Diameter of bolts \( d_b = 24 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( F_{Rd} = k_2 * f_{ub} * A_t / (\gamma_M 2 * 10^3) = 203.33 \text{ kN} \)

End plate details

Thickness of plate \( t_p = 25 \text{ mm} \)
Distance to first row of bolts \( e_x = 95 \text{ mm} \)
Distance to beam top flange \( x = 40 \text{ mm} \)
Pitch of tension bolts \( p_1 = 90 \text{ mm} \)
Edge distance to bottom of flange \( x_2 = 42.9 \text{ mm} \)
Edge distance to shear bolts \( e_{1'} = 100 \text{ mm} \)
Bolt cross-centres \( p_3 = 100 \text{ mm} \)
Width of end plate \( b_p = 260 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Row</th>
<th>Pr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top row of bolts</td>
<td>Pr1=406.66 kN</td>
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<tr>
<td>Row 2</td>
<td>Pr2=347.13 kN</td>
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<td>Row 3</td>
<td>Pr3=287.6 kN</td>
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<td>Row 4</td>
<td>Pr4=228.07 kN</td>
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<tr>
<td>Row 5</td>
<td>Pr5=168.54 kN</td>
</tr>
<tr>
<td>Row 6</td>
<td>Pr6=109.01 kN</td>
</tr>
</tbody>
</table>

Beam tension flange welds

Weld design strength \( \frac{f_{wd}}{\sqrt{3}} / (\beta_{aw} \gamma_{M2}) \) = 222.79 N/mm²
Weld size required \( s_{fr} = \frac{F_{wEdf} \times 10^3}{(0.7 \times f_{wd})} = 9.7783 \text{ mm} \)
Selected fillet weld size \( s_{wt} = 10 \text{ mm} \)

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, \( t_{wb} \).
Selected fillet weld size \( s_{fw} = 10 \text{ mm} \)
Length of web weld \( L_{wt} = (ex-x-t_{fb}-rb)+(t_{row}-1) \times p_{1}+1.73 \times p_{3}/2 = 560.1 \text{ mm} \)
Shear zone - assume an 10 mm FW continued for the full depth.
Length of weld for shear \( L_{ws} = hb-2 \times (t_{fb}+rb)-L_{wt} = 55 \text{ mm} \)
Resistance of weld for shear \( F_{wRds} = 2 \times 0.7 \times s_{fw}^{*} f_{wd}^{*} L_{ws}/10^3 = 171.55 \text{ kN} \)
Vertical shear \( V_{Ed} = 60 \text{ kN} \)
An 10 mm fillet weld size is therefore considered suitable.

Compression welds

Assuming a bearing fit between the flange and end plate, the use of an 8 mm fillet weld will suffice for the beam compression flange.
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts: 14
Bolt diameter: 24 mm
Bolt grade: 8.8
Welds:
- Tension flange fillet weld: 10 mm
- Web fillet weld: 10 mm
- Compression flange fillet weld: 8 mm

Beam / Rafter section: 686 x 254 x 125 UKB
Flush beam splice

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice two-way (reversible moment)

Factored moment \( M_{Ed} = 280 \text{ kNm} \)
Factored shear force \( V_{Ed} = 238 \text{ kN} \)
Axial force (+ve compression) \( N_{Ed} = 0 \text{ kN} \)

Supported beam details

533 x 210 x 92 UKB

Bolt details

Diameter of bolts \( d_b = 24 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( F_{T,Rd} = k_2 f_{ub} A_t / (\gamma_M 10^3) = 203.33 \text{ kN} \)

End plate details

Thickness of plate \( t_p = 25 \text{ mm} \)
Total number of bolts \( b_n = 8 \)
Distance to first row of bolts \( e_x = 90 \text{ mm} \)
Edge distance \( x = 36 \text{ mm} \)
Pitch of tension bolts \( p_l = 90 \text{ mm} \)
Bolt cross-centres \( p_3 = 140 \text{ mm} \)
Width of end plate \( b_p = 220 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts  
Pr1=406.66 kN

Row 2  
Pr2=265.66 kN

Beam tension flange welds

Weld design strength  
fvwd=fu/SQR(3)/(betaw*gamM2)  
=222.79 N/mm²

Weld size required  
swfr=FwEdf*10^3/(0.7*fvwd)=10.553 mm

Selected fillet weld size  
swt=12 mm

Beam web welds

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, twb.

Selected fillet weld size  
sfw=8 mm

Length of web weld  
Lwt=(ex-x-tfb-rb)+(trow-1)*p1+1.73*p3/2=236.8 mm

Shear zone - assume an 8 mm FW continued for the full depth.

Length of weld for shear  
Lws=hb-2*(tfb+rb)-Lwt=239.7 mm

Resistance of weld for shear  
FwRds=2*0.7*sfw*fvwd*Lws/10^3  
=598.11 kN

Vertical shear  
VED=238 kN

An 8 mm fillet weld size is therefore considered suitable.

Compression welds

As moment is reversible tension and compression surfaces are interchangeable, welding must be the same for both flanges.
**CONNECTION SUMMARY** - all dimensions in diagram are in mm

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>T</td>
<td>25</td>
</tr>
<tr>
<td>ex</td>
<td>90</td>
</tr>
<tr>
<td>p1</td>
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<td>245</td>
</tr>
<tr>
<td>p1</td>
<td>90</td>
</tr>
<tr>
<td>ex</td>
<td>90</td>
</tr>
<tr>
<td>1</td>
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</tr>
<tr>
<td></td>
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<td>140</td>
</tr>
<tr>
<td></td>
<td>140</td>
</tr>
<tr>
<td>Number of bolts</td>
<td>8</td>
</tr>
<tr>
<td>Bolt diameter</td>
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</tr>
<tr>
<td>Bolt grade</td>
<td>8.8</td>
</tr>
<tr>
<td>Welds</td>
<td></td>
</tr>
<tr>
<td>Tension flange fillet weld</td>
<td>12 mm</td>
</tr>
<tr>
<td>Web fillet weld</td>
<td>8 mm</td>
</tr>
<tr>
<td>Compression flange fillet weld</td>
<td>12 mm</td>
</tr>
<tr>
<td>Beam / Rafter section</td>
<td>533 x 210 x 92 UKB</td>
</tr>
</tbody>
</table>
Flush beam splice

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Bolted end plate beam splice two-way (reversible moment)

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (y-y axis) of the beam
- moment is reversible
- moment is carried by half of the bolts
- shear is carried by the other half of the bolts
- the connection consists of two identical beam & end plate arrangements.

Factored moment \( M_{Ed}=33 \text{ kNm} \)
Factored shear force \( V_{Ed}=129 \text{ kN} \)
Axial force (+ve compression) \( N_{Ed}=0 \text{ kN} \)

Supported beam details

178 x 102 x 19 UKB

Bolt details

Diameter of bolts \( d_b=24 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( F_{Rd}=k_2*f_{ub}*A_t/(\gamma M_2*10^3)=203.33 \text{ kN} \)

End plate details

Thickness of plate \( t_p=25 \text{ mm} \)
Total number of bolts \( b_n=4 \)
Distance to first row of bolts \( e_x=80 \text{ mm} \)
Edge distance \( x=30 \text{ mm} \)
Bolt cross-centres \( p_3=60 \text{ mm} \)
Width of end plate \( b_p=125 \text{ mm} \)
Assumed web fillet weld size \( s_{ww}=8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf}=8 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts  Pr1=406.66 kN

**Beam tension flange welds**

Weld design strength  \( fvwd=\frac{fu}{\sqrt{3}}/\beta_{aw}\times \gamma_{M2} \)  
=222.79 N/mm²

Weld size required  \( swfr=\frac{F_{wEdf}\times 10^3}{0.7\times fvwd} \)=7.1345 mm

Selected fillet weld size  swt=8 mm

**Beam web welds**

Tension zone - a full strength weld is recommended in the web tension zone with the sum of the throat thicknesses ≥ the web thickness, twb.

Selected fillet weld size  sfw=8 mm

Length of web weld  \( L_{wt}=\left( e_x-x-t_{fb}-r_{b}\right)+1.73\times p_3/2=86.4 \) mm

Shear zone - assume an 8 mm FW continued for the full depth.

Length of weld for shear  \( L_{ws}=h_b-2\times \left(t_{fb}+r_{b}\right)-L_{wt}=60.4 \) mm

Resistance of weld for shear  \( F_{wRds}=2\times 0.7 \times sfw\times fvwd\times L_{ws}/10^3 \)

=150.71 kN

Vertical shear  \( V_{Ed}=129 \) kN

An 8 mm fillet weld size is therefore considered suitable.

**Compression welds**

As moment is reversible tension and compression surfaces are interchangeable, welding must be the same for both flanges.
 CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 4
Bolt diameter 24 mm
Bolt grade 8.8
Welds
  Tension flange fillet weld 8 mm
  Web fillet weld 8 mm
  Compression flange fillet weld 8 mm

Beam / Rafter section 178 x 102 x 19 UKB
Welded beam to column connection

Calculations are in accordance with BS5950-1:2000. The analysis follows the recommended design procedure checks given in the BCSA/BSI publication entitled 'Joints in Steel Construction - Moment Connections'.

The connection is attached to the column flange.

There is only one connection.

Factored moment $M=170$ kNm
Factored shear force $V=57$ kN
Axial force (+ve compression) $N=0$ kN

533 x 210 x 82 UB.
305 x 165 x 46 UB.

Connection of stiffeners to column web

For electrode grade 35
Design strength $pw=\text{TABLE 37 for cgrade}=275, \ Es=35 =220$ N/mm$^2$
Weld size $ss=6$ mm
Adopt 2/80 mm x 15 mm plate stiffeners Grade S 275 on line with the beam compression flange connected by 4/6 mm fillet welds.

Tension stiffener welds

Flange:
Provide full strength welds to the flange based on the thickness of the beam flange.
Required weld size $sfw'=Tb/(2\times0.7)=8.4286$ mm
Selected weld size $sfw=10$ mm

Web:
Assume the total force is transferred to the web via the welds.
Selected fillet weld size $stw=6$ mm
Weld chosen is considered suitable.
## CONNECTION SUMMARY

### Beam details

<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam section</td>
<td>305 x 165 x 46 UB</td>
</tr>
<tr>
<td>Fin plate details</td>
<td>80 x 6 x 220 long.</td>
</tr>
<tr>
<td></td>
<td>2 / M 20 bolts in slotted holes</td>
</tr>
<tr>
<td></td>
<td>HSFG Bolt General grade (8.8) to BS4395 Part 1. Nuts grade 10.</td>
</tr>
</tbody>
</table>

### Welds

- Tension and compression flange:
- Full penetration butt weld 12 mm
- Web butt weld 7 mm

### Column details

<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column section</td>
<td>533 x 210 x 82 UB</td>
</tr>
<tr>
<td>Compression stiffeners</td>
<td>2/80 mm x 15 mm plate Grade S 275 connected by 4/6 mm fillet welds.</td>
</tr>
<tr>
<td>Tension stiffeners</td>
<td>2/75 mm x 16 mm plate Grade S 275 connected by 4/6 mm fillet welds to the web and 4/10 mm fillet welds to the flanges.</td>
</tr>
</tbody>
</table>
Welded beam to column connection

Calculations are in accordance with BS5950-1:2000. The analysis follows the recommended design procedure checks given in the BCSA/BSI publication entitled 'Joints in Steel Construction - Moment Connections'.

The connection is attached to the column flange.

There is only one connection.

Factored moment \( M = 256 \text{ kNm} \)
Factored shear force \( V = 184 \text{ kN} \)
Axial force (+ve compression) \( N = 96 \text{ kN} \)

254 x 254 x 89 UC.
457 x 191 x 82 UB.

Connection of stiffeners to column web

For electrode grade 35
Design strength \( \text{pw}=\text{TABLE 37 for cgrade}=275, \text{Es}=35 \)
\[ =220 \text{ N/mm}^2 \]
Weld size \( \text{ss}=6 \text{ mm} \)
Adopt 2/110 mm x 15 mm plate stiffeners Grade S 275 on line with the beam compression flange connected by 4/6 mm fillet welds.

Tension stiffener welds

Flange:
Provide full strength welds to the flange based on the thickness of the beam flange.
Required weld size \( \text{sfw}'=\text{Tb}/(2\times0.7)=11.429 \text{ mm} \)
Selected weld size \( \text{sfw}=14 \text{ mm} \)

Web:
Assume the total force is transferred to the web via the welds.
Selected fillet weld size \( \text{stw}=8 \text{ mm} \)
Weld chosen is considered suitable.
Weld size $sc = 6\, \text{mm}$

Adopt $2/110\, \text{mm} \times 15\, \text{mm}$ diagonal stiffeners Grade S 275 on line between the beam tension and compression flanges connected by $4/6\, \text{mm}$ fillet welds.

Selected fillet weld size $swt = 12\, \text{mm}$

Bolt diameter $db = 20\, \text{mm}$

Thickness of fin plate $tfin = 10\, \text{mm}$

Pitch of bolts vertically $p = 70\, \text{mm}$

Vertical edge distance $e1 = 40\, \text{mm}$

Horizontal edge distance $e2 = 40\, \text{mm}$

Eccentricity of bolt group $af = 60\, \text{mm}$

Supported beam end clearance $ec = 20\, \text{mm}$

Slip factor for preloaded bolts $\mu = 0.5$

Coefficient for type of hole $Ks = 1$

Weld leg length $s = 8\, \text{mm}$

Selected fillet weld size $sfc = 12\, \text{mm}$

**CONNECTION SUMMARY**

**Beam details**

- **Beam section**: $457 \times 191 \times 82\, \text{UB}$
- **Fin plate details**: $100 \times 10 \times 290\, \text{long.}$
  - $4\, \text{M 20 bolts in clearance holes}$
  - $8\, \text{mm fillet welding}$
  - HSFG Bolt General grade (8.8)
  - to BS4395 Part 1.
  - Nuts grade 10.

**Welds**

- Tension flange F.W. $12\, \text{mm}$
- Compression flange F.W. $12\, \text{mm}$

**Column details**

- **Column section**: $254 \times 254 \times 89\, \text{UC}$
- **Compression stiffeners**: $2/110\, \text{mm} \times 15\, \text{mm}$ plate Grade S 275 connected by $4/6\, \text{mm}$ fillet welds.
- **Tension stiffeners**: $2/110\, \text{mm} \times 15\, \text{mm}$ plate Grade S 275 connected by $4/8\, \text{mm}$ fillet welds to the web and $4/14\, \text{mm}$ fillet welds to the flanges.
- **Diagonal stiffeners**: $2/110\, \text{mm} \times 15\, \text{mm}$ Grade S 275 connected by $4/6\, \text{mm}$ fillet welds.
Location: Ex3 - Shop welded stub

Welded beam to column connection

Calculations are in accordance with BS5950-1:2000. The analysis follows the recommended design procedure checks given in the BCSA/BSI publication entitled 'Joints in Steel Construction - Moment Connections'.

The connection is attached to the column flange.

There is a similar beam on each column face.

Factored moment \( M = 383 \text{ kNm} \)
Factored shear force \( V = 176 \text{ kN} \)
Axial force (+ve compression) \( N = -34 \text{ kN} \)

254 x 254 x 107 UC.
533 x 210 x 109 UB.

Connection of stiffeners to column web

For electrode grade 35
Design strength \( pw = \text{TABLE 37 for cgrade=275, Es=35} = 220 \text{ N/mm}^2 \)
Weld size \( ss = 8 \text{ mm} \)
Adopt 2/110 mm x 15 mm plate stiffeners Grade S 275 on line with the beam compression flange connected by 4/8 mm fillet welds.

Tension stiffener welds

Flange:
Provide full strength welds to the flange based on the thickness of the beam flange.
Required weld size \( sfw' = Tb/(2 \times 0.7) = 13.429 \text{ mm} \)
Selected weld size \( sfw = 14 \text{ mm} \)

Web:
Assume the total force is transferred to the web via the welds.
Selected fillet weld size \( stw = 8 \text{ mm} \)
Weld chosen is considered suitable.
Selected fillet weld size  swt=12 mm
Selected fillet weld size  ww=9 mm

CONNECTION SUMMARY

Beam details - beam on either side of the stanchion

Beam section  533 x 210 x 109 UB
Welds
Tension flange F.W.  12 mm
Web fillet weld  9 mm
Compression flange F.W.  8 mm

Column details

Column section  254 x 254 x 107 UC
Compression stiffeners  2/110 mm x 15 mm plate Grade S 275
connected by 4/8 mm fillet welds.
Tension stiffeners  2/110 mm x 25 mm plate Grade S 275
connected by 4/8 mm fillet welds
to the web and 4/14 mm fillet welds
to the flanges.
Location: Ex4 - High shear, grade 355 steel

Welded beam to column connection

Calculations are in accordance with BS5950-1:2000. The analysis follows the recommended design procedure checks given in the BCSA/BSI publication entitled 'Joints in Steel Construction - Moment Connections'.

The connection is attached to the column flange.

There is a similar beam on each column face.

Factored moment \( M = 569 \text{ kNm} \)
Factored shear force \( V = 334 \text{ kN} \)
Axial force (+ve compression) \( N = 189 \text{ kN} \)

305 x 305 x 198 UC.
610 x 229 x 113 UB.

Bolt diameter \( d_b = 20 \text{ mm} \)
Thickness of fin plate \( t_{\text{fin}} = 10 \text{ mm} \)
Pitch of bolts vertically \( p = 90 \text{ mm} \)
Vertical edge distance \( e_1 = 40 \text{ mm} \)
Horizontal edge distance \( e_2 = 40 \text{ mm} \)
Eccentricity of bolt group \( a_f = 60 \text{ mm} \)
Supported beam end clearance \( e_c = 20 \text{ mm} \)
Slip factor for preloaded bolts \( \mu = 0.5 \)
Coefficient for type of hole \( K_s = 1 \)
Weld leg length \( s = 8 \text{ mm} \)

CONNECTION SUMMARY

Beam details - beam on either side of the stanchion

Beam section 610 x 229 x 113 UB
Fin plate details
5 / M 20 bolts in clearance holes
8 mm fillet welding.
HSFG Bolt General grade (8.8) to BS4395 Part 1. Nuts grade 10.

Welds

Tension and compression flange:
Full penetration butt weld 18 mm
Column details

Column section          305 x 305 x 198 UC
Location: Ex1 - Example from Moment Connections publication

Welded beam to column connection

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

The connection is attached to the supporting column flange. All loads and moments are factored.

One sided beam to column connection will be considered.

Design moment \( M_{Edc} = 170 \text{ kNm} \)

Design shear force \( V_{Ed} = 57 \text{ kN} \)

Axial force (+ve compression) \( N_{Ed} = 0 \text{ kN} \)

Supporting column details

533 x 210 x 82 UKB

Supported beam details

305 x 165 x 46 UKB

Column web in transverse compression - EC3 Part 1-8

Design axial load on column \( N_{Edc} = 500 \text{ kN} \)

Connection of stiffeners to column web

Load on welds \( F_{WEds} = F_{EdCf} \times 10^3 / (L_w \times 10^3) \)

Weld design shear strength \( f_{vwd1} = f_u / \sqrt{3} / (\beta_{aw} \times \gamma_{M2}) \)

Weld size \( ss = 6 \text{ mm} \)

Adopt 2/80 mm x 15 mm plate stiffeners Grade S 275 on line with the beam compression flange connected by 4/6 mm fillet welds.
Column tension stiffener welds

Flange welds: Provide full strength welds to the flange based on the thickness of the beam flange.

Required weld size \( s_{fw}' = \frac{t_{fb}}{2 \times 0.7} = 8.4286 \) mm

Selected weld size \( s_{fw} = 10 \) mm

Web welds: Assume the force is transferred to the web via the welds.

Selected fillet weld size \( s_{tw} = 6 \) mm

Weld chosen is considered suitable.

**CONNECTION SUMMARY**

**Beam details**

<table>
<thead>
<tr>
<th>Beam section</th>
<th>305 x 165 x 46 UKB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fin plate backing strip</td>
<td>80 x 6 x 220 long. Provide 2 / M 20 grade 8.8 bolts in slotted holes for adjustment.</td>
</tr>
</tbody>
</table>

**Beam to column welds**

| Tension & compression flange | Full penetration butt weld 12 mm |
| Web butt weld                | 7 mm |

**Column details**

<table>
<thead>
<tr>
<th>Column section</th>
<th>533 x 210 x 82 UKB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression stiffeners</td>
<td>2/80 mm x 15 mm plate Grade S 275 connected by 4/6 mm fillet welds.</td>
</tr>
<tr>
<td>Tension stiffeners</td>
<td>2/75 mm x 16 mm plate Grade S 275 connected by 4/6 mm fillet welds to the web and 4/10 mm fillet welds to the flanges.</td>
</tr>
</tbody>
</table>
**Location: Ex2 - Example with fin plate and diagonal stiffeners**

**Welded beam to column connection**

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

![Diagram of welded beam to column connection]

The connection is attached to the supporting column flange. All loads and moments are factored.

One sided beam to column connection will be considered.

- **Design moment** \( \text{MyEd} = 256 \text{ kNm} \)
- **Design shear force** \( \text{VEd} = 184 \text{ kN} \)
- **Axial force (+ve compression)** \( \text{NEd} = 96 \text{ kN} \)

**Supporting column details**

- 254 x 254 x 89 UKC

**Supported beam details**

- 457 x 191 x 82 UKB

**Column web in transverse compression - EC3 Part 1-8**

- **Design axial load on column** \( \text{NEdc} = 500 \text{ kN} \)

**Connection of stiffeners to column web**

- **Load on welds** \( \text{FwEds} = \text{FEdCf} \times 10^3/(\text{Lw} \times 10^3) = 0.85 \text{ kN/mm} \)
- **Weld design shear strength** \( \text{fvwd1} = \text{fu}/\text{SQR}(3)/(\text{betaw} \times \text{gamM2}) \)
  \[ = 222.79 \text{ N/mm}^2 \]
- **Weld size** \( \text{ss} = 6 \text{ mm} \)

Adopt 2/115 mm x 15 mm plate stiffeners Grade S 275 on line with the beam compression flange connected by 4/6 mm fillet welds.
Column tension stiffener welds

Flange welds: Provide full strength welds to the flange based on the thickness of the beam flange.

Required weld size               \( sfw' = \frac{tfb}{2 \times 0.7} = 11.429 \) mm
Selected weld size               \( sfw = 14 \) mm

Web welds: Assume the force is transferred to the web via the welds.

Selected fillet weld size        \( stw = 8 \) mm
Weld chosen is considered suitable.
Weld size                        \( sc = 6 \) mm

Adopt 2/115 mm x 15 mm diagonal stiffeners Grade S 275
on line between the beam tension and compression flanges
connected by 4/6 mm fillet welds.

Selected fillet weld size        \( swt = 12 \) mm
Selected fillet weld size         \( ww = 8 \) mm
Selected fillet weld size         \( sfc = 12 \) mm

**CONNECTION SUMMARY**

**Beam details**

**Beam section**

457 x 191 x 82 UKB

**Beam to column welds**

<table>
<thead>
<tr>
<th>Flange Type</th>
<th>Weld Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension flange F.W.</td>
<td>12 mm</td>
</tr>
<tr>
<td>Web fillet weld</td>
<td>8 mm</td>
</tr>
<tr>
<td>Compression flange F.W.</td>
<td>12 mm</td>
</tr>
</tbody>
</table>

**Column details**

**Column section**

254 x 254 x 89 UKC

**Compression stiffeners**

2/115 mm x 15 mm plate Grade S 275
connected by 4/6 mm fillet welds.

**Tension stiffeners**

2/115 mm x 15 mm plate Grade S 275
connected by 4/8 mm fillet welds
to the web and 4/14 mm fillet welds
to the flanges.

**Diagonal stiffeners**

2/115 mm x 15 mm Grade S 275
connected by 4/6 mm fillet welds.
Location: Ex3 - Shop welded stub

Welded beam to column connection

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

There is a similar beam on each column face.
Design moment MyEd=383 kNm
Design shear force VEd=176 kN
Axial force (+ve compression) NEd=-34 kN

Supporting column details

254 x 254 x 107 UKC

Supported beam details

533 x 210 x 109 UKB

Column web in transverse compression - EC3 Part 1-8

Design axial load on column NEdc=500 kN

Connection of stiffeners to column web

Load on welds FwEds=FEdCf*10^3/(Lw*10^3)
=0.97788 kN/mm
Weld design shear strength fvwd1=fu/SQR(3)/(betaw*gamM2)
=222.79 N/mm²
Weld size ss=8 mm
Adopt 2/110 mm x 15 mm plate stiffeners Grade S 275 on line with the beam compression flange connected by 4/8 mm fillet welds.
Column tension stiffener welds

Flange welds: Provide full strength welds to the flange based on the thickness of the beam flange.

Required weld size \( sfw' = \frac{t_{fb}}{2 \times 0.7} = 13.429 \text{ mm} \)

Selected weld size \( sfw = 14 \text{ mm} \)

Web welds: Assume the force is transferred to the web via the welds.

Selected fillet weld size \( stw = 8 \text{ mm} \)

Weld chosen is considered suitable.

Selected fillet weld size \( swt = 12 \text{ mm} \)

Selected fillet weld size \( ww = 9 \text{ mm} \)

Selected fillet weld size \( sfc = 8 \text{ mm} \)

Connection summary

Beam details - beam on either side of the stanchion

Beam section 533 x 210 x 109 UKB

Beam to column welds

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension flange F.W.</td>
<td>12 mm</td>
</tr>
<tr>
<td>Web fillet weld</td>
<td>9 mm</td>
</tr>
<tr>
<td>Compression flange F.W.</td>
<td>8 mm</td>
</tr>
</tbody>
</table>

Column details

Column section 254 x 254 x 107 UKC

Compression stiffeners

2/110 mm x 15 mm plate Grade S 275 connected by 4/8 mm fillet welds.

Tension stiffeners

2/110 mm x 25 mm plate Grade S 275 connected by 4/8 mm fillet welds

to the web and 4/14 mm fillet welds to the flanges.
Location: Ex4 - High shear, grade 355 steel

Welded beam to column connection

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

The connection is attached to the supporting column flange.

All loads and moments are factored.

There is a similar beam on each column face.

- Design moment: MyEd=569 kNm
- Design shear force: VEd=334 kN
- Axial force (+ve compression): NEd=189 kN

Supporting column details

- 305 x 305 x 198 UKC

Supported beam details

- 610 x 229 x 113 UKB

Column web in transverse compression - EC3 Part 1-8

- Design axial load on column: NEEdc=500 kN
- Selected fillet weld size: ww=8 mm
CONNECTION SUMMARY

Beam details - beam on either side of the stanchion

**Beam section** 610 x 229 x 113 UKB

**Beam to column welds**
- Tension & compression flange
  - Full penetration butt weld 18 mm
- Web fillet weld 8 mm

Column details

**Column section** 305 x 305 x 198 UKC
Location: Ex5 - No column stiffeners (Lawrence Martin book Ex7.7)

Welded beam to column connection

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

The connection is attached to the supporting column flange.

All loads and moments are factored.

One sided beam to column connection will be considered.

Design moment \( \text{MyEd}=243 \text{ kNm} \)

Design shear force \( \text{VEd}=405 \text{ kN} \)

Axial force (+ve compression) \( \text{NEd}=0 \text{ kN} \)

Supporting column details

305 x 305 x 198 UKC

Supported beam details

838 x 292 x 226 UKB

Column web in transverse compression - EC3 Part 1-8

Design axial load on column \( \text{NEdc}=500 \text{ kN} \)

Selected fillet weld size \( \text{swt}=6 \text{ mm} \)

Selected fillet weld size \( \text{ww}=12 \text{ mm} \)

Selected fillet weld size \( \text{sfc}=6 \text{ mm} \)
## CONNECTION SUMMARY

### Beam details

**Beam section**
838 x 292 x 226 UKB

### Beam to column welds

- **Tension flange F.W.** 6 mm
- **Web fillet weld** 12 mm
- **Compression flange F.W.** 6 mm

### Column details

**Column section**
305 x 305 x 198 UKC
Flush beam splice with end plates and single flange plate

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions

- all end plate bolts are subject to tension
- shear is evenly distributed between the bolts in the end plate arrangement
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible
- flange plate (FP) on the compression surface.

Factored moment  
M=405 kNm

Factored shear force  
V=60 kN

Axial force (+ve compression)  
N=0 kN

Beam details

533 x 210 x 92 UB.

Bolt details

Diameter of bolts  
bd=24 mm

Tensile capacity  
Pt' = pt*At/10^3 = 197.68 kN

End plate details

Thickness of plate  
tp=25 mm

Number of tension bolts  
bn=8

Distance to first row of bolts  
ex=75 mm

Distance to beam flange  
x=15 mm

Pitch of tension bolts  
p2=90 mm

Bolt cross-centres  
g=100 mm

Width of end plate  
bp=250 mm

Assumed web fillet weld size  
sww=8 mm

Assumed flange fillet weld size  
swf=12 mm
### SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Top row of bolts</th>
<th>Pr1=395.36 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Row 2</td>
<td>Pr2=318.89 kN</td>
</tr>
<tr>
<td>Row 3</td>
<td>Pr3=242.42 kN</td>
</tr>
<tr>
<td>Row 4</td>
<td>Pr4=165.94 kN</td>
</tr>
</tbody>
</table>

#### Beam tension flange welds

- Selected fillet weld size: swt=12 mm

#### Beam web welds

- Selected fillet weld size: sfw=8 mm
- Vertical shear: V=60 kN

#### FLANGE SPLICE

**Assumptions:**
- the flange plate(s) resist a compressive force, Fcc
- the splice uses HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

### Flange plate details

- Thickness of flange plate: tcp=20 mm
- Width of flange cover plate: Bfp=210 mm

### Flange plate bolt details

- Bolt diameter in flange plate: bdf=24 mm
- Bolt spacing (across the beam): crs=140 mm
- Total no. of bolts in cover plate: np=16
- Pitch of bolts (along the beam): pit=70 mm
- Spacing of bolts across beams: spac=150 mm
- End distance in flange plate: end=50 mm
- Edge distance: ed=(Bfp-crs)/2=35 mm
- Projection of flange cover plate: P=(np/4-1)*pit+spac/2+end=335 mm
- Length of flange cover plate: Dfp=2*P=670 mm

### Slip resistance of one fastener

- Slip factor for preloaded bolts: mu=0.5
- Coefficient for type of hole: Ks=1
- Proof load (BS4395): Po=207 kN
- Slip resistance: PsL=1.1*Ks*mu*Po=113.85 kN
### CONNECTION SUMMARY - all dimensions in diagram are in mm

<table>
<thead>
<tr>
<th></th>
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<tbody>
<tr>
<td>ex</td>
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<td>p2</td>
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<td></td>
</tr>
<tr>
<td>p3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTE:** Bottom flange cover plate omitted for clarity.

- **Bolt grade:** 8.8
- **Welds:**
  - Tension flange fillet weld 12 mm
  - Web fillet weld 8 mm
  - Compression flange fillet weld 8 mm
- **Beam / Rafter section:** 533 x 210 x 92 UB
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plate: 210 mm x 20 mm x 670 mm
Number of bolts: 16
Bolt diameter: 24 mm
Bolt grade: HSFG
General grade (8.8) to BS4395 Part 1
Nut grade: 10
**Location: Ex2 - Double plates using 8.8 bolts**

Flush beam splice with end plates and single flange plate

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

**Design assumptions**

- all end plate bolts are subject to tension
- shear is evenly distributed between the bolts in the end plate arrangement
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible
- flange plate (FP) on the compression surface.

**Factored moment** \( M = 380 \text{ kNm} \)

**Factored shear force** \( V = 60 \text{ kN} \)

**Axial force (+ve compression)** \( N = 0 \text{ kN} \)

**Beam details**

533 x 210 x 92 UB.

**Bolt details**

- Diameter of bolts \( b_d = 24 \text{ mm} \)
- Tensile capacity \( P_t' = P_t \times A_t / 10^3 = 197.68 \text{ kN} \)

**End plate details**

- Thickness of plate \( t_p = 25 \text{ mm} \)
- Number of tension bolts \( b_n = 8 \)
- Distance to first row of bolts \( e_x = 75 \text{ mm} \)
- Distance to beam flange \( x = 15 \text{ mm} \)
- Pitch of tension bolts \( p_2 = 90 \text{ mm} \)
- Bolt cross-centres \( g = 100 \text{ mm} \)
- Width of end plate \( b_p = 250 \text{ mm} \)
- Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
- Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)
SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts                  Pr1=395.36 kN
Row 2                             Pr2=318.89 kN
Row 3                             Pr3=242.42 kN
Row 4                             Pr4=165.94 kN

Beam tension flange welds

Selected fillet weld size         swt=12 mm

Beam web welds

Selected fillet weld size         sfw=8 mm
Vertical shear                   V=60 kN

FLANGE SPLICE

Assumptions:
• the flange plate(s) resist a compressive force, Fcc
• the splice uses HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

Flange plate details

Thickness of flange plate         tcp=15 mm
Width of flange cover plate       Bfp=210 mm

Flange plate bolt details

Bolt spacing (across the beam)    crs=140 mm
Total no. of bolts in cover plate np=20
Pitch of bolts (along the beam)   pit=70 mm
Spacing of bolts across beams     spac=150 mm
End distance in flange plate      end=50 mm
Edge distance                     ed=(Bfp-crs)/2=35 mm
Projection of flange cover plate  P=(np/4-1)*pit+spac/2+end=405 mm
Length of flange cover plate      Dfp=2*P=810 mm

Slip resistance of one fastener

Slip factor for preloaded bolts   mu=0.5
Coefficient for type of hole      Ks=1
Proof load (BS4395)               Po=207 kN
Slip resistance                   PsL=1.1*Ks*mu*Po=113.85 kN
CONNECTION SUMMARY - all dimensions in diagram are in mm

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<table>
<thead>
<tr>
<th></th>
<th>25</th>
<th></th>
<th>250</th>
</tr>
</thead>
<tbody>
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</tr>
<tr>
<td>p3</td>
<td></td>
<td>203</td>
<td>548</td>
</tr>
</tbody>
</table>

NOTE: Bottom flange cover plate omitted for clarity.

Bolt grade       8.8
Welds            Tension flange fillet weld 12 mm
                  Web fillet weld 8 mm
                  Compression flange fillet weld 8 mm
Beam / Rafter section 533 x 210 x 92 UB
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plate 210 mm x 15 mm x 810 mm
Number of bolts 20
Bolt diameter 24 mm
Bolt grade HSFG
Nut grade General grade (8.8) to BS4395 Part 1

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Ref No: SC427 BS

Steel design to BS5950-1:2000 and Eurocode 3
Made by: IFB
Flush beam splice w end plates & single flange plate
Date: 02/12/19

Sample output for SCALE Proforma 427. (ans=2)
Flush beam splice w end plates & single flange plate

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions
- all end plate bolts are subject to shear
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible
- flange plate (FP) on the tension surface.

Factored moment \( M = 30 \) kNm
Factored shear force \( V = 20 \) kN
Axial force (+ve compression) \( N = 0 \) kN

Beam details
152 x 152 x 37 UC.

End plate details
- Diameter of bolts \( b_d = 20 \) mm
- Thickness of plate \( t_p = 16 \) mm
- Number of shear bolts \( b_n = 4 \)
- Distance to first row of bolts \( e_x = 72 \) mm
- Distance to beam flange \( x = 16 \) mm
- Pitch of remaining bolts \( p_2 = 50 \) mm
- Bolt cross-centres \( g = 90 \) mm
- Width of end plate \( b_p = 156 \) mm
- Assumed web fillet weld size \( s_w_w = 8 \) mm
- Assumed flange fillet weld size \( s_w_f = 8 \) mm
Beam tension flange welds

Beam web welds

Full strength weld used
Selected fillet weld size $s_{fw}=8$ mm

**FLANGE SPLICE**

Assumptions:

- the flange plate(s) resist a tensile force, $F_t$
- the splice uses HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

**Flange plate details**

- Thickness of flange plate $t_{cp}=16$ mm
- Width of flange cover plate $B_{fp}=154$ mm

**Flange plate bolt details**

- Bolt spacing (across the beam) $c_{rs}=90$ mm
- Total no. of bolts in cover plate $n_p=8$
- Pitch of bolts (along the beam) $p_{it}=50$ mm
- Spacing of bolts across beams $s_{pac}=120$ mm
- End distance in flange plate $e_{nd}=75$ mm
- Edge distance $e_d=(B_{fp}-c_{rs})/2=32$ mm
- Projection of flange cover plate $P=(n_p/4-1)*p_{it}+s_{pac}/2+e_{nd}=185$ mm
- Length of flange cover plate $D_{fp}=2*P=370$ mm

**Slip resistance of one fastener**

- Slip factor for preloaded bolts $\mu=0.4$
- Coefficient for type of hole $K_s=1$
- Proof load (BS4395) $P_o=144$ kN
- Slip resistance $P_{sl}=1.1*K_s*\mu*P_o=63.36$ kN
CONNECTION SUMMARY - all dimensions in diagram are in mm

NOTE: Bottom flange cover plate omitted for clarity.
Bolt grade 8.8
Welds
- Tension flange fillet weld 8 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Beam / Rafter section 152 x 152 x 37 UC

FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plate 154 mm x 16 mm x 370 mm
Number of bolts 8
Bolt diameter 20 mm
Bolt grade HSFG
Nut grade General grade (8.8) to BS4395 Part 1
Flush beam splice with end plates and single flange plate

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions
• all end plate bolts are subject to shear
• the connection consists of two identical beam and end plate arrangements
• moment is not reversible
• flange plate (FP) on the tension surface.

Factored moment \( M = 30 \text{ kNm} \)
Factored shear force \( V = 20 \text{ kN} \)
Axial force (+ve compression) \( N = 0 \text{ kN} \)

Beam details
152 x 152 x 37 UC.

End plate details
Diameter of bolts \( b_d = 20 \text{ mm} \)
Thickness of plate \( t_p = 16 \text{ mm} \)
Number of shear bolts \( b_n = 4 \)
Distance to first row of bolts \( e_x = 72 \text{ mm} \)
Distance to beam flange \( x = 16 \text{ mm} \)
Pitch of remaining bolts \( p_2 = 50 \text{ mm} \)
Bolt cross-centres \( g = 90 \text{ mm} \)
Width of end plate \( b_p = 156 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 8 \text{ mm} \)
Beam tension flange welds

Beam web welds

Selected fillet weld size \( sfw = 6 \text{ mm} \)
Capacity of weld for shear \( P_{sw} = 2 \times 0.7 \times sfw \times pw \times Lws / 10^3 = 228.41 \text{ kN} \)
Vertical shear \( V = 20 \text{ kN} \)

FLANGE SPLICE

Assumptions:

• the flange plate(s) resist a tensile force, \( F_t \)
• the splice uses HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

Flange plate details

Thickness of flange plate \( tcp = 16 \text{ mm} \)
Width of flange cover plate \( Bfp = 154 \text{ mm} \)

Flange plate bolt details

Bolt spacing (across the beam) \( crs = 90 \text{ mm} \)
Total no. of bolts in cover plate \( np = 8 \)
Pitch of bolts (along the beam) \( pit = 50 \text{ mm} \)
Spacing of bolts across beams \( spac = 120 \text{ mm} \)
End distance in flange plate \( end = 75 \text{ mm} \)
Edge distance \( ed = (Bfp - crs) / 2 = 32 \text{ mm} \)
Projection of flange cover plate \( P = (np / 4 - 1) \times pit + spac / 2 + end = 185 \text{ mm} \)
Length of flange cover plate \( Dfp = 2 \times P = 370 \text{ mm} \)

Slip resistance of one fastener

Slip factor for preloaded bolts \( \mu = 0.4 \)
Coefficient for type of hole \( K_s = 1 \)
Proof load (BS4395) \( P_o = 144 \text{ kN} \)
Slip resistance \( P_{sL} = 1.1 \times K_s \times \mu \times P_o = 63.36 \text{ kN} \)
CONNECTION SUMMARY - all dimensions in diagram are in mm

NOTE: Bottom flange cover plate omitted for clarity.
Bolt grade 8.8
Welds
- Tension flange fillet weld 8 mm
- Web fillet weld 6 mm
- Compression flange fillet weld 8 mm
Beam / Rafter section 152 x 152 x 37 UC

FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plate 154 mm x 16 mm x 370 mm
Number of bolts 8
Bolt diameter 20 mm
Bolt grade HSFG
General grade (8.8) to BS4395 Part 1
Nut grade 10
Flush beam splice with end plates and single flange plate

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions
- all bolts are in tension with the exception of the bottom two which will resist all the applied shear force
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible
- compression flange plate (CFP) as shown in diagram.

Factored moment $MyEd=405\ kNm$
Factored shear force $VeEd=60\ kN$
Axial force (+ve compression) $NeEd=0\ kN$

Beam details
533 x 210 x 92 UKB

Bolt details
Diameter of bolts $db=24\ mm$
Tensile resistance (EC3 Part 1-8) $FtRd=k2*fub*At/(gamM2*10^3)=203.33\ kN$

End plate details
Thickness of plate $tp=25\ mm$
Number of bolts (in tension) $bn=8$
Distance to first row of bolts $el=75\ mm$
Distance to beam flange $x=15\ mm$
Pitch of tension bolts $p1=90\ mm$
End distance to shear bolts $el'=100\ mm$
Bolt cross-centres $p3=100\ mm$
Width of end plate $bp=250\ mm$
Assumed web fillet weld size $sww=8\ mm$
Assumed flange fillet weld size $swf=12\ mm$
## SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Row</th>
<th>Pr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top row of bolts</td>
<td>406.66 kN</td>
</tr>
<tr>
<td>Row 2</td>
<td>328 kN</td>
</tr>
<tr>
<td>Row 3</td>
<td>249.34 kN</td>
</tr>
<tr>
<td>Row 4</td>
<td>170.69 kN</td>
</tr>
</tbody>
</table>

### Beam tension flange welds

- Selected fillet weld size
  - \( swt = 12 \text{ mm} \)

### Beam web welds

- **Selected fillet weld size**
  - \( sfw = 8 \text{ mm} \)
- **Resistance of weld for shear**
  - \( FwRds = 2 \times 0.7 \times sfw \times fvwd \times Lws / 10^3 = 220.33 \text{ kN} \)
- **Vertical shear**
  - \( VEd = 60 \text{ kN} \)

### FLANGE SPLICE

**Assumptions:**
- the flange plate(s) resist a compressive force, \( FcEdp \)
- the splice uses preloaded HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

### Flange cover plate details

- **Thickness of flange cover plate**
  - \( tfp = 20 \text{ mm} \)
- **Width of flange cover plate**
  - \( bfp = 210 \text{ mm} \)

### Flange cover plate bolt details

- **Bolt diameter in flange plate**
  - \( dbfp = 24 \text{ mm} \)
- **Bolt cross-centres**
  - \( p3fp = 140 \text{ mm} \)
- **Total no. of bolts in cover plate**
  - \( np = 16 \)
- **Pitch of bolts (along the beam)**
  - \( plfp = 70 \text{ mm} \)
- **Spacing of bolts across beams**
  - \( spac = 150 \text{ mm} \)
- **End distance in flange plate**
  - \( elfp = 50 \text{ mm} \)
- **Edge distance at cross centres**
  - \( e2fp = (bfp - p3fp) / 2 = 35 \text{ mm} \)
- **Projection of flange cover plate**
  - \( P = (np / 4 - 1) \times plfp + spac / 2 + elfp = 335 \text{ mm} \)
- **Length of flange cover plate**
  - \( hfp = 2 \times P = 670 \text{ mm} \)

### Slip resistance of one fastener - EC3 Part 1-8

- **Slip resistance**
  - \( FsRd = ks \times n \times \mu \times Fpc / 1.25 = 98.84 \text{ kN} \)
CONNECTION SUMMARY - all dimensions in diagram are in mm

NOTE: Bottom flange cover plate omitted for clarity.

Bolt grade 8.8
Bolt diameter 24 mm
Welds
- Tension flange fillet weld 12 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm

Beam / Rafter section 533 x 210 x 92 UKB
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

- Flange cover plate: 210 mm x 20 mm x 670 mm
- Number of bolts: 16
- Bolt diameter: 24 mm
- Bolt grade: Preloaded (HSFG), higher grade (10.9) with grade 12 nuts.
Flush beam splice with end plates and single flange plate

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions
- all bolts are in tension with the exception of the bottom two which will resist all the applied shear force
- the connection consists of two identical beam and end plate arrangements
- moment is not reversible
- compression flange plate (CFP) as shown in diagram.

Factored moment \( M_{Ed} = 380 \) kNm
Factored shear force \( V_{Ed} = 60 \) kN
Axial force (+ve compression) \( N_{Ed} = 0 \) kN

Beam details
- 533 x 210 x 92 UKB

Bolt details
- Diameter of bolts \( d_b = 24 \) mm
- Tensile resistance (EC3 Part 1-8) \( F_{T,Rd} = k_2 * f_{ub} * A_t / (\gamma_{M2} * 10^3) = 203.33 \) kN

End plate details
- Thickness of plate \( t_{p} = 25 \) mm
- Number of bolts (in tension) \( b_n = 8 \)
- Distance to first row of bolts \( e_{l} = 75 \) mm
- Distance to beam flange \( x = 15 \) mm
- Pitch of tension bolts \( p_{l} = 90 \) mm
- End distance to shear bolts \( e_{l}' = 100 \) mm
- Bolt cross-centres \( p_{3} = 100 \) mm
- Width of end plate \( b_{p} = 250 \) mm
- Assumed web fillet weld size \( s_{ww} = 8 \) mm
- Assumed flange fillet weld size \( s_{wf} = 12 \) mm
SUMMARY OF BOLT POTENTIAL RESISTANCES

<table>
<thead>
<tr>
<th>Top row of bolts</th>
<th>Pr1=406.66 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Row 2</td>
<td>Pr2=328 kN</td>
</tr>
<tr>
<td>Row 3</td>
<td>Pr3=249.34 kN</td>
</tr>
<tr>
<td>Row 4</td>
<td>Pr4=170.69 kN</td>
</tr>
</tbody>
</table>

Beam tension flange welds

Selected fillet weld size \( \text{swt}=12 \text{ mm} \)

Beam web welds

Selected fillet weld size \( \text{sfw}=8 \text{ mm} \)

Resistance of weld for shear \( F_{wRd} = 2 \times 0.7 \times \text{sfw} \times \text{fvwd} \times L_{ws}/10^3 = 220.33 \text{ kN} \)

Vertical shear \( V_{Ed}=60 \text{ kN} \)

FLANGE SPLICE

Assumptions:
- the flange plate(s) resist a compressive force, \( \text{FcEdp} \)
- the splice uses preloaded HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

Flange cover plate details

Thickness of flange cover plate \( \text{tfp}=15 \text{ mm} \)
Width of flange cover plate \( \text{bfp}=210 \text{ mm} \)

Flange cover plate bolt details

Bolt cross-centres \( \text{p3fp}=140 \text{ mm} \)
Total no. of bolts in cover plate \( \text{np}=20 \)
Pitch of bolts (along the beam) \( \text{plfp}=70 \text{ mm} \)
Spacing of bolts across beams \( \text{spac}=150 \text{ mm} \)
End distance in flange plate \( \text{elfp}=50 \text{ mm} \)
Edge distance at cross centres \( \text{e2fp}= (\text{bfp}-\text{p3fp})/2=35 \text{ mm} \)
Projection of flange cover plate \( \text{P}=(\text{np}/4-1) \times \text{plfp} + \text{spac}/2 + \text{elfp}=405 \text{ mm} \)
Length of flange cover plate \( \text{hfp}=2 \times \text{P}=810 \text{ mm} \)

Slip resistance of one fastener - EC3 Part 1-8

Slip resistance \( F_{sRd}= k_s \times n \times \mu \times F_{pc}/1.25=79.072 \text{ kN} \)
CONNECTION SUMMARY - all dimensions in diagram are in mm

NOTE: Bottom flange cover plate omitted for clarity.
Bolt grade 8.8
Bolt diameter 24 mm
Welds
- Tension flange fillet weld 12 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Beam / Rafter section 533 x 210 x 92 UKB
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plate: 210 mm x 15 mm x 810 mm
Number of bolts: 20
Bolt diameter: 24 mm
Bolt grade: Preloaded (HSFG), general grade (8.8) with grade 10 nuts.
Flush beam splice with end plates and single flange plate

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

**Design assumptions**
- all bolts will resist the applied shear force
- connection consists of two identical beam & end plate arrangements
- moment is not reversible
- tension flange plate (TFP) as shown in diagram.

**Factored moment**  \( My_{Ed} = 30 \, \text{kNm} \)
**Factored shear force**  \( V_{Ed} = 20 \, \text{kN} \)
**Axial force (+ve compression)**  \( N_{Ed} = 0 \, \text{kN} \)

**Beam details**
152 x 152 x 37 UKC

**End plate details**
- Diameter of bolts  \( d_b = 20 \, \text{mm} \)
- Thickness of plate  \( t_p = 16 \, \text{mm} \)
- Number of bolts (in shear)  \( b_n = 4 \)
- Distance to first row of bolts  \( e_1 = 72 \, \text{mm} \)
- Distance to beam flange  \( x = 16 \, \text{mm} \)
- Pitch of remaining bolts  \( p_1 = 50 \, \text{mm} \)
- Bolt cross-centres  \( p_3 = 90 \, \text{mm} \)
- Width of end plate  \( b_p = 156 \, \text{mm} \)
- Assumed web fillet weld size  \( s_{ww} = 8 \, \text{mm} \)
- Assumed flange fillet weld size  \( s_{wf} = 8 \, \text{mm} \)

**Beam tension flange welds**

**Beam web welds**
- Full strength weld used
- Selected fillet weld size  \( s_{fw} = 8 \, \text{mm} \)
FLANGE SPlice

Assumptions:
• the flange plate(s) resist a tensile force, FtEdp
• the splice uses preloaded HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

Flange cover plate details

Thickness of flange cover plate tfp=16 mm
Width of flange cover plate bfp=154 mm

Flange cover plate bolt details

Bolt cross-centres p3fp=90 mm
Total no. of bolts in cover plate np=8
Pitch of bolts (along the beam) plfp=50 mm
Spacing of bolts across beams spac=120 mm
End distance in flange plate elfp=75 mm
Edge distance at cross centres e2fp=(bfp-p3fp)/2=32 mm
Projection of flange cover plate P=(np/4-1)*plfp+spac/2+elfp=185 mm
Length of flange cover plate hfp=2*P=370 mm

Slip resistance of one fastener - EC3 Part 1-8

Slip resistance FsRd=ks*n*mu*Fpc/1.25=54.88 kN

CONNECTION SUMMARY - all dimensions in diagram are in mm

NOTE: Bottom flange cover plate omitted for clarity.
Bolt grade 8.8
Bolt diameter 20 mm
Welds
- Tension flange fillet weld 8 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Beam / Rafter section 152 x 152 x 37 UKC
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plate  154 mm x 16 mm x 370 mm
Number of bolts  8
Bolt diameter  20 mm
Bolt grade  Preloaded (HSFG), higher grade (10.9) with grade 12 nuts.
Flush beam splice with end plates and single flange plate

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions

- all bolts will resist the applied shear force
- connection consists of two identical beam & end plate arrangements
- moment is not reversible
- tension flange plate (TFP) as shown in diagram.

Factored moment $M_{Ed}=30 \, \text{kNm}$
Factored shear force $V_{Ed}=20 \, \text{kN}$
Axial force (+ve compression) $N_{Ed}=0 \, \text{kN}$

Beam details

152 x 152 x 37 UKC

End plate details

- Diameter of bolts $d_b=20 \, \text{mm}$
- Thickness of plate $t_p=16 \, \text{mm}$
- Number of bolts (in shear) $b_n=4$
- Distance to first row of bolts $e_1=72 \, \text{mm}$
- Distance to beam flange $x=16 \, \text{mm}$
- Pitch of remaining bolts $p_1=50 \, \text{mm}$
- Bolt cross-centres $p_3=90 \, \text{mm}$
- Width of end plate $b_p=156 \, \text{mm}$
- Assumed web fillet weld size $s_{ww}=8 \, \text{mm}$
- Assumed flange fillet weld size $s_{wf}=8 \, \text{mm}$

Beam tension flange welds

Beam web welds

- Selected fillet weld size $s_{fw}=6 \, \text{mm}$
- Resistance of weld for shear $F_{W_{Rd}}=2\times0.7\times s_{fw}\times f_{vwd}\times L_{ws}/10^3=231.31 \, \text{kN}$
- Vertical shear $V_{Ed}=20 \, \text{kN}$
FLANGE SPLICE

Assumptions:
• the flange plate(s) resist a tensile force, \( F_{tEdp} \)
• the splice uses preloaded HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

Flange cover plate details

- Thickness of flange cover plate \( t_{fp} = 16 \text{ mm} \)
- Width of flange cover plate \( b_{fp} = 154 \text{ mm} \)

Flange cover plate bolt details

- Bolt cross-centres \( p_{3fp} = 90 \text{ mm} \)
- Total no. of bolts in cover plate \( n_p = 8 \)
- Pitch of bolts (along the beam) \( p_{1fp} = 50 \text{ mm} \)
- Spacing of bolts across beams \( \text{spac} = 120 \text{ mm} \)
- End distance in flange plate \( e_{1fp} = 75 \text{ mm} \)
- Edge distance at cross centres \( e_{2fp} = (b_{fp} - p_{3fp})/2 = 32 \text{ mm} \)
- Projection of flange cover plate \( P = (n_p/4 - 1) \cdot p_{1fp} + \text{spac}/2 + e_{1fp} = 185 \text{ mm} \)
- Length of flange cover plate \( h_{fp} = 2 \cdot P = 370 \text{ mm} \)

Slip resistance of one fastener - EC3 Part 1-8

- Slip resistance \( F_{sRd} = k_s \cdot n \cdot \mu \cdot F_{pc}/1.25 = 54.88 \text{ kN} \)

CONNECTION SUMMARY - all dimensions in diagram are in mm

NOTE: Bottom flange cover plate omitted for clarity.

- Bolt grade 8.8
- Bolt diameter 20 mm
- Welds:
  - Tension flange fillet weld 8 mm
  - Web fillet weld 6 mm
  - Compression flange fillet weld 8 mm

Beam / Rafter section 152 x 152 x 37 UKC
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plate: 154 mm x 16 mm x 370 mm
Number of bolts: 8
Bolt diameter: 20 mm
Bolt grade: Preloaded (HSFG), higher grade (10.9) with grade 12 nuts.
Flush beam splice with end plates and single flange plate

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

**Design assumptions**
- all bolts will resist the applied shear force
- connection consists of two identical beam & end plate arrangements
- moment is not reversible
- tension flange plate (TFP) as shown in diagram.

Factored moment \( M_{Ed} = 30 \ \text{kNm} \)
Factored shear force \( V_{Ed} = 20 \ \text{kN} \)
Axial force (+ve compression) \( N_{Ed} = 0 \ \text{kN} \)

**Beam details**
457 x 191 x 67 UKB

**End plate details**
- Diameter of bolts \( d_b = 20 \ \text{mm} \)
- Thickness of plate \( t_p = 16 \ \text{mm} \)
- Number of bolts (in shear) \( b_n = 6 \)
- Distance to first row of bolts \( e_1 = 72 \ \text{mm} \)
- Distance to beam flange \( x = 16 \ \text{mm} \)
- Pitch of remaining bolts \( p_1 = 50 \ \text{mm} \)
- Bolt cross-centres \( p_3 = 90 \ \text{mm} \)
- Width of end plate \( b_p = 190 \ \text{mm} \)
- Assumed web fillet weld size \( s_{ww} = 8 \ \text{mm} \)
- Assumed flange fillet weld size \( s_{wf} = 8 \ \text{mm} \)

**Slip resistance of one fastener - EC3 Part 1-8**

Slip resistance \( F_{sRd} = k_s \cdot n \cdot \mu \cdot F_{pc} / 1.25 = 43.904 \ \text{kN} \)
Beam tension flange welds

Beam web welds

Selected fillet weld size \( sfw = 6 \text{ mm} \)

Resistance of weld for shear

\[
F_{wwd} = 2 \times 0.7 \times sfw \times fvwd \times Lws / 10^3
\]
\[= 762.79 \text{ kN} \]

Vertical shear \( V_{Ed} = 20 \text{ kN} \)

FLANGE SPLICE

Assumptions:
- the flange plate(s) resist a tensile force, \( F_{tEdp} \)
- the splice uses preloaded HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

Flange cover plate details

Thickness of flange cover plate \( t_{fp} = 16 \text{ mm} \)

Width of flange cover plate \( b_{fp} = 190 \text{ mm} \)

Flange cover plate bolt details

Bolt cross-centres \( p_{3fp} = 90 \text{ mm} \)

Total no. of bolts in cover plate \( n_p = 8 \)

Pitch of bolts (along the beam) \( p_{1fp} = 50 \text{ mm} \)

Spacing of bolts across beams \( \text{spac} = 120 \text{ mm} \)

End distance in flange plate \( e_{1fp} = 100 \text{ mm} \)

Edge distance at cross centres \( e_{2fp} = (b_{fp} - p_{3fp}) / 2 = 50 \text{ mm} \)

Projection of flange cover plate \( P = (n_p / 4 - 1) \times p_{1fp} + \text{spac} / 2 + e_{1fp} = 210 \text{ mm} \)

Length of flange cover plate \( h_{fp} = 2 \times P = 420 \text{ mm} \)

Slip resistance of one fastener - EC3 Part 1-8

Slip resistance \( F_{sRd} = k_s \times n \times \mu \times F_{pc} / 1.25 = 43.904 \text{ kN} \)
CONNECTION SUMMARY - all dimensions in diagram are in mm

NOTE: Bottom flange cover plate omitted for clarity.
Bolt grade Preloaded (HSFG), general grade (8.8) with grade 10 nuts.
Bolt diameter 20 mm
Welds Tension flange fillet weld 8 mm
Web fillet weld 6 mm
Compression flange fillet weld 8 mm
Beam / Rafter section 457 x 191 x 67 UKB

FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plate 190 mm x 16 mm x 420 mm
Number of bolts 8
Bolt diameter 20 mm
Bolt grade Preloaded (HSFG), general grade (8.8) with grade 10 nuts.
Flush beam splice w end plates & single flange plate

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions

- all bolts will resist the applied shear force
- connection consists of two identical beam & end plate arrangements
- moment is not reversible
- tension flange plate (TFP) as shown in diagram.

Factored moment MyEd = 30 kNm
Factored shear force VEd = 20 kN
Axial force (+ve compression) NEd = 0 kN

Beam details

152 x 152 x 37 UKC

End plate details

<table>
<thead>
<tr>
<th>Diameter of bolts</th>
<th>db = 20 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of plate</td>
<td>tp = 16 mm</td>
</tr>
<tr>
<td>Number of bolts (in shear)</td>
<td>bn = 4</td>
</tr>
<tr>
<td>Distance to first row of bolts</td>
<td>el = 72 mm</td>
</tr>
<tr>
<td>Distance to beam flange</td>
<td>x = 16 mm</td>
</tr>
<tr>
<td>Pitch of remaining bolts</td>
<td>pl = 50 mm</td>
</tr>
<tr>
<td>Bolt cross-centres</td>
<td>p3 = 90 mm</td>
</tr>
<tr>
<td>Width of end plate</td>
<td>bp = 156 mm</td>
</tr>
<tr>
<td>Assumed web fillet weld size</td>
<td>sww = 8 mm</td>
</tr>
<tr>
<td>Assumed flange fillet weld size</td>
<td>swf = 8 mm</td>
</tr>
</tbody>
</table>

Beam tension flange welds

Beam web welds

<table>
<thead>
<tr>
<th>Selected fillet weld size</th>
<th>sfw = 6 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistance of weld for shear</td>
<td>FwRds = 2<em>0.7</em>sfw<em>fvwd</em>Lws/10^3 = 231.31 kN</td>
</tr>
<tr>
<td>Vertical shear</td>
<td>VEd = 20 kN</td>
</tr>
</tbody>
</table>
FLANGE SPLICE

Assumptions:
• the flange plate(s) resist a tensile force, \( F_{Edp} \)
• the splice uses 8.8 bolts as excessive beam deflection is considered to be unlikely.

Flange cover plate details

Thickness of flange cover plate \( t_{fp} = 16 \text{ mm} \)
Width of flange cover plate \( b_{fp} = 154 \text{ mm} \)

Flange cover plate bolt details

<table>
<thead>
<tr>
<th>Bolt cross-centres</th>
<th>( p_{3fp} = 90 \text{ mm} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total no. of bolts in cover plate</td>
<td>( n_p = 8 )</td>
</tr>
<tr>
<td>Pitch of bolts (along the beam)</td>
<td>( p_{1fp} = 50 \text{ mm} )</td>
</tr>
<tr>
<td>Spacing of bolts across beams</td>
<td>( \text{spac} = 120 \text{ mm} )</td>
</tr>
<tr>
<td>End distance in flange plate</td>
<td>( e_{1fp} = 75 \text{ mm} )</td>
</tr>
<tr>
<td>Edge distance at cross centres</td>
<td>( e_{2fp} = (b_{fp} - p_{3fp})/2 = 32 \text{ mm} )</td>
</tr>
<tr>
<td>Projection of flange cover plate</td>
<td>( P = (n_p/4-1) \times p_{1fp} + \text{spac}/2 + e_{1fp} = 185 \text{ mm} )</td>
</tr>
<tr>
<td>Length of flange cover plate</td>
<td>( h_{fp} = 2 \times P = 370 \text{ mm} )</td>
</tr>
</tbody>
</table>

CONNECTION SUMMARY - all dimensions in diagram are in mm

![Diagram](image)

NOTE: Bottom flange cover plate omitted for clarity.

Bolt grade 8.8
Bolt diameter 20 mm
Welds
- Tension flange fillet weld 8 mm
- Web fillet weld 6 mm
- Compression flange fillet weld 8 mm

Beam / Rafter section 152 x 152 x 37 UKC
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

154 mm x 16 mm x 370 mm

Number of bolts: 8
Bolt diameter: 20 mm
Bolt grade: 8.8
Cantilevered beam

Calculations are in accordance with BS5950 BS 5950-1:2000 clause 4.3.5.4 or 4.3.5.5.

Cantilever span \( L = 2 \text{ m} \)

- 254 x 146 x 43 UB.
- Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Dead load (including self weight) \( w_d = 8 \text{ kN/m} \)
Imposed load \( w_i = 10 \text{ kN/m} \)
Dead load \( W_d = 10 \text{ kN} \)
Imposed load \( W_i = 6 \text{ kN} \)

**UNIVERSAL BEAM**

- Design shear force \( 78 \text{ kN} \)
- Shear capacity \( 308.4 \text{ kN} \)
- Design moment \( 101.6 \text{ kNm} \)
- Moment capacity \( 155.65 \text{ kNm} \)
- Buckling resistance \( 108.32 \text{ kNm} \)
- Deflection \( 5.8676 \text{ mm} \)
Location: Ex1 - Default example

\[ W = W_d + W_i \]

Cantilevered beam

Calculations are in accordance with EC3 Part 1-1 : 2005.

Cantilever span \( L = 2 \text{ m} \)

254 x 146 x 43 UKB

Dimensions (mm): \( h = 259.6 \ b = 147.3 \ tw = 7.2 \ tf = 12.7 \ r = 7.6 \)
Properties (cm): \( I_y = 6540 \ I_z = 677 \ W_{p y} = 566 \ W_{p z} = 141 \ I_t = 23.9 \)

Permanent load (SW included) \( W_d = 8 \text{ kN/m} \)
Variable load \( W_i = 10 \text{ kN/m} \)
Permanent load \( W_d = 10 \text{ kN} \)
Variable load \( W_i = 6 \text{ kN} \)
Buckling length for LTB \( L_T = 3.2 \text{ m} \)

UNIVERSAL BEAM 254 x 146 x 43 UKB Grade S 275

DESIGN SUMMARY

Section yield strength 275 N/mm²
Design moment 96.6 kNm
Buckling resistance BM 125.95 kNm
Design shear force 74.1 kN
Shear plastic resistance 321.2 kN
Deflection to span ratio 1:349
Interaction factors
\[ 0.62062 \leq 1 \]
\[ 0.23069 \leq 1 \]
\[ 0.76699 \leq 1 \]
**Location: Ex2 - Channel section**

\[
W = W_d + W_i \\
\]

\[
W = W_d + W_i \\
\]

Cantilevered beam

Calculations are in accordance with EC3 Part 1-1 : 2005.

Cantilever span \( L = 3 \text{ m} \)

380 x 100 UKPFC

Dimensions (mm): \( h = 380 \), \( b = 100 \), \( t_w = 9.5 \), \( t_f = 17.5 \), \( r = 15 \)

Properties (cm): \( I_y = 15000 \), \( I_z = 643 \), \( W_{ply} = 933 \), \( W_{plz} = 161 \), \( I_t = 45.7 \)

Permanent load (SW included) \( W_d = 9 \text{ kN/m} \)

Variable load \( W_i = 12 \text{ kN/m} \)

Buckling length for LTB \( L_T = 3 \text{ m} \)

**PARALLEL FLANGE CHANNEL 380 x 100 Grade S 275**

**DESIGN SUMMARY**

- Section yield strength \( 265 \text{ N/mm}^2 \)
- Design moment \( 135.68 \text{ kNm} \)
- Buckling resistance BM \( 158.06 \text{ kNm} \)
- Design shear force \( 90.45 \text{ kN} \)
- Shear plastic resistance \( 581.2 \text{ kN} \)
- Deflection to span ratio \( 1:444 \)
- Interaction factors
  - \( 0.54875 \leq 1 \)
  - \( 0.15563 \leq 1 \)
  - \( 0.85836 \leq 1 \)
Location: Ex1 - Default example

\[ w = w_d + w_i \]

Cantilevered beam

Calculations are in accordance with BS5950 and 'Steelwork Design Guide' by SCI.

Design moment about z-z axis \( M_z = 109.95 \text{ kNm} \)
Design shear force \( F_v = 101.6 \text{ kN} \)
Design axial load (+ve comp) \( F = 0 \text{ kN} \)
Length of member \( L = 1.5 \text{ m} \)

254 x 146 x 43 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

UNIVERSAL Beam

DESIGN SUMMARY

254 x 146 x 43 UB Grade S 275

Design shear force \( 101.6 \text{ kN} \)
Shear capacity \( 308.4 \text{ kN} \)
Design moment \( 109.95 \text{ kNm} \)
Moment capacity \( 155.65 \text{ kNm} \)
Buckling resistance \( 128.11 \text{ kNm} \)
Moment factor mLT \( 1 \)
Resistance (Mb/mLT) \( 128.11 \text{ kNm} \)
Deflection \( 1.2115 \text{ mm} \)
Limiting deflection \( 8.3333 \text{ mm} \)
Location: Ex2 - Default example plus axial load

\[ w = w_d + w_i \]

Cantilevered beam - alt linking with NL-STRESS

Calculations are in accordance with BS5950 and 'Steelwork Design Guide' by SCI.

Design moment about z-z axis \( M_z = 109.95 \text{ kNm} \)
Design shear force \( F_v = 101.6 \text{ kN} \)
Design axial load (+ve comp) \( F = 44 \text{ kN} \)
Length of member \( L = 1.5 \text{ m} \)

254 x 146 x 43 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Length between restraints z axis \( L_z = 1500 \text{ mm} \)
Length between restraints y axis \( L_y = 1500 \text{ mm} \)
Quarter point \( m_{z2} = 74.2 \text{ kNm} \)
Mid-span \( m_{z3} = 44.1 \text{ kNm} \)
Three quarter point \( m_{z4} = 19.4 \text{ kNm} \)
Maximum moment on central half \( M_{24} = 74.2 \text{ kNm} \)

Values to applied factored values \( R_f = 0.6 \)

UNIVERSAL BEAM
DESIGN SUMMARY

254 x 146 x 43 UB Grade S 275
Design shear force \( 101.6 \text{ kN} \)
Shear capacity \( 308.4 \text{ kN} \)
Design moment \( 109.95 \text{ kNm} \)
Moment capacity \( 155.65 \text{ kNm} \)
Design axial force \( 44 \text{ kN} \)
Local capacity check \( 0.73559 \leq 1 \)
Buckling resistance \( 128.11 \text{ kNm} \)
Overall buckling \( 0.6384 \leq 1 \)
\( 0.89078 \leq 1 \)
Deflection \( 1.2115 \text{ mm} \)
Limiting deflection \( 8.3333 \text{ mm} \)
Heated perimeter \( 946.7 \text{ mm}^2 \)

FIRE RESISTANCE
SUMMARY

Area of section \( 54.8 \text{ cm}^2 \)
Section factor \( 170 /\text{m} \)
Load Ratio (R) \( 0.53447 \)
Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided.
Location: Ex3 - Channel section

\[ w = w_d + w_i \]

Cantilevered beam

Calculations are in accordance with BS5950 and 'Steelwork Design Guide' by SCI.

Design moment about z-z axis \( M_z = 130 \text{ kNm} \)
Design shear force \( F_v = 89 \text{ kN} \)
Design axial load (+ve comp) \( F = 40 \text{ kN} \)
Length of member \( L = 1.5 \text{ m} \)
430 x 100 Parallel Flange Channel.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Length between restraints z axis \( L_z = 1500 \text{ mm} \)
Length between restraints y axis \( L_y = 1500 \text{ mm} \)
Quarter point \( m_z = 64 \text{ kNm} \)
Mid-span \( m_z = 42 \text{ kNm} \)
Three quarter point \( m_z = 18 \text{ kNm} \)
Maximum moment on central half \( M_2 = 64 \text{ kNm} \)

PARALLEL FLANGE CHANNEL 430 x 100 Grade S 275
DESIGN SUMMARY

Design shear force 89 kN
Shear capacity 752.07 kN
Design moment 130 kNm
Moment capacity 323.3 kNm
Design axial force 40 kN
Local capacity check 0.42049 ≤ 1
Buckling resistance 217.32 kNm
Overall buckling 0.36332 ≤ 1 0.62116 ≤ 1
Location: Ex4 - Channel section plus fire check

\[ w = w_d + w_i \]

Cantilevered beam

Calculations are in accordance with BS5950 and 'Steelwork Design Guide' by SCI.

Design moment about z-z axis \( M_z = 130 \text{ kNm} \)
Design shear force \( F_v = 89 \text{ kN} \)
Design axial load (+ve comp) \( F = 40 \text{ kN} \)
Length of member \( L = 1.5 \text{ m} \)

430 x 100 Parallel Flange Channel.

Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Length between restraints z axis \( L_z = 1500 \text{ mm} \)
Length between restraints y axis \( L_y = 1500 \text{ mm} \)
Quarter point \( m_{z2} = 64 \text{ kNm} \)
Mid-span \( m_{z3} = 42 \text{ kNm} \)
Three quarter point \( m_{z4} = 18 \text{ kNm} \)
Maximum moment on central half \( M_{z4} = 64 \text{ kNm} \)

Values to applied factored values \( R_f = 0.5 \)

**PARALLEL FLANGE CHANNEL 430 x 100 Grade S 275**

**DESIGN SUMMARY**

- Design shear force 89 kN
- Shear capacity 752.07 kN
- Design moment 130 kNm
- Moment capacity 323.3 kNm
- Design axial force 40 kN
- Local capacity check 0.42049 \( \leq 1 \)
- Buckling resistance 217.32 kNm
- Overall buckling 0.36332 \( \leq 1 \), 0.62116 \( \leq 1 \)

**FIRE RESISTANCE SUMMARY**

- Heated perimeter 1238 mm²
- Area of section 82.1 cm²
- Section factor 150 /m
- Load Ratio (R) 0.31058

Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided.
Location: Ex1 - UB section in bending and axial load

\[
w = w_d + w_i
\]

**Cantilevered beam**

Calculations are in accordance with EC3 Part 1-1 : 2005.

Design moment about major axis \( M_y\text{Ed} = 109.95 \text{ kNm} \)
Design shear force \( V_z\text{Ed} = 101.6 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 0 \text{ kN} \)
Length of member \( L = 1.5 \text{ m} \)

Dimensions (mm): \( h = 259.6 \text{ b} = 147.3 \text{ tw} = 7.2 \text{ tf} = 12.7 \text{ r} = 7.6 \)

Properties (cm): \( I_y = 6540 \text{ I}_z = 677 \text{ W}_{ply} = 566 \text{ W}_{plz} = 141 \text{ I}_{t} = 23.9 \)

Buckling length for LTB \( L_T = 2.4 \text{ m} \)
Stiff bearing length \( s_s = 100 \text{ mm} \)
Stiff bearing position \( C = 0 \text{ mm} \)

**UNIVERSAL BEAM**

254 x 146 x 43 UKB Grade S 275

**DESIGN SUMMARY**

Section yield strength \( 275 \text{ N/mm}^2 \)
Design moment \( 109.95 \text{ kNm} \)
Buckling resistance BM \( 139.3 \text{ kNm} \)
Design shear force \( 101.6 \text{ kN} \)
Shear plastic resistance \( 321.2 \text{ kN} \)
Variable load deflection \( 1.1826 \text{ mm} \)
Limiting deflection \( 8.3333 \text{ mm} \)
Interaction factors
\[
0.70639 \leq 1 \\
0.31631 \leq 1 \\
0.7893 \leq 1
\]
Location: Ex2 - UB section with compressive axial load

\[ w = w_d + w_i \]

Cantilevered beam
Calculations are in accordance with EC3 Part 1-1 : 2005.

Design moment about major axis \( M_{Ed}=109.95 \, \text{kNm} \)
Design shear force \( V_{zEd}=101.6 \, \text{kN} \)
Design axial load (+ve comp) \( N_{Ed}=44 \, \text{kN} \)
Length of member \( L=1.5 \, \text{m} \)
Dimensions (mm): \( h=259.6 \, b=147.3 \, t_w=7.2 \, t_f=12.7 \, r=7.6 \)
Properties (cm): \( I_y=6540 \, I_z=677 \, W_{pl y}=566 \, W_{pl z}=141 \, I_t=23.9 \)

Section classification

Classify outstand element of compression flange:
As \( c/t_f \leq 9e \) (4.9173 \leq 8.3197), outstand element of compression flange is classified as Class 1 plastic.
Classify web element of section:
Depth between fillets \( d=h-2*(t_f+r)=219 \, \text{mm} \)
Ratio \( c't_w=d/t_w=30.417 \)
Limiting ratio \( c't_1=396*e/(13*ad-1)=59.431 \)
Web is Class 1 plastic.
Buckling length for LTB \( L_T=2.4 \, \text{m} \)

UNIVERSAL BEAM 254 x 146 x 43 UKB Grade S 275
DESIGN SUMMARY
Section yield strength 275 N/mm\(^2\)
Design moment 109.95 kNm
Buckling resistance BM 139.3 kNm
Design shear force 101.6 kNm
Shear plastic resistance 321.2 kN
Variable load deflection 1.1826 mm
Limiting deflection 8.3333 mm
Interaction factors 0.70639 \leq 1
0.31631 \leq 1
0.7893 \leq 1
0.8185 \leq 1
0.81699 \leq 1
Location: Ex3 - Channel section with compressive axial load

\[ w = w_d + w_i \]

Cantilevered beam

Calculations are in accordance with EC3 Part 1-1: 2005.

Design moment about major axis \( M_{Ed} = 130 \text{ kNm} \)
Design shear force \( V_{zEd} = 89 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 40 \text{ kN} \)
Length of member \( L = 1.5 \text{ m} \)

430 x 100 UKPFC

Dimensions (mm): \( h = 430 \quad b = 100 \quad tw = 11 \quad tf = 19 \quad r = 15 \)
Properties (cm): \( I_y = 21900 \quad I_z = 722 \quad W_{ply} = 1220 \quad W_{plz} = 176 \quad I_t = 63 \)
Buckling length for LTB \( LT = 2.73 \text{ m} \)

PARALLEL FLANGE CHANNEL

DESIGN SUMMARY

430 x 100 Grade S 275

Section yield strength \( 265 \text{ N/mm}^2 \)
Design moment \( 130 \text{ kNm} \)
Buckling resistance BM \( 193.27 \text{ kNm} \)
Design shear force \( 89 \text{ kN} \)
Shear plastic resistance \( 750.3 \text{ kN} \)
Interaction factors \( 0.4021 \leq 1 \)
\[ 0.11862 \leq 1 \]
\[ 0.67262 \leq 1 \]
\[ 0.69101 \leq 1 \]
\[ 0.69006 \leq 1 \]
Location: Ex4 - Channel section no axial load

\[ w = w_d + w_i \]

Cantilevered beam

Calculations are in accordance with EC3 Part 1-1: 2005.

Design moment about major axis \( M_{yEd} = 130 \text{ kNm} \)
Design shear force \( V_{zEd} = 89 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 0 \text{ kN} \)
Length of member \( L = 1.5 \text{ m} \)

430 x 100 UKPFC
Dimensions (mm): \( h = 430 \), \( b = 100 \), \( tw = 11 \), \( tf = 19 \), \( r = 15 \)
Properties (cm): \( I_y = 21900 \), \( I_z = 722 \), \( W_{ply} = 1220 \), \( W_{plz} = 176 \), \( I_t = 63 \)
Buckling length for LTB \( L_T = 2.73 \text{ m} \)

PARALLEL FLANGE CHANNEL
DESIGN SUMMARY

430 x 100 Grade S 275
Section yield strength \( 265 \text{ N/mm}^2 \)
Design moment \( 130 \text{ kNm} \)
Buckling resistance BM \( 193.27 \text{ kNm} \)
Design shear force \( 89 \text{ kN} \)
Shear plastic resistance \( 750.3 \text{ kN} \)
Interaction factors
\[
0.4021 \leq 1 \\
0.11862 \leq 1 \\
0.67262 \leq 1
\]
Location: Ex5 - UB section in bending and tension

\[ w = w_d + w_i \]

Cantilevered beam

Calculations are in accordance with EC3 Part 1-1 : 2005.

Design moment about major axis \( M_{Ed} = 109.95 \text{ kNm} \)
Design shear force \( V_{zEd} = 101.6 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = -10 \text{ kN} \)
Length of member \( L = 1.5 \text{ m} \)
254 x 146 x 43 UKB

Dimensions (mm): \( h = 259.6 \) \( b = 147.3 \) \( t_w = 7.2 \) \( t_f = 12.7 \) \( r = 7.6 \)
Properties (cm): \( I_y = 6540 \) \( I_z = 677 \) \( W_{ply} = 566 \) \( W_{plz} = 141 \) \( I_t = 23.9 \)

Section classification

Classify outstand element of compression flange:
As \( c/t_f \leq 9e \) \((4.9173 \leq 8.3197)\), outstand element of compression flange is classified as Class 1 plastic.
Classify web element of section:
Depth between fillets \( d = h - 2*(t_f + r) = 219 \text{ mm} \)
Ratio \( c't_w = d/t_w = 30.417 \)
Limiting ratio \( c't_1 = 36*e/ad = 68.129 \)
Web is Class 1 plastic.
Buckling length for LTB \( L_T = 2.4 \text{ m} \)
Estimated net area (tension) \( A_{e} = 50 \text{ cm}^2 \)

UNIVERSAL BEAM 254 x 146 x 43 UKB Grade S 275
DESIGN SUMMARY
Section yield strength 275 N/mm²
Design moment 109.95 kNm
Buckling resistance BM 139.3 kNnm
Design shear force 101.6 kN
Shear plastic resistance 321.2 kN
Variable load deflection 1.1826 mm
Limiting deflection 8.3333 mm
Interaction factors 0.70639 ≤ 1
0.31631 ≤ 1
0.7893 ≤ 1
0.71367 ≤ 1
Location: Ex1 - Beam with biaxial bending and axial load

Design moment about z-z axis \( M_z = 530 \, \text{kNm} \)
Design shear force \( F_v = 95 \, \text{kN} \)
Design axial load (+ve comp) \( F = 160 \, \text{kN} \)
Length of member \( L = 5.6 \, \text{m} \)
Design moment about minor axis \( M_y = 0 \, \text{kNm} \)
Design shear force // flanges \( F_{vz} = 0 \, \text{kN} \)

533 x 210 x 82 UB.
Young's Modulus \( E = 205 \, \text{kN/mm}^2 \)

Length between restraints z axis \( L_z = 5600 \, \text{mm} \)
Length between restraints y axis \( L_y = 1600 \, \text{mm} \)
Length between restraints \( L_T = 1.6 \, \text{m} \)
Maximum moment on segment \( M_{LT} = 530 \, \text{kNm} \)
Far end BM \( \beta_M = 378.6 \, \text{kNm} \)
Maximum moment on segment \( M_z' = 530 \, \text{kNm} \)
Far end BM \( \beta_{Mz} = 0 \, \text{kNm} \)

Member buckling resistance

Overall buckling check - Clause 4.8.3.3.2

For major axis in-plane buckling

\[
\frac{F}{P_{cz}} + \frac{mz \cdot M_z}{M_{cz}} \left[ 1 + 0.5 \frac{F}{P_{cz}} \right] \leq 1.0
\]

Unity relationship

\[
\text{unity} = \frac{F}{P_{cz}} + mz \cdot M_z \cdot M_{cz} \cdot (1 + 0.5 \cdot F/P_{cz}) = 0.48843
\]

The unity relationship is satisfied.

For out-of-plane buckling,

\[
\frac{F_c}{P_{cy}} + \frac{mLT \cdot MLT}{M_b} \leq 1.0
\]
Unity relationship \[ ufy = F / P_c y + mL_T * mL_T / M_b = 0.6948 \]

The unity relationship is satisfied.

**UNIVERSAL BEAM**

533 x 210 x 82 UB Grade S 355

**SECTION**

Section is satisfactory for bending, axial load, local capacity, and overall buckling check.

**SUMMARY**

**DESIGN**

Axial load (+ve comp) 160 kN

**SUMMARY**

Compressive resistance 3402.2 kN

Maximum moment (z-axis) 530 kNm

Moment resistance 731.3 kNm

Local capacity check 0.76766 ≤ 1

Overall buckling check 0.48843 ≤ 1

0.6948 ≤ 1

Deflection check is beyond the scope of this proforma. It has been assumed that the beam deflection is satisfactory or not critical.
Location: Ex2 As Ex1 but with reduced axial load - no restraint

Open Section Beam

Calculations in accordance with BS5950-1:2000.

All loads and moments are factored.

Design moment about z-z axis \( M_z = 400 \text{ kNm} \)
Design shear force \( F_v = 95 \text{ kN} \)
Design axial load (+ve comp) \( F = 160 \text{ kN} \)
Length of member \( L = 5.6 \text{ m} \)
Design moment about minor axis \( M_y = 0 \text{ kNm} \)
Design shear force // flanges \( F_{vz} = 0 \text{ kN} \)

533 x 210 x 82 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Length between restraints z axis \( L_z = 5600 \text{ mm} \)
Length between restraints y axis \( L_y = 5600 \text{ mm} \)
Length between restraints \( L_T = 5.6 \text{ m} \)
Maximum moment on segment \( M_{LT} = 400 \text{ kNm} \)
Far end BM \( \beta M = 0 \text{ kNm} \)
Maximum moment on segment \( M_z' = 530 \text{ kNm} \)
Far end BM \( \beta M_z = 0 \text{ kNm} \)

Member buckling resistance

Overall buckling check - Clause 4.8.3.3.2

For members with moments about major axis only
For major axis in-plane buckling

\[
\frac{F}{P_{cz}} + \frac{mz.M_z}{Mc_z} \left[ 1 + 0.5 \frac{F}{P_{cz}} \right] \leq 1.0
\]

Unity relationship

\[
\text{unity} = \frac{F}{P_{cz}} + mz \cdot M_z / Mc_z \cdot (1 + 0.5 \cdot F / P_{cz}) = 0.37942
\]

The unity relationship is satisfied.
For out-of-plane buckling,

\[
\frac{F_c}{P_{cy}} + \frac{mLT.MLT}{Mb} \leq 1.0
\]
Unity relationship \[ u_{fy} = \frac{F}{P_{cy} + mLT \cdot MLT / Mb} = 0.99052 \]
The unity relationship is satisfied.

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<thead>
<tr>
<th>UNIVERSAL BEAM</th>
<th>533 x 210 x 82 UB Grade S 355</th>
</tr>
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<tbody>
<tr>
<td>SECTION</td>
<td>Section is satisfactory for bending,</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>axial load, local capacity, and overall buckling check.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Axial load (+ve comp) 160 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Compressive resistance 1086.1 kN</td>
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<tr>
<td></td>
<td>Maximum moment(z-axis) 400 kNm</td>
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<tr>
<td></td>
<td>Moment resistance 731.3 kNm</td>
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<tr>
<td></td>
<td>Local capacity check 0.5899 ≤ 1</td>
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<td></td>
<td>Overall buckling check 0.37942 ≤ 1</td>
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<tr>
<td></td>
<td>0.99052 ≤ 1</td>
</tr>
</tbody>
</table>

Deflection check is beyond the scope of this proforma. It has been assumed that the beam deflection is satisfactory or not critical.
Location: Ex3 - Semi-compact section

Open Section Beam

Calculations in accordance with BS5950-1:2000.

All loads and moments are factored.

Design moment about z-z axis $M_z=45$ kNm
Design shear force $F_v=23$ kN
Design axial load (+ve comp) $F=525$ kN
Length of member $L=5$ m
Design moment about minor axis $M_y=0$ kNm
Design shear force // flanges $F_vz=0$ kN

406 x 140 x 39 UB.
Young's Modulus $E=205$ kN/mm²

Length between restraints z axis $L_z=5000$ mm
Length between restraints y axis $L_y=2500$ mm
Length between restraints $L_T=2.5$ m
Maximum moment on segment $M_{LT}=45$ kNm
Far end BM $\beta M=22.5$ kNm
Maximum moment on segment $M_z'=45$ kNm
Far end BM $\beta M_z=0$ kNm

Member buckling resistance

Overall buckling check - Clause 4.8.3.3.2

For members with moments about major axis only
For major axis in-plane buckling

$$\frac{F}{P_{cz}} + \frac{mz.M_z}{Mcz} \left[ \frac{F}{1 + 0.5 \frac{F}{P_{cz}}} \right] \leq 1.0$$

Unity relationship

$$\frac{F}{P_{cz}} + \frac{mz.M_z}{Mcz} \left( 1 + 0.5 \frac{F}{P_{cz}} \right) = 0.57479$$

The unity relationship is satisfied.
For out-of-plane buckling,

$$\frac{F_c}{P_{cy}} + \frac{mLT.MLT}{Mb} \leq 1.0$$
Unity relationship
\[ u_{fy} = F/P_{cy} + mLT*MLT/Mb = 0.91792 \]
The unity relationship is satisfied.

<table>
<thead>
<tr>
<th>UNIVERSAL BEAM</th>
<th>406 x 140 x 39 UB Grade S 275</th>
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<tbody>
<tr>
<td>SECTION</td>
<td>Section is satisfactory for bending,</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>axial load, local capacity, and overall buckling check.</td>
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</table>

<table>
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<tr>
<th>DESIGN</th>
<th>Axial load (+ve comp) 525 kN</th>
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<tbody>
<tr>
<td>SUMMARY</td>
<td>Compressive resistance 828.12 kN</td>
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<td></td>
<td>Maximum moment(z-axis) 45 kNm</td>
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<td></td>
<td>Moment resistance 181.81 kNm</td>
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<tr>
<td></td>
<td>Local capacity check 0.63163 ≤ 1</td>
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<tr>
<td></td>
<td>Overall buckling check 0.57479 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>0.91792 ≤ 1</td>
</tr>
</tbody>
</table>

Deflection check is beyond the scope of this proforma. It has been assumed that the beam deflection is satisfactory or not critical.
**Location: Ex4 - Biaxial bending**

![Diagram](image)

**Open Section Beam**

Calculations in accordance with BS5950-1:2000.

All loads and moments are factored.

- **Design moment about z-z axis**: $M_z = 230$ kNm
- **Design shear force**: $F_v = 65$ kN
- **Design axial load (+ve comp)**: $F = 1500$ kN
- **Length of member**: $L = 4.3$ m
- **Design moment about minor axis**: $M_y = 87$ kNm
- **Design shear force // flanges**: $F_{vz} = 0$ kN

305 x 305 x 118 UC.

**Young's Modulus**: $E = 205$ kN/mm²

- **Length between restraints z axis**: $L_z = 4300$ mm
- **Length between restraints y axis**: $L_y = 4300$ mm
- **Length between restraints**: $L_T = 4.3$ m
- **Maximum moment on segment**: $M_{LT} = 230$ kNm
- **Far end BM**: $\beta_M = -100$ kNm
- **Maximum moment on segment**: $M_{z'} = 230$ kNm
- **Far end BM**: $\beta_M z = -100$ kNm
- **Maximum moment on segment**: $M_{y'} = 87$ kNm
- **Far end BM**: $\beta_M y = 23$ kNm
- **Maximum moment on segment**: $M_{maxy} = 87$ kNm
- **Far end BM**: $\beta_y = 23$ kNm

**Member buckling resistance**

**Overall buckling check - Clause 4.8.3.3.2**

For members with moments about both axes

For major axis in-plane buckling

\[
\frac{F}{P_{cz}} + \frac{mz.M_z}{M_{cz}} \left[ \frac{1 + 0.5}{P_{cz}} \right] + 0.5 \frac{myz.M_y}{M_{cy}} \leq 1.0
\]

Unity relationship

\[
unity = \frac{F}{P_{cz}} + mz \frac{M_z}{M_{cz}} (1 + 0.5 \frac{F}{P_{cz}}) + 0.5 \frac{myz.M_y}{M_{cy}} = 0.5985
\]

The unity relationship is satisfied.
For lateral torsional buckling,

$$\frac{F}{P_{cy}} + \frac{mLT \cdot MLT}{M_b} + \frac{m_y \cdot M_y}{M_{cy}} \left[ 1 + \frac{F}{P_{cy}} \right] \leq 1.0$$

Unity relationship

$$u_f_y = \frac{F}{P_{cy}} + \frac{mLT \cdot MLT}{M_b} + \frac{m_y \cdot M_y}{M_{cy}} \left( 1 + \frac{F}{P_{cy}} \right)$$

The unity relationship is satisfied.

For interactive buckling

$$\frac{m_z \cdot M_z (1+0.5F/P_{cz})}{M_{cz}(1-F/P_{cz})} + \frac{m_y \cdot M_y (1+F/P_{cy})}{M_{cy}(1-F/P_{cy})} \leq 1.0$$

Unity relationship z term

$$u_z = \frac{m_z \cdot M_z (1+0.5F/P_{cz})}{M_{cz}(1-F/P_{cz})}$$

Unity relationship y term

$$u_y = \frac{m_y \cdot M_y (1+F/P_{cy})}{M_{cy}(1-F/P_{cy})}$$

Unity relationship

$$U_n = u_z + u_y = 0.69183$$

The unity relationship is satisfied.

**UNIVERSAL COLUMN**

305 x 305 x 118 UC Grade S 355

**SECTION**

Section is satisfactory for bending, axial load, local capacity, and overall buckling check.

**SUMMARY**

Axial load (+ve comp) 1500 kN
Compressive resistance 4442 kN
Maximum moment (z-axis) 230 kNm
Moment resistance 676.2 kNm
Maximum moment (y-axis) 87 kNm
Moment resistance 305.05 kNm
Local capacity check 0.91519 \( \leq 1 \)
Overall buckling check 0.5985 \( < 1 \)
0.77781 \( \leq 1 \)
0.69183 \( \leq 1 \)

Deflection check is beyond the scope of this proforma. It has been assumed that the beam deflection is satisfactory or not critical.
Location: Ex5 - With fire check

Open Section Beam

Calculations in accordance with BS5950-1:2000.

All loads and moments are factored.

Design moment about z-z axis \( M_z = 530 \text{ kNm} \)
Design shear force \( F_v = 95 \text{ kN} \)
Design axial load (+ve comp) \( F = 160 \text{ kN} \)
Length of member \( L = 5.6 \text{ m} \)
Design moment about minor axis \( M_y = 0 \text{ kNm} \)
Design shear force // flanges \( F_{vz} = 0 \text{ kN} \)

533 x 210 x 82 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Length between restraints z axis \( L_z = 5600 \text{ mm} \)
Length between restraints y axis \( L_y = 1600 \text{ mm} \)
Length between restraints \( L_T = 1.6 \text{ m} \)
Maximum moment on segment \( M_{LT} = 530 \text{ kNm} \)
Far end BM \( \beta M = 378.6 \text{ kNm} \)
Maximum moment on segment \( M'_z = 530 \text{ kNm} \)
Far end BM \( \beta M'_z = 0 \text{kNm} \)

Member buckling resistance

Overall buckling check - Clause 4.8.3.3.2

For members with moments about major axis only
For major axis in-plane buckling

\[
\frac{F}{Pcz} + \frac{mz.Mz}{Mcz} \left[ \frac{F}{Pcz} + 0.5 \frac{F}{Pcz} \right] \leq 1.0
\]

Unity relationship

\[
\begin{align*}
\text{unity} &= \frac{F}{Pcz} + mz.Mz/Mcz (1+0.5*F/Pcz) \\
&= 0.48843
\end{align*}
\]

The unity relationship is satisfied.
For out-of-plane buckling,

\[
\frac{F_c}{P_{cy}} + \frac{mLT.MLT}{Mb} \leq 1.0
\]
Unity relationship \( u_fy = \frac{F}{P_{cy} + mLT \cdot MLT / M_b} = 0.6948 \)
The unity relationship is satisfied.

**UNIVERSAL BEAM**
533 x 210 x 82 UB Grade S 355

**SECTION**
Section is satisfactory for bending,

**SUMMARY**
axial load, local capacity, and overall buckling check.

**DESIGN**
Axial load (+ve comp) 160 kN

**SUMMARY**
Compressive resistance 3402.2 kN
Maximum moment(z-axis) 530 kNm
Moment resistance 731.3 kNm
Local capacity check \( 0.76766 \leq 1 \)
Overall buckling check \( 0.48843 \leq 1 \)
\( 0.6948 \leq 1 \)

Deflection check is beyond the scope of this proforma. It has been assumed that the beam deflection is satisfactory or not critical.
Location: Ex1 - Beam with biaxial bending and axial load

Open Section Beam

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis \( M_{yEd} = 30 \text{ kNm} \)
Design shear force \( V_{zEd} = 10 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 590 \text{ kN} \)
Length of member \( L = 5 \text{ m} \)
Moment about the minor axis \( M_{zEd} = 1 \text{ kNm} \)
Design shear force // to flanges \( V_{yEd} = 0.6 \text{ kN} \)

203 x 203 x 46 UKC
Dimensions (mm): \( h = 203.2, b = 203.6, tw = 7.2, tf = 11, r = 10.2 \)
Properties (cm): \( I_y = 4570, I_z = 1550, W_{pl_y} = 497, W_{pl_z} = 231, I_t = 22.2 \)
Effective length parameter \( K = 1 \)
Length for LTB \( L_T = 5 \text{ m} \)
Maximum moment on section \( M = 30 \text{ kNm} \)
Far end moment \( p_M(1) = 0 \text{ kNm} \)
Far end bending moment \( p_M(2) = 0 \text{ kNm} \)
Far end bending moment \( p_M(3) = 0 \text{ kNm} \)

Interaction equations

\[
\frac{N_{Ed}}{\chi_y NR_d} + \frac{k_{yy} M_{yEd}}{\chi_{LT} My_{Rd}} + \frac{k_{yz} M_{zEd}}{M_{zRd}} \leq 1
\]

Unity factor \( U_{n1} = \frac{N_{Ed}}{(\chi_y NR_d)} + \frac{k_{yy} M_{yEd}}{(\chi_{LT} My_{Rd})} + \frac{k_{yz} M_{zEd}}{M_{zRd}} = 0.62179 \)
Section chosen is suitable.

\[
\frac{N_{Ed}}{\chi_z NR_d} + \frac{k_{zy} M_{yEd'}}{\chi_{LT} My_{Rd}} + \frac{k_{zz} M_{zEd}}{M_{zRd}} \leq 1
\]
where, the maximum moment on segment \( M_{yEd'} = 30 \text{ kNm} \)

Unity factor \( U_{n2} = \frac{N_{Ed}}{(\chi_z NR_d)} + \frac{k_{zy} M_{yEd'}}{(\chi_{LT} My_{Rd})} + \frac{k_{zz} M_{zEd}}{M_{zRd}} = 0.96296 \)
Section chosen is suitable.
<table>
<thead>
<tr>
<th><strong>UNIVERSAL COLUMN</strong></th>
<th><strong>203 x 203 x 46 UKC Grade S 275</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DESIGN</strong></td>
<td></td>
</tr>
<tr>
<td>Design BM y-y axis</td>
<td>30 kNm</td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td></td>
</tr>
<tr>
<td>Moment resistance (y-y)</td>
<td>136.68 kNm</td>
</tr>
<tr>
<td>Design BM z-z axis</td>
<td>1 kNm</td>
</tr>
<tr>
<td>Moment resistance (z-z)</td>
<td>63.525 kNm</td>
</tr>
<tr>
<td>Design compression load</td>
<td>590 kN</td>
</tr>
<tr>
<td>Axial resistance</td>
<td>1614.3 kN</td>
</tr>
<tr>
<td>Interaction factors</td>
<td>0.62179 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>0.96296 ≤ 1</td>
</tr>
</tbody>
</table>

Deflection check is beyond the scope of this proforma. It has been assumed that the beam deflection is satisfactory or not critical.
Location: Ex2 - Moment about major axis and load

Open Section Beam

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis MyEd=60 kNm
Design shear force VzEd=30 kN
Design axial load (+ve comp) NEd=600 kN
Length of member L=6 m
Moment about the minor axis MzEd=0 kNm
Design shear force // to flanges VyEd=0 kN

254 x 254 x 89 UKC
Dimensions (mm): h=260.3 b=256.3 tw=10.3 tf=17.3 r=12.7
Properties (cm): Iy=14300 Iz=4860 Wply=1220 Wplz=575 It=102
Effective length parameter K=1
Length for LTB LT=6 m
Maximum moment on section M=60 kNm
Far end moment psiM=-30 kNm
Far end bending moment pM(1)=-30 kNm
Far end bending moment pM(3)=-30 kNm

Interaction equations

\[ \frac{NEd}{\chi_y.NRd} + kyy \frac{MyEd}{\chi_LT.MyRd} + kyz \frac{MzEd}{MzRd} \leq 1 \]

Unity factor \( Un1=NEd/(chiy*NRd)+kyy*MyEd/(ChiLT*MyRd)+kyz*MzEd/MzRd =0.32123 \)
Section chosen is suitable.

\[ \frac{NEd}{\chi_z.NRd} + kzy \frac{MyEd'}{\chi_LT.MyRd} + kzz \frac{MzEd}{MzRd} \leq 1 \]

where, the maximum moment on segment \( MyEd'=60 \) kNm

Unity factor \( Un2=NEd/(chiz*NRd)+kzy*MyEd'/(ChiLT*MyRd)+kzz*MzEd/MzRd =0.52312 \)
Section chosen is suitable.
**UNIVERSAL COLUMN**

<table>
<thead>
<tr>
<th>Design BM y-y axis</th>
<th>60 kNm</th>
</tr>
</thead>
</table>

**SUMMARY**

<table>
<thead>
<tr>
<th>Moment resistance (y-y)</th>
<th>323.3 kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design compression load</td>
<td>600 kN</td>
</tr>
<tr>
<td>Axial resistance</td>
<td>2994.5 kN</td>
</tr>
<tr>
<td>Interaction factors</td>
<td>0.32123 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>0.52312 ≤ 1</td>
</tr>
</tbody>
</table>

Deflection check is beyond the scope of this proforma. It has been assumed that the beam deflection is satisfactory or not critical.
Location: Ex3 - Moment about minor axis and load

Calculation in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis $M_{y,Ed}=0$ kNm
Design shear force $V_{y,Ed}=0$ kN
Design axial load (+ve comp) $N_{Ed}=800$ kN
Length of member $L=6$ m
Moment about the minor axis $M_{z,Ed}=30$ kNm
Design shear force // to flanges $V_{y,Ed}=16$ kN

Dimensions (mm): $h=260.3$ b=$256.3$ tw=$10.3$ tf=$17.3$ r=$12.7$
Properties (cm): $I_y=14300$ $I_z=4860$ $W_{pl, y}=1220$ $W_{pl, z}=575$ $I_t=102$

Far end bending moment $pM(2)=0$ kNm

Interaction equations

$$\frac{N_{Ed}}{\chi_y N_{Rd}} + \frac{k_{yz} M_{y, Ed}}{M_{z, Rd}} \leq 1$$

Unity factor $Un_1=N_{Ed}/(\chi_y N_{Rd})+k_{yz} M_{y, Ed}/M_{z, Rd} = 0.4416$

Section chosen is suitable.

$$\frac{N_{Ed}}{\chi_z N_{Rd}} + \frac{k_{zz} M_{z, Ed}}{M_{z, Rd}} \leq 1$$

Unity factor $Un_2=N_{Ed}/(\chi_z N_{Rd})+k_{zz} M_{z, Ed}/M_{z, Rd} = 0.71666$

Section chosen is suitable.

UNIVERSAL COLUMN

254 x 254 x 89 UKC Grade S 275

DESIGN

Design BM y-y axis 0 kNm
Design BM z-z axis 30 kNm
Design compression load 800 kN
Axial resistance 2994.5 kN

SUMMARY

Moment resistance (y-y) 323.3 kNm
Moment resistance (z-z) 152.38 kNm
Design compression load 800 kN

Interaction factors $0.4416 \leq 1$

Deflection check is beyond the scope of this proforma. It has been assumed that the beam deflection is satisfactory or not critical.
Location: Ex4 - Tensile load and moments

Open Section Beam

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis $M_{yEd}=60 \text{ kNm}$
Design shear force $V_{zEd}=30 \text{ kN}$
Design axial load (+ve comp) $N_{Ed}=-1000 \text{ kN}$
Length of member $L=6 \text{ m}$
Moment about the minor axis $M_{zEd}=30 \text{ kNm}$
Design shear force // to flanges $V_{yEd}=16 \text{ kN}$

Dimensions (mm): $h=260.3 \quad b=256.3 \quad t_w=10.3 \quad t_f=17.3 \quad r=12.7$
Properties (cm): $I_y=14300 \quad I_z=4860 \quad W_{pl_y}=1220 \quad W_{pl_z}=575 \quad I_t=102$
Effective length parameter $K=1$
Length for LTB $L_T=6 \text{ m}$
Maximum moment on section $M=60 \text{ kNm}$
Far end moment $p_s M=0 \text{ kNm}$

Interaction equation

\[
\frac{N_{Ed}}{N_{Rd}} + \frac{M_{yEd}}{\chi_{LT} \cdot M_{yRd}} + \frac{M_{zEd}}{M_{zRd}} \leq 1
\]

Unity factor $U_{n1}=N_{tEd}/N_{Rd}+M_{yEd}/(\chi_{LT} \cdot M_{yRd})+M_{zEd}/M_{zRd}=0.71641$

Section chosen is suitable.

**UNIVERSAL COLUMN**

- **DESIGN**
  - Design BM y-y axis $60 \text{ kNm}$
- **SUMMARY**
  - Moment resistance (y-y) $323.3 \text{ kNm}$
  - Design BM z-z axis $30 \text{ kNm}$
  - Moment resistance (z-z) $152.38 \text{ kNm}$
  - Design tensile load $1000 \text{ kN}$
  - Axial resistance $2994.5 \text{ kN}$
  - Interaction factors $0.71641 \leq 1$

Deflection check is beyond the scope of this proforma. It has been assumed that the beam deflection is satisfactory or not critical.
**Weighting**

- **Fully restrained SS I beam with UDL & point load/s**
- **Date**: 02/12/19

---

**Location: Example 4.1 SCI examples**

---

**Simply supported**

**fully restrained beam**

- **Point load n is shown**
- **Wn = Wnd + Wni**
- **where**, 
  - **Wnd = dead point load**
  - **Wni = imposed point load**

---

**Beam span**

- **L = 6.5 m**

---

**457 x 191 x 67 UB.**

**Young's Modulus**

- **E = 205 kN/mm²**

---

**Dead UDL (including S.W)**

- **wd = 15 kN/m**

**Imposed UDL**

- **wi = 20 kN/m**

---

**Dead point load 1 (+ve down)**

- **Wld = 40 kN**

**Imposed point load 1**

- **Wli = 50 kN**

---

**Distance from L.H. supp to load 1 x1 = 3.25 m**

---

**Force applied through flange**

- **Fbw = 240.25 kN**

**Stiff bearing length**

- **b1 = 50 mm**

---

**Distance to nearer end**

- **be = 0 mm**

**Distance to nearer end of member**

- **ae = 25 mm**

---

**Permanent load**

- **wp = 25 kN/m**

**Permanent load**

- **Wlp = 56 kN**
**SECTION**  
457 x 191 x 67 UKB Grade S 355

**DESIGN**  
Shear force  
240.25 kN

**SUMMARY**  
Shear capacity  
820.88 kN

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied moment</td>
<td>500.91 kNm</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>521.85 kNm</td>
</tr>
<tr>
<td>Deflection</td>
<td>12.459 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>18.056 mm</td>
</tr>
<tr>
<td>Applied flange force</td>
<td>240.25 kN</td>
</tr>
<tr>
<td>Local web capacity</td>
<td>289.08 kN</td>
</tr>
<tr>
<td>Web buckling capacity</td>
<td>148.79 kN</td>
</tr>
</tbody>
</table>

Bearing stiffeners are required and possibly at positions of high point loads.

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL shear at A</td>
<td>68.75 kN</td>
</tr>
<tr>
<td>LL shear at A</td>
<td>90 kN</td>
</tr>
<tr>
<td>DL shear at B</td>
<td>68.75 kN</td>
</tr>
<tr>
<td>LL shear at B</td>
<td>90 kN</td>
</tr>
</tbody>
</table>

**FIRE**  
Exposed surface area  
1459.5 mm²

**RESISTANCE**  
Area of section  
85.5 cm²

**SUMMARY**  
Section factor  
170 /m

Load Ratio (R)  
0.59305

Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided.
Location: Moment capacity method of fire resistance

Simply supported
fully restrained beam

Point load n is shown
Wn = Wnd + Wni
where,
Wnd=dead point load
Wni=imposed point load

<xn--> Wn
vvvvvvvvvvvvvvvvvvvvvvvvvvv

w = wd + wi

>bl<

Beam span L=6.5 m

457 x 191 x 67 UB.
Young's Modulus E=205 kN/mm²

Dead UDL (including S.W) wd=15 kN/m
Imposed UDL wi=20 kN/m
Dead point load 1 (+ve down) W1d=40 kN
Imposed point load 1 W1i=50 kN
Distance from L.H. supp to load 1 x1=3.25 m

Permanent load wp=25 kN/m
Permanent load W1p=56 kN
<table>
<thead>
<tr>
<th>SECTION</th>
<th>457 x 191 x 67 UKB Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESIGN</td>
<td></td>
</tr>
<tr>
<td>Shear force</td>
<td>240.25 kN</td>
</tr>
<tr>
<td>Shear capacity</td>
<td>820.88 kN</td>
</tr>
<tr>
<td>SUMMARY</td>
<td></td>
</tr>
<tr>
<td>Applied moment</td>
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<tr>
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</tr>
<tr>
<td>DL shear at B</td>
<td>68.75 kN</td>
</tr>
<tr>
<td>LL shear at B</td>
<td>90 kN</td>
</tr>
<tr>
<td>Unfactored end shears</td>
<td></td>
</tr>
<tr>
<td>FIRE</td>
<td></td>
</tr>
<tr>
<td>Exposed surface area</td>
<td>1459.5 mm²</td>
</tr>
<tr>
<td>RESISTANCE</td>
<td></td>
</tr>
<tr>
<td>Area of section</td>
<td>85.5 cm²</td>
</tr>
<tr>
<td>SUMMARY</td>
<td></td>
</tr>
<tr>
<td>Section factor</td>
<td>170 /m</td>
</tr>
</tbody>
</table>

Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided.
**Location:** Example 4.1 SCI examples (with FR check)

**Simply supported**

**fully restrained beam**

Point load \( n \) is shown

\[ W_n = W_{nd} + W_{ni} \]

where,

\( W_{nd} \) = permanent point load

\( W_{ni} \) = variable point load

---

**Beam span**  
\( L = 6.5 \text{ m} \)

**Permanent UDL (including S.W)**  
\( w_d = 15 \text{ kN/m} \)

**Variable UDL**  
\( w_i = 20 \text{ kN/m} \)

**Permanent point load 1 (+ve down)**  
\( W_{1d} = 40 \text{ kN} \)

**Variable point load 1**  
\( W_{1i} = 50 \text{ kN} \)

**Distance from L.H. supp to load 1**  
\( x_1 = 3.25 \text{ m} \)

**457 x 191 x 67 UKB**

**Dimensions (mm):**  
\( h = 453.4 \)  
\( b = 189.9 \)  
\( t_w = 8.5 \)  
\( t_f = 12.7 \)  
\( r = 10.2 \)

**Properties (cm):**  
\( I_y = 29400 \)  
\( I_z = 1450 \)  
\( W_{p_l y} = 1470 \)  
\( W_{p_l z} = 237 \)  
\( I_t = 37.1 \)

**Stiff bearing length**  
\( C = 50 \text{ mm} \)

**Stiff bearing position**  
\( C = 0 \text{ mm} \)

**SECTION**  
457 x 191 x 67 UKB Grade S 355

**DESIGN**

**SUMMARY**

**Maximum moment**  
460.59 kNm

**Moment resistance**  
521.85 kNm

**Unity factor bending**  
0.8826 \( \leq 1 \)

**Maximum shear force**  
220.94 kN

**Shear plastic resistance**  
839.02 kN

**Unity factor shear**  
0.26333 \( \leq 1 \)

**Deflection to span ratio**  
1: 534

**Web resistance**  
273.37 kN

---

**Fire resistance without applied protection**

The critical temperature method given in BS EN 1993-1-2 Clause 4.2.4 will be adopted. For further clarification refer to thomas telford publication entitled 'Designer's guide to EN 1991-1-2, EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2'.

Factor for variable actions \( \psi_1 \)  
\( \psi_1 = 0.5 \)
Maximum moment: MyEd = M = 237.66 kNm
Maximum shear force: VzEd = Vb = 113.75 kN

Section classification

Classify outstand element of compression flange:
As 9e < c/tf ≤ 10e (6.3386 ≤ 6.9157), outstand element of compression flange is classified as Class 2 compact.

Classify web element of section:
As C/tw ≤ 72e (47.953 ≤ 49.793), web element in bending is classified as Class 1 plastic.

Critical temperature of carbon structural steel

Design moment (fire limit state): MfEd = MyEd = 237.66 kNm
Design buckling resistance: MbRdfi = McRd = 521.85 kNm
Design moment resistance: MfRd = ky*(gamM0/gamfi)*MbRdfi/(k1 *k2) = 745.5 kNm
Critical temperature θacr: taCR = 39.19*LOG(comp-1)+482 = 654.55 °C
Fire resistance period: 21 minutes

FIRE RESISTANCE

SUMMARY

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Location: High shear

\[ w = w_d + w_i \]

Simply supported

fully restrained beam

Location: High shear

Beam span \( L = 2 \) m

Permanent UDL (including S.W) \( w_d = 1.2 \) kN/m
Variable UDL \( w_i = 0 \) kN/m
Permanent point load 1 (+ve down) \( W_1d = 240 \) kN
Variable point load 1 \( W_1i = 180 \) kN
Distance from L.H. supp to load 1 \( x_1 = 1.5 \) m

457 x 191 x 67 UKB
Dimensions (mm): \( h = 453.4 \) b = 189.9 \( t_w = 8.5 \) tf = 12.7 \( r = 10.2 \)
Properties (cm): \( I_y = 29400 \) \( I_z = 1450 \) \( W_{ply} = 1470 \) \( W_{plz} = 237 \) \( I_t = 37.1 \)

SECTION 457 x 191 x 67 UKB Grade S 275
DESIGN

Maximum moment 214.31 kNm
Moment resistance 363.42 kNm
Unity factor bending 0.58971 \( \leq 1 \)
MAXIMUM SHEAR FORCE 429 kN
Shear plastic resistance 649.94 kN
Unity factor shear 0.66006 \( \leq 1 \)
Deflection to span ratio 1: 5987
Location: Example 3, 'Steelwork Design Guide'

\[ w = wd + wi \]

Simply supported steel
beam restrained at ends

Calculations are in accordance with BS5950-1:2000 Clause 4.3.5.1. Beam has no intermediate lateral restraints.

Beam span \( L = 9 \text{ m} \)

686 x 254 x 125 UB.

Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Dead UDL (including S.W) \( wd = 10 \text{ kN/m} \)

Imposed UDL \( wi = 15 \text{ kN/m} \)

Length between restraints \( LT = 9 \text{ m} \)

Permanent load \( wp = 16 \text{ kN/m} \)

**SECTION**

686 x 254 x 125 UKB Grade S 275

Shear force \( 171 \text{ kN} \)

**DESIGN**

Shear capacity \( 1261.1 \text{ kN} \)

Applied moment \( 384.75 \text{ kNm} \)

Moment capacity \( 1057.3 \text{ kNm} \)

Buckling resistance \( 480.33 \text{ kNm} \)

Moment factor mLT \( 0.925 \)

Resistance (Mb/mLT) \( 519.27 \text{ kNm} \)

Deflection \( 5.2974 \text{ mm} \)

Limiting deflection \( 25 \text{ mm} \)

DL shear at A \( 45 \text{ kN} \)

LL shear at A \( 67.5 \text{ kN} \)

DL shear at B \( 45 \text{ kN} \)

LL shear at B \( 67.5 \text{ kN} \)

**FIRE RESISTANCE**

Heated perimeter \( 2091.4 \text{ mm}^2 \)

Area of section \( 159 \text{ cm}^2 \)

Section factor \( 130 \)

Load Ratio (R) \( 0.45236 \)

Section does not have an inherent fire resistance of 30 minutes.

Fire protection should be provided.
Location: Example 2

\[ w = w_d + w_i \]

**Simply supported steel**

**beam restrained at ends**

Calculations are in accordance with BS5950-1:2000 Clause 4.3.5.1. Beam has no intermediate lateral restraints.

Beam span \( L = 5.7 \text{ m} \)

457 x 152 x 52 UB.

Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Dead UDL (including S.W) \( w_d = 9.6 \text{ kN/m} \)

Imposed UDL \( w_i = 6.25 \text{ kN/m} \)

Length between restraints \( LT = 5.7 \text{ m} \)

<table>
<thead>
<tr>
<th>SECTION</th>
<th>457 x 152 x 52 UKB Grade S 275</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESIGN</td>
<td>Shear force 66.804 kN</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Shear capacity 564.05 kN</td>
</tr>
<tr>
<td></td>
<td>Applied moment 95.196 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment capacity 302.5 kNm</td>
</tr>
<tr>
<td></td>
<td>Buckling resistance 124.83 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment factor mLT 0.925</td>
</tr>
<tr>
<td></td>
<td>Resistance (Mb/mLT) 134.95 kNm</td>
</tr>
<tr>
<td></td>
<td>Deflection 1.9582 mm</td>
</tr>
<tr>
<td></td>
<td>Limiting deflection 15.833 mm</td>
</tr>
</tbody>
</table>

**Unfactored end shears**

- DL shear at A 27.36 kN
- LL shear at A 17.813 kN
- DL shear at B 27.36 kN
- LL shear at B 17.813 kN
Location: Example using EC3 (with FR check)

w = wd + wi

Simply supported beam

restrained at ends

wd=permanent load

wi=variable load

Beam span L=5.7 m

Permanent UDL (including S.W) wd=9.6 kN/m

Variable UDL wi=6.25 kN/m

457 x 152 x 52 UKB

Dimensions (mm): h=449.8 b=152.4 tw=7.6 tf=10.9 r=10.2

Properties (cm): Iy=21400 Iz=645 Wply=1100 Wplz=133 It=21.4

Lateral torsional buckling

Critical moment Mcr=C1*Et*Sr/10^6=139.86 kNm

Design buckling resistance moment MbRd=ChiLT*Wply*fy/10^3=121.5 kNm

Unity factor unitb=MyEd/MbRd=0.71446

Section chosen is considered suitable.

SECTION 457 x 152 x 52 UKB Grade S 275

DESIGN

Maximum moment 86.809 kNm

SUMMARY

Moment resistance 302.5 kNm

Unity factor bending 0.28697 ≤ 1

Maximum shear force 60.919 kN

Shear plastic resistance 578.38 kN

Unity factor shear 0.10533 ≤ 1

Deflection to span ratio 1: 2982

Buckling resistance BM 121.5 kNm

Unity factor buckling 0.71446 ≤ 1

Fire resistance without applied protection

The critical temperature method given in BS EN 1993-1-2 Clause 4.2.4 will be adopted. For further clarification refer to thomas telford publication entitled 'Designer's guide to EN 1991-1-2, EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2'.
Factor for variable actions $p_1$  $psi_1=0.5$
Maximum moment $MyEd=M=51.679$ kNm
Maximum shear force $VzEd=Vb=36.266$ kN

Section classification

Classify outstand element of compression flange:
As $c/tf \leq 9e$ (5.7064 $\leq$ 7.0718), outstand element of compression flange is classified as Class 1 plastic.
Classify web element of section:
As $C/tw \leq 72e$ (53.632 $\leq$ 56.574), web element in bending is classified as Class 1 plastic.

Critical temperature of carbon structural steel

Design moment (fire limit state) $MfiEd=MyEd=51.679$ kNm
Design buckling resistance $MbRdfi=ChiLTf*McRd=88.637$ kNm
Design moment resistance
$MfiRd=ky*(gamM0/gamfi)*MbRdfi/(k1 *k2)=126.62$ kNm
Critical temperature $\theta_{acr}$ $taCR=39.19*LOG(comp-1)+482=616.68$ °C
Fire resistance period 19 minutes

FIRE RESISTANCE
SUMMARY
The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Location: Example 2, 'Steelwork Design Guide'

Simply supported steel beam subject to lateral torsional buckling

Calculations are in accordance with BS5950-1:2000. Simple beam with intermediate lateral restraints in accordance with Clause 4.3.5.2.

The n'th point load is shown:

\[ W_n = W_{nd} + W_{ni} \]

where,

- \( W_{nd} \) = dead pint load
- \( W_{ni} \) = imposed point load

Beam span \( L = 9 \) m

457 x 191 x 89 UB.

Young's Modulus \( E = 205 \) kN/mm²

Dead UDL (including S.W) \( w_d = 3 \) kN/m

Imposed UDL \( w_i = 0 \) kN/m

Dead point load 1 (+ve down) \( W_{1d} = 40 \) kN

Imposed point load 1 \( W_{1i} = 60 \) kN

Distance from L.H. supp to load 1 \( x_1 = 3 \) m

Dead point load 2 \( W_{2d} = 20 \) kN

Imposed point load 2 \( W_{2i} = 30 \) kN

Distance from L.H. supp to load 2 \( x_2 = 6 \) m

Maximum moment on segment \( M_e = 417.8 \) kNm

Far end BM \( \beta M = 341.8 \) kNm

Permanent load \( w_p = 3 \) kN/m

Permanent load \( W_{1p} = 65 \) kN

Permanent load \( W_{2p} = 35 \) kN

SECTION 457 x 191 x 89 UKB Grade S 275

Shear force 145.57 kN

Shear capacity 773.65 kN

Max. applied moment 417.8 kNm

Moment capacity 532.65 kNm

Buckling resistance 428.61 kNm

Moment factor (mLT) 0.92724

Resistance (Mb/mLT) 462.25 kNm

Deflection 13.853 mm

Limiting deflection 45 mm

Unfactored DL shear at A 46.833 kN

Unfactored LL shear at A 50 kN

Unfactored DL shear at B 40.167 kN

Unfactored LL shear at B 40 kN

SCALE 5.48 Office 1007 Proforma 436
FIRE RESISTANCE
SUMMARY

Heated perimeter 1673.4 mm²
Area of section 114 cm²
Section factor 145
Load Ratio (R) 0.56247

Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided.
Location: Ex1 - Default example (with FR check)

Simply supported beam

\[ W_n = \text{permanent point load} + \text{variable point load} \]

Beam span \( L = 10 \text{ m} \)

Permanent UDL (including S.W) \( \text{wd} = 1.44 \text{ kN/m} \)
Variable UDL \( \text{wi} = 0 \text{ kN/m} \)
Permanent point load 1 (+ve down) \( W_{1d} = 140 \text{ kN} \)
Variable point load 1 \( W_{1i} = 85 \text{ kN} \)
Distance from L.H. supp to load 1 \( x_1 = 5 \text{ m} \)

Dimensions (mm): \( h = 539.5 \text{ b=210.8 tw=11.6 tf=18.8 r=12.7} \)
Properties (cm): \( I_y = 66800 \text{ Iz=2940 Wply=2830 Wplz=436 It=126} \)

Coefficient C1

The beam is restrained at intervals by other steel members.
The member is not loaded between restraints (only beam self weight).
Eff.length between restraints \( LT = 5 \text{ m} \)

Lateral torsional buckling

Critical moment \( M_c = C_1 \times E_t \times S_r / 10^6 = 1427.8 \text{ kNm} \)
Design buckling resistance moment \( M_{bRd} = C_1 \times L \times W_{ply} \times f_y / 10^3 = 832.35 \text{ kNm} \)
Section chosen is considered suitable.

SECTION 533 x 210 x 109 UKB Grade S 355
DESIGN
SUMMARY
Maximum moment 778.75 \text{ kNm}
Moment resistance 976.35 \text{ kNm}
Unity factor bending 0.79761 \leq 1
Maximum shear force 160.25 \text{ kN}
Shear plastic resistance 1328.5 \text{ kN}
Unity factor shear 0.12063 \leq 1
Maximum BM on segment 778.75 \text{ kNm}
Buckling resistance BM 832.35 \text{ kNm}
Unity factor buckling 0.93561 \leq 1
Deflection to span ratio 1: 792
Fire resistance without applied protection

The critical temperature method given in BS EN 1993-1-2 Clause 4.2.4 will be adopted. For further clarification refer to Thomas Telford publication entitled 'Designer's guide to EN 1991-1-2, EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2'.

Factor for variable actions $\psi_l = 0.5$

Maximum moment $M_{yEd} = M = 474.25$ kNm

Maximum shear force $V_{zEd} = V_b = 98.45$ kN

Section classification

Classify outstanding element of compression flange:
As $c/t_f \leq 9e$ (4.6223 $\leq$ 6.3137), outstanding element of compression flange is classified as Class 1 plastic.

Classify web element of section:
As $C/t_w \leq 72e$ (41.078 $\leq$ 50.51), web element in bending is classified as Class 1 plastic.

Critical temperature of carbon structural steel

Design moment (fire limit state) $M_{fiEd} = M_{yEd} = 474.25$ kNm

Design buckling resistance $M_{bRdfi} = \chi_{LTf} * M_{crd} = 563.46$ kNm

Design moment resistance $M_{fiRd} = k_y (\gamma_{M0} / \gamma_{fi}) * M_{bRdfi} / (k_1 * k_2) = 804.94$ kNm

Critical temperature $\theta_{acr}$ $t_aCR = 39.19 \times \log(\text{comp} - 1) + 482 = 557.43^\circ$C

Fire resistance period 17 minutes

FIRE RESISTANCE SUMMARY

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Simply supported steel beam

Calculations are in accordance with BS5950-1:2000 and 'Design Examples to BS5950' published by British Steel Tubes Division. The beam is assumed to be a square or rectangular hollow section.

\[ w = wd + wi \]

The n'th point load is shown:

\[ W_n = W_{nd} + W_{ni} \]

where,

\[ W_{nd} = \text{dead point load} \]
\[ W_{ni} = \text{imposed point load} \]

Beam span \( L = 5 \text{ m} \)

250 x 150 x 8 RHS - Hot finished.

Properties (cm): \( A = 60.8 \), \( r_x = 9.17 \), \( Z_x = 409 \), \( S_x = 501 \), \( I_x = 5110 \)

\( J = 5020 \), \( C = 506 \), \( Z_y = 306 \), \( S_y = 350 \), \( I_y = 2300 \), \( r_y = 6.15 \)

Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Dead UDL (including S.W) \( wd = 2 \text{ kN/m} \)

Imposed UDL \( wi = 3 \text{ kN/m} \)

Dead point load 1 (+ve down) \( W_{1d} = 40 \text{ kN} \)

Imposed point load 1 \( W_{1i} = 40 \text{ kN} \)

Distance from L.H. supp to load 1 \( x_1 = 2.5 \text{ m} \)

Check section for combined moment and shear

Maximum moment and co-existent shear

Shear capacity \( P_v = 809.4 \text{ kN} \)

Design shear force \( F_v = 60 \text{ kN} \)

Moment capacity for compact sec \( M_c = p_y S_x / 10^3 = 177.86 \text{ kNm} \)

Reduce \( M_c \) to limiting value \( M_c = 1.2 p_y Z_x / 10^3 = 174.23 \text{ kNm} \)

Since \( M \leq M_c \) \( (173.75 \text{ kNm} \leq 174.23 \text{ kNm} \) applied moment within moment capacity.

Limiting value of slenderness - Clause 4.3.6.1

Dist. betwn torsional restraints \( L_y = 5 \text{ m} \)
Effective length (Table 13) \( Le = 0.8 \times Ly \times 1000 = 4000 \) mm
Slenderness (minor axis) \( \lambda = \frac{Le}{(ry \times 10)} = 65.041 \)
Side ratio D/B (Table 15) \( D'B = D/B = 1.6667 \)
Limiting slenderness \( \lambda_{ml} = \frac{d'b \times 275}{py} = 338.19 \)
As actual slenderness is less than limiting slenderness
Lateral Torsional Buckling is not critical and no further check is required.

**Fire resistance of beam**

Calculations in accordance with BS 5950-8:2003 and 'Fire Resistant Design of Steel Structures' published by SCI.

**Fire resistance without applied protection**

The design temperature \( \text{temp} = \text{TABLE 10 for } T = 16.4, \text{ column } = 30 = 731.8 \) deg.C.
The load ratio, \( R \), at the fire limit state is to be calculated from:

\[
R = \frac{\text{Moment at the fire limit state}}{\text{Moment capacity at 20 deg.C.}}
\]

Permanent load \( wp = 3 \text{ kN/m} \)
Permanent load \( Wlp = 40 \text{ kN} \)
Moment capacity \( Mc = 174.23 \text{ kNm} \)
Load ratio \( R = \frac{Mf}{Mc} = 0.59905 \)

**HOT FINISHED**

**RECTANGULAR HOLLOW SECTION** 250 x 150 x 8 RHS Grade S 355

**HOLLOW SECTION**

**SUMMARY**

Unfactored end shears

**FIRE RESISTANCE SUMMARY**

Exposed surface area \( 650 \text{ mm}^2 \)
Area of section \( 60.8 \text{ cm}^2 \)
Section factor \( 105 \)
Load Ratio (R) \( 0.59905 \)
Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided.
Location: Moment applied about minor axis

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000 and 'Design Examples to BS5950' published by British Steel Tubes Division. The beam is assumed to be a square or rectangular hollow section.

[Diagram: \[ w = wd + wi \]

The n'th point load is shown:

\[ Wn = Wnd + Wni \]

where,

\[ Wnd = \text{dead point load} \]
\[ Wni = \text{imposed point load} \]

Beam span \( L = 5 \text{ m} \)

200 x 400 x 8 RHS - Cold formed.

Properties (cm): \( A = 91.2 \) \( rx = 8.45 \) \( Zx = 652 \) \( Sx = 728 \) \( Ix = 6520 \)

\( J = 15800 \) \( C = 1130 \) \( Zy = 949 \) \( Sy = 1170 \) \( Iy = 19000 \) \( ry = 14.4 \)

Young's Modulus \( E = 205 \text{ kN/mm}\^2 \)

Dead UDL (including S.W) \( wd = 2 \text{ kN/m} \)

Imposed UDL \( wi = 3 \text{ kN/m} \)

Dead point load 1 (+ve down) \( Wld = 30 \text{ kN} \)

Imposed point load 1 \( Wli = 30 \text{ kN} \)

Distance from L.H. supp to load 1 \( x1 = 2.5 \text{ m} \)

Check section for combined moment and shear

Maximum moment and co-existent shear

Shear capacity \( P_v = 647.52 \text{ kN} \)

Design shear force \( F_v = F = 45 \text{ kN} \)

Moment capacity based on effective elastic section

Since \( F_v < 0.6 P_v \) \( M_c = p_y * Z_{eff} / 10^3 = 179.38 \text{ kNm} \)

Since \( M \leq M_c \) (136.25 kNm ≤ 179.38 kNm) applied moment within moment capacity.
<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
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<tbody>
<tr>
<td>COLD FORMED</td>
<td>In accordance with EN 10219</td>
</tr>
<tr>
<td>RECTANGULAR HOLLOW SECTION</td>
<td>200 x 400 x 8 RHS Grade S 355</td>
</tr>
<tr>
<td>HOLLOW</td>
<td>Applied moment 136.25 kNm</td>
</tr>
<tr>
<td>SECTION</td>
<td>Moment capacity 179.38 kNm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Deflection 7.6716 mm</td>
</tr>
<tr>
<td></td>
<td>Limiting deflection 25 mm</td>
</tr>
<tr>
<td></td>
<td>DL shear at A 20 kN</td>
</tr>
<tr>
<td>Unfactored end shears</td>
<td>LL shear at A 22.5 kN</td>
</tr>
<tr>
<td></td>
<td>DL shear at B 20 kN</td>
</tr>
<tr>
<td></td>
<td>LL shear at B 22.5 kN</td>
</tr>
</tbody>
</table>
Simply supported steel beam

Calculations are in accordance with BS5950-1:2000 and 'Design Examples to BS5950' published by British Steel Tubes Division. The beam is assumed to be a square or rectangular hollow section.

\[ w = w_d + w_i \]

\[ W_n = W_{nd} + W_{ni} \]

where,

\[ W_{nd} = \text{dead point load} \]

\[ W_{ni} = \text{imposed point load} \]

Beam span \( L = 12 \text{ m} \)

457 dia x 8 thick CHS - Cold formed.

Properties (cm): \( A = 112.85 \) \( r = 15.877 \)

\[ Z = Z_x = 1244.9 \quad S = S_x = 1613 \quad I = I_x = 28446 \]

Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Dead UDL (including S.W) \( w_d = 2 \text{ kN/m} \)

Imposed UDL \( w_i = 0 \text{ kN/m} \)

Dead point load 1 (+ve down) \( W_{1d} = 30 \text{ kN} \)

Imposed point load 1 \( W_{1i} = 30 \text{ kN} \)

Distance from L.H. supp to load 1 \( x_1 = 3 \text{ m} \)

Dead point load 2 \( W_{2d} = 30 \text{ kN} \)

Imposed point load 2 \( W_{2i} = 30 \text{ kN} \)

Distance from L.H. supp to load 2 \( x_2 = 9 \text{ m} \)

Check section for combined moment and shear

Maximum moment and co-existent shear

Shear capacity \( P_v = 1442.2 \text{ kN} \)

Design shear force \( F_v = F = 0 \text{ kN} \)

Moment capacity based on effective plastic modulus

Since \( F_v < 0.6 P_v \)

\[ M_c = py \times (S_x \text{eff})/10^3 = 515.26 \text{ kNm} \]

Since \( M \leq M_c \) ( 320.4 kNm \( \leq 515.26 \text{ kNm} \) ) applied moment within moment capacity.
<table>
<thead>
<tr>
<th>COLD FORMED</th>
<th>In accordance with EN 10219</th>
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</thead>
<tbody>
<tr>
<td>CIRCULAR HOLLOW SECTION</td>
<td>457 O.D. x 8 CHS Grade S 355</td>
</tr>
<tr>
<td>HOLLOW</td>
<td>Applied moment 320.4 kNm</td>
</tr>
<tr>
<td>SECTION</td>
<td>Moment capacity 515.26 kNm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Deflection 25.465 mm</td>
</tr>
<tr>
<td></td>
<td>Limiting deflection 60 mm</td>
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<tr>
<td>Unfactored end shears</td>
<td>DL shear at A 42 kN</td>
</tr>
<tr>
<td></td>
<td>LL shear at A 30 kN</td>
</tr>
<tr>
<td></td>
<td>DL shear at B 42 kN</td>
</tr>
<tr>
<td></td>
<td>LL shear at B 30 kN</td>
</tr>
</tbody>
</table>
Hollow section

.simply supported beam

Point load n is shown
\[ W_n = W_{nd} + W_{ni} \]
where,
\[ W_{nd} = \text{permanent point load} \]
\[ W_{ni} = \text{variable point load} \]

Beam span \( L = 10 \text{ m} \)

Permanent UDL (including S.W) \( w_d = 1.44 \text{ kN/m} \)
Variable UDL \( w_i = 0 \text{ kN/m} \)
Permanent point load 1 (+ve down) \( W_{1d} = 100 \text{ kN} \)
Variable point load 1 \( W_{1i} = 45 \text{ kN} \)
Distance from L.H. supp to load 1 \( x_1 = 5 \text{ m} \)
400 x 400 x 12.5 SHS - Hot finished.
Properties (cm): \( A = 192 \) \( i_y = 15.8 \)
\( W_{ely} = 2390 \) \( W_{ply} = 2780 \) \( I_y = 47800 \) \( I_t = 73900 \)
\( W_t = 3530 \)

Moment resistance

\[ \text{Moment resistance} \quad M_{Cr} = W_{ply} \times f_y / 10^3 = 986.9 \text{ kNm} \]
\[ \text{Unity factor} \quad \text{unity} = M_{Ed} / M_{Cr} = 0.51044 \]
Section chosen is suitable.

Shear plastic resistance

\[ \text{Shear plastic resistance} \quad V_{plRd} = A_{vy} \times f_y / \sqrt{3} / 10^3 = 1967.6 \text{ kN} \]
\[ \text{Unity factor} \quad \text{Unity} = V_{zEd} / V_{plRd} = 0.053491 \]
Section chosen is suitable.
Note: No M-V interaction has to be considered as since the maximum moment occurs near the mid-span and the maximum shear force is obtained at the supports.

Coefficient \( C_1 \)

The beam is simply supported and is not restrained at point loads.
Effective length parameter \( K = 1.0 \)
Length for LTB \( L_T = 10 \text{ m} \)
Coefficient \( C_1 = 1.13 \)

Lateral torsional buckling

\[ \text{Critical moment} \quad M_{cr} = C_1 \times E_t \times G_t^{0.5} / 10^6 = 27479 \text{ kNm} \]
As \( \lambda a_{LT} \leq \lambda a_{LT0} \),
The segment of the beam is not subject to lateral torsional buckling.
Design buckling resistance moment \( M_{bRd} = W_{ply} \times f_y / 10^3 = 986.9 \text{ kNm} \)
Unity factor \( \frac{\text{unitb}}{\text{MyEd/MbRd}} = 0.51044 \)

Section chosen is suitable.

**HOT FINISHED SQUARE HOLLOW SECTION**

- DESIGN
  - Maximum moment: 503.75 kNm
  - Moment resistance: 986.9 kNm
  - Unity factor bending: \( 0.51044 \leq 1 \)
  - Maximum shear force: 105.25 kN
  - Shear plastic resistance: 1967.6 kN
  - Unity factor shear: \( 0.053491 \leq 1 \)
  - Deflection/span ratio: 1071
  - Buckling resistance: 986.9 kNm

**Fire resistance without applied protection**

The critical temperature method given in BS EN 1993-1-2 Clause 4.2.4 will be adopted. For further clarification refer to Thomas Telford publication entitled 'Designer's guide to EN 1991-1-2, EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2'.

- Factor for variable actions \( \psi_1 \)
  - \( \psi_1 = 0.5 \)
- Maximum moment
  - MyEd = M = 324.25 kNm
- Maximum shear force
  - VzEd = Vb = 68.45 kN

**Section classification**

- Internal part subject to compression (flange): As \( C/t \leq 42e \left( 29 \leq 29.046 \right) \), internal element in compression is classified as Class 3 semi-compact.
- Internal part subject to bending (web): As \( C/t \leq 72e \left( 29 \leq 49.793 \right) \), web element in bending is classified as Class 1 plastic.

Section classification is therefore Class 3.

**Critical temperature of carbon structural steel**

- Design moment (fire limit state)
  - MfiEd = MyEd = 324.25 kNm
- Design buckling resistance
  - MbRdfi = ChiLTf*McRd = 768.81 kNm
- Design moment resistance
  - MfiRd = ky*(gamM0/gamfi)*MbRdfi/(k1 *k2) = 1098.3 kNm
- Critical temperature \( \theta_{acr} \)
  - \( \theta_{acr} = 39.19 \times \text{LOG(comp-1)} + 482 = 666.21 ^\circ \text{C} \)
- Fire resistance period
  - 22 minutes

**FIRE RESISTANCE SUMMARY**

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Location: Example RHS

Hollow section
simply supported beam

Point load n is shown
WN = Wn + Wni
where,
Wn = permanent point load
Wni = variable point load

Beam span L = 5 m

Permanent UDL (including S.W) Wd = 3 kN/m
Variable UDL Wi = 3 kN/m
Permanent point load 1 (+ve down) W1d = 40 kN
Variable point load 1 W1i = 50 kN
Distance from L.H. supp to load 1 x1 = 2.5 m
250 x 150 x 16 RHS - Hot finished.
Properties (cm): A = 115 iz = 5.8 Wely = 710 Wply = 906 Iy = 8880
It = 8870 Wt = 849 Welz = 516 Wplz = 625 Iz = 3870 iy = 8.79

Moment resistance

Moment resistance Mrd = Wply * fy / 10^3 = 321.63 kNm
Unity factor unity = MyEd / Mrd = 0.56596
Section chosen is suitable.

Shear plastic resistance

Shear plastic resistance VplRd = Avz * fy / SQR(3) / 1000 = 1473.1 kN
Unity factor Unity = VzEd / VplRd = 0.056427
Section chosen is suitable.
Note: No M-V interaction has to be considered as since the maximum
moment occurs near the mid-span and the maximum shear force
is obtained at the supports.

Coefficient C1

The beam is assumed to be restrained at point loads.
Length for LTB LT = 2.5 m
Eff.length between restraints LT = 2.5 m
Coefficient C1 = TABLE 3.1 for psi = 0
= 1.77
**Lateral torsional buckling**

Critical moment: \[ \text{M}_{cr} = C_1 \times E_t \times G_t^{0.5}/10^6 = 16972 \text{ kNm} \]

As \( \text{lam}_{LT} \leq \text{lam}_{LT0} \),

The segment of the beam is not subject to lateral torsional buckling.

Design buckling resistance moment: \[ \text{M}_{br} = W_{ply} \times f_y/10^3 = 321.63 \text{ kNm} \]

Unity factor: \[ \text{unit}_b = \frac{M_y E_d}{M_{br}} = 0.56596 \]

Section chosen is suitable.

Stiff bearing length: \( b_1 = 75 \text{ mm} \)

**HOT FINISHED**

**RECTANGULAR HOLLOW SECTION**

**DESIGN**

- Maximum moment: 182.03 kNm
- Moment resistance: 321.63 kNm
- Unity factor bending: 0.56596 ≤ 1
- Maximum shear force: 83.125 kN
- Shear plastic resistance: 1473.1 kN
- Unity factor shear: 0.056427 ≤ 1
- Deflection/span ratio: 603
- Buckling resistance: 321.63 kNm
- Web resistance: 1215.5 kN

**In accordance with EN 10210**

- 250 x 150 x 16 RHS
- Grade S 355
Location: Cold formed CHS

Hollow section

simply supported beam

Point load n is shown

\[ W_n = W_{nd} + W_{ni} \]

where,

- \( W_{nd} \) = permanent point load
- \( W_{ni} \) = variable point load

Beam span \( L = 10 \) m

Permanent UDL (including S.W) \( wd = 1.44 \) kN/m
Variable UDL \( wi = 1 \) kN/m
219.1 dia x 4.5 thick CHS - Cold formed.
Properties (cm): \( A = 30.338 \) iy = 7.5889 \( W_{ely} = 159.49 \) \( W_{ply} = 207.27 \) \( I_y = 1747.2 \) \( I_t = 3494.5 \)

Moment resistance

Moment resistance \( M_{Rd} = W_{ely} \cdot f_y / 10^3 = 56.62 \) kNm
Unity factor \( \text{unity} = M_{Ed} / M_{Rd} = 0.72854 \)
Section chosen is suitable.

Shear plastic resistance

Shear plastic resistance \( V_{plRd} = A v_z \cdot f_y / \sqrt{3} / 1000 = 395.86 \) kN
Unity factor \( \text{unity} = V_{zEd} / V_{plRd} = 0.041682 \)
Section chosen is suitable.

Note: No M-V interaction has to be considered as since the maximum moment occurs near the mid-span and the maximum shear force is obtained at the supports.

Coefficient C1

The beam is simply supported and is not restrained at point loads.
Effective length parameter \( K = 1.0 \)
Length for LTB \( LT = 10 \) m
Coefficient \( C_1 = 1.13 \) (based on UDL)

Lateral torsional buckling

Critical moment \( M_{cr} = C_1 \cdot E_t \cdot G_t \cdot 0.5 / 10^6 = 1142.4 \) kNm
As \( \lambda_{LT} \leq \lambda_{LT0} \), the segment of the beam is not subject to lateral torsional buckling.
Design buckling resistance moment \( M_{bRd} = W_{ely} \cdot f_y / 10^3 = 56.62 \) kNm
Unity factor \( \text{unitb} = M_{Ed} / M_{bRd} = 0.72854 \)
Section chosen is suitable.
<table>
<thead>
<tr>
<th>COLD FORMED</th>
<th>In accordance with EN 10219</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIRCULAR HOLLOW SECTION</td>
<td>219.1 O.D. x 4.5 CHS Grade S 355</td>
</tr>
<tr>
<td>DESIGN</td>
<td>Maximum moment</td>
</tr>
<tr>
<td></td>
<td>41.25 kNm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Moment resistance</td>
</tr>
<tr>
<td></td>
<td>56.62 kNm</td>
</tr>
<tr>
<td></td>
<td>Unity factor bending</td>
</tr>
<tr>
<td></td>
<td>0.72854 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>Maximum shear force</td>
</tr>
<tr>
<td></td>
<td>16.5 kN</td>
</tr>
<tr>
<td></td>
<td>Shear plastic resistance</td>
</tr>
<tr>
<td></td>
<td>395.86 kN</td>
</tr>
<tr>
<td></td>
<td>Unity factor shear</td>
</tr>
<tr>
<td></td>
<td>0.041682 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>Deflection/span ratio</td>
</tr>
<tr>
<td></td>
<td>282</td>
</tr>
<tr>
<td></td>
<td>Buckling resistance</td>
</tr>
<tr>
<td></td>
<td>56.62 kNm</td>
</tr>
</tbody>
</table>
**Location: Example 6 'Structural Steelwork Design'**

Calculations are in accordance with BS 5950 Part 1:2000 and follow Ex 6 of 'Structural Steelwork Design' by L.J.Morris & D.R.Plum.

**Purlin on sloping roof**

Purlin centres  
**crs=2 m**  

Purlin span (rafter centres)  
**L=6 m**

Angle of rafter with horizontal  
**theta=20°**

Dead load (cladding+insulation)  
**wd=0.15 kN/m²**

Imposed load (on plan)  
**wi=0.75 kN/m²**

Wind load (suction)  
**ww=0.4 kN/m²**

152 x 76 Tapered Channel.

Young's Modulus  
**E=205 kN/mm²**

Unity factor  
**uf=mx*(-Mx)/(py*Zx/10^3)+my*My/(py*Zy/10^3) =0.21342**

**Check for deflection under serviceability loads**

Total deflection  
**DEL=5*Wix*L^3*10^5/(384*E*Ix)**

=12.797 mm

Limiting deflection  
**DELlim=L*1000/200=30 mm**

Since **DEL ≤ DELlim ( 12.797 mm ≤ 30 mm )**, deflection within limiting value.

**PURLIN SECTION SUMMARY**

152 x 76  
Tapered flange Channel Grade S 275  
Section is satisfactory for bending, shear, combined bending & shear, and deflection.
Location: Ex1 - Roof purlin with wind suction load using UKPFC

Calculations are in accordance with EC3 Part 1-1:2005.

\[ L = \text{purlin span} \]
\[ \text{crs} = \text{purlin centres} \]
\[ \theta = \text{rafter slope} \]

Purlin centres \( \text{crs} = 2 \text{ m} \)
Purlin span (rafter centres) \( L = 6 \text{ m} \)
Angle of rafter with horizontal \( \theta = 20^\circ \)
Permanent load (cladding+insulation) \( g_k = 0.15 \text{ kN/m}^2 \)
Variable load (on plan) \( q_k = 0.75 \text{ kN/m}^2 \)
Wind suction (input as positive) \( w_k = 0.4 \text{ kN/m}^2 \)

150 x 75 UKPFC

The rafter slope \( \theta = 20^\circ \) results in the purlins at the same angle. Components of load are used to calculate moments about \( yy \) & \( zz \) axes, i.e. normal and tangential to the rafter. It would be possible to ignore bending in the plane of the cladding, but in practice, biaxial bending is usually considered in purlin design.

Length for LTB \( LT = 6 \text{ m} \)
Equivalent uniform moment factor \( C_{my} = 1 \)
Equivalent uniform moment factor \( C_{mz} = 1 \)

DESIGN 150 x 75 UKPFC
SUMMARY Parallel flange Channel Grade S 275
Section is satisfactory for bending, shear, combined bending & shear, and deflection.
Location: Ex2 - Roof purlin w. wind suction load, Tapered flange

Purlin on sloping roof

Calculations are in accordance with EC3 Part 1-1:2005.

L = purlin span

crs = purlin centres

theta = rafter slope

Purlin centres crs=2 m
Purlin span (rafter centres) L=6 m
Angle of rafter with horizontal theta=20°
Permanent load (cladd+insulation) gk=0.15 kN/m²
Variable load (on plan) qk=0.75 kN/m²
Wind suction (input as positive) wk=0.4 kN/m²

152 x 76 Tapered Channel.

The rafter slope theta=20 degrees results in the purlins at the same angle. Components of load are used to calculate moments about yy & zz axes, i.e. normal and tangential to the rafter. It would be possible to ignore bending in the plane of the cladding, but in practice, biaxial bending is usually considered in purlin design.

Length for LTB LT=6 m
Equivalent uniform moment factor Cmy=1
Equivalent uniform moment factor Cmz=1

DESIGN
152 x 76
Tapered flange Channel Grade S 275
Section is satisfactory for bending, shear, combined bending & shear, and deflection.

SUMMARY
Location: Ex3 - Roof purlin user defined section

Cladding+Insulation

<table>
<thead>
<tr>
<th>Purlin on sloping roof</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculations are in accordance with EC3 Part 1-1:2005.</td>
</tr>
<tr>
<td>L = purlin span</td>
</tr>
<tr>
<td>crs = purlin centres</td>
</tr>
<tr>
<td>theta = rafter slope</td>
</tr>
</tbody>
</table>

Purlin centres crs=2 m
Purlin span (rafter centres) L=6 m
Angle of rafter with horizontal theta=20°
Permanent load (clad+insulation) gk=0.15 kN/m²
Variable load (on plan) qk=0.75 kN/m²
Wind suction (input as positive) wk=0.4 kN/m²

The rafter slope theta=20 degrees results in the purlins at the same angle. Components of load are used to calculate moments about yy & zz axes, i.e. normal and tangential to the rafter. It would be possible to ignore bending in the plane of the cladding, but in practice, biaxial bending is usually considered in purlin design.

Length for LTB LT=6 m
Equivalent uniform moment factor Cmy=1
Equivalent uniform moment factor Cmz=1

DESIGN 150 x 75 User defined channel section
tf=10 mm tw=5.5 mm Grade S 275
Section is satisfactory for bending, shear, combined bending & shear, and deflection.

SUMMARY tf=10 mm tw=5.5 mm Grade S 275
Location: Example 7 in Structural Steelwork Design

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Side rail

Calculations are in accordance with BS5950-1: 2000 and follow Example 7 of 'Structural Steelwork Design' by L.J. Morris and D.R. Plum.

X-X = major axis

Y-Y = minor axis

---

Side rail span \( L=5 \text{ m} \)
Side rail centres \( \text{crs}=2 \text{ m} \)
Characteristic wind load \( \text{ww}=0.8 \text{ kN/m}^2 \)
125 x 75 x 10 mm Unequal Angle.
Young's Modulus \( E=205 \text{ kN/mm}^2 \)

Loading

Wind load \( W_w=\text{crs} \times L \times \text{ww}=8 \text{ kN} \)
Ultimate wind load \( W_x=1.4 \times W_w=11.2 \text{ kN} \)

Applied moment and shear

Max moment occurs at centre \( M_x=W_x \times L/8=7 \text{ kNm} \)
Max shear occurs at ends \( F_x=W_x/2=5.6 \text{ kN} \)

Shear capacity

Shear area (bending about xx) \( A_{vx}=0.9 \times A \times t=1125 \text{ mm}^2 \)
Shear capacity \( P_{vx}=0.6 \times p_y \times A_{vx}/1000=185.63 \text{ kN} \)
Since \( F_x \leq P_{vx} \) (5.6 kN \( \leq 185.63 \text{ kN} \)) shear force in web within shear capacity.

Moment capacity

Moment capacity \( M_{cx}=p_y \times Z/10^3=10.042 \text{ kNm} \)
Since \( M_x \leq M_{cx} \) (7 kNm \( \leq 10.042 \text{ kNm} \)) section is satisfactory.
## Check for deflection under serviceability loads

Total deflection: $DEL = \frac{5 \times Ww \times L^3 \times 10^5}{384 \times E \times I_x} = 21.032\ mm$

Limiting deflection: $DEL_{\text{lim}} = \frac{L}{1000} = 25\ mm$

Since $DEL \leq DEL_{\text{lim}}$ (21.032 mm ≤ 25 mm), deflection within limiting value.

<table>
<thead>
<tr>
<th>SIDE RAIL</th>
<th>125 x 75 x 10 Angle Grade S 275</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>Shear force 5.6 kN</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Shear capacity 185.63 kN</td>
</tr>
<tr>
<td></td>
<td>Applied moment 7 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment capacity 10.042 kNm</td>
</tr>
<tr>
<td></td>
<td>Deflection 21.032 mm</td>
</tr>
<tr>
<td></td>
<td>Limiting deflection 25 mm</td>
</tr>
</tbody>
</table>
Location: Ex1 - Cladding side rail design

Cladding side rail

Calculations are in accordance with EC3 Part 1-1:2005

crs = rail centres

Y-Y = major axis

Z-Z = minor axis

L = column centres = side rail span

Side rail span \( L = 5 \, \text{m} \)

Side rail centres \( \text{crs} = 2 \, \text{m} \)

Characteristic wind load \( w_k = 0.8 \, \text{kN/m}^2 \)

125 x 75 x 10 mm Unequal Angle (Advance UKA)

Partial factors for actions and resistance

Variable action (horizontal wind) \( \gamma_Q = 1.5 \)

Partial factor \( \gamma_{M0} = 1.0 \)

Partial factor \( \gamma_{M1} = 1.0 \)

Design moment and shear

Max moment occurs at centre \( M_{yEd} = \frac{W_y \cdot L}{8} = 7.5 \, \text{kNm} \)

Max shear occurs at ends \( V_{yEd} = \frac{W_y}{2} = 6 \, \text{kN} \)

Design shear resistance

Shear area (bending about y-axis) \( A_v = 0.9 \cdot h \cdot t = 1125 \, \text{mm}^2 \)

Shear resistance \( V_{plRdy} = A_v \cdot (f_y/3^{0.5})/(\gamma_{M0} \cdot 1000) = 178.62 \, \text{kN} \)

Since \( V_{yEd} \leq V_{plRdy} \) ( 6 kN ≤ 178.62 kN ) design shear force in web is within the shear resistance.

Moment resistance

Moment resistance \( M_{crdy} = W_e1 \cdot f_y/(\gamma_{M0} \cdot 10^3) = 10.042 \, \text{kNm} \)

Since \( M_{yEd} \leq M_{crdy} \) ( 7.5 kNm ≤ 10.042 kNm ) section is satisfactory.
Check for deflection under serviceability loads

Total deflection \( \text{DEL} = 5 \times Wk \times L^3 \times 10^8 / (384 \times E \times I_y) = 20.531 \text{ mm} \)

Since \( \text{DEL} \leq \text{DELlim} \) (20.531 mm \( \leq \) 25 mm) deflection OK.

<table>
<thead>
<tr>
<th>SIDE RAIL</th>
<th>125 x 75 x 10 Angle Grade S 275</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESIGN</td>
<td>Design shear force 6 kN</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Shear resistance 178.62 kN</td>
</tr>
<tr>
<td></td>
<td>Design moment 7.5 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment resistance 10.042 kNm</td>
</tr>
<tr>
<td></td>
<td>Deflection 20.531 mm</td>
</tr>
<tr>
<td></td>
<td>Limiting deflection 25 mm</td>
</tr>
</tbody>
</table>
Location: Example 2, 'Steel Designer's Manual' - with restraint

I Section Column

Calculations in accordance with BS5950-1:2000 Sections 4.7 and 4.8.

All loads and moments are factored.

Design moment about z-z axis \( M_z = 530 \) kNm
Design shear force \( F_v = 95 \) kN
Design axial load (+ve comp) \( F = 160 \) kN
Length of member \( L = 5.6 \) m
Design moment about minor axis \( M_y = 0 \) kNm
Design shear force // flanges \( F_{vz} = 0 \) kN

533 x 210 x 82 UB.
Young's Modulus \( E = 205 \) kN/mm²

Length between restraints z-axis \( L_z = 5600 \) mm
Length between restraints y-axis \( L_y = 1600 \) mm
Length between restraints \( L_T = 1.6 \) m
Maximum moment on segment \( M_{LT} = 530 \) kNm
Far end BM \( \beta M = 378.6 \) kNm
Maximum moment on segment \( M_z' = 530 \) kNm
Far end BM \( \beta M_z = 0 \) kNm

UNIVERSAL BEAM

\( 533 \times 210 \times 82 \) UB Grade S 355
SECTION
Section is satisfactory for bending, axial load, local capacity, and overall buckling check.

DESIGN
Axial load (+ve comp) \( 160 \) kN
Compressive resistance \( 3402.2 \) kN
Maximum moment(z-axis) \( 530 \) kNm
Moment resistance \( 731.3 \) kNm
Local capacity check \( 0.76766 \leq 1 \)
Overall buckling check \( 0.48843 \leq 1 \)
\( 0.6948 \leq 1 \)
Location: Ex2 As Ex1 but with reduced axial load - no restraint

I Section Column

Calculations in accordance with BS5950-1:2000 Sections 4.7 and 4.8.

All loads and moments are factored.

Design moment about z-z axis $M_z = 400 \text{ kNm}$
Design shear force $F_v = 95 \text{ kN}$
Design axial load (+ve comp) $F = 160 \text{ kN}$
Length of member $L = 5.6 \text{ m}$
Design moment about minor axis $M_y = 0 \text{ kNm}$
Design shear force // flanges $F_{vz} = 0 \text{ kN}$

533 x 210 x 82 UB.
Young's Modulus $E = 205 \text{ kN/mm}^2$

Length between restraints z-axis $L_z = 5600 \text{ mm}$
Length between restraints y-axis $L_y = 5600 \text{ mm}$
Length between restraints $L_T = 5.6 \text{ m}$
Maximum moment on segment $M_{LT} = 400 \text{ kNm}$
Far end BM $\beta_{M} = 0 \text{ kNm}$
Maximum moment on segment $M_{z'} = 530 \text{ kNm}$
Far end BM $\beta_{Mz} = 0 \text{ kNm}$

UNIVERSAL BEAM 533 x 210 x 82 UB Grade S 355
SECTION Section is satisfactory for bending, axial load, local capacity, and overall buckling check.
SUMMARY

DESIGN Axial load (+ve comp) 160 kN
SUMMARY Compressive resistance 1086.1 kN
Maximum moment(z-axis) 400 kNm
Moment resistance 731.3 kNm
Local capacity check $0.5899 < 1$
Overall buckling check $0.37942 < 1$
$0.99052 < 1$
Location: Semi-compact section

I Section Column

Calculations in accordance with BS5950-1:2000 Sections 4.7 and 4.8.

All loads and moments are factored.

Design moment about z-z axis $M_z=45$ kNm
Design shear force $F_v=23$ kN
Design axial load (+ve comp) $F=525$ kN
Length of member $L=5$ m
Design moment about minor axis $M_y=0$ kNm
Design shear force // flanges $F_{vz}=0$ kN

406 x 140 x 39 UB.
Young's Modulus $E=205$ kN/mm²

Length between restraints z-axis $L_z=5000$ mm
Length between restraints y-axis $L_y=2500$ mm
Length between restraints $L_T=2.5$ m
Maximum moment on segment $M_{LT}=45$ kNm
Far end BM $\beta M=22.5$ kNm
Maximum moment on segment $M_z'=45$ kNm
Far end BM $\beta M_z=0$ kNm

UNIVERSAL BEAM 406 x 140 x 39 UB Grade S 275
SECTION
Section is satisfactory for bending, axial load, local capacity, and overall buckling check.

SUMMARY

DESIGN
Axial load (+ve comp) 525 kN

SUMMARY
Compressive resistance 828.12 kN
Maximum moment (z-axis) 45 kNm
Moment resistance 181.81 kNm
Local capacity check $0.63163 \leq 1$
Overall buckling check $0.57479 \leq 1$
$0.91792 \leq 1$
Location: Biaxial bending

I Section Column

Calculations in accordance with BS5950-1:2000 Sections 4.7 and 4.8.

All loads and moments are factored.

Design moment about z-z axis \( M_z = 230 \text{ kNm} \)
Design shear force \( F_v = 65 \text{ kN} \)
Design axial load (+ve comp) \( F = 1500 \text{ kN} \)
Length of member \( L = 4.3 \text{ m} \)
Design moment about minor axis \( M_y = 87 \text{ kNm} \)
Design shear force // flanges \( F_vz = 0 \text{ kN} \)

305 x 305 x 118 UC.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Length between restraints z-axis \( L_z = 4300 \text{ mm} \)
Length between restraints y-axis \( L_y = 4300 \text{ mm} \)
Length between restraints \( L_T = 4.3 \text{ m} \)
Maximum moment on segment \( M_{LT} = 230 \text{ kNm} \)
Far end BM \( \beta M = -100 \text{ kNm} \)
Maximum moment on segment \( M_z' = 230 \text{ kNm} \)
Far end BM \( \beta M_z = -100 \text{ kNm} \)
Maximum moment on segment \( M_y' = 87 \text{ kNm} \)
Far end BM \( \beta M_y = 23 \text{ kNm} \)
Maximum moment on segment \( M_{maxy} = 87 \text{ kNm} \)
Far end BM \( \beta y = 23 \text{ kNm} \)

UNIVERSAL COLUMN

SECTION 305 x 305 x 118 UC Grade S 355
SUMMARY Section is satisfactory for bending, axial load, local capacity, and overall buckling check.

DESIGN Axial load (+ve comp) \( 1500 \text{ kN} \)
SUMMARY Compressive resistance \( 4442 \text{ kN} \)
Maximum moment(z-axis) \( 230 \text{ kNm} \)
Moment resistance \( 676.2 \text{ kNm} \)
Maximum moment(y-axis) \( 87 \text{ kNm} \)
Moment resistance \( 305.05 \text{ kNm} \)
Local capacity check \( 0.91519 \leq 1 \)
Overall buckling check \( 0.5985 < 1 \)

0.77781 \leq 1
0.69183 \leq 1
Location: Example 2, 'Steel Designer's Manual' - fire check

I Section Column

Calculations in accordance with BS5950-1:2000 Sections 4.7 and 4.8.

All loads and moments are factored.

Design moment about z-z axis $M_z = 530 \text{kNm}$
Design shear force $V = 95 \text{kN}$
Design axial load (positive component) $F = 160 \text{kN}$
Length of member $L = 5.6 \text{ m}$
Design moment about minor axis $M_y = 0 \text{kNm}$
Design shear force parallel to flanges $V_{yz} = 0 \text{kN}$

$533 \times 210 \times 82 \text{ UB}$.
Young's Modulus $E = 205 \text{kN/mm}^2$

Length between restraints z-axis $L_z = 5600 \text{ mm}$
Length between restraints y-axis $L_y = 1600 \text{ mm}$
Length between restraints $L_T = 1.6 \text{ m}$
Maximum moment on segment $M_{LT} = 530 \text{kNm}$
Far end BM $\beta M = 378.6 \text{kNm}$
Maximum moment on segment $M_z' = 530 \text{kNm}$
Far end BM $\beta M_z = 0 \text{kNm}$

Values to applied factored values $R_f = 0.6$

UNIVERSAL BEAM $533 \times 210 \times 82 \text{ UB Grade S 355}$
SECTION Section is satisfactory for bending, axial load, local capacity, and overall buckling check.
SUMMARY

DESIGN Axial load (+ve comp) $160 \text{kN}$
Compressive resistance $3402.2 \text{kN}$
Maximum moment (z-axis) $530 \text{kNm}$
Moment resistance $731.3 \text{kNm}$
Local capacity check $0.76766 \leq 1$
Overall buckling check $0.48843 \leq 1$
$0.6948 \leq 1$
FIRE RESISTANCE

SUMMARY

Exposed surface area  1872.6 mm²
Area of section       105 cm²
Section factor        175 /m
Load Ratio (R)        0.4606

Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided.
Location: Ex1 - SCI example

**I Section Column**

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

- Design moment about major axis: \( M_{yEd} = 30 \text{ kNm} \)
- Design shear force: \( V_{zEd} = 10 \text{ kN} \)
- Design axial load (+ve comp): \( N_{Ed} = 590 \text{ kN} \)
- Length of member: \( L = 5 \text{ m} \)
- Moment about the minor axis: \( M_{zEd} = 1 \text{ kNm} \)
- Design shear force // to flanges: \( V_{yEd} = 0.6 \text{ kN} \)
- Dimensions (mm): \( h = 203.2 \text{ b} = 203.6 \text{ tw} = 7.2 \text{ tf} = 11 \text{ r} = 10.2 \)
- Properties (cm): \( I_y = 4570 \text{ Iz} = 1550 \text{ W_{ply}} = 497 \text{ W_{plz}} = 231 \text{ It} = 22.2 \)
- Effective length parameter: \( K = 1 \)
- Length for LTB: \( L_T = 5 \text{ m} \)
- Maximum moment on section: \( M = 30 \text{ kNm} \)
- Far end moment: \( \psi_{iM} = 0 \text{ kNm} \)
- Far end bending moment: \( pM(1) = 0 \text{ kNm} \)
- Far end bending moment: \( pM(2) = 0 \text{ kNm} \)
- Far end bending moment: \( pM(3) = 0 \text{ kNm} \)

**UNIVERSAL COLUMN**

- **DESIGN**
  - Design BM y-y axis: \( 30 \text{ kNm} \)
  - Moment resistance (y-y): \( 136.68 \text{ kNm} \)
  - Design BM z-z axis: \( 1 \text{ kNm} \)
  - Moment resistance (z-z): \( 63.525 \text{ kNm} \)
  - Design compression load: \( 590 \text{ kN} \)
  - Axial resistance: \( 1614.3 \text{ kN} \)
  - Interaction factors: \( 0.62179 \leq 1 \)
    \( 0.96296 \leq 1 \)
Location: Ex2 - Moment about major axis and load

I Section Column

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis \( M_y = 60 \text{ kNm} \)
Design shear force \( V_z = 30 \text{ kN} \)
Design axial load (+ve comp) \( N_E = 600 \text{ kN} \)
Length of member \( L = 6 \text{ m} \)
Moment about the minor axis \( M_z = 0 \text{ kNm} \)
Design shear force // to flanges \( V_y = 0 \text{ kN} \)

Dimensions (mm): \( h = 260.3 \), \( b = 256.3 \), \( t_w = 10.3 \), \( t_f = 17.3 \), \( r = 12.7 \)

Properties (cm): \( I_y = 14300 \), \( I_z = 4860 \), \( W_{pl,y} = 1220 \), \( W_{pl,z} = 575 \), \( I_t = 102 \)

Effective length parameter \( K = 1 \)
Length for LTB \( L_T = 6 \text{ m} \)
Maximum moment on section \( M = 60 \text{ kNm} \)
Far end moment \( p_M(1) = -30 \text{ kNm} \)
Far end bending moment \( p_M(3) = -30 \text{ kNm} \)

UNIVERSAL COLUMN

DESIGN

Design BM \( y-y \) axis \( 60 \text{ kNm} \)

SUMMARY

Moment resistance \( y-y \) \( 323.3 \text{ kNm} \)
Design compression load \( 600 \text{ kN} \)
Axial resistance \( 2994.5 \text{ kN} \)
Interaction factors \( 0.32123 \leq 1 \), \( 0.52312 \leq 1 \)
Location: Ex3 - Moment about minor axis and load

I Section Column

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis \( M_{Ed}=0 \) kNm
Design shear force \( V_{zEd}=0 \) kN
Design axial load (+ve comp) \( N_{Ed}=800 \) kN
Length of member \( L=6 \) m
Moment about the minor axis \( M_{zEd}=30 \) kNm
Design shear force // to flanges \( V_{yEd}=16 \) kN
254 x 254 x 89 UKC
Dimensions (mm): \( h=260.3 \) b=256.3 tw=10.3 tf=17.3 r=12.7
Properties (cm): \( I_{y}=14300 \) \( I_{z}=4860 \) \( W_{ply}=1220 \) \( W_{plz}=575 \) \( I_{t}=102 \)
Far end bending moment \( pM(2)=0 \) kNm

**UNIVERSAL COLUMN**

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>254 x 254 x 89 UKC Grade S 275</td>
<td>Design BM y-y axis 0 kNm</td>
</tr>
<tr>
<td><strong>DESIGN</strong></td>
<td>Moment resistance (y-y) 323.3 kNm</td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td>Design BM z-z axis 30 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment resistance (z-z) 152.38 kNm</td>
</tr>
<tr>
<td></td>
<td>Design compression load 800 kN</td>
</tr>
<tr>
<td></td>
<td>Axial resistance 2994.5 kN</td>
</tr>
<tr>
<td></td>
<td>Interaction factors ( 0.4416 \leq 1 )</td>
</tr>
</tbody>
</table>

0.71666 \leq 1
Location: Ex4 - Tensile load and moments

I Section Column

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis $M_{Ed}=60 \text{ kNm}$
Design shear force $V_{zEd}=30 \text{ kN}$
Design axial load (+ve comp) $N_{Ed}=-1000 \text{ kN}$
Length of member $L=6 \text{ m}$
Moment about the minor axis $M_{zEd}=30 \text{ kNm}$
Design shear force // to flanges $V_{yEd}=16 \text{ kN}$
254 x 254 x 89 UKC
Dimensions (mm): $h=260.3 \quad b=256.3 \quad tw=10.3 \quad tf=17.3 \quad r=12.7$
Properties (cm): $I_y=14300 \quad I_z=4860 \quad W_{pl,y}=1220 \quad W_{pl,z}=575 \quad I_t=102$
Effective length parameter $K=1$
Length for LTB $L_T=6 \text{ m}$
Maximum moment on section $M=60 \text{ kNm}$
Far end moment $psiM=0 \text{ kNm}$

UNIVERSAL COLUMN

DESIGN

254 x 254 x 89 UKC Grade S 275
Design BM $y-y$ axis 60 kNm
Moment resistance ($y-y$) 323.3 kNm
Design BM $z-z$ axis 30 kNm
Moment resistance ($z-z$) 152.38 kNm
Design tensile load 1000 kN
Axial resistance 2994.5 kN
Interaction factors 0.71641 ≤ 1
Location: Ex5 - Class 4 section

I Section Column

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis.

All forces and moments are factored.

Design moment about major axis \( M_{yEd} = 30 \text{ kNm} \)
Design shear force \( V_{zEd} = 30 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 7000 \text{ kN} \)
Length of member \( L = 5 \text{ m} \)
Moment about the minor axis \( M_{zEd} = 0 \text{ kNm} \)
Design shear force // to flanges \( V_{yEd} = 0.6 \text{ kN} \)

Dimensions (mm): \( h = 970.3 \), \( b = 300 \), \( t_w = 16 \), \( t_f = 21.1 \), \( r = 30 \)
Properties (cm): \( I_y = 408000 \), \( I_z = 9550 \), \( W_{ply} = 9810 \), \( W_{plz} = 1020 \), \( I_t = 390 \)

Determine the effective section properties - EN 1992-1-5, Cl. 4.4

The process is iterative as the amount of web not considered is a function of the stresses at the top and bottom of the web.

Iteration 1:
Gross area of section \( A = 2b * t_f + h_w * t_w = 27510 \text{ mm}^2 \)
Gross 2nd moment of area \( I_y = (b^2 * (h_w + 2 * t_f)^3) - (b - t_w) * h_w^3 - 12 = 3.918E9 \text{ mm}^4 \)
Stress at top of web \( s_1 = \frac{M_{yEd} * 10^6 * h_w}{2 * I_y} = 3.5532 \text{ N/mm}^2 \)
Stress at bottom of web \( s_2 = \frac{-M_{yEd} * 10^6 * h_w}{2 * I_y} = -3.5532 \text{ N/mm}^2 \)

Iteration 10:
Position of effective centroid:
\( z_{eff} = \frac{(A * h/2 - A_w * (z_{effp} + b * 2 + l_w/2))}{A_{eff}} = 485.15 \text{ mm} \)
Effective 2nd moment of area:
\( I_{y_{eff}} = I_y + A * (h/2 - z_{eff})^2 - t_w * l_w^3/12 - A_w * (z_{effp} + b * 2 + l_w/2 - z_{eff})^2 \)
\( = 3.918E9 \text{ mm}^4 \)
Top of web stress \( s_{1(10)} = \frac{M_{yEd} * 10^6 * (h - t_f - z_{eff})}{I_{y_{eff}}} = 3.5532 \text{ N/mm}^2 \)
Bottom of web stress \( s_{2(10)} = \frac{-M_{yEd} * 10^6 * (z_{eff} - t_f)}{I_{y_{eff}}} = -3.5532 \text{ N/mm}^2 \)
Net loss area \( A_w = l_w * t_w = 0 \text{ mm}^2 \)
Effective area of section \( A_{eff} = A - A_w = 27510 \text{ mm}^2 \)
Effective section modulus \( W_{eff} = \frac{I_{y_{eff}}}{(1000 * (h - z_{eff}))} = 8075.8 \text{ cm}^3 \)
Effective length parameter                  K=1
Length for LTB                                 LT=5 m
Maximum moment on section                      M=30 kNm
Far end moment                                  psiM=0 kNm
Far end bending moment                         pM(1)=0 kNm
Far end bending moment                         pM(3)=0 kNm

UNIVERSAL BEAM                               1016 x 305 x 222 UKB Grade S 275
DESIGN                                      Design BM y-y axis         30 kNm
SUMMARY                                      Moment resistance (y-y)  2140.1 kNm
                                              Design compression load   7000 kN
                                              Axial resistance          729004 kN
                                              Interaction factors      0.018014 ≤ 1
                                              0.92731 ≤ 1
Location: Example 8.7.2 'Structural Steelwork'

Cased I Section Column

Calculations in accordance with BS5950-1:2000
Sections 4.7, 4.8 and 4.14

All loads and moments are factored.

Design moment about z-z axis \( M_z = 85 \text{ kNm} \)
Design shear force \( F_v = 0 \text{ kN} \)
Design axial load (+ve comp) \( F = 1200 \text{ kN} \)
Length of member \( L = 7 \text{ m} \)
Factored moment about minor axis \( M_y = 0 \text{ kNm} \)

203 x 203 x 86 UC.
Depth of concrete section \( d_c = 325 \text{ mm} \)
Width of concrete section \( b_c = 325 \text{ mm} \)
Concrete Grade \( f_{cu} = 30 \text{ N/mm}^2 \)
Modular ratio \( \alpha_e = 10 \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Length between restraints z axis \( L_z = 7000 \text{ mm} \)
Length between restraints y axis \( L_y = 7000 \text{ mm} \)
Length between restraints \( L_T = 7 \text{ m} \)
Maximum moment on segment \( M_{LT} = 85 \text{ kNm} \)
Far end BM \( \beta M = -51 \text{ kNm} \)
Maximum moment on segment \( M_z' = 85 \text{ kNm} \)
Far end BM \( \beta M_z = -51 \text{ kNm} \)

UNIVERSAL COLUMN
DEVELOPMENT SUMMARY

Axial load \( 1200 \text{ kN} \)
Compressive resistance \( 2162.4 \text{ kN} \)
Moment (z-z axis) \( 85 \text{ kNm} \)
Moment capacity \( 258.9 \text{ kNm} \)
Local capacity check \( 0.65937 < 1 \)
Buckling resistance \( 230.5 \text{ kNm} \)
Overall buckling check \( 0.70335 < 1 \)
\( 0.96037 < 1 \)

CASED SECTION DETAILS

Concrete grade \( C 30 \)
Cased section dimensions \( 325 \text{ mm} \times 325 \text{ mm} \)

STEEL

Fabric to comply with BS 4483, reference D98 (5 mm dia. wire at maximum spacing of 200 mm)
Location: Ex1 - Cased I-Section column (uni-axial bending)

Cased I-Section column


All loads and moments are factored.

Design moment about major axis \( M_{yEd} = 85 \) kNm
Design shear force \( V_{zEd} = 13 \) kN
Design axial load (+ve comp) \( N_{Ed} = 1200 \) kN
Length of member \( L = 7 \) m
Maximum BM about the minor axis \( M_{zEd} = 0 \) kNm

203 x 203 x 86 UKC
Permanent design load \( N_{Ed} = 800 \)
Depth of concrete section \( h_c = 325 \) mm
Width of concrete section \( b_c = 325 \) mm
Length btwn restraints yy axis \( L_y = 7 \) m
Length btwn restraints zz axis \( L_z = 7 \) m
Effective length factor y-y axis \( K_y = 0.85 \)
Effective length factor z-z axis \( K_z = 0.85 \)
Far end BM \( M_{yEd1} = 0 \) kNm

UNIVERSAL COLUMN

203 x 203 x 86 UKC Grade S 275
Design axial load \( 1200 \) kN
Design moment y-y axis \( 121.93 \) kNm
Moment capacity y-y axis \( 308.69 \) kNm

CASED SECTION DETAILS

Cylinder strength (NWC) \( 25 \) N/mm\(^2\)
Cased section dimensions \( 325 \) mm x \( 325 \) mm

Reinforcement

The resistance of the cross-section was evaluated without taking into consideration the contribution of any reinforcement. For fully encased members any longitudinal & transverse reinforcement that is to be provided needs to be in accordance with EC4 Part 1-1, Clause 6.7.5.2.
Location: Ex2 - Axial load only

Cased I-Section column


All loads and moments are factored.

Design moment about major axis \( M_{yEd} = 0 \) kNm
Design shear force \( V_{zEd} = 0 \) kN
Design axial load (+ve comp) \( N_{Ed} = 1800 \) kN
Length of member \( L = 7 \) m
Maximum BM about the minor axis \( M_{zEd} = 0 \) kNm

254 x 254 x 89 UKC
Permanent design load \( N_{gEd} = 1200 \)
Depth of concrete section \( h_c = 400 \) mm
Width of concrete section \( b_c = 400 \) mm
Length btwn restraints yy axis \( L_y = 7 \) m
Length btwn restraints zz axis \( L_z = 7 \) m
Effective length factor y-y axis \( K_y = 1 \)
Effective length factor z-z axis \( K_z = 1 \)

UNIVERSAL COLUMN 254 x 254 x 89 UKC Grade S 355
DESIGN SUMMARY Design axial load 1800 kN
Column axial resistance 2584.9 kN
CASED SECTION DETAILS Cylinder strength (NWC) 25 N/mm²
Cased section dimensions 400 mm x 400 mm

Reinforcement The resistance of the cross-section was evaluated without taking into consideration the contribution of any reinforcement. For fully encased members any longitudinal & transverse reinforcement that is to be provided needs to be in accordance with EC4 Part 1-1, Clause 6.7.5.2.
Location: Ex3 - Uni-axial bending case (see Thomas Telford publ.)

Cased I-Section column


All loads and moments are factored.

Design moment about major axis \( \text{MyEd}=380 \text{ kNm} \)
Design shear force \( \text{VzEd}=54 \text{ kN} \)
Design axial load (+ve comp) \( \text{NEd}=1800 \text{ kN} \)
Length of member \( L=7 \text{ m} \)
Maximum BM about the minor axis \( \text{MzEd}=0 \text{ kNm} \)

254 x 254 x 89 UKC
Permanent design load \( \text{NgEd}=1200 \)
Depth of concrete section \( hc=400 \text{ mm} \)
Width of concrete section \( bc=400 \text{ mm} \)
Length btwn restraints yy axis \( Ly=7 \text{ m} \)
Length btwn restraints zz axis \( Lz=7 \text{ m} \)
Effective length factor y-y axis \( Ky=1 \)
Effective length factor z-z axis \( Kz=1 \)
Far end BM \( \text{MyEd1}=0 \text{ kNm} \)

UNIVERSAL COLUMN
254 x 254 x 89 UKC Grade S 355

DESIGN SUMMARY
Design axial load \( 1800 \text{ kN} \)
Design moment y-y axis \( 413.93 \text{ kNm} \)
Moment capacity y-y axis \( 518.52 \text{ kNm} \)

CASED SECTION DETAILS
Cylinder strength (NWC) \( 25 \text{ N/mm}^2 \)
Cased section dimensions \( 400 \text{ mm} \times 400 \text{ mm} \)

Reinforcement
The resistance of the cross-section was evaluated without taking into consideration the contribution of any reinforcement. For fully encased members any longitudinal & transverse reinforcement that is to be provided needs to be in accordance with EC4 Part 1-1, Clause 6.7.5.2.
Location: Ex4 - Biaxial bending case (see Thomas Telford publ.)

Cased I-Section column


All loads and moments are factored.

Design moment about major axis \( M_{yEd} = 380 \text{ kNm} \)
Design shear force \( V_{zEd} = 54 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 1800 \text{ kN} \)
Length of member \( L = 7 \text{ m} \)
Maximum BM about the minor axis \( M_{zEd} = 50 \text{ kNm} \)

254 x 254 x 89 UKC
Permanent design load \( N_{Ed} = 1200 \)
Depth of concrete section \( h_c = 400 \text{ mm} \)
Width of concrete section \( b_c = 400 \text{ mm} \)
Length between restraints yy axis \( L_y = 7 \text{ m} \)
Length between restraints zz axis \( L_z = 7 \text{ m} \)
Effective length factor yy axis \( k_y = 1 \)
Effective length factor zz axis \( k_z = 1 \)
Far end BM \( My_{Ed1} = 0 \text{ kNm} \)
Far end BM \( Mz_{Ed1} = 0 \text{ kNm} \)

Universal column 254 x 254 x 89 UKC Grade S 355
Design axial load 1800 kN
Design moment y-y axis 380 kNm
Moment capacity y-y axis 518.52 kNm
Design moment z-z axis 50 kNm
Moment capacity z-z axis 325.01 kNm
Interaction factors 0.98514 ≤ 1

Cased section details Cylinder strength (NWC) 25 N/mm²
Cased section dimensions 400 mm x 400 mm

Reinforcement The resistance of the cross-section was evaluated without taking into consideration the contribution of any reinforcement. For fully encased members any longitudinal & transverse reinforcement that is to be provided needs to be in accordance with EC4 Part 1-1, Clause 6.7.5.2.
Location: Example 2, 'Steel Designer's Manual' - with restraint

H Section Column

Calculations in accordance with BS5950-1:2000 Sections 4.7 and 4.8.

All loads and moments are factored.
Axis references are GLOBAL.

Design moment about y-y axis \( M_y = 530 \text{ kNm} \)
Design shear force \( F_y = 0 \text{ kN} \)
Design axial load (+ve comp) \( F = 160 \text{ kN} \)
Length of member \( L = 5.6 \text{ m} \)
Factored moment about z-z axis \( M_z = 0 \text{ kNm} \)
Accompanying shear force \( F_v = 95 \text{ kN} \)

533 x 210 x 82 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Length between restraints z axis \( L_z = 1600 \text{ mm} \)
Length between restraints y axis \( L_y = 5600 \text{ mm} \)
Length between restraints \( L_T = 1.6 \text{ m} \)
Maximum moment on segment \( M_{LT} = 530 \text{ kNm} \)
Far end BM \( \beta M = 378 \text{ kNm} \)
Maximum moment on segment \( M_y' = 530 \text{ kNm} \)
Far end BM \( \beta M_y = 0 \text{ kNm} \)

UNIVERSAL BEAM 533 x 210 x 82 UB Grade S 355
SECTION
Summary Section is satisfactory for bending, axial load, local capacity, and overall buckling check.

DESIGN
Axial load (+ comp.) 160 kN
Compressive resistance 3402.2 kN
Maximum moment (y-axis) 530 kNm
Moment capacity 731.3 kNm
Local capacity check \( 0.76766 \leq 1 \)
Overall buckling check \( 0.48843 \leq 1 \)
\( 0.69447 \leq 1 \)
Location: Moment about z-z axis only

**H Section Column**

Calculations in accordance with BS5950-1:2000
Sections 4.7 and 4.8.

All loads and moments are factored.
Axis references are GLOBAL.

Design moment about y-y axis \( M_y = 0 \) kNm
Design shear force \( F_y = 0 \) kN
Design axial load (+ve comp) \( F = 470 \) kN
Length of member \( L = 5.6 \) m
Factored moment about z-z axis \( M_z = 48 \) kNm
Accompanying shear force \( F_v = 65 \) kN

203 x 203 x 86 UC.
Young's Modulus \( E = 205 \) kN/mm²

Length between restraints z axis \( L_z = 5600 \) mm
Length between restraints y axis \( L_y = 5600 \) mm
Maximum moment on segment \( M_z' = 48 \) kNm
Far end BM \( \beta M_z = 0 \) kNm
Maximum moment on segment \( M_{maxy} = 48 \) kNm
Far end BM \( \beta_{maxy} = 0 \) kNm

**UNIVERSAL COLUMN**

203 x 203 x 86 UC Grade S 275

**SECTION**

Section is satisfactory for bending, axial load, local capacity, and overall buckling check.

**DESIGN**

Axial load (+ comp.) \( 470 \) kN

**SUMMARY**

Compressive resistance \( 1269.5 \) kN
Maximum moment(z-axis) \( 48 \) kNm
Moment capacity \( 119 \) kNm
Local capacity check \( 0.56459 \leq 1 \)
Overall buckling check \( 0.70182 \leq 1 \)
\( 0.3216 \leq 1 \)
Location: Semi-compact

H Section Column

Calculations in accordance with BS5950-1:2000 Sections 4.7 and 4.8.
All loads and moments are factored.
Axis references are GLOBAL.

Design moment about y-y axis $M_y=0$ kNm
Design shear force $F_y=0$ kN
Design axial load (+ve comp) $F=500$ kN
Length of member $L=2.8$ m
Factored moment about z-z axis $M_z=12$ kNm
Accompanying shear force $F_v=46$ kN

406 x 140 x 39 UB.
Young's Modulus $E=205$ kN/mm²

Length between restraints z axis $L_z=2800$ mm
Length between restraints y axis $L_y=1400$ mm
Maximum moment on segment $M_{z'}=12$ kNm
Far end BM $\beta M_z=0$ kNm
Maximum moment on segment $M_{max y}=12$ kNm
Far end BM $\beta y=0$ kNm

UNIVERSAL BEAM 406 x 140 x 39 UB Grade S 355
SECTION
SUMMARY Section is satisfactory for bending, axial load, local capacity, and overall buckling check.

DESIGN Axial load (+ comp.) 500 kN
SUMMARY Compressive resistance 803.15 kN
Maximum moment(z-axis) 12 kNm
Moment capacity 32.234 kNm
Local capacity check $0.65567 \leq 1$
Overall buckling check $0.98497 \leq 1$
$0.39507 \leq 1$
Location: SCI example

H Section Column

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis. All forces and moments are factored.

Design moment about minor axis $M_{zEd}=1$ kNm
Design shear force $V_{yEd}=0.6$ kN
Design axial load (+ve comp) $N_{Ed}=590$ kN
Length of member $L=5$ m
Moment about the y-y axis $M_{yEd}=30$ kNm
Accompanying shear force $V_{zEd}=10$ kN
203 x 203 x 46 UKC
Dimensions (mm): $h=203.2$, $b=203.6$, $tw=7.2$, $tf=11$, $r=10.2$
Properties (cm): $I_y=4570$, $I_z=1550$, $W_{ply}=497$, $W_{plz}=231$, $I_t=22.2$
Effective length parameter $K=1$
Length for LTB $L_T=5$ m
Maximum moment on section $M=30$ kNm
Far end moment $pM(1)=0$ kNm
Far end bending moment $pM(2)=0$ kNm
Far end bending moment $pM(3)=0$ kNm

UNIVERSAL COLUMN
203 x 203 x 46 UKC Grade S 275
DESIGN
Design BM z-z axis $1$ kNm
Moment resistance (MzRd) $63.525$ kNm
Design BM y-y axis $30$ kNm
Moment resistance (MyRd) $136.68$ kNm
Buckling resistance $136.68$ kNm
Design compression load $590$ kN
Axial resistance $1614.3$ kN
Interaction factors $0.62179 \leq 1$

SUMMARY

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design BM z-z axis</td>
<td>1 kNm</td>
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<tr>
<td>Design BM y-y axis</td>
<td>30 kNm</td>
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<tr>
<td>Moment resistance (MzRd)</td>
<td>63.525 kNm</td>
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<tr>
<td>Moment resistance (MyRd)</td>
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<td>Buckling resistance</td>
<td>136.68 kNm</td>
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<td>Design compression load</td>
<td>590 kN</td>
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<tr>
<td>Axial resistance</td>
<td>1614.3 kN</td>
</tr>
<tr>
<td>Interaction factors</td>
<td>$0.62179 \leq 1$</td>
</tr>
</tbody>
</table>

SCALE 5.48 Office 1007 Proforma 442
Location: Moment about major axis and load

H Section Column

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis. All forces and moments are factored.

Design moment about minor axis \( M_{zEd} = 0 \) kNm
Design shear force \( V_{yEd} = 0 \) kN
Design axial load (+ve comp) \( N_{Ed} = 600 \) kN
Length of member \( L = 6 \) m
Moment about the y-y axis \( M_{yEd} = 60 \) kNm
Accompanying shear force \( V_{zEd} = 30 \) kN
254 x 254 x 89 UKC
Dimensions (mm): \( h = 260.3 \) b = 256.3 \( tw = 10.3 \) tf = 17.3 r = 12.7
Properties (cm): \( I_y = 14300 \) \( I_z = 4860 \) \( W_{ply} = 1220 \) \( W_{plz} = 575 \) \( I_t = 102 \)
Effective length parameter \( K = 1 \)
Length for LTB \( L_T = 6 \) m
Maximum moment on section \( M = 60 \) kNm
Far end moment \( \psi M = -30 \) kNm
Far end bending moment \( pM(1) = -30 \) kNm
Far end bending moment \( pM(3) = -30 \) kNm

UNIVERSAL COLUMN

DESIGN

Design BM z-z axis 0 kNm

SUMMARY

Moment resistance (MzRd) 152.38 kNm
Design BM y-y axis 60 kNm
Moment resistance (MyRd) 323.3 kNm
Buckling resistance 323.3 kNm
Design compression load 600 kN
Axial resistance 2994.5 kN
Interaction factors \( 0.32123 \leq 1 \)
\( 0.52148 \leq 1 \)
Location: Moment about minor axis and load

H Section Column

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis. All forces and moments are factored.

Design moment about minor axis \( M_{zEd} = 30 \text{ kNm} \)
Design shear force \( V_{yEd} = 16 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 800 \text{ kN} \)
Length of member \( L = 6 \text{ m} \)
Moment about the y-y axis \( M_{yEd} = 0 \text{ kNm} \)
Accompanying shear force \( V_{zEd} = 0 \text{ kN} \)

Dimensions (mm): \( h = 260.3 \text{ b} = 256.3 \text{ tw} = 10.3 \text{ tf} = 17.3 \text{ r} = 12.7 \)
Properties (cm): \( I_y = 14300 \text{ I}_z = 4860 \text{ W}_{ply} = 1220 \text{ W}_{plz} = 575 \text{ I}_t = 102 \)
Far end bending moment \( pM(2) = 0 \text{ kNm} \)

UNIVERSAL COLUMN 254 x 254 x 89 UKC Grade S 275
DESIGN Design BM z-z axis 30 kNm
SUMMARY Moment resistance (MzRd) 152.38 kNm
Design compression load 800 kN
Axial resistance 2994.5 kN
Interaction factors 0.4416 ≤ 1
0.71666 ≤ 1
Location: Tensile load and moments

H Section Column

Calculations in accordance with BS EN 1993-1-1:2005. Internal forces and moments have been determined by elastic global analysis factored. All forces and moments are factored.

Design moment about minor axis \( M_{zEd} = 30 \text{ kNm} \)
Design shear force \( V_{yEd} = 16 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = -1000 \text{ kN} \)
Length of member \( L = 6 \text{ m} \)
Moment about the y-y axis \( M_{yEd} = 60 \text{ kNm} \)
Accompanying shear force \( V_{zEd} = 30 \text{ kN} \)
254 x 254 x 89 UKC
Dimensions (mm): \( h = 260.3 \text{ b = 256.3 tw = 10.3 tf = 17.3 r = 12.7} \)
Properties (cm): \( I_y = 14300 \text{ Iz = 4860 W}_{pl_y} = 1220 \text{ W}_{pl_z} = 575 \text{ It = 102} \)
Effective length parameter \( K = 1 \)
Length for LTB \( L_T = 6 \text{ m}\)
Maximum moment on section \( M = 60 \text{ kNm} \)
Far end moment \( psiM = 0 \text{ kNm} \)

**UNIVERSAL COLUMN**

**DESIGN**

Design BM z-z axis \( 30 \text{ kNm} \)

**SUMMARY**

Moment resistance (MzRd) \( 152.38 \text{ kNm} \)
Moment resistance (MyRd) \( 323.3 \text{ kNm} \)
Buckling resistance \( 323.3 \text{ kNm} \)
Design tensile load \( 1000 \text{ kN} \)
Axial resistance \( 2994.5 \text{ kN} \)
Interaction factors \( 0.71641 \leq 1 \)
Location: Default Example

Annex E Effective lengths of compression members

The effective length LE for in-plane buckling of a column in a continuous structure with moment-resisting joints

The column being designed is Kc.

K = stiffness coefficients
k = distribution factors

Figure E.3

K2

Take 75% of inertia for pin ended members.
Take zero inertia and unit length for missing members.

305 x 305 x 118 UC.
Inertia of column being designed Ic=27700 cm^4
Length of column being designed lc=4 m

305 x 305 x 118 UC.
Inertia of upper column Iu=27700 cm^4
Length of upper column lu=4 m

305 x 305 x 118 UC.
Inertia of lower column Il=27700 cm^4
Length of lower column ll=6 m

610 x 229 x 125 UB.
Inertia of top left beam Itl=98600 cm^4
Length of top left beam ltl=10 m
Inertia of top right beam Itr=0 cm^4
Inertia of bottom left beam Ibl=0 cm^4

610 x 229 x 125 UB.
Inertia of bottom right beam Ibr=98600 cm^4
Length of bottom right beam lbr=10 m

Joint restraint coefficients

The frame under consideration is unbraced against sidesway.
Upper formula Figure E.2 uf=1-0.2*(k1+k2)-0.12*k1*k2=0.738
Lower formula lf=1-0.8*(k1+k2)+0.6*k1*k2=0.29
Ratio effective/actual length LE'/L=(uf/lf)^0.5=1.59
Location: Lowest length of a column

Annex E Effective lengths of compression members

\[
\begin{array}{c}
\begin{array}{c}
\begin{array}{c}
K1 \\
K11 \quad k1 \quad K21 \\
K12 \quad / \\
Kc \\
K22 \\
k2 \\
\end{array}
\end{array}
\end{array}
\]

The effective length \( LE \) for in-plane buckling of a column in a continuous structure with moment-resisting joints.

The column being designed is \( Kc \).

\( K = \) stiffness coefficients
\( k = \) distribution factors

Take 75% of inertia for pin ended members.
Take zero inertia and unit length for missing members.

305 x 305 x 118 UC.
Inertia of column being designed \( Ic=27700 \text{ cm}^4 \)
Length of column being designed \( lc=4 \text{ m} \)

305 x 305 x 118 UC.
Inertia of upper column \( Iu=27700 \text{ cm}^4 \)
Length of upper column \( lu=4 \text{ m} \)

610 x 229 x 125 UB.
Inertia of top left beam \( Itl=98600 \text{ cm}^4 \)
Length of top left beam \( ltl=10 \text{ m} \)
Inertia of top right beam \( Itr=0 \text{ cm}^4 \)

Joint restraint coefficients

The frame under consideration is braced against sidesway.
Sum of distribution factors \( ks=k1+k2=1.57 \)
Ratio effective/actual length \( LE'L=0.5+0.14*ks+0.055*ks^2=0.856 \)
Location: Using E.3 and E.4

Annex E Effective lengths of compression members

\[
\begin{align*}
| K1 | K11 & k1 & K12 \rangle \\
| Kc | K21 & K22 \rangle \\
| k2 |
\end{align*}
\]

The effective length LE for in-plane buckling of a column in a continuous structure with moment-resisting joints.

The column being designed is Kc.

K = stiffness coefficients

k = distribution factors

Take 75% of inertia for pin ended members.

Take zero inertia and unit length for missing members.

305 x 305 x 118 UC.
Inertia of column being designed \( I_c = 27700 \text{ cm}^4 \)
Length of column being designed \( l_c = 4 \text{ m} \)

305 x 305 x 118 UC.
Inertia of upper column \( I_u = 27700 \text{ cm}^4 \)
Length of upper column \( l_u = 4 \text{ m} \)

305 x 305 x 118 UC.
Inertia of lower column \( I_l = 27700 \text{ cm}^4 \)
Length of lower column \( l_l = 5 \text{ m} \)

610 x 229 x 125 UB.
Inertia of top left beam \( I_{tl} = 98600 \text{ cm}^4 \)
Length of top left beam \( l_{tl} = 10 \text{ m} \)

610 x 229 x 125 UB.
Inertia of bottom left beam \( I_{bl} = 98600 \text{ cm}^4 \)
Length of bottom left beam \( l_{bl} = 10 \text{ m} \)

Joint restraint coefficients

Taking into account the stiffening effects of wall panels in the storey in accordance with Annex E.3.

Stiffness of columns

Number of columns in storey \( a = 28 \)
Column inertia \( I_c(1) = 27600 \text{ cm}^4 \)
Length of column \( l_c = 4 \text{ m} \)
No. of columns with stiffness \( n_1(1) = 14 \)
Column inertia \( I_c(2) = 22200 \text{ cm}^4 \)
Length of column \( l_c = 4 \text{ m} \)
No. of columns with stiffness \( n_1(2) = 14 \)
E.3.4 Stiffness of panels

Number of wall panels in storey    wall=4
Width of panel                   b(1)=6 m
Thickness of panel               T(1)=150 mm
Modulus of elasticity of panel   Ep(1)=16 kN/mm²
No. of panels with stiffness     n2(1)=4
Total spring stiffness           Sp=1840 kN/mm

Relative stiffness

Modulus of elasticity of steel   E=205 kN/mm²
Interpolating between Tables E.4 and E.5 for kp=1.03
Ratio of length with kp as 1     LE1=TABLE 4 for k1=0.584, k2=0.558 =1.43
Ratio of length with kp as 2     LE2=TABLE 5 for k1=0.584, k2=0.558 =1.3
Ratio effective/actual length    LE'=LE1-(kp-1)*(LE1-LE2)=1.42
**Location: Ex1 - Default example**

**Effective lengths of compression members - NCCI SN008a-EN-EU**

The buckling length 'Lcr' for in-plane buckling of a column in a continuous structure with moment-resisting joints.

Kc = stiffness coefficient of column being designed.
K1, K2, K11, K12, K21, K22 are the column and beam stiffness coefficients.
Take 75% of inertia for pin ended members and zero inertia & unit length for missing members.

### Inertias and Lengths

- **305 x 305 x 118 UKC**
  - Inertia of column being designed: \( I_c = 27700 \text{ cm}^4 \)
  - Length of column being designed: \( l_c = 4 \text{ m} \)
- **305 x 305 x 118 UKC**
  - Inertia of upper column: \( I_u = 27700 \text{ cm}^4 \)
  - Length of upper column: \( l_u = 4 \text{ m} \)
- **305 x 305 x 118 UKC**
  - Inertia of lower column: \( I_l = 27700 \text{ cm}^4 \)
  - Length of lower column: \( l_l = 6 \text{ m} \)
- **610 x 229 x 125 UKB**
  - Inertia of top left beam: \( I_{tl} = 98600 \text{ cm}^4 \)
  - Length of top left beam: \( l_{tl} = 10 \text{ m} \)
  - Inertia of top right beam: \( I_{tr} = 0 \text{ cm}^4 \)
  - Inertia of bottom left beam: \( I_{bl} = 0 \text{ cm}^4 \)
- **610 x 229 x 125 UKB**
  - Inertia of bottom right beam: \( I_{br} = 98600 \text{ cm}^4 \)
  - Length of bottom right beam: \( l_{br} = 10 \text{ m} \)

### Joint restraint coefficients

The frame under consideration is unbraced against sidesway.

- **Upper formula**
  \[ u_f = 1 - 0.2 \cdot (\eta_1 + \eta_2) - 0.12 \cdot \eta_1 \cdot \eta_2 \]
  \[ = 0.738 \]
- **Lower formula**
  \[ l_f = 1 - 0.8 \cdot (\eta_1 + \eta_2) + 0.6 \cdot \eta_1 \cdot \eta_2 \]
  \[ = 0.29 \]
- **Ratio effective/actual length**
  \[ L_{cr}'L = (u_f/l_f)^{0.5} = 1.59 \]
- **Buckling length**
  \[ L_{cr} = L_{cr}'L \cdot l_c = 6.38 \text{ m} \]
Location: Ex2 - Lowest length of a column

Effective lengths of compression members - NCCI SN008a-EN-EU

The buckling length 'Lcr' for in-plane buckling of a column in a continuous structure with moment-resisting joints. Kc = stiffness coefficient of column being designed. K1, K2, K11, K12, K21, K22 are the column and beam stiffness coefficients. Take 75% of inertia for pin ended members and zero inertia & unit length for missing members.

305 x 305 x 118 UKC
Inertia of column being designed \( I_c = 27700 \text{ cm}^4 \)
Length of column being designed \( l_c = 4 \text{ m} \)
305 x 305 x 118 UKC
Inertia of upper column \( I_u = 27700 \text{ cm}^4 \)
Length of upper column \( l_u = 4 \text{ m} \)
610 x 229 x 125 UKB
Inertia of top left beam \( I_{tl} = 98600 \text{ cm}^4 \)
Length of top left beam \( l_{tl} = 10 \text{ m} \)
Inertia of top right beam \( I_{tr} = 0 \text{ cm}^4 \)

Joint restraint coefficients

The frame under consideration is braced against sidesway.
Sum of distribution factors \( \eta = \eta_1 + \eta_2 = 1.57 \)
Ratio effective/actual length \( Lcr'L = 0.5 + 0.14 \eta + 0.055 \eta^2 = 0.856 \)
Buckling length \( Lcr = Lcr'L \times l_c = 3.42 \text{ m} \)
Location: Ex3 - Beam with direct load

Effective lengths of compression members - NCCI SN008a-EN-EU

The buckling length 'Lcr' for in-plane buckling of a column in a continuous structure with moment-resisting joints. 
Kc = stiffness coefficient of column being designed. 
K1, K2, K11, K12, K21, K22 are the column and beam stiffness coefficients. 
Take 75% of inertia for pin ended members and zero inertia & unit length for missing members.

305 x 305 x 118 UKC
Inertia of column being designed  Ic=27700 cm^4
Length of column being designed  lc=4 m

305 x 305 x 118 UKC
Inertia of upper column  Iu=27700 cm^4
Length of upper column  lu=4 m

305 x 305 x 118 UKC
Inertia of lower column  Il=27700 cm^4
Length of lower column  ll=5 m

610 x 229 x 125 UKB
Inertia of top left beam  Itl=98600 cm^4
Length of top left beam  ltl=10 m

610 x 229 x 125 UKB
Inertia of top right beam  Itr=0 cm^4
Length of top right beam  lbr=0 cm^4

Joint restraint coefficients

Taking into account the stiffening effects of wall panels in the storey.

Stiffness of columns

Number of columns in storey  a=28
Column inertia  Ic(1)=27700 cm^4
Length of column  lc=4 m
No. of columns with stiffness  n1(1)=14
Column inertia  Ic(2)=22200 cm^4
Length of column  lc=4 m
No. of columns with stiffness  n1(2)=14
**Stiffness of panels**

- Number of wall panels in storey: wall=4
- Storey height: h=lc=4 m
- Width of panel: b(1)=6 m
- Thickness of panel: T(1)=150 mm
- Modulus of elasticity of panel: Ep(1)=16 kN/mm²
- No. of panels with stiffness: n2(1)=4
- Total spring stiffness: Sp=1840 kN/mm

**Relative stiffness**

- Modulus of elasticity of steel: E=210 kN/mm²
- Ratio of length with kp as 1: LE1=1.43
- Ratio of length with kp as 2: LE2=1.3
- Ratio effective/actual length: Lcr'L=LE1-(kp-1)*(LE1-LE2)=1.42
- Buckling length: Lcr=Lcr'L*lc=5.7 m
Location: Default example bi-axial bending

 Structural Hollow Section Column


All loads and moments are factored.

Factored bending moment axis zz $M_z = 14 \text{ kNm}$
Factored shear force y-y direct. $F_v = 0 \text{ kN}$
Axial load (+ve compression) $F = 500 \text{ kN}$
Length of member $L = 3.5 \text{ m}$
200 x 100 x 5 RHS - Hot finished.
Properties (cm): $A = 28.7$, $r_x = 7.21$, $Z_x = 149$, $S_x = 185$, $I_x = 1500$
$J = 1200$, $C = 172$, $Z_y = 101$, $S_y = 114$, $I_y = 505$, $r_y = 4.19$
(where $r_z = r_x$, $Z_z = Z_x$, $S_z = S_x$ & $I_z = I_x$)
Factored bending moment axis yy $M_y = 1 \text{ kNm}$
Young’s Modulus $E = 205 \text{ kN/mm}^2$
Length between restraints z axis $L_z = 3500 \text{ mm}$
Length between restraints y axis $L_y = 3500 \text{ mm}$
Far end BM $\beta_{M} = 0 \text{ kNm}$
Maximum moment on segment $M_z = 14 \text{ kNm}$
Far end BM $\beta_{M_z} = 0 \text{ kNm}$
Far end BM $\beta_{M_y} = 0 \text{ kNm}$
Maximum moment on segment $M_{max_y} = 1 \text{ kNm}$
Far end BM $\beta_{y} = 0 \text{ kNm}$
Dist. betwn torsional restraints $L_T = 3.5 \text{ m}$

HOT FINISHED
RECTANGULAR HOLLOW SECTION
200 x 100 x 5 RHS Grade S 355
SECTION
SUMMARY
In accordance with EN 10210
Section is satisfactory for axial load, and overall buckling check.
Axial load $500 \text{ kN}$
Compression resistance $764.58 \text{ kN}$
Maximum moment z axis $14 \text{ kNm}$
Moment capacity $65.675 \text{ kNm}$
Maximum moment y axis $1 \text{ kNm}$
Moment capacity $40.47 \text{ kNm}$
Local capacity check $0.72863 \leq 1$
Overall buckling checks $0.69532 \leq 1$
$0.73758 \leq 1$
$0.39795 \leq 1$
Location: Example - bending about minor axis only

### Structural Hollow Section Column


All loads and moments are factored.

Factored bending moment axis \( zz \) \( M_z = 0 \) kNm

Factored shear force \( y-y \) direct. \( F_v = 23 \) kN

Axial load (+ve compression) \( F = 1500 \) kN

Length of member \( L = 6 \) m

300 x 200 x 8 RHS - Cold formed.

Properties (cm): \( A = 75.2 \) \( r_x = 11.2 \) \( Z_x = 626 \) \( S_x = 757 \) \( I_x = 9390 \)

\( J = 10600 \) \( C = 838 \) \( Z_y = 504 \) \( S_y = 574 \) \( I_y = 5040 \) \( r_y = 8.19 \)

(where \( r_z = r_x \), \( Z_z = Z_x \), \( S_z = S_x \) & \( I_z = I_x \))

Factored bending moment axis \( yy \) \( M_y = 14 \) kNm

Young's Modulus \( E = 205 \) kN/mm²

Length between restraints z axis \( L_z = 6000 \) mm

Length between restraints y axis \( L_y = 6000 \) mm

COLD FORMED

RECTANGULAR HOLLOW SECTION 300 x 200 x 8 RHS Grade S 355

SECTION

SUMMARY

Section is satisfactory for axial load, and overall buckling check.

Axial load \( 1500 \) kN

Compression resistance \( 1771.1 \) kN

Maximum moment y axis \( 14 \) kNm

Moment capacity \( 172.84 \) kNm

Local capacity check \( 0.65958 \leq 1 \)

Overall buckling checks \( 0.96224 \leq 1 \)

\( 0.75202 \leq 1 \)
Location: Bending about x-x axis only

Structural Hollow Section Column


All loads and moments are factored.

Factored bending moment axis zz \( M_z = 320 \text{ kNm} \)
Factored shear force y-y direct. \( F_v = 327 \text{ kN} \)
Axial load (+ve compression) \( F = 1000 \text{ kN} \)
Length of member \( L = 6 \text{ m} \)
400 x 200 x 10 RHS - Cold formed.

Properties (cm): \( A = 113 \), \( r_x = 14.3 \)
\( Z_x = 1150 \), \( S_x = 1430 \), \( I_x = 23000 \)
\( J = 19400 \), \( C = 1370 \), \( Z_y = 786 \), \( S_y = 888 \), \( I_y = 7860 \), \( r_y = 8.36 \)

(where \( r_z = r_x \), \( Z_z = Z_x \), \( S_z = S_x \) & \( I_z = I_x \))

Factored bending moment axis yy \( M_y = 0 \text{ kNm} \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Length between restraints z axis \( L_z = 6000 \text{ mm} \)
Length between restraints y axis \( L_y = 6000 \text{ mm} \)
Far end BM \( \beta M = 0 \text{ kNm} \)
Maximum moment on segment \( M_z = 320 \text{ kNm} \)
Far end BM \( \beta M_z = 0 \text{ kNm} \)
Dist. betwn torsional restraints \( L_T = 6 \text{ m} \)

COLD FORMED
RECTANGULAR HOLLOW SECTION
SECTION
SUMMARY

In accordance with EN 10219
400 x 200 x 10 RHS Grade S 355
Section is satisfactory for axial load, and overall buckling check.
Axial load \( 1000 \text{ kN} \)
Compression resistance \( 2393 \text{ kN} \)
Maximum moment z axis \( 320 \text{ kNm} \)
Moment capacity \( 507.65 \text{ kNm} \)
Local capacity check \( 0.87964 \leq 1 \)
Overall buckling checks \( 0.71632 \leq 1 \)
\( 0.60699 \leq 1 \)
Location: Default example bi-axial bending

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**Steel design to BS5950-1:2000 and Eurocode 3**  
Made by: IFB  
Date: 02/12/19  
Ref No: SC444 BS

---

**Structural Hollow Section Column**


All loads and moments are factored.

Factored bending moment axis zz: \( M_z = 14 \text{ kNm} \)
Factored shear force y-y direct: \( F_{v} = 0 \text{ kN} \)
Axial load (+ve compression): \( F = 500 \text{ kN} \)
Length of member: \( L = 3.5 \text{ m} \)

200 x 100 x 5 RHS - Hot finished.
Properties (cm): \( A = 28.7 \text{ cm}^2 \), \( r_x = 7.21 \text{ cm} \), \( Z_x = 149 \text{ cm}^3 \), \( S_x = 185 \text{ cm}^2 \), \( I_x = 1500 \text{ cm}^4 \)
\( J = 1200 \text{ cm}^4 \), \( C = 172 \text{ cm} \), \( Z_y = 101 \text{ cm}^3 \), \( S_y = 114 \text{ cm}^2 \), \( I_y = 505 \text{ cm}^4 \)
\( r_y = 4.19 \text{ cm} \)

Factored bending moment axis yy: \( M_y = 1 \text{ kNm} \)
Young's Modulus: \( E = 205 \text{ kN/mm}^2 \)
Length between restraints z axis: \( L_z = 3500 \text{ mm} \)
Length between restraints y axis: \( L_y = 3500 \text{ mm} \)
Far end BM: \( \beta M = 0 \text{ kNm} \)
Maximum moment on segment: \( M_z = 14 \text{ kNm} \)
Far end BM: \( \beta M_z = 0 \text{ kNm} \)
Far end BM: \( \beta M_y = 0 \text{ kNm} \)
Maximum moment on segment: \( M_{\text{max}y} = 1 \text{ kNm} \)
Far end BM: \( \beta y = 0 \text{ kNm} \)
Dist. betw. torsional restraints: \( L_T = 3.5 \text{ m} \)

values to applied factored values \( R_f = 0.6 \)

---

**RECTANGULAR HOLLOW SECTION**

- **Summary**
  - Section is satisfactory for axial load, and overall buckling check.
  - Axial load: \( 500 \text{ kN} \)
  - Compression resistance: \( 764.58 \text{ kN} \)
  - Maximum moment z axis: \( 14 \text{ kN} \)
  - Moment capacity: \( 65.675 \text{ kNm} \)
  - Maximum moment y axis: \( 1 \text{ kN} \)
  - Moment capacity: \( 40.47 \text{ kNm} \)
  - Local capacity check: \( 0.72863 \leq 1 \)
  - Overall buckling checks: \( 0.69532 \leq 1 \)
  - \( 0.73758 \leq 1 \)
  - \( 0.39795 \leq 1 \)

---

SCALE 5.48  
Office 1007  
Proforma 444
<table>
<thead>
<tr>
<th>FIRE RESISTANCE</th>
<th>Exposed surface area</th>
<th>600 mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Area of section</td>
<td>28.7 cm²</td>
</tr>
<tr>
<td></td>
<td>Section factor</td>
<td>205 /m</td>
</tr>
<tr>
<td></td>
<td>Load Ratio (R)</td>
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</tr>
</tbody>
</table>

Section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Hollow Section Column Design

Calculations are in accordance with EN 1993-1-1 & EN 1993-1-2.

Internal forces and moments have been determined by elastic global analysis.

All loads and moments are factored.

Moment about y-y axis       MyEd=20 kNm
Shear force                 VzEd=10 kN
Axial load (+ve compression) NEd=600 kN
Length of member             L=3.5 m
Moment about the z-z axis   MzEd=5 kNm
Accompanying shear force     VyEd=6 kN

150 x 150 x 6.3 SHS - Hot finished.

Properties (cm): A=35.8 iy=5.85 Wely=163 Wply=192 Iy=1220 It=1910 Wt=240
Length between restraints (y-y) Ly=3.5 m
Length between restraints (z-z) Lz=3.5 m
Far end bending moment        pM(1)=0 kNm
Far end bending moment        pM(2)=0 kNm
Effective length parameter   K=1
Length for LTB                LT=3.5 m
Maximum moment on section     M=20 kNm
Far end moment                psiM=0 kNm

HOT FINISHED
SQUARE HOLLOW SECTION 150 x 150 x 6.3 SHS Grade S 355
DESIGN
Moment resistance (y-y) 68.16 kNm
Moment resistance (z-z) 68.16 kNm
Axial resistance         1270.9 kN
Interaction factors 0.85889 ≤ 1
                      0.78794 ≤ 1

Fire resistance without applied protection

Unfactored permanent axial force Np=200 kN
Unfactored variable axial force Nv=220 kN
Design axial compressive force NfiEd=1.0*Np+1.0*Nv=420 kN
**Determine critical temperature of steel in fire situation**

Buckling length in fire situation $L_{fi}=2450$ mm  
Critical temperature $\theta_{cr}$  
$t_{CR}=TABLE 254$ for $m_o=0.33047$, $\lambda_{fi}=0.64492$  
$=579.41 \degree C$

Fire resistance period 18 minutes

**Interaction of axial loads and moments in fire situation**

The interaction of axial compression and bending is given by the following expression (EN 1993-1-2 Clause 4.2.3.5):  

Unity expression  
$R=N_{fi}E_d/N_{fi}R_d+k_{fi}L*T_{fi}M_{fi}E_d/M_{y_b}R_d+k_z*M_{fi}E_dz/M_zR_d$  
$=0.67281$

As $R \leq 1$, section capacity is OK for a FR period of 18 minutes.

**FIRE RESISTANCE SUMMARY**  
The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Location: Method of Clause 5.3.3

Plastic design of stanchions

Calculations in accordance with BS5950:Part 1:2000
Sections 5.3 and Annex G.

All loads and moments are factored.

Note: Plastic design may be used where loading is predominantly static and fatigue will not be a design criterion.

Design moment about z-z axis $M_z=614$ kNm
Design shear force $F_v=94.9$ kN
Design axial load (+ve comp) $F=173$ kN
Length of member $L=6.47$ m

533 x 210 x 92 UB.
Young's Modulus $E=205$ kN/mm²

Far end bending moment value $M_f=462$ kNm
Maximum moment on segment $M_{LT}=462$ kNm
Far end BM $\beta M=0$ kNm
Effective length $L_e=4870$ mm

maximum moment on segment $M'_z=462$ kNm
Far end BM $\beta M'_z=0$ kNm

UNIVERSAL BEAM 533 x 210 x 92 UB Grade S 275
DESIGN SUMMARY Section is satisfactory for plastic design in accordance with BS5950-1.
Provide lateral restraint to both flanges at plastic hinge location.
Intermediate torsional restraint(s) provided 1600 mm below hinge.

Compressive load 173 kN
Compressive resistance 1485 kN
Factored moment 614 kNm
Local capacity check 1 ≤ 1
Overall buckling check 0.639 ≤ 1
0.89 ≤ 1
**Location: Method of Annex G**

**Plastic design of stanchions**

Calculations in accordance with BS5950: Part 1:2000 Sections 5.3 and Annex G.

All loads and moments are factored.

Note: Plastic design may be used where loading is predominantly static and fatigue will not be a design criterion.

- Design moment about z-z axis: \( M_z = 820 \text{ kNm} \)
- Design shear force: \( F_v = 102 \text{ kN} \)
- Design axial load (+ve comp): \( F = 193 \text{ kN} \)
- Length of member: \( L = 8 \text{ m} \)

610 x 229 x 113 UB.

- Young's Modulus: \( E = 205 \text{ kN/mm}^2 \)
- Distance between ref. axes (fig G1): \( a' = 365 \text{ mm} \)
- Smaller end moment: \( M_s = -82 \text{ kNm} \)

**UNIVERSAL BEAM DESIGN SUMMARY**

610 x 229 x 113 UB Grade S 275

Section is satisfactory for plastic design in accordance with BS5950-1.

Provide lateral restraint to both flanges at plastic hinge location.

Intermediate torsional restraint(s) to be provided 2000 mm below hinge.

No other additional restraint needed

- Compressive load: 193 kN
- Compressive resistance: 1705 kN
- Factored moment: 820 kNm
- Local capacity check: \( 0.994 \leq 1 \)
Location: Method of 5.3.4

Plastic design of stanchions

Calculations in accordance with BS5950:Part 1:2000
Sections 5.3 and Annex G.

All loads and moments are factored.

Note: Plastic design may be used where loading is predominantly static and fatigue will not be a design criterion.

Design moment about z-z axis \( M_z = 635 \text{ kNm} \)
Design shear force \( F_v = 108 \text{ kN} \)
Design axial load (+ve comp) \( F = 206 \text{ kN} \)
Length of member \( L = 6 \text{ m} \)

457 x 191 x 89 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Distance about z-z axis \( L_z = 6000 \text{ mm} \)
Maximum moment on segment \( M_{LT} = 402 \text{ kNm} \)
Far end BM \( \beta_m = -64 \text{ kNm} \)
Effective length \( L_e = 4500 \text{ mm} \)

Maximum moment on segment \( M'_z = 635 \text{ kNm} \)
Far end BM \( \beta_mz = -64 \text{ kNm} \)

UNIVERSAL BEAM
DESIGN SUMMARY

457 x 191 x 89 UB Grade S 355
Section is satisfactory for plastic design in accordance with BS5950-1.
Provide lateral restraint to both flanges at plastic hinge location.
Intermediate torsional restraint(s) provided 2000 mm below hinge

Compressive load \( 206 \text{ kN} \)
Compressive resistance \( 1940 \text{ kN} \)
Factored moment \( 635 \text{ kNm} \)
Local capacity check \( 0.968 \leq 1 \)
Overall buckling check \( 0.709 \leq 1 \) \( 0.688 \leq 1 \)
Location: Method of Clause 5.3.3 - UC section

Plastic design of stanchions

Calculations in accordance with BS5950:Part 1:2000
Sections 5.3 and Annex G.

All loads and moments are factored.

Note: Plastic design may be used where loading is predominantly static and fatigue will not be a design criterion.

Design moment about z-z axis \( M_z = 414 \text{ kNm} \)
Design shear force \( F_v = 94.9 \text{ kN} \)
Design axial load (+ve comp) \( F = 575 \text{ kN} \)
Length of member \( L = 4.27 \text{ m} \)

254 x 254 x 132 UC.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

**UNIVERSAL COLUMN 254 x 254 x 132 UC Grade S 275**

**DESIGN SUMMARY**

Section is satisfactory for plastic design in accordance with BS5950-1.
Provide lateral restraint to both flanges at plastic hinge location.
Location: Ex1 - Based on example 13, 'Steelwork Design Guide'

Plastic design of stanchions

Calculations in accordance with BS EN 1993-1-1:2005
Clause 6.3.5 and Annex BB.3.

All loads and moments are factored.

Note: Plastic design may be used where loading is predominantly static and fatigue will not be a design criterion.

Design moment about major axis \( \text{MyEd} = 608 \text{ kNm} \)
Design shear force \( \text{VzEd} = 94.9 \text{ kN} \)
Design axial load (+ve comp) \( \text{NEd} = 173 \text{ kN} \)
Length of member \( L = 6.47 \text{ m} \)

533 x 210 x 92 UKB

Moment resistance - Clause 6.2.5

Design moment resistance yy axis \( \text{McRd} = \text{fy} \times \text{Wply} / (\text{gamM0} \times 10^3) = 649 \text{ kNm} \)
Since \( \text{MyEd} \leq \text{McRd} \) (608 kNm \( \leq \) 649 kNm) design moment within the moment resistance.

Cross-section resistance - Clause 6.2(7)

\[
\frac{\text{NEd}}{A \times \text{fy}} + \frac{\text{MyEd}}{\text{McRd}} \leq 1
\]

Unity factor \( \text{uf} = \frac{\text{NEd} \times 10}{(A \times \text{fy})} + \frac{\text{MyEd}}{\text{McRd}} = 0.991 \)
The interaction formula is satisfied.

Column stability

At the column plastic hinge location (i.e. at the bottom of the haunch lateral restraint to both flanges needs to be provided (Annex BB.3). The maximum distance from the plastic hinge to the adjacent lateral restraint is derived from the following expression:

\[
\text{Lm}' = 38 \text{ iz } \left[ \frac{1}{57.4} \left( \frac{\text{NEd}'}{A} \right) + \frac{1}{756} \left( \frac{(\text{Wply})^2}{A \times \text{It}} \right) \left( \frac{\text{fy}}{235} \right)^2 \right]^{0.5}
\]

Axial load at hinge position \( \text{NEd}' = 173 \text{ kN} \)
Provide a restraint at \( \text{Lm} = 1125 \text{ mm below plastic hinge} \)
CALCULATIONS FOR LOWER COLUMN SEGMENT AB

Column segment length AB
Le=L*1000-Lm=5345 mm
Moment at column base
MyEdA=0 kNm (pinned base)
Moment at restraint position B
MyEdB=502 kNm

Lateral torsional buckling - lower column segment

Length for LTB
LT=Lz/1000=5.35 m
Critical moment
Mcr=C1*Et*Sr/10^6=981 kNm
Modified chiLT factor
ChiLT=0.862
Design buckling resistance moment
MbRd=ChiLT*Wply*fy/(gamM1*10^3)=560 kNm

As MyEdB ≤ MbRd ( 502 kNm ≤ 560 kNm ), design moment at
point B is within the buckling resistance moment. Hence OK.

As MyEdB ≤ MbRd ( 502 kNm ≤ 560 kNm ), design moment at
point B is within the buckling resistance moment. Hence OK.

CALCULATIONS FOR UPPER COLUMN SEGMENT BC

Upper column segment length
Le=Lm=1125 mm
Lengths for buckling:
about y-y axis (major axis) Ly=6470 mm (whole col.length)
about z-z axis (minor axis) Lz=1125 mm

Lateral torsional buckling

Length for LTB
LT=Lz/1000=1.12 m
Critical moment
Mcr=C1*Et*Sr/10^6=11245 kNm
Lateral torsional buckling effects are ignored.
Modification factor
ChiLT=1.0
Design buckling resistance moment
MbRd=ChiLT*Wply*fy/(gamM1*10^3)=649 kNm

As MyEd ≤ MbRd ( 608 kNm ≤ 649 kNm ), design moment at
point B is within the buckling resistance moment. Hence OK.

As MyEd ≤ MbRd ( 608 kNm ≤ 649 kNm ), design moment at
point B is within the buckling resistance moment. Hence OK.
UNIVERSAL BEAM

DESIGN SUMMARY

533 x 210 x 92 UKB Grade S 275

Section is satisfactory for plastic design to EC3 Part 1-1.

Lateral restraint to both flanges assumed at points B and C.

Design axial load 173 kN
Buckling resistance 1281 kN
Design moment at point C 608 kNm
Design moment at point B 502 kNm
Cross-section resistance 0.991 ≤ 1
Interaction factors 0.711 ≤ 1
0.997 ≤ 1
0.62 ≤ 1
0.884 ≤ 1
Location: Ex2 - I Column section with moment at base

Plastic design of stanchions

Calculations in accordance with BS EN 1993-1-1:2005
Clause 6.3.5 and Annex BB.3.

All loads and moments are factored.

Note: Plastic design may be used where loading is predominantly static and fatigue will not be a design criterion.

Design moment about major axis \( M_{yEd} = 820 \text{ kNm} \)
Design shear force \( V_{zEd} = 102 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 193 \text{ kN} \)
Length of member \( L = 8 \text{ m} \)

610 x 229 x 125 UKB

Moment resistance - Clause 6.2.5

Design moment resistance yy axis \( M_{crd} = f_y \cdot W_{ply} / (\gamma M_0 \cdot 10^3) = 975 \text{ kNm} \)
Since \( M_{yEd} \leq M_{crd} \) (820 kNm \( \leq 975 \text{ kNm} \)) design moment within the moment resistance.

Cross-section resistance - Clause 6.2(7)

\[
\frac{N_{Ed}}{A \cdot f_y} + \frac{M_{yEd}}{M_{crd}} \leq 1
\]

Unity factor \( u_f = \frac{N_{Ed} \cdot 10}{A \cdot f_y} + \frac{M_{yEd}}{M_{crd}} = 0.887 \)
The interaction formula is satisfied.

Column stability

At the column plastic hinge location (i.e. at the bottom of the haunch lateral restraint to both flanges needs to be provided (Annex BB.3)). The maximum distance from the plastic hinge to the adjacent lateral restraint is derived from the following expression:

\[
L_{m'} = \frac{1}{57.4} \left[ \frac{N_{Ed'}^2}{A} \right] + \frac{1}{756(C1)^2} \left[ \frac{(W_{ply})^2}{A \cdot I_t} \right] \cdot \left[ \frac{f_y}{235} \right]^2 \cdot 0.5
\]

Axial load at hinge position \( N_{Ed'} = 193 \text{ kN} \)
Provide a restraint at \( L_{m'} = 1300 \text{ mm below plastic hinge} \)
CALCULATIONS FOR LOWER COLUMN SEGMENT AB

Column segment length AB          Le=L*1000-Lm=6700 mm
Maximum moment on segment         MyEdB=673 kNm
Far end bending moment value      MyEdA=-82 kNm (fixed base)

Lateral torsional buckling - lower column segment

Length for LTB                    LT=Lz/1000=6.7 m
Critical moment                   Mcr=C1*Et*Sr/10^6=1368 kNm
Modified chiLT factor             ChiLT=chiLT/f=0.85
Design buckling resistance moment MbRd=ChiLT*Wply*fy/(gamM1*10^3) =829 kNm

As MyEdB ≤ MbRd ( 673 kNm ≤ 829 kNm ), design moment at point B is within the buckling resistance moment. Hence OK.
Far end bending moment            pM(1)=-82 kNm
Far end bending moment            pM(2)=-82 kNm
Interaction factor                kyy=0.564
Interaction factor                kzy=0.954
Section chosen is suitable.
Section chosen is suitable.

CALCULATIONS FOR UPPER COLUMN SEGMENT BC

Upper column segment length       Le=Lm=1300 mm
Lengths for buckling:
about y-y axis (major axis)       Ly=8000 mm (whole col.length)
about z-z axis (minor axis)       Lz=1300 mm

Lateral torsional buckling

Length for LTB                    LT=Lz/1000=1.3 m
Critical moment                   Mcr=C1*Et*Sr/10^6=15940 kNm
Lateral torsional buckling effects are ignored.
Modification factor               ChiLT=1.0
Design buckling resistance moment MbRd=ChiLT*Wply*fy/(gamM1*10^3) =975 kNm

As MyEd ≤ MbRd ( 820 kNm ≤ 975 kNm ), design moment at point B is within the buckling resistance moment. Hence OK.
Far end bending moment            pM(3)=673 kNm
Interaction factor                kyy=0.564
Interaction factor                kzy=0.896
Section chosen is suitable.
Section chosen is suitable.
### UNIVERSAL BEAM

#### DESIGN SUMMARY

- **Section**: 610 x 229 x 125 UKB Grade S 275
- **Design is satisfactory for plastic design to EC3 Part 1-1.**
- **Lateral restraint to both flanges assumed at points B and C.**
- **Design axial load**: 193 kN
- **Buckling resistance**: 1406 kN
- **Design moment at point C**: 820 kNm
- **Design moment at point B**: 673 kNm
- **Cross-section resistance**: $0.887 \leq 1$
- **Interaction factors**:
  - $0.604 \leq 1$
  - $0.912 \leq 1$
  - $0.52 \leq 1$
  - $0.799 \leq 1$
Location: Ex3 - I Column section with moment at base

Plastic design of stanchions

Calculations in accordance with BS EN 1993-1-1:2005 Clause 6.3.5 and Annex BB.3. All loads and moments are factored.

Note: Plastic design may be used where loading is predominantly static and fatigue will not be a design criterion.

Design moment about major axis \( \text{MyEd}=635 \text{ kNm} \)
Design shear force \( \text{VzEd}=108 \text{ kN} \)
Design axial load (+ve comp) \( \text{NEd}=206 \text{ kN} \)
Length of member \( \text{L}=6 \text{ m} \)

457 x 191 x 89 UKB

Moment resistance - Clause 6.2.5

Design moment resistance yy axis \( \text{McRd}=\text{fy*Wply}/(\text{gamM0*10^3})=693 \text{ kNm} \)
Since \( \text{MyEd} \leq \text{McRd} \) ( 635 kNm ≤ 693 kNm ) design moment within the moment resistance.

Cross-section resistance - Clause 6.2(7)

\[
\frac{\text{NEd}}{\text{A*fy}} + \frac{\text{MyEd}}{\text{McRd}} \leq 1
\]

Unity factor \( \text{uf}=\text{NEd*10/(A*fy)}+\text{MyEd/McRd}=0.968 \)
The interaction formula is satisfied.

Column stability

At the column plastic hinge location (i.e. at the bottom of the haunch lateral restraint to both flanges needs to be provided (Annex BB.3). The maximum distance from the plastic hinge to the adjacent lateral restraint is derived from the following expression:

\[
\text{Lm'} = \frac{1}{57.4} \left[ \frac{\text{NEd'}}{\text{A}} \right] + \frac{1}{756(C1)^2} \left[ \frac{(\text{Wply})^2}{\text{A.It}} \right] \left[ \frac{\text{fy}}{235} \right]^{2}^{0.5}
\]

Axial load at hinge position \( \text{NEd'}=206 \text{ kN} \)
Provide a restraint at \( \text{Lm}=950 \text{ mm below plastic hinge} \)
CALCULATIONS FOR LOWER COLUMN SEGMENT AB

Column segment length AB  \( L_e = L \times 1000 - L_m = 5050 \text{ mm} \)
Maximum moment on segment \( M_{YEdB} = 524 \text{ kNm} \)
Far end bending moment value \( M_{YEdA} = -64 \text{ kNm} \) (fixed base)

**Lateral torsional buckling - lower column segment**

Length for LTB  \( L_T = L_z / 1000 = 5.05 \text{ m} \)
Critical moment  \( M_{cr} = C_1 \times E_t \times S_r / 10^6 = 987 \text{ kNm} \)
Modified chiLT factor  \( \text{ChiLT} = \text{chiLT} / f = 0.855 \)
Design buckling resistance moment  \( M_{Brd} = \text{ChiLT} \times W_{ply} \times f_y / (\gamma_M \times 10^3) = 593 \text{ kNm} \)

As \( M_{YEdB} \leq M_{Brd} \leq 524 \text{ kNm} \leq 593 \text{ kNm} \), design moment at point B is within the buckling resistance moment. Hence OK.

Far end bending moment  \( pM(1) = -64 \text{ kNm} \)
Far end bending moment  \( pM(2) = -64 \text{ kNm} \)
Interaction factor  \( k_{yy} = 0.566 \)
Interaction factor  \( k_{zy} = 0.948 \)
Section chosen is suitable.

CALCULATIONS FOR UPPER COLUMN SEGMENT BC

Upper column segment length  \( L_e = L_m = 950 \text{ mm} \)
Lengths for buckling:
about y-y axis (major axis)  \( L_y = 6000 \text{ mm} \) (whole col.length)
about z-z axis (minor axis)  \( L_z = 950 \text{ mm} \)

**Lateral torsional buckling**

Length for LTB  \( L_T = L_z / 1000 = 0.95 \text{ m} \)
Critical moment  \( M_{cr} = C_1 \times E_t \times S_r / 10^6 = 11919 \text{ kNm} \)
Lateral torsional buckling effects are ignored.
Modification factor  \( \text{ChiLT} = 1.0 \)
Design buckling resistance moment  \( M_{Brd} = \text{ChiLT} \times W_{ply} \times f_y / (\gamma_M \times 10^3) = 693 \text{ kNm} \)

As \( M_{YEd} \leq M_{Brd} \leq 635 \text{ kNm} \leq 693 \text{ kNm} \), design moment at point B is within the buckling resistance moment. Hence OK.

Far end bending moment  \( pM(3) = 524 \text{ kNm} \)
Interaction factor  \( k_{yy} = 0.566 \)
Interaction factor  \( k_{zy} = 0.886 \)
Section chosen is suitable.
Section chosen is suitable.
**UNIVERSAL BEAM**  
**DESIGN SUMMARY**  

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
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<tbody>
<tr>
<td>Section design to EC3 Part 1-1</td>
<td></td>
</tr>
<tr>
<td>Lateral restraint to both flanges assumed at B and C</td>
<td></td>
</tr>
<tr>
<td>Design axial load</td>
<td>206 kN</td>
</tr>
<tr>
<td>Buckling resistance</td>
<td>1315 kN</td>
</tr>
<tr>
<td>Design moment at point C</td>
<td>635 kNm</td>
</tr>
<tr>
<td>Design moment at point B</td>
<td>524 kNm</td>
</tr>
<tr>
<td>Cross-section resistance</td>
<td>0.968 ≤ 1</td>
</tr>
<tr>
<td>Interaction factors</td>
<td>0.659 ≤ 1</td>
</tr>
<tr>
<td></td>
<td>0.995 ≤ 1</td>
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<tr>
<td></td>
<td>0.57 ≤ 1</td>
</tr>
<tr>
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<td>0.864 ≤ 1</td>
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</tbody>
</table>
Location: Ex4 - Using UC section with moment at base

Plastic design of stanchions

Calculations in accordance with BS EN 1993-1-1:2005
Clause 6.3.5 and Annex BB.3.

All loads and moments are factored.

Note: Plastic design may be used where loading is predominantly static and fatigue will not be a design criterion.

Design moment about major axis \( M_{Ed} = 414 \text{ kNm} \)
Design shear force \( V_{zEd} = 94.9 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 575 \text{ kN} \)
Length of member \( L = 4.27 \text{ m} \)

254 x 254 x 132 UKC

Moment resistance - Clause 6.2.5

Design moment resistance \( yy \) axis \( M_{crd} = f_y \frac{W_{pl} y}{(\gamma M_0 \times 10^3)} = 496 \text{ kNm} \)

Since \( M_{Ed} \leq M_{crd} \) (414 kNm \( \leq 496 \text{ kNm} \)) design moment within the moment resistance.

Cross-section resistance - Clause 6.2(7)

\[
\frac{N_{Ed}}{A f_y} + \frac{M_{Ed}}{M_{crd}} \leq 1
\]

Unity factor \( u_f = \frac{N_{Ed} \times 10}{A f_y} + \frac{M_{Ed}}{M_{crd}} = 0.965 \)

The interaction formula is satisfied.

Column stability

At the column plastic hinge location (i.e. at the bottom of the haunch lateral restraint to both flanges needs to be provided (Annex BB.3)). The maximum distance from the plastic hinge to the adjacent lateral restraint is derived from the following expression:

\[
38 iz
\]

\[
L_m' = \left[ \frac{1}{57.4} \left( \frac{N_{Ed}'}{A} \right) + \frac{1}{756(C1)^2} \left( \frac{(W_{pl}y)^2}{A \cdot I_t} \right) \right] \left( \frac{f_y}{235} \right)^2 ^{0.5}
\]

Axial load at hinge position \( N_{Ed}' = 575 \text{ kN} \)
Provide a restraint at \( L_m = 1825 \text{ mm below plastic hinge} \)
CALCULATIONS FOR LOWER COLUMN SEGMENT AB

Column segment length AB
Le=L*1000-Lm=2445 mm

Maximum moment on segment
MyEdB=186 kNm

Far end bending moment value
MyEdA=-120 kNm (fixed base)

Lateral torsional buckling - lower column segment

Length for LTB
LT=Lz/1000=2.45 m

Critical moment
Mcr=C1*Et*Sr/10^6=10319 kNm

Design buckling resistance moment
MbRd=ChiLT*Wply*fy/(gamM1*10^3)
        =496 kNm

As MyEdB ≤ MbRd ( 186 kNm ≤ 496 kNm ), design moment at point B is within the buckling resistance moment. Hence OK.

Far end bending moment
pM(1)=-120 kNm

Far end bending moment
pM(2)=-120 kNm

Interaction factor
kyy=0.498

Interaction factor
kzy=0.964

Section chosen is suitable.

Section chosen is suitable.

CALCULATIONS FOR UPPER COLUMN SEGMENT BC

Upper column segment length
Le=Lm=1825 mm

Lengths for buckling:

about y-y axis (major axis)    Ly=4270 mm (whole col.length)
about z-z axis (minor axis)    Lz=1825 mm

Lateral torsional buckling

Length for LTB
LT=Lz/1000=1.82 m

Critical moment
Mcr=C1*Et*Sr/10^6=9242 kNm

Lateral torsional buckling effects are ignored.

Modification factor
ChiLT=1.0

Design buckling resistance moment
MbRd=ChiLT*Wply*fy/(gamM1*10^3)
        =496 kNm

As MyEd ≤ MbRd ( 414 kNm ≤ 496 kNm ), design moment at point B is within the buckling resistance moment. Hence OK.

Far end bending moment
pM(3)=186 kNm

Interaction factor
kyy=0.498

Interaction factor
kzy=0.908

Section chosen is suitable.

Section chosen is suitable.

UNIVERSAL COLUMN

DESIGN SUMMARY

254 x 254 x 132 UKC Grade S 275

Section is satisfactory for plastic design to EC3 Part 1-1.

Lateral restraint to both flanges assumed at points B and C.

Design axial load
575 kN

Buckling resistance
4452 kN

Design moment at point C
414 kNm

Design moment at point B
186 kNm

Cross-section resistance
0.965 ≤ 1
Interaction factors

0.545 ≤ 1
0.491 ≤ 1
0.545 ≤ 1
0.888 ≤ 1
Location: Ex5 - Based on example in SCI P397

\[
\begin{align*}
\text{Plastic design of stanchions} \\
\text{Calculations in accordance with BS EN 1993-1-1:2005} \\
\text{Clause 6.3.5 and Annex BB.3.}
\end{align*}
\]

All loads and moments are factored.

Note: Plastic design may be used where loading is predominantly static and fatigue will not be a design criterion.

Design moment about major axis \( \text{MyEd}=1034 \text{kNm} \)
Design shear force \( \text{VzEd}=100 \text{kN} \)
Design axial load (+ve comp) \( \text{NEd}=271 \text{kN} \)
Length of member \( \text{L}=12.3 \text{m} \)

686 x 254 x 125 UKB

**Moment resistance - Clause 6.2.5**

Design moment resistance yy axis \( \text{McRd}=fy*Wply/(\text{gamM0}*10^3)=1377 \text{kNm} \)
Since \( \text{MyEd} \leq \text{McRd} \) (1034 kNm \( \leq \) 1377 kNm) design moment within the moment resistance.

**Cross-section resistance - Clause 6.2(7)**

\[
\frac{\text{NEd}}{A*fy} + \frac{\text{MyEd}}{\text{McRd}} \leq 1
\]

Unity factor \( \text{uf} = \frac{\text{NEd} * 10}{(A*fy)} + \frac{\text{MyEd}}{\text{McRd}} = 0.801 \)
The interaction formula is satisfied.

**Column stability**

At the column plastic hinge location (i.e. at the bottom of the haunch lateral restraint to both flanges needs to be provided (Annex BB.3)). The maximum distance from the plastic hinge to the adjacent lateral restraint is derived from the following expression:

\[
\text{Lm}' = \left[ \frac{1}{57.4} \left( \frac{\text{NEd}'}{A} \right) + \frac{1}{756(C1)^2} \left( \frac{(Wply)^2}{A*It} \right) \right] \left( \frac{\text{fy}^2}{235} \right)^{0.5}
\]

Axial load at hinge position \( \text{NEd}'=271 \text{kN} \)
Provide a restraint at \( \text{Lm}=1050 \text{mm} \text{ below plastic hinge} \)
CALCULATIONS FOR LOWER COLUMN SEGMENT AB

Column segment length AB \( Le = 6450 \text{ mm} \)
Moment at column base \( MyEdA = 0 \text{ kNm} \) (pinned base)
Moment at restraint position C \( MyEdC = 946 \text{ kNm} \)
Moment at restraint position B \( MyEdB = 542 \text{ kNm} \)

**Lateral torsional buckling - lower column segment**

Length for LTB \( LT = Lz / 1000 = 6.45 \text{ m} \)
Critical moment \( Mcr = C1 * Et * Sr / 10^6 = 1508 \text{ kNm} \)
Modified chiLT factor \( ChiLT = chiLT / f = 0.756 \)
Design buckling resistance moment \( MbRd = ChiLT * Wply * fy / (gamM1 * 10^3) = 1040 \text{ kNm} \)

As \( MyEdB \leq MbRd \) ( 542 kNm \( \leq 1040 \text{ kNm} \) ), design moment at point B is within the buckling resistance moment. Hence OK.

Far end bending moment \( pM(1) = 0 \text{ kNm} \)
Far end bending moment \( pM(2) = 0 \text{ kNm} \)
Interaction factor \( kyy = 0.611 \)
Interaction factor \( kzy = 0.955 \)
Section chosen is suitable.

Section chosen is suitable.

CALCULATIONS FOR INTERMEDIATE COLUMN SEGMENT BC

Lengths for buckling:
about y-y axis (major axis) \( Ly = 12300 \text{ mm} \) (whole col.length)
about z-z axis (minor axis) \( Lz = 4800 \text{ mm} \)

**Lateral torsional buckling**

Length for LTB \( LT = Lz / 1000 = 4.8 \text{ m} \)
Critical moment \( Mcr = C1 * Et * Sr / 10^6 = 1813 \text{ kNm} \)
Limiting slenderness value \( lamLT0 = 0.4 \)
Modified chiLT factor \( ChiLT = chiLT / f = 0.766 \)
Design buckling resistance moment \( MbRd = ChiLT * Wply * fy / (gamM1 * 10^3) = 1054 \text{ kNm} \)

As \( MyEdC \leq MbRd \) ( 946 kNm \( \leq 1054 \text{ kNm} \) ), design moment at point B is within the buckling resistance moment. Hence OK.

Far end bending moment \( pM(4) = 542 \text{ kNm} \)
Interaction factor \( kyy = 0.611 \)
Interaction factor \( kzy = 0.983 \)
Section chosen is suitable.

Unity factor \( Un6 = NEd / (chiz * NRd) + kzy * MyEdC / (ChiLT * McRd) = 0.983 \)
Section chosen is suitable.

CALCULATIONS FOR UPPER COLUMN SEGMENT CD
Upper column segment length
Le=Lm=1050 mm

Lengths for buckling:
about y-y axis (major axis) Ly=12300 mm (whole col.length)
about z-z axis (minor axis) Lz=1050 mm

Lateral torsion buckling

Length for LTB LT=Lz/1000=1.05 m
Critical moment Mcr=C1*Et*Sr/10^6=28689 kNm
Lateral torsional buckling effects are ignored.
Modification factor ChiLT=1.0
Design buckling resistance moment MbRd=ChiLT*Wply*fy/(gamM1*10^3)
=1377 kNm
As MyEd ≤ MbRd (1034 kNm ≤ 1377 kNm), design moment at point B is within the buckling resistance moment. Hence OK.
Far end bending moment pM(3)=946 kNm
Interaction factor kyy=0.611
Interaction factor kzy=0.858
Section chosen is suitable.

UNIVERSAL BEAM 686 x 254 x 125 UKB Grade S 355
DESIGN SUMMARY Section is satisfactory for plastic design to EC3 Part 1-1.
Lateral restraint to both flanges assumed at points B, C and D.
Design axial load 271 kN
Buckling resistance 1715 kN
Design moment at point D 1034 kNm
Design moment at point C 946 kNm
Design moment at point B 542 kNm
Cross-section resistance 0.801 ≤ 1
Interaction factors 0.657 ≤ 1
0.656 ≤ 1
0.509 ≤ 1
0.694 ≤ 1
0.649 ≤ 1
0.983 ≤ 1
Location: Example 14, 'Steelwork Design Guide'
Location: single bolt connection

**Design of Axially Loaded Section**

formed from single angle
In accordance with BS5950-1:2000

Loads are factored.

Factored axial compressive load $F_c = 15 \text{ kN}$
Factored axial tensile load $F_t = 14 \text{ kN}$
Length $L = 2000 \text{ mm}$

75 x 50 x 6 mm Unequal Angle.
Young's Modulus $E = 205 \text{ kN/mm}^2$

Diameter of bolts $d_b = 12 \text{ mm}$
About axis $//$ to connected leg $L_a = 2000 \text{ mm}$
About axis $\perp$ to connected leg $L_b = 2000 \text{ mm}$
About v-v axis $L_v = 2000 \text{ mm}$

**UNEQUAL ANGLE SECTION SUMMARY**

75 x 50 x 6 Grade S 275
Section is satisfactory for axial load.
Long leg forming connection
One bolt M 12 at each end
Compressive load 15 kN
Compressive resistance 27.297 kN
Tensile load 14 kN
Tensile capacity 157.77 kN
Location: Slender section

Design of Axially Loaded Section

formed from single angle
In accordance with BS5950-1:2000

Loads are factored.

Factored axial compressive load \( F_c = 73 \text{ kN} \)
Factored axial tensile load \( F_t = 0 \text{ kN} \)
Length \( L = 3500 \text{ mm} \)

125 x 75 x 8 mm Unequal Angle.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Diameter of bolts \( d_b = 16 \text{ mm} \)
About axis // to connected leg \( L_a = 3500 \text{ mm} \)
About axis ⊥ to connected leg \( L_b = 3500 \text{ mm} \)
About v-v axis \( L_v = 3500 \text{ mm} \)

UNEQUAL ANGLE
SECTION
SUMMARY

125 x 75 x 8 Grade S 275
Section is satisfactory for axial load.
Short leg forming connection
At least two bolts M 16 at each end
Compressive load 73 kN
Compressive resistance 73.828 kN
Location: Ex1 - Based on 'Steelwork Design Guide'

Design of axially loaded angle

Calculations are in accordance with BS EN 1993-1-1:2005.

All loads are factored.

- Design axial compressive load \( N_{Ed} = 43 \text{ kN} \)
- Design axial tensile load \( N_{EdT} = 12 \text{ kN} \)
- Length of member \( L = 2154 \text{ mm} \)

60 x 60 x 8 mm Equal Angle (Advance UKA)

**EQUAL ANGLE DESIGN SUMMARY**

- Section is satisfactory for axial load.
- Welding equivalent to at least two bolts at each end.
- Design compression load \( 43 \text{ kN} \)
- Compressive resistance \( 60.006 \text{ kN} \)
- Design tensile load \( 12 \text{ kN} \)
- Tensile resistance \( 185.12 \text{ kN} \)
Location: Ex2 - Single bolt connection

Design of axially loaded angle

Calculations are in accordance with BS EN 1993-1-1:2005.

All loads are factored.

Design axial compressive load \( N_{Ed} = 15 \) kN
Design axial tensile load \( N_{EdT} = 14 \) kN
Length of member \( L = 2000 \) mm

75 x 50 x 6 mm Unequal Angle (Advance UKA)
Diameter of bolts \( d_b = 12 \) mm
Edge distance \( e_2 = 37.5 \) mm

75 x 50 x 6 Grade S 275
Section is satisfactory for axial load.
Long leg forming connection
One bolt M 12 at each end
Design compression load 15 kN
Compressive resistance 38.176 kN
Design tensile load 14 kN
Tensile resistance 136.42 kN
Location: Ex3 - Class 3 section

Design of axially loaded angle

Calculations are in accordance with BS EN 1993-1-1:2005.

All loads are factored.

Design axial compressive load \( \text{NEd}=73 \text{ kN} \)
Design axial tensile load \( \text{NEdT}=0 \text{ kN} \)
Length of member \( L=3500 \text{ mm} \)

125 x 75 x 10 mm Unequal Angle (Advance UKA)
Diameter of bolts \( d_b=16 \text{ mm} \)
Pitch \( p_1=90 \text{ mm} \)

UNEQUAL ANGLE

125 x 75 x 10 Grade S 275

DESIGN

Section is satisfactory for axial load.

Short leg forming connection

At least two bolts M 16 at each end

Design compression load 73 kN

Compressive resistance 101.17 kN
Location: Design Example 6 Curved Steel publication

I section beam curved on plan

Calculations in accordance with BS5950-1:2000 and SCI publication entitled "Design of Curved Steel".

Moment connections are assumed in the vertical plane and pin connections on plan.

Loads and moments are factored.

838 x 292 x 226 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Radius of curvature of beam \( R = 5 \text{ m} \)
Accompanying shear force \( F_v = 77.6 \text{ kN} \)
Maximum radial load \( F_a = 25.6 \text{ kN} \)
Max. horiz. moment in compr. flange \( M_{ycf} = 90.4 \text{ kNm} \)
Quarter point \( M_{x2} = -34.5 \text{ kNm} \)
Mid-span \( M_{x3} = 65 \text{ kNm} \)
Three quarter point \( M_{x4} = -34.5 \text{ kNm} \)
Maximum moment on central half \( M_{24} = 65 \text{ kNm} \)
Quarter point \( M_{y2} = 86.8 \text{ kNm} \)
Mid-span \( M_{y3} = 43.7 \text{ kNm} \)
Three quarter point \( M_{y4} = -33.4 \text{ kNm} \)
Maximum moment on central half \( M_{24y} = 86.8 \text{ kNm} \)
Maximum moment \( M_{LT} = 274.1 \text{ kNm} \)
Quarter point \( M_{m2} = 127 \text{ kNm} \)
Mid-span \( M_{m3} = 34.5 \text{ kNm} \)
Three quarter point \( M_{m4} = 39 \text{ kNm} \)

UNIVERSAL BEAM SECTION
838 x 292 x 226 UB Grade S 275

DESIGN
Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.

Factored shear force \( 77.6 \text{ kN} \)
Shear capacity \( 2178.2 \text{ kN} \)
Vertical moment \( 274.1 \text{ kNm} \)
Moment capacity \( 2117.8 \text{ kNm} \)
Horizontal moment \( 90.4 \text{ kNm} \)
Local capacity check \( 1.0086 \leq 1 \)
Buckling resistance \( 1034.7 \text{ kNm} \)
Overall buckling check \( 0.71636 \leq 1 \)

0.79188 \leq 1
Location: Curved I-beam on plan

I section beam curved on plan

Beam is curved on plan & design calculations are in accordance with BS EN 1993-1-1:2005.

Moment connections are assumed in the vertical plane and pin connections on plan. All loads and moments are factored.

838 x 292 x 226 UKB
Dimensions (mm): h=850.9 b=293.8 tw=16.1 tf=26.8 r=17.8
Properties (cm): Iy=340000 Iz=11400 Wply=9160 Wplz=1210 It=514
Radius of curvature of beam R=5 m
Accompanying design shear force VzEd=77.6 kN
Maximum radial load Fa=25.6 kN
Design horiz.BM in compr.flange MzEd=90.4 kNm
Effective length parameter K=1
Correction factor kc=0.9

UNIVERSAL BEAM 838 x 292 x 226 UB Grade S 275
DESIGN Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.
SUMMARY Factored shear force 77.6 kN
Design shear resistance 2224.3 kN
Design vert.bending moment 274.1 kNm
Design moment resistance 2117.8 kNm
Buckling resistance moment 2117.8 kNm
Horizontal design moment 90.4 kNm
Interaction factors 0.89591 ≤ 1
0.26277 ≤ 1
0.38273 ≤ 1
Location: Design Example 6 Curved Steel publication

Structural Hollow Section Design

Beam curved in plan.
Calculations are in accordance with BS5950-1:2000 and SCI publication "Design of Curved Steel"
Moment connections are assumed in the vertical plane, pin connections on plan.
All loads and moments are factored.

300 x 200 x 12.5 RHS - Hot finished.
Properties (cm): A=117 rx=11 Zx=952 Sx=1170 Ix=14300
J=15700 C=1220 Zy=754 Sy=877 Iy=7540 ry=8.02
Radius of curvature of beam
R=5 m
about major axis
Mx=253 kNm
Accompanying shear force
Fv=77.6 kN
Maximum radial load
Fa=25.6 kN
Young's Modulus
E=205 kN/mm²
Maximum torque
Tm=37.6 kNm
Quarter point
mx2=-34.5 kNm
Mid-span
mx3=65 kNm
Three quarter point
mx4=-34.5 kNm
Maximum moment on central half
M24=65 kNm
Quarter point
m2=127 kNm
Mid-span
m3=34.5 kNm
Three quarter point
m4=39 kNm

HOT FINISHED
RECTANGULAR HOLLOW SECTION DESIGN
SUMMARY
In accordance with EN 10210
300 x 200 x 12.5 RHS Grade S 355
Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.
Factored shear force
77.6 kN
Shear capacity
1495.3 kN
Factored moment
260.36 kNm
Moment capacity
330.83 kNm
Local capacity check
0.78699 ≤ 1
Buckling resistance
330.83 kNm
Overall buckling check
0.25443 ≤ 1
0.34627 ≤ 1
**Location: Ex1 - Steel beam curved on plan**

**Structural hollow section design**

Beam is curved on plan and design calculations are in accordance with BS EN 1993-1-1:2005.

The ends of the curved beam are assumed to be fixed in the vertical plane and pinned on plan. All loads and moments are factored.

300 x 200 x 12.5 RHS - Hot finished.

Properties (cm): A=117 iz=8.02 Wely=952 Wply=1170 Iy=14300 
It=15700 Wt=1220 Welz=754 Wplz=877 Iz=7540 iy=11

Radius of curvature of beam R=5 m
Accompanying design shear force VzEd=77.6 kN
Maximum radial load Fa=25.6 kN
Design torque Tm=37.6 kNm
Effective length parameter K=1
Correction factor kc=0.9

**HOT FINISHED**

**RECTANGULAR HOLLOW SECTION 300 x 200 x 12.5 RHS  Grade S 355**

Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.

Design shear force 77.6 kN
Design shear resistance 1438.8 kN
Design bending moment 260.36 kNm
Moment resistance 330.83 kNm
Buckling resistance 330.83 kNm
Overall buckling check 0.78699 ≤ 1
Location: Design Example 6 Curved Steel publication

Circular hollow section

Section is curved on plan. Calculations in accordance with BS5950: Part 1:2000 and SCI publication "Design of Curved Steel". Moment connections are assumed in the vertical plane and pin connections on plan. All loads and moments are factored.

Radius of curvature of beam about the major axis
Accompanying shear force
Maximum radial load
Young's Modulus
Maximum torque
Quarter point
Mid-span
Three quarter point
Maximum moment on central half
Quarter point
Mid-span
Three quarter point

R=5 m
Mx=253 kNm
Fv=77.6 kN
Fa=25.6 kN
E=205 kN/mm²
Tm=37.6 kN
mx2=-34.5 kNm
mx3=65 kNm
mx4=-34.5 kNm
M24=65 kNm
m2=127 kNm
m3=34.5 kNm
m4=39 kNm

HOT FINISHED
CIRCULAR HOLLOW SECTION
DESIGN
SUMMARY

In accordance with EN 10210
323.9 O.D. x 10 CHS Grade S 355
Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.

Factored shear force
Shear capacity
Factored moment
Moment capacity
Local capacity check
Buckling resistance
Overall buckling check

77.6 kN
1260.3 kN
261.04 kNm
261.86 kNm
0.99687 ≤ 1
261.86 kNm
0.32196 ≤ 1
0.43862 ≤ 1
Location: Ex1 - Curved CHS with bending about the major axis

Circular hollow section design

Beam is curved on plan and calculations are in accordance with BS EN 1993-1-1:2005.

The ends of the curved beam are assumed to be fixed in the vertical plane and pinned on plan. All loads and moments are factored.

323.9 dia x 10 thick CHS - Hot finished.
Properties (cm): A=98.615 iy=11.104 Wely=750.75 Wply=985.67 Iy=12158 It=24317
Radius of curvature of beam R=5 m
Accompanying design shear force VzEd=77.6 kN
Max radial load from analysis Fa=25.6 kN
Design torque from analysis Tm=37.6 kNm
Effective length parameter K=1
Correction factor kc=0.9

HOT FINISHED
CIRCULAR HOLLOW SECTION
DESIGN SUMMARY

In accordance with EN 10210
323.9 O.D. x 10 CHS Grade S 355
Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.
Design shear force 77.6 kN
Design shear resistance 1286.7 kN
Design bending moment 261.04 kNm
Moment resistance 262.04 kNm
Buckling resistance 262.04 kNm
Interaction factor 0.99617 ≤ 1
Location: Example including EC2 summary

Axially loaded column

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by SCI.

Compressive load is factored.

Factored axial compressive load $F = 2000$ kN
254 x 254 x 107 UC.
Young's Modulus $E = 205$ kN/mm$^2$

Effective length factor (x axis) $efx = 1$
Effective length factor (y axis) $efy = 0.7$

UNIVERSAL COLUMN SECTION 254 x 254 x 107 UC Grade S 275
Section is satisfactory for axial load check.

DESIGN Axial compressive load 2000 kN
SUMMARY Compression resistance 2019.7 kN
**Location: Example 6.1**

**Axially loaded column**

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by SCI.

Compressive load is factored.

Factored axial compressive load $F=700$ kN

457 x 191 x 67 UB.

Young's Modulus $E=205$ kN/mm²

Effective length factor (x axis) $efx=0.85$

Effective length factor (y axis) $efy=0.85$

**UNIVERSAL BEAM SECTION**

457 x 191 x 67 UB Grade S 355

Section is satisfactory for axial load check.

**DESIGN**

Axial compressive load 700 kN

**SUMMARY**

Compression resistance 2552.9 kN
**Location: Example 6.2**

Axially loaded column

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by SCI.

Compressive load is factored.

Factored axial compressive load $F=1840$ kN

$457 \times 191 \times 67$ UB.

Young's Modulus $E=205$ kN/mm²

Effective length factor (x axis) $efx=0.85$

Effective length factor (y axis) $efy=0.85$

**UNIVERSAL BEAM SECTION**

$457 \times 191 \times 67$ UB Grade S 355

Section is satisfactory for axial load check.

**DESIGN**

Axial compressive load $1840$ kN

**SUMMARY**

Compression resistance $2323.8$ kN
Location: Access steel example

Axially loaded column

Calculations are in accordance with BS EN 1993-1-1:2005.

Compressive load is factored.

Factored axial compressive load NEd=2000 kN

Dimensions (mm): h=307.9 b=305.3 tw=9.9 tf=15.4 r=15.2

Properties (cm): Iy=22300 Iz=7310 Wply=1590 Wplz=726 It=91.2

Factor about y-y axis fLy=1

Factor about z-z axis fLz=0.7

SECTION 305 x 305 x 97 UKC Grade S 275
DESIGN Axial compressive load 2000 kN
SUMMARY Compression resistance 2161.8 kN
Unity factor 0.92515 ≤ 1.0
**Location:** EX2 - Class 4 section

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**Axially loaded column**

Calculations are in accordance with BS EN 1993-1-1:2005.

Compressive load is factored.

Factored axial compressive load \( \text{NEd}=2000 \text{ kN} \)

1016 x 305 x 222 UKB

Dimensions (mm): \( h=970.3 \) \( b=300 \) \( tf=16 \) \( tw=21.1 \) \( r=30 \)

Properties (cm): \( I_y=408000 \) \( I_z=9550 \) \( W_{ply}=9810 \) \( W_{plz}=1020 \) \( I_t=390 \)

**Determine the effective section properties - EN 1992-1-5, Cl. 4.4**

The process is iterative as the amount of web not considered is a function of the stresses at the top and bottom of the web.

**Iteration 1:**

Gross area of section \( A=2*b*tf+hw*tw=27510 \text{ mm}^2 \)

Gross 2nd moment of area \( I_y=(b*(hw+2*tf)^3-(b-tw)*hw^3)/12=3.918E9 \text{ mm}^4 \)

Stress at top of web \( s_1=MyEd*10^6*hw/2/Iy=0 \text{ N/mm}^2 \)

Stress at bottom of web \( s_2=-MyEd*10^6*(hw/2)/Iy=0 \text{ N/mm}^2 \)

**Iteration 10:**

Position of effective centroid:

\[ z_{eff}=(A*h/2-Aw*(zeffp+be2+lw/2))/Aeff=485.15 \text{ mm} \]

Effective 2nd moment of area:

\[ I_{yeff}=I_y+A*(h/2-zeff)^2-tw*lw^2/12-Aw*(zeffp+be2+lw/2-zeff)^2=3.918E9 \text{ mm}^4 \]

Top of web stress \( s_{1(10)}=MyEd*10^6*(h-tf-zeff)/I_{yeff}=0 \text{ N/mm}^2 \)

Bottom of web stress \( s_{2(10)}=-MyEd*10^6*(zeff-tf)/I_{yeff}=0 \text{ N/mm}^2 \)

Net loss area \( Aw=lw*tw=0 \text{ mm}^2 \)

Effective area of section \( A_{eff}=A-Aw=27510 \text{ mm}^2 \)

Effective section modulus \( W_{eyff}=I_{yeff}/(1000*(h-zeff))=8075.8 \text{ cm}^3 \)

Factor about y-y axis \( f_{Ly}=1 \)

Factor about z-z axis \( f_{Lz}=0.7 \)

**SECTION** 1016 x 305 x 222 UKB Grade S 275

**DESIGN** Axial compressive load 2000 kN

**SUMMARY** Compression resistance 6120.1 kN

Unity factor \( 0.32679 \leq 1.0 \)
Location: Example 1 'SHS Design Examples'

Axially loaded SHS column

Calculations are in accordance with BS5950-1:2000.

Load is factored and for compression members length between restraints is the same for both axes.

Design load (+ve compression) F=1200 kN
Length between restraints L=6000 mm

200 x 200 x 6.3 SHS - Hot finished.
Properties (cm): A=48.4 rx=7.89 Zx=301 Sx=350 Ix=3010 J=4650 C=444
Young's Modulus E=205 kN/mm²

Effective length factor (x axis) efx=1
Effective length factor (y axis) efy=1

HOT FINISHED In accordance with EN 10210-1
SQUARE HOLLOW SECTION 200 x 200 x 6.3 SHS Grade S 355
SECTION Section is satisfactory for axial compression load.
SUMMARY
DESIGN Axial compressive load 1200 kN
SUMMARY Compressive Resistance 1206.6 kN
Location: slender section

**Axially loaded SHS column**

Calculations are in accordance with BS5950-1:2000.

Load is factored and for compression members length between restraints is the same for both axes.

Design load (+ve compression) \( F = 1550 \text{ kN} \)
Length between restraints \( L = 6000 \text{ mm} \)

300 x 200 x 6.3 RHS - Hot finished.
Properties (cm): \( A = 61 \), \( r_x = 11.3 \), \( Z_x = 522 \), \( S_x = 624 \), \( I_x = 7830 \)
\( J = 8480 \), \( C = 681 \), \( Z_y = 419 \), \( S_y = 472 \), \( I_y = 4190 \), \( r_y = 8.29 \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Effective length factor (x axis) \( e_{fx} = 1 \)
Effective length factor (y axis) \( e_{fy} = 1 \)

**WARNING:**
As \( F > P_c \) (1550 kN > 1480.4 kN) design axial compressive load exceeds compression resistance and section size must be increased.
Location: SCI example

Axially loaded column

Calculations are in accordance with BS EN 1993-1-1:2005.

Compressive load is factored.

Factored axial compressive load  \( N_{Ed} = 920 \text{ kN} \)
200 x 200 x 6.3 SHS - Hot finished.
Properties (cm): \( A = 48.4 \), \( i_y = 7.89 \), \( W_{ely} = 301 \), \( W_{ply} = 350 \), \( I_y = 3010 \), \( I_t = 4650 \), \( W_t = 444 \)
Factor about \( y-y \) axis  \( f_{Ly} = 1 \)
Factor about \( z-z \) axis  \( f_{Lz} = 1 \)

HOT FINISHED  In accordance with EN 10210-1
SQUARE HOLLOW SECTION 200 x 200 x 6.3 SHS  Grade S 355
DESIGN Axial compressive load 920 kN
SUMMARY Compression resistance 1148.8 kN
Unity factor 0.80087 ≤ 1
Location: RHS cold formed

Axially loaded column

Calculations are in accordance with BS EN 1993-1-1:2005.

Compressive load is factored.

Factored axial compressive load $N_{Ed}=2200$ kN
buckling about major axis $L_y=6000$ mm
buckling about minor axis $L_z=3000$ mm

$300 \times 200 \times 10$ RHS - Cold formed.

Properties (cm): $A=92.6$ iy=$11.1$ Wely=$754$ Wply=$921$ Iy=$11300$ It=$13000$
Welz=$606$ Wplz=$698$ Iz=$6060$ iz=$8.09$ Wt=$1010$
Factor about y-y axis $f_{Ly}=1$
Factor about z-z axis $f_{Lz}=1$

COLD FORMED  In accordance with EN 10219-1
RECTANGULAR HOLLOW SECTION  $300 \times 200 \times 10$ RHS  Grade S 355
DESIGN  Axial compressive load  2200 kN
SUMMARY  Compression resistance  2360.2 kN
Unity factor  $0.93213 \leq 1$
Location: CHS cold formed

Axially loaded column

Calculations are in accordance with BS EN 1993-1-1:2005.

Compressive load is factored.

Factored axial compressive load \( N_{Ed} = 2200 \text{ kN} \)

- Buckling about major axis \( L_y = 6000 \text{ mm} \)
- Buckling about minor axis \( L_z = 3000 \text{ mm} \)

323.9 dia x 10 thick CHS - Cold formed.

Properties (cm): \( A = 98.615 \) \( i_y = 11.104 \) \( W_{ely} = 750.75 \) \( W_{ply} = 985.67 \) \( I_y = 12158 \) \( I_t = 24317 \)

- Factor about y-y axis \( f_{Ly} = 1 \)
- Factor about z-z axis \( f_{Lz} = 1 \)

COLD FORMED
CIRCULAR HOLLOW SECTION 323.9 OD x 10 CHS Grade S 355
DESIGN Axial compressive load 2200 kN
SUMMARY Compression resistance 2521.4 kN
Unity factor 0.87253 ≤ 1
### Location: Column 1-2

#### I column in 'simple' construction

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by SCI.

All beams supported by the column are assumed to be fully loaded. Based on Clause 4.7.7 it is not necessary to consider the effect of pattern loading.

<table>
<thead>
<tr>
<th>Number of storeys</th>
<th>Storey height (bottom storey)</th>
<th>Storey height (elsewhere)</th>
<th>Column centres (average)</th>
<th>Frame centres</th>
</tr>
</thead>
<tbody>
<tr>
<td>storey=2</td>
<td>Ht=3 m</td>
<td>ht=6 m</td>
<td>crs=6 m</td>
<td>frs=5 m</td>
</tr>
</tbody>
</table>

| Beam 1 | dr1=20 kN  |
| Beam 2 | dr2=24 kN  |
| Beam 3 | dr3=20 kN  |
| Beam 4 | dr4=0 kN   |

| Beam 1 | ir1=20 kN  |
| Beam 2 | ir2=80 kN  |
| Beam 3 | ir3=20 kN  |
| Beam 4 | ir4=0 kN   |

| Beam 1 | df1=30 kN  |
| Beam 2 | df2=30 kN  |
| Beam 3 | df3=30 kN  |
| Beam 4 | df4=0 kN   |

| Beam 1 | if1=100 kN |
| Beam 2 | if2=200 kN |
| Beam 3 | if3=100 kN |
| Beam 4 | if4=0 kN   |

Assume self weight for the lowest storey of the column. For variations in column heights this value will be proportioned. Assumed self wt. of column/storey sw=7 kN
203 x 203 x 60 UC.
Young's Modulus \( E = 205 \text{kN/mm}^2 \)

Assumed eccentricity of beam loading from column face \( e = 100 \text{ mm} \)
Effective length factor \( e_f = 0.85 \)

SECTION
203 x 203 x 60 UC Grade S 275
Design strength 275 N/mm²

DESIGN
Compressive strength 222.18 N/mm²

SUMMARY
Buckling strength 275 N/mm²
Buckling check 0.85944 \( \leq 1 \)
Section is satisfactory for bending, axial load, and overall buckling.

FIRE RESISTANCE
Heated perimeter 1223.6 mm²

SUMMARY
Area of section 76.4 cm²
Section factor 160 /m
Load ratio (R) 0.46746
Section does not have an inherent fire resistance of 30 minutes.
Fire protection should be provided.
Location: Column 1-2

I column in simple construction

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by SCI.

All beams supported by the column are assumed to be fully loaded. Based on Clause 4.7.7 it is not necessary to consider the effect of pattern loading.

Factored axial compressive load \( F = 993.8 \, \text{kN} \)
Factored BM about major axis \( x-x \) \( M_x = 26 \, \text{kNm} \)
Factored BM about minor axis \( y-y \) \( M_y = 0 \, \text{kNm} \)
Length between restraints \( L = 3000 \, \text{mm} \)

203 x 203 x 60 UC.
Young's Modulus \( E = 205 \, \text{kN/mm}^2 \)

Effective length factor \( ef = 0.85 \)

SECTION
203 x 203 x 60 UKC Grade S 275

DESIGN

Design strength 275 N/mm²

SUMMARY

Compressive strength 222.18 N/mm²
Buckling strength 275 N/mm²

Buckling check \( 0.72959 \le 1 \)
Section is satisfactory for bending, axial load, and overall buckling.

Axial compressive load \( F_f = 549.4 \, \text{kN} \)
BM about major axis \( x-x \) \( M_{fx} = 26 \, \text{kNm} \)
BM about minor axis \( y-y \) \( M_{fy} = 0 \, \text{kNm} \)

FIRE RESISTANCE

SUMMARY

Heated perimeter 1223.6 mm²
Area of section 76.4 cm²
Section factor 160 /m
Load ratio (R) 0.46779

Section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Location: Semi-compact column

I column in 'simple' construction

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by SCI.

All beams supported by the column are assumed to be fully loaded. Based on Clause 4.7.7 it is not necessary to consider the effect of pattern loading.

Factored axial compressive load \( F = 525 \text{ kN} \)
Factored BM about major axis \( x-x \) \( M_x = 45 \text{ kNm} \)
Factored BM about minor axis \( y-y \) \( M_y = 0 \text{ kNm} \)
Length between restraints \( L = 2500 \text{ mm} \)

406 x 140 x 39 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Effective length factor \( ef = 1 \)

SECTION 406 x 140 x 39 UKB Grade S 275
Design strength 275 \text{ N/mm}^2

DESIGN Compressive strength 166.62 \text{ N/mm}^2
Buckling strength 253.56 \text{ N/mm}^2
Buckling check 0.9024 \leq 1

SUMMARY Section is satisfactory for bending, axial load, and overall buckling.
Location: Slender column failing

I column in 'simple' construction

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by SCI.

All beams supported by the column are assumed to be fully loaded. Based on Clause 4.7.7 it is not necessary to consider the effect of pattern loading.

Factored axial compressive load $F = 1600$ kN
Factored BM about major axis $M_x = 34$ kNm
Factored BM about minor axis $M_y = 2$ kNm
Length between restraints $L = 1500$ mm

457 x 191 x 67 UB.
Young's Modulus $E = 205$ kN/mm$^2$

Effective length factor $ef = 1$

SECTION

457 x 191 x 67 UB Grade S 275
Design strength 275 N/mm$^2$

DESIGN

Compressive strength 254.41 N/mm$^2$

SUMMARY

Buckling strength 275 N/mm$^2$
Buckling check $0.87853 \leq 1$
Section is satisfactory for bending, axial load, and overall buckling.
Location: Access steel example (with FR check)

Column in simple construction

Calculations are in accordance with EN 1993-1-1 and EN 1993-1-2.

- Column is continuous and forms part of a structure of simple construction
- Column is nominally pinned at the base
- Beams are connected to the column by "simple" connections.

All loads and moments are factored.

Factored axial compressive load \( N_{Ed} = 589 \text{ kN} \)
Factored BM about y-y axis \( M_{yEd} = 11.11 \text{ kNm} \)
Factored BM about z-z axis \( M_{zEd} = 0.35 \text{ kNm} \)
Length between restraints \( L = 5000 \text{ mm} \)
203 x 203 x 46 UKC
Dimensions (mm): \( h=203.2 \text{ b}=203.6 \text{ tw}=7.2 \text{ tf}=11 \text{ r}=10.2 \)
Properties (cm): \( I_y=4570 \text{ I}_z=1550 \text{ W}_{ply}=497 \text{ W}_{plz}=231 \text{ I}_t=22.2 \)

SECTION 203 x 203 x 46 UKC Grade S 275
DESIGN Design axial load 589 kN
SUMMARY Buckling resistance 764.02 kN
Design BM y-y axis 11.11 kNm
LTB resistance 136.68 kNm
Design BM z-z axis 0.35 kNm
Moment resistance (z-z) 63.525 kNm
Unity factor 0.86048 \( \leq 1 \)

Fire resistance without applied protection

Unfactored permanent axial force \( N_p = 200 \text{ kN} \)
Unfactored variable axial force \( N_v = 210 \text{ kN} \)
Design axial compressive force \( N_{fiEd} = 1.0*N_p + 1.0* N_v = 410 \text{ kN} \)

Determine critical temperature of steel in fire situation

Buckling length in fire situation \( L_{fi} = 3500 \text{ mm} \)
Critical temperature \( \theta_{cr} \) \( t_{CR} = \text{TABLE 253 for } \Theta = 0.25399, \text{ lamfi} = 0.92314 \)
\( = 566.28 \text{ °C} \)
Fire resistance period 17 minutes

Interaction of axial loads and moments in fire situation

The interaction of axial compression and bending is given by the following expression (EN 1993-1-2 Clause 4.2.3.5):

\[
R = \frac{N_{fiEd}}{N_{fiRd} + k_L T M_{fiEd} y / M_{byRd} + k_z M_{fiEd} z / M_{zRd}} = 0.33058
\]
As $R \leq 1$, section capacity is OK for a FR period of 17 minutes.

<table>
<thead>
<tr>
<th>FIRE RESISTANCE</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.</td>
<td></td>
</tr>
</tbody>
</table>
Location: Access steel example

Column in simple construction

Calculations are in accordance with EN 1993-1-1 and EN 1993-1-2.

- Column is continuous and forms part of a structure of simple construction
- Column is nominally pinned at the base
- Beams are connected to the column by "simple" connections.

All loads and moments are factored.

Beam 1 \( N_1 = 37 \text{ kN} \)
Beam 2 \( N_2 = 147 \text{ kN} \)
Beam 3 \( N_3 = 28 \text{ kN} \)
Beam 4 \( N_4 = 0 \text{ kN} \)
Loading from upper floors \( N_5 = 377 \text{ kN} \)

Dimensions (mm): \( h=203.2 \ b=203.6 \ tw=7.2 \ tf=11 \ r=10.2 \)
Properties (cm): \( I_y=4570 \ Iz=1550 \ W_{ply}=497 \ W_{plz}=231 \ It=22.2 \)
At level above \( L_b=3000 \text{ mm} \)

SECTION \( 203 \times 203 \times 46 \text{ UKC Grade S 275} \)
DESIGN
- Design axial load \( 589 \text{ kN} \)
- Buckling resistance \( 764.02 \text{ kN} \)
- Design BM y-y axis \( 11.113 \text{ kNm} \)
- LTB resistance \( 136.68 \text{ kNm} \)
- Design BM z-z axis \( 0.34965 \text{ kNm} \)
SUMMARY
- Moment resistance (z-z) \( 63.525 \text{ kNm} \)
- Unity factor \( 0.86049 \leq 1 \)
Location: Corner column intermediate length

Columns in simple construction

Calculations are in accordance with EN 1993-1-1 and EN 1993-1-2.

- Column is continuous and forms part of a structure of simple construction
- Column is nominally pinned at the base
- Beams are connected to the column by "simple" connections.

All loads and moments are factored.

Beam 1 N1=200 kN
Beam 2 N2=300 kN
Beam 3 N3=0 kN
Beam 4 N4=0 kN
Loading from upper floors N5=1250 kN
254 x 254 x 107 UKC
Dimensions (mm): h=266.7 b=258.8 tw=12.8 tf=20.5 r=12.7
Properties (cm): Iy=17500 Iz=5930 Wply=1480 Wplz=697 It=172
At level above Lb=4000 mm
Far end BM psiM=-30 kNm

SECTION 254 x 254 x 107 UKC Grade S 275
DESIGN Design axial load 1750 kN
SUMMARY Buckling resistance 2259.5 kN
Design BM y-y axis 35.003 kNm
LTB resistance 392.2 kNm
Design BM z-z axis 10.64 kNm
Moment resistance (z-z) 184.71 kNm
Unity factor 0.95015 ≤ 1
Location: Ground to first floor column

SHS column in 'simple' construction

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple construction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Number of storeys: storey=2
Storey height (bottom storey): Ht=3.5 m
Storey height (elsewhere): ht=3 m
Column centres (average): crs=5 m
Frame centres: frs=5 m

Beam 1: dr1=23 kN
Beam 2: dr2=12.5 kN
Beam 3: dr3=0 kN
Beam 4: dr4=13 kN
Beam 1: ir1=30 kN
Beam 2: ir2=15 kN
Beam 3: ir3=0 kN
Beam 4: ir4=20 kN
Beam 1: df1=41 kN
Beam 2: df2=26 kN
Beam 3: df3=0 kN
Beam 4: df4=26 kN
Beam 1: if1=52 kN
Beam 2: if2=31 kN
Beam 3: if3=0 kN
Beam 4: if4=40 kN

Assume self weight for the lowest storey of the column.
For variations in column heights this value will be proportioned.
Assumed self wt. of column/storey sw=1.65 kN
200 x 100 x 5 RHS - Hot finished.  
Properties (cm): A=28.7 rx=7.21 Zx=149 Sx=185 Ix=1500  
J=1200 C=172 Zy=101 Sy=114 Iy=505 ry=4.19  
Young's Modulus E=205 kN/mm²  
Effective length factor ef=0.85  
Assumed eccentricity of beam loading from column face ecc=100 mm  

<table>
<thead>
<tr>
<th>HOT FINISHED</th>
<th>200 x 100 x 5 RHS Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>Section is satisfactory for axial load, buckling resistance and overall buckling check.</td>
</tr>
<tr>
<td>SUMMARY</td>
<td></td>
</tr>
<tr>
<td>DESIGN</td>
<td>Axial compressive load 474.39 kN</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Compressive resistance 764.58 kN</td>
</tr>
<tr>
<td></td>
<td>Moment about major axis 14.06 kNm</td>
</tr>
<tr>
<td></td>
<td>Buckling resistance 65.675 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment about minor axis 1.08 kNm</td>
</tr>
<tr>
<td></td>
<td>Minor axis resistance 35.855 kNm</td>
</tr>
<tr>
<td></td>
<td>Overall buckling check 0.86466 ≤ 1</td>
</tr>
</tbody>
</table>
SHS column in 'simple' construction

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple construction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Factored axial compressive load  \( F = 474.39 \text{ kN} \)
Factored BM about major axis \( x-x \)  \( M_x = 14.06 \text{ kNm} \)
Factored BM about minor axis \( y-y \)  \( M_y = 1.08 \text{ kNm} \)
Length between restraints  \( L = 3500 \text{ mm} \)

200 x 100 x 5 RHS - Hot finished.
Properties (cm): \( A = 28.7 \text{ cm}^2 \)  \( r_x = 7.21 \text{ cm} \)  \( Z_x = 149 \text{ cm} \)  \( S_x = 185 \text{ cm} \)  \( I_x = 1500 \text{ cm}^4 \)  \( J = 1200 \text{ cm}^4 \)  \( C = 172 \text{ cm} \)  \( Z_y = 101 \text{ cm} \)  \( S_y = 114 \text{ cm} \)  \( I_y = 505 \text{ cm}^4 \)  \( r_y = 4.19 \text{ cm} \)
Young's Modulus  \( E = 205 \text{ kN/mm}^2 \)

Effective length factor  \( e_f = 0.85 \)

In accordance with EN 10210

200 x 100 x 5 RHS  Grade S 355
Section is satisfactory for axial load, buckling resistance and overall buckling check.

Axial compressive load  474.39 kN
Compressive resistance  764.58 kN
Moment about major axis  14.06 kNm
Buckling resistance  65.675 kNm
Moment about minor axis  1.08 kNm
Minor axis resistance  35.855 kNm
Overall buckling check  0.86466 ≤ 1
Location: Bending about x-x axis only

SHS column in 'simple' construction

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple construction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Factored axial compressive load \( F = 1800 \text{ kN} \)
Factored BM about major axis \( x-x \) \( M_x = 34 \text{ kNm} \)
Factored BM about minor axis \( y-y \) \( M_y = 0 \text{ kNm} \)
Length between restraints \( L = 6000 \text{ mm} \)

300 x 200 x 8 RHS - Hot finished.
Properties (cm): \( A = 76.8 \text{ cm}^2 \), \( rx = 11.3 \text{ cm} \), \( Zx = 648 \text{ cm}^3 \), \( Sx = 779 \text{ cm}^2 \), \( lx = 9720 \text{ cm} \), \( J = 10600 \text{ cm}^4 \), \( C = 840 \text{ cm} \), \( Zy = 518 \text{ cm}^3 \), \( Sy = 589 \text{ cm}^2 \), \( Iy = 5180 \text{ cm} \), \( ry = 8.22 \text{ cm} \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Effective length factor \( ef = 0.85 \)

HOT FINISHED
RECTANGULAR HOLLOW SECTION
SECTION
SUMMARY
DESIGN
SUMMARY

In accordance with EN 10210
300 x 200 x 8 RHS Grade S 355
Section is satisfactory for axial load, buckling resistance and overall buckling check.
Axial compressive load \( 1800 \text{ kN} \)
Compressive resistance \( 2249.4 \text{ kN} \)
Moment about major axis \( 34 \text{ kNm} \)
Buckling resistance \( 276.55 \text{ kNm} \)
Overall buckling check \( 0.92316 \leq 1 \)
Location:

**SHS column in 'simple' construction**

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple construction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Factored axial compressive load \( F = 1500 \text{ kN} \)
Factored BM about major axis \( x-x \) \( M_x = 0 \text{ kNm} \)
Factored BM about minor axis \( y-y \) \( M_y = 12 \text{ kNm} \)
Length between restraints \( L = 6000 \text{ mm} \)

300 x 200 x 8 RHS - Cold formed.
Properties (cm): \( A = 75.2 \) \( r_x = 11.2 \) \( Z_x = 626 \) \( S_x = 757 \) \( I_x = 9390 \)
\( J = 10600 \) \( C = 838 \) \( Z_y = 504 \) \( S_y = 574 \) \( I_y = 5040 \) \( r_y = 8.19 \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Effective length factor \( e_f = 0.85 \)

**COLD FORMED**
**RECTANGULAR HOLLOW SECTION**
**SECTION**
**SUMMARY**
**DESIGN**
**SUMMARY**

Axial compressive load \( 1500 \text{ kN} \)
Compressive resistance \( 1803.6 \text{ kN} \)
Moment about minor axis \( 12 \text{ kNm} \)
Minor axis resistance \( 178.92 \text{ kNm} \)
Overall buckling check \( 0.89874 \leq 1 \)
Location: CHS section - biaxial bending

SHS column in 'simple' construction

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple construction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Factored axial compressive load \( F = 1300 \text{ kN} \)
Factored BM about major axis \( M_x = 24 \text{ kNm} \)
Factored BM about minor axis \( M_y = 12 \text{ kNm} \)
Length between restraints \( L = 6000 \text{ mm} \)

244.5 dia x 8 thick CHS - Hot finished.
Properties (cm): \( A = 59.439 \text{ cm}^2 \), \( r = 8.3663 \text{ cm} \), \( Z = 340.32 \text{ cm} \), \( S = 447.63 \text{ cm} \), \( I = 4160.4 \text{ cm}^4 \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Effective length factor \( ef = 0.85 \)

HOT FINISHED
CIRCULAR HOLLOW SECTION 244.5.D. x 8 CHS Grade S 355
SECTION Section is satisfactory for axial load, buckling resistance and overall buckling check.
SUMMARY

DESIGN
SUMMARY Axial compressive load 1300 kN
Compressive resistance 1757.4 kN
Moment about major axis 24 kNm
Buckling resistance 158.91 kNm
Moment about minor axis 12 kNm
Overall buckling check \( 0.96629 \leq 1 \)
Location: Ground to first floor column

SHS column in 'simple' construction

Calculations are in accordance with BS5950 and 'SHS Design Examples to BS5950' published by British Steel General Steels.

The column is part of simple construction and in accordance with 4.7.7 it is not necessary to consider the effect of pattern loading. All beams supported by the column are assumed to be fully loaded.

It is assumed that all elements of the column remain in compression.

Number of storeys
Storey height (bottom storey)
Storey height (elsewhere)
Column centres (average)
Frame centres
Beam 1
Beam 2
Beam 3
Beam 4
Beam 1
Beam 2
Beam 3
Beam 4
Beam 1
Beam 2
Beam 3
Beam 4

Assume self weight for the lowest storey of the column.
For variations in column heights this value will be proportioned.
Assumed self wt. of column/storey sw=1.65 kN
200 x 100 x 5 RHS - Hot finished.
Properties (cm): A=28.7 rx=7.21 Zx=149 Sx=185 Ix=1500
J=1200 C=172 Zy=101 Sy=114 Iy=505 ry=4.19
Young's Modulus  E=205 kN/mm²

Effective length factor  ef=0.85
Assumed eccentricity of beam loading from column face  ecc=100 mm

<table>
<thead>
<tr>
<th>HOT FINISHED</th>
<th>In accordance with EN 10210</th>
</tr>
</thead>
<tbody>
<tr>
<td>RECTANGULAR HOLLOW SECTION</td>
<td>200 x 100 x 5 RHS Grade S 355</td>
</tr>
<tr>
<td>SECTION</td>
<td>Section is satisfactory for axial load, buckling resistance and overall buckling check.</td>
</tr>
<tr>
<td>SUMMARY</td>
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<tr>
<td>DESIGN</td>
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<td>Minor axis resistance 35.855 kNm</td>
</tr>
<tr>
<td></td>
<td>Overall buckling check 0.86466 ≤ 1</td>
</tr>
</tbody>
</table>

| FIRE RESISTANCE               | Heated perimeter 500 mm² |
| SUMMARY                       | Area of section 28.7 cm² |
|                               | Section factor 170 /m |
|                               | Load ratio (R) 0.50778 |
|                               | Section does not have an inherent fire resistance of 30 minutes. |
|                               | Fire protection should be provided. |
Location: SCI example (SHS)

Column in simple construction

Calculations are in accordance with EN 1993-1-1 and EN 1993-1-2.

- Column is continuous and forms part of a structure of simple construction
- Column is nominally pinned at the base
- Beams are connected to the column by "simple" connections

All loads and moments are factored.

Beam 1
Beam 2
Beam 3
Beam 4
Loading from upper floors
150 x 150 x 5 SHS - Hot finished.

Properties (cm): A=28.7 iy=5.9 Wely=134 Wply=156 Iy=1000 It=1550 Wt=197

At level above

HOT FINISHED
SQUARE HOLLOW SECTION
SUMMARY

Design axial load 470 kN
Buckling resistance 824.38 kN
Design BM y-y axis 11.375 kNm
Moment resistance (y-y) 55.38 kNm
Design BM z-z axis 1.3125 kNm
Moment resistance (z-z) 55.38 kNm
Unity factor 0.81108 ≤ 1

Fire resistance without applied protection

Unfactored permanent axial force Np=150 kN
Unfactored variable axial force Nv=150 kN
Design axial compressive force NfiEd=1.0*Np+1.0*Nv=300 kN

Determine critical temperature of steel in fire situation

Buckling length in fire situation Lfi=2450 mm
Critical temperature θcr tCR=TABLE 254 for mo=0.29445, lamfi =0.63945
=598.71 °C

Fire resistance period 18 minutes
Interaction of axial loads and moments in fire situation

The interaction of axial compression and bending is given by the following expression (EN 1993-1-2 Clause 4.2.3.5):

Unity expression \[ R = \frac{N_{f}E_{d}}{N_{f}R_{d}} + kLT \frac{M_{f}E_{dy}}{M_{b}R_{d}} + kz \frac{M_{f}E_{dz}}{M_{z}R_{d}} \]

= 0.51012

As \( R \leq 1 \), section capacity is OK for a FR period of 18 minutes.

FIRE RESISTANCE

SUMMARY

The section does not have an inherent fire resistance of 30 minutes. Fire protection should be provided.
Location: RHS example (SHS)

Column in simple construction

Calculations are in accordance with EN 1993-1-1 and EN 1993-1-2.

- Column is continuous and forms part of a structure of simple construction
- Column is nominally pinned at the base
- Beams are connected to the column by "simple" connections

All loads and moments are factored.

Beam 1 N1=37 kN
Beam 2 N2=147 kN
Beam 3 N3=28 kN
Beam 4 N4=0 kN
Loading from upper floors N5=377 kN

300 x 150 x 10 RHS - Hot finished.

Properties (cm): A=84.9 iz=6.18 Wely=648 Wply=811 Iy=9270
It=7840 Wt=736 Welz=433 Wplz=496 Iz=3250 iy=10.7

At level above Lb=3000 mm

HOT FINISHED
RECTANGULAR HOLLOW SECTION
DESIGN
SUMMARY

In accordance with EN 10210
300 x 150 x 10 RHS Grade S 355
Design axial load 589 kN
Buckling resistance 1884.6 kN
Design BM y-y axis 13.781 kNm
Moment resistance (y-y) 287.9 kNm
Design BM z-z axis 0.59063 kNm
Moment resistance (z-z) 176.08 kNm
Unity factor 0.36543 ≤ 1
Location: Corner column intermediate length (CHS)

Column in simple construction

Calculations are in accordance with EN 1993-1-1 and EN 1993-1-2.

- Column is continuous and forms part of a structure of simple construction
- Column is nominally pinned at the base
- Beams are connected to the column by "simple" connections

All loads and moments are factored.

Factored axial compressive load \( \text{NEd}=1200 \text{ kN} \)
Factored BM about \( y-y \) axis \( \text{MyEd}=56 \text{ kNm} \)
Factored BM about \( z-z \) axis \( \text{MzEd}=56 \text{ kNm} \)
Length between restraints \( L=5000 \text{ mm} \)
323.9 dia x 8 thick CHS - Hot finished.
Properties (cm): \( A=79.394 \), \( iy=11.172 \), \( Wely=611.92 \), \( Wply=798.51 \), \( Iy=9910.1 \), \( It=19820 \)
Far end BM \( \text{psiM}=-56 \text{ kNm} \)

HOT FINISHED
CIRCULAR HOLLOW SECTION 323.9 O.D. x 8 CHS Grade S 355
DESIGN
Design axial load 1200 kN
Buckling resistance 2523.5 kN
Design BM \( y-y \) axis 56 kNm
Moment resistance \( (y-y) \) 283.47 kNm
Design BM \( z-z \) axis 56 kNm
Moment resistance \( (z-z) \) 283.47 kNm
Unity factor 0.96941 ≤ 1
Location: Default example

Simply supported steel beam

Calculations in accordance with BS5950-1:2000.

Beam span: \( L = 5.2 \text{ m} \)

432 x 102 Tapered Channel.

Young's Modulus: \( E = 205 \text{ kN/mm}^2 \)

Dist. from left support to start: \( L_{au(1)} = 0 \text{ m} \)
Dist. from left support to end: \( L_{bu(1)} = 5.2 \text{ m} \)

Dead load (unfactored): \( G_{ku(1)} = 4 \text{ kN/m} \)
Imposed load (unfactored): \( Q_{ku(1)} = 1 \text{ kN/m} \)

Maximum span bending moment: \( 24.336 \text{ kNm} \)

Design shear force: \( F_v = 18.72 \text{ kN} \)

**CHANNEL**

**SECTION**

432 x 102

**DESIGN**

Tapered flange Channel Grade S 275

**SUMMARY**

Shear force: 18.72 kN
Shear capacity: 488.51 kN
Maximum moment: 24.336 kNm
Moment capacity: 25.472 kNm
Deflection due to IL: 7.4068 mm
Limiting deflection: 26 mm

Unfactored end shears

- DL shear at LHE: 10.4 kN
- LL shear at LHE: 2.6 kN
- DL shear at RHE: 10.4 kN
- LL shear at RHE: 2.6 kN
Location: Example using parallel flange channel

Simply supported steel beam

Calculations in accordance with BS5950-1:2000.

Channel section bending

<table>
<thead>
<tr>
<th>compression</th>
<th>about minor axis</th>
</tr>
</thead>
<tbody>
<tr>
<td>tension</td>
<td></td>
</tr>
</tbody>
</table>

Beam span L=5.2 m

430 x 100 Parallel Flange Channel.

Young's Modulus E=205 kN/mm²

Distance from left support to start Lau(1)=0 m

Distance from left support to end Lbu(1)=5 m

Dead load (unfactored) Gku(1)=4 kN/m

Imposed load (unfactored) Qku(1)=1 kN/m

Maximum span bending moment 24.264 kNm

Design shear force Fv=18.692 kN

<table>
<thead>
<tr>
<th>CHANNEL</th>
<th>430 x 100</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>Parallel flange Channel Grade S 275</td>
</tr>
<tr>
<td>DESIGN</td>
<td>Shear force 18.692 kN</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Shear capacity 543.78 kN</td>
</tr>
<tr>
<td></td>
<td>Maximum moment 24.264 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment capacity 31.132 kNm</td>
</tr>
<tr>
<td></td>
<td>Deflection due to IL 6.4094 mm</td>
</tr>
<tr>
<td></td>
<td>Limiting deflection 26 mm</td>
</tr>
<tr>
<td>Unfactored [</td>
<td>DL shear at LHE 10.385 kN</td>
</tr>
<tr>
<td>end shears [</td>
<td>LL shear at LHE 2.5962 kN</td>
</tr>
<tr>
<td></td>
<td>DL shear at RHE 9.6154 kN</td>
</tr>
<tr>
<td></td>
<td>LL shear at RHE 2.4038 kN</td>
</tr>
</tbody>
</table>
Location: Ex1 - Default example

Simply supported steel beam

\[ w = w_d + w_i \]

Simply supported beam restrained at ends

where suffices:
- \( d \) denotes permanent load
- \( i \) denotes variable load

Beam span \( L = 5.2 \text{ m} \)

Permanent UDL (including S.W.) \( w_d = 4 \text{ kN/m} \)
Variable UDL \( w_i = 1 \text{ kN/m} \)

Channel orientation

Channel section bending about minor axis \( zz \)

432 x 102 Tapered Channel.

<table>
<thead>
<tr>
<th>CHANNEL</th>
<th>432 x 102</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>Tapered flange Channel Grade S 275</td>
</tr>
<tr>
<td>DESIGN</td>
<td>Design shear force 17.94 kN</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Shear resistance 470.07 kN</td>
</tr>
<tr>
<td></td>
<td>Design moment 23.322 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment resistance 40.545 kNm</td>
</tr>
<tr>
<td></td>
<td>Var.load deflection 7.2304 mm</td>
</tr>
<tr>
<td></td>
<td>Limiting deflection 26 mm</td>
</tr>
<tr>
<td></td>
<td>Design shear at LHE 17.94 kN</td>
</tr>
<tr>
<td></td>
<td>Design shear at RHE 17.94 kN</td>
</tr>
</tbody>
</table>
Location: Ex2 - Example using parallel flange channel

Simply supported steel beam

\[ w = w_d + w_i \]

Simply supported beam restrained at ends

where suffices:
- \( d \) denotes permanent load
- \( i \) denotes variable load

Beam span \( L = 5.2 \text{ m} \)

Permanent UDL (including S.W.) \( w_d = 4 \text{ kN/m} \)

Variable UDL \( w_i = 1 \text{ kN/m} \)

Channel orientation

Compression

Channel section bending about minor axis \( zz \)

tension

430 x 100 UKPFC

<table>
<thead>
<tr>
<th>CHANNEL</th>
<th>430 x 100 UKPFC</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION</td>
<td>Parallel flange Channel Grade S 275</td>
</tr>
<tr>
<td>DESIGN</td>
<td>Design shear force 17.94 kN</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Shear resistance 523.25 kN</td>
</tr>
<tr>
<td></td>
<td>Design moment 23.322 kNm</td>
</tr>
<tr>
<td></td>
<td>Moment resistance 46.64 kNm</td>
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<tr>
<td></td>
<td>Var.load deflection 6.2791 mm</td>
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<tr>
<td></td>
<td>Limiting deflection 26 mm</td>
</tr>
<tr>
<td></td>
<td>Design shear at LHE 17.94 kN</td>
</tr>
<tr>
<td></td>
<td>Design shear at RHE 17.94 kN</td>
</tr>
</tbody>
</table>
**Location:** Ex1 - Staircase with flat plate stringers

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**Staircase stringer**

- **Flat plate stringer**
  - Calculations are in accordance with EC3 Part 1-1:2005
  - $crs =$ stringer centres
  - $Y-Y =$ major axis
  - $Z-Z =$ minor axis
  - $L =$ Stringer span

### Stringer span
- $L = 4.5 \text{ m}$

### Distance between stringers
- $crs = 2 \text{ m}$

### Depth of stringer
- $h = 220 \text{ mm}$

### Thickness of plate stringer
- $t = 15 \text{ mm}$

### Permanent action partial factor
- $\gamma_G = 1.35$

### Variable action partial factor
- $\gamma_Q = 1.5$

### Staircase loading

- **Characteristic permanent load** $w_{kp} = 1.7 \text{ kN/m}^2$
- **Characteristic variable load** $w_{kv} = 4 \text{ kN/m}^2$
- **Char permanent load on stringer** $W_{kp} = W_{ks} + w_{kp} * crs * L / 2 = 8.8157 \text{ kN}$
- **Char variable load on stringer** $W_{kv} = w_{kv} * crs * L / 2 = 18 \text{ kN}$

**Design load on stringer:**

$$Wy = \gamma_G * W_{ks} + (\gamma_G * w_{kp} + \gamma_Q * w_{kv}) * crs * L / 2 = 38.901 \text{ kN}$$

### Design moment and shear

- **Design moment (at centre)** $My_{Ed} = Wy * L / 8 = 21.882 \text{ kNm}$
- **Design shear (at ends)** $Vy_{Ed} = Wy / 2 = 19.451 \text{ kN}$

### Design shear resistance

- **Shear resistance** $V_{plRdy} = Av * (fy / 3^{0.5}) / (\gamma_M0 * 1000)$
  - $= 471.55 \text{ kN}$

Since $Vy_{Ed} \leq V_{plRdy}$ (19.451 kN $\leq$ 471.55 kN) design shear force in web is within the shear resistance.
Design moment resistance

(BS EN 1993-1-1, Clause 6.2.5(2))
Design moment resistance \( McRd = \frac{Wel \cdot f_y}{(\gamma M_0 \cdot 10^3)} = 33.275 \text{ kNm} \)
Since \( MyEd \leq McRd \) (21.882 kNm \( \leq 33.275 \text{ kNm} \)) section is satisfactory.

Lateral torsional buckling check

(BS EN 1993-1-1, Clause 6.3.2.2(1))
The buckling resistance of the stringer over a buckling length equal to the tread spacing will be evaluated here, assuming Class 3 stringer. Refer to SCI AD 391 for further clarification as BS EN 1993-1-1 does not explicitly cover LTB of rectangular plates.
Tread spacing \( L_{tr} = 300 \text{ mm} \)
Design buckling resistance moment \( MbRd = \frac{\chi L_{tr} \cdot Wel \cdot f_y}{(\gamma M_1 \cdot 10^3)} \)
\( = 27.249 \text{ kNm} \)
Since \( MyEd \leq MbRd \) (21.882 kNm \( \leq 27.249 \text{ kNm} \)) section is satisfactory.

Check for deflection under serviceability loads

Variable load deflection \( \Delta E = 5 \cdot W_{kv} \cdot L^3 \cdot 10^8 / (384 \cdot E \cdot I_y) = 7.641 \text{ mm} \)
Total stringer deflection \( d_{Total} = \Delta E \cdot W_{kp} / W_{kv} + \Delta E = 11.383 \text{ mm} \)
Limiting deflection (var. load) \( \Delta E_{lim} = L \cdot 1000 / 250 = 12.5 \text{ mm} \)
Limiting deflection (total load) \( \Delta E_{lim} = L \cdot 1000 / 360 = 18 \text{ mm} \)
Since \( \Delta E \leq \Delta E_{lim} \) (7.641 mm \( \leq 12.5 \text{ mm} \)) deflection within limiting value for imposed loading only.

FLAT PLATE STRINGER          220 x 15 Flat plate, Grade S 275
DESIGN SUMMARY               Design shear force        19.451 kN
                                Shear resistance        471.55 kN
                                Design moment             21.882 kNm
                                Buckling resistance BM     27.249 kNm
                                Variable load deflection  7.641 mm
                                Total load deflection     11.383 mm
                                Limiting deflection       12.5 mm
**Location: Example 10, 'Steelwork Design Guide'**

**Axially loaded struts**

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by The Steel Construction Institute.

Compressive load is factored and length between restraints is the same for both axes.

Factored axial compressive load \( F = 910 \) kN

Overall length \( L = 8 \) m

203 x 76 Tapered Channel.

Young's Modulus \( E = 205 \) kN/mm²

Overall width combined elements \( s = 300 \) mm

Effective length factor (y axis) \( e_fy = 1 \)

Effective length factor (x axis) \( e_{fx} = 0.85 \)

Width of lacings \( bl = 40 \) mm

Thickness of lacing \( tl = 8 \) mm

Inclined angle of lacing \( \text{ang} = 45° \)

Length of batten \( l_{eng} = 260 \) mm

Width of batten \( bb = 260 \) mm

Thickness of batten \( tb = 6 \) mm

**TAPERED FLANGE CHANNEL**

2/203 x 76 Grade S 275

**DESIGN**

Axial compressive load \( 910 \) kN

**SUMMARY**

Compression resistance \( 924.21 \) kN

**LACINGS**

40 mm x 8 mm plate

at 45 degrees to horizontal

**TIE BATTENS**

260 mm x 260 mm x 6 mm plate
**Location:** Example 11, 'Steelwork Design Guide'

### Axially loaded struts

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by The Steel Construction Institute.

Compressive load is factored and length between restraints is the same for both axes.

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Factored axial compressive load  \( F = 906 \text{ kN} \)
Overall length  \( L = 8 \text{ m} \)

203 x 76 Tapered Channel.
Young's Modulus  \( E = 205 \text{ kN/mm}^2 \)

Overall width combined elements  \( s = 300 \text{ mm} \)
Effective length factor (y axis)  \( e_{fy} = 1 \)
Effective length factor (x axis)  \( e_{fx} = 0.85 \)
Assumed spacing of battens  \( l = 1000 \text{ mm} \)
Length of batten  \( a' = 260 \text{ mm} \)
Width of batten  \( b_{we} = 260 \text{ mm} \)
Thickness of batten  \( t_{b} = 6 \text{ mm} \)
Width of batten  \( b_{wi} = 160 \text{ mm} \)

**TAPERED FLANGE CHANNEL**

**DESIGN**
Axial compressive load  \( 906 \text{ kN} \)

**SUMMARY**
Compression resistance  \( 924.21 \text{ kN} \)

**BATTENS**
End 260 mm x 260 mm x 6 mm plate
Intermediate 260 mm x 160 mm x 6 mm plate
Centres 1000 mm
Location: Example using UB sections

F kN

Axially loaded struts

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by The Steel Construction Institute.

Compressive load is factored and length between restraints is the same for both axes.

Factored axial compressive load \( F = 850 \text{ kN} \)
Overall length \( L = 6.5 \text{ m} \)

305 x 102 x 28 UB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Overall width combined elements \( s = 300 \text{ mm} \)

Effective length factor (y axis) \( e_{fy} = 1.5 \)
Effective length factor (x axis) \( e_{fx} = 1.5 \)
Width of lacings \( b_l = 50 \text{ mm} \)
Thickness of lacing \( t_l = 6 \text{ mm} \)
Inclined angle of lacing \( \text{ang} = 45^\circ \)
Length of batten \( l_{eng} = 200 \text{ mm} \)
Width of batten \( b_b = 210 \text{ mm} \)
Thickness of batten \( t_b = 6 \text{ mm} \)

UNIVERSAL BEAM
2/305 x 102 x 28 UB's Grade S 275

DESIGN
Axial compressive load \( 850 \text{ kN} \)

SUMMARY
Compression resistance \( 920.56 \text{ kN} \)

LACINGS
50 mm x 6 mm plate
at 45 degrees to horizontal

TIE BATTENS
210 mm x 200 mm x 6 mm plate
Location: Example using parallel flange channels

Axially loaded struts

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by The Steel Construction Institute.

Compressive load is factored and length between restraints is the same for both axes.

Factored axial compressive load  \( F = 3400 \) kN
Overall length  \( L = 8 \) m

430 x 100 Parallel Flange Channel.
Young's Modulus  \( E = 205 \) kN/mm²

Overall width combined elements  \( s = 470 \) mm
Effective length factor (y axis)  \( e_y = 1 \)
Effective length factor (x axis)  \( e_x = 1 \)
Width of lacings  \( b_l = 50 \) mm
Thickness of lacing  \( t_l = 15 \) mm
Inclined angle of lacing  \( \text{ang} = 45^\circ \)
Length of batten  \( l_{\text{eng}} = 350 \) mm
Width of batten  \( b_b = 325 \) mm
Thickness of batten  \( t_b = 8 \) mm

PARALLEL FLANGE CHANNEL  2/430 x 100 Grade S 275
DESIGN  Axial compressive load  3400 kN
SUMMARY  Compression resistance  3849.4 kN

LACINGS  50 mm x 15 mm plate
at 45 degrees to horizontal

TIE BATTENS  325 mm x 350 mm x 8 mm plate
Axially loaded struts

Calculations are in accordance with EC3 Part 1-1:2005.

Compression load is factored and length between restraints is the same for both yy and zz axes.

Design axial compressive load \( N_{Ed} = 910 \text{ kN} \)
Overall length \( L = 8 \text{ m} \)

200 x 75 UKPFC
Dimensions (mm): \( h = 200 \) \( b = 75 \) \( tw = 6 \) \( tf = 12.5 \) \( r = 12 \)
Properties (cm): \( I_y = 1960 \) \( I_z = 170 \) \( W_{ply} = 227 \) \( W_{plz} = 60.6 \) \( I_t = 11.1 \)
Overall width combined elements \( s = 300 \text{ mm} \)
Effective length factor (yy axis) \( e_{fy} = 0.85 \)
Effective length factor (zz axis) \( e_{fz} = 1 \)
Inclined angle of lacing \( \theta = 45^\circ \)
Width of lacings \( bl = 40 \text{ mm} \)
Thickness of lacing \( tl = 8 \text{ mm} \)
Length of batten \( leng = 260 \text{ mm} \)
Width of batten \( bb = 260 \text{ mm} \)
Thickness of batten \( tb = 8 \text{ mm} \)

TAPERED FLANGE CHANNEL 2/200 x 75 Grade S 275
DESIGN SUMMARY
Applied compressive load \( 910 \text{ kN} \)
Design compressive load \( 927.05 \text{ kN} \)
Compression resistance \( 1155.5 \text{ kN} \)

LACINGS
40 mm x 8 mm plate at 45 degrees to horizontal

TIE BATTENS AT ENDS
260 mm x 260 mm x 8 mm plate
Location: Ex2 - 'Steelwork Design Guide' Ex11, battens on two sides

Axially loaded struts

Calculations are in accordance with EC3 Part 1-1:2005.

Compression load is factored and length between restraints is the same for both yy and zz axes.

Design axial compressive load \( N_{Ed} = 906 \text{kN} \)
Overall length \( L = 8 \text{ m} \)

200 x 75 UKPFC
Dimensions (mm): \( h = 200 \) \( b = 75 \) \( tw = 6 \) \( tf = 12.5 \) \( r = 12 \)
Properties (cm): \( I_y = 1960 \) \( I_z = 170 \) \( W_{ply} = 227 \) \( W_{plz} = 60.6 \) \( I_t = 11.1 \)
Overall width combined elements \( s = 300 \text{ mm} \)
Effective length factor (yy axis) \( \varepsilon_{fy} = 0.85 \)
Effective length factor (zz axis) \( \varepsilon_{fz} = 1 \)
Length of end batten \( a' = 260 \text{ mm} \)
Width of end batten \( b_{we} = 260 \text{ mm} \)
Thickness of end batten \( t_b = 8 \text{ mm} \)
Intermediate batten width \( b_{wi} = 160 \text{ mm} \)
Spacing of battens \( a = 1000 \text{ mm} \)

TAPERED FLANGE CHANNEL 2/200 x 75 Grade S 275

DESIGN SUMMARY
Applied compressive load 906 kN
Design compressive load 925.12 kN
Compression resistance 1168.8 kN

BATTENS
End 260 mm x 260 mm x 8 mm plate
Intermediate 260 mm x 160 mm x 8 mm plate
Centres 1000 mm
**Location: Ex3 - UB sections with lacings on two sides**

![Axially loaded struts](image.png)

**Calculations are in accordance with EC3 Part 1-1:2005.**

Compression load is factored and length between restraints is the same for both yy and zz axes.

- **Design axial compressive load**: $N_{Ed} = 850 \text{ kN}$
- **Overall length**: $L = 6.5 \text{ m}$

**Dimensions (mm):**
- $h = 304.4$, $b = 123.4$, $tw = 7.1$, $tf = 10.7$, $r = 8.9$

**Properties (cm):**
- $I_y = 7170$, $I_z = 336$, $W_{pl y} = 539$, $W_{pl z} = 85.4$, $I_t = 14.8$

**Overall width combined elements**: $s = 300 \text{ mm}$

**Effective length factor (yy axis)**: $e_{ff y} = 1.5$

**Effective length factor (zz axis)**: $e_{ff z} = 1.5$

**Inclined angle of lacing**: $\theta = 45^\circ$

**Width of lacings**: $b_l = 50 \text{ mm}$

**Thickness of lacing**: $t_l = 6 \text{ mm}$

**Length of batten**: $l_{en g} = 200 \text{ mm}$

**Width of batten**: $b_b = 250 \text{ mm}$

**Thickness of batten**: $t_b = 8 \text{ mm}$

**UNIVERSAL BEAM**

**2/305 x 127 x 37 UB's Grade S 275**

**DESIGN SUMMARY**

- **Applied compressive load**: $850 \text{ kN}$
- **Design compressive load**: $877.73 \text{ kN}$
- **Compression resistance**: $1040.7 \text{ kN}$

**LACINGS**

- $50 \text{ mm x 6 mm plate}$ at 45 degrees to horizontal

**TIE BATTENS AT ENDS**

- $250 \text{ mm x 200 mm x 8 mm plate}$
Location: Ex4 - Parallel flange channels (PFC), lacings on two sides

Axially loaded struts

Calculations are in accordance with EC3 Part 1-1:2005.

Compression load is factored and length between restraints is the same for both yy and zz axes.

Design axial compressive load  \( N_{Ed} = 3400 \text{ kN} \)

Overall length  \( L = 8 \text{ m} \)

Dimensions (mm):  \( h=430 \quad b=100 \quad tw=11 \quad tf=19 \quad r=15 \)

Properties (cm):  \( I_y=21900 \quad I_z=721 \quad W_{p,ly}=1220 \quad W_{plz}=165 \quad I_t=63 \)

Overall width combined elements  \( s=470 \text{ mm} \)

Effective length factor (yy axis)  \( \text{eff}_y=1 \)

Effective length factor (zz axis)  \( \text{eff}_z=1 \)

Inclined angle of lacing  \( \text{ang}=45^\circ \)

Width of lacings  \( \text{bl}=50 \text{ mm} \)

Thickness of lacing  \( \text{tl}=15 \text{ mm} \)

Length of batten  \( \text{leng}=350 \text{ mm} \)

Width of batten  \( \text{bb}=325 \text{ mm} \)

Thickness of batten  \( \text{tb}=10 \text{ mm} \)

**PARALLEL FLANGE CHANNEL**  2/430 x 100 Grade S 275

\( \text{tw}=11 \quad \text{tf}=19 \quad \text{r}=15 \text{ mm} \)

**DESIGN SUMMARY**

- Applied compressive load 3400 kN
- Design compressive load 3446.7 kN
- Compression resistance 3510.8 kN

**LACINGS**

- 50 mm x 15 mm plate
  - at 45 degrees to horizontal

**TIE BATTENS AT ENDS**

- 325 mm x 350 mm x 10 mm plate
Location: Example 3 'Design of Curved Steel'

Structural hollow section design

Section is curved in elevation. Calculations are in accordance with BS5950:Part 1:2000 and SCI publication entitled "Design of Curved Steel". All loads and moments are factored.

Factored bending moment axis \( zz \) \( M_z' = 7 \text{ kNm} \)
Factored SF in y direction \( F_v = 2 \text{ kN} \)
Axial load (+ve is compression) \( F = 621 \text{ kN} \)
Length of member \( L = 3.264 \text{ m} \)
180 x 180 x 6.3 SHS - Hot finished.
Properties (cm): \( A = 43.3 \) \( r_x = 7.07 \) \( Z_x = 241 \) \( S_x = 281 \) \( I_x = 2170 \) \( J = 3360 \) \( C = 355 \)
Radius of curvature of beam \( R = 55 \text{ m} \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Length between restraints \( z \) axis \( L_z = 3264 \text{ mm} \)
Length between restraints \( y \) axis \( L_y = 3264 \text{ mm} \)

HOT FINISHED
SQUARE HOLLOW SECTION
DESIGN SUMMARY

In accordance with EN 10210
180 x 180 x 6.3 SHS Grade S 355
Section is satisfactory for axial load, and overall buckling check.
Axial load \( 621 \text{ kN} \)
Compression resistance \( 1372.9 \text{ kN} \)
Factored shear force \( 2 \text{ kN} \)
Shear capacity \( 461.15 \text{ kN} \)
Maximum moment \( z \) axis \( 22.036 \text{ kNm} \)
Moment capacity \( 97.361 \text{ kNm} \)
Local capacity check \( 0.64026 \leq 1 \)
Overall buckling checks \( 0.71622 \leq 1 \)
\( 0.67865 \leq 1 \)
Location: Based on example 1

Structural hollow section design

Section is curved in elevation. Calculations are in accordance with BS5950:Part 1:2000 and SCI publication entitled "Design of Curved Steel". All loads and moments are factored.

Factored bending moment axis \( M_z' \) \( = \) 546 kNm
Factored SF in y direction \( F_y \) \( = \) 155 kN
Axial load (+ve is compression) \( F \) \( = \) 0 kN
Length of member \( L \) \( = \) 5 m
450 x 250 x 10 RHS - Hot finished.
Properties (cm): \( A = 135 \) \( \), \( r_x = 16.5 \) \( \), \( Z_x = 1640 \) \( \), \( S_x = 2000 \) \( \), \( I_x = 36900 \) \( \)
\( J = 33300 \) \( \), \( C = 1990 \) \( \), \( Z_y = 1190 \) \( \), \( S_y = 1330 \) \( \), \( I_y = 14800 \) \( \), \( r_y = 10.5 \) \( \)
Radius of curvature of beam \( R = 17.05 \) m
Young's Modulus \( E = 205 \) kN/mm²

Dist. betwn torsional restraints \( L_T = 5 \) m

HOT FINISHED
RECTANGULAR HOLLOW SECTION
450 x 250 x 10 RHS Grade S 355
DESIGN SUMMARY
In accordance with EN 10210
Section is satisfactory for axial load, and overall buckling check.
Factored shear force \( 155 \) kN
Shear capacity \( 1848.5 \) kN
Maximum moment z axis \( 546 \) kNm
Moment capacity \( 603.3 \) kNm

HOT FINISHED
RECTANGULAR HOLLOW SECTION
450 x 250 x 10 RHS Grade S 355
DESIGN SUMMARY
In accordance with EN 10210
Section is satisfactory for axial load, and overall buckling check.
Factored shear force \( 155 \) kN
Shear capacity \( 1848.5 \) kN
Maximum moment z axis \( 546 \) kNm
Moment capacity \( 603.3 \) kNm
**Location: Ex1 - Based on 'Design of Curved Steel' Example 3**

---

### Structural hollow section design

Section is curved in elevation and calculations are in accordance with BS EN 1993-1-1:2005. All loads and moments are factored.

In the calculations the length of member (L) refers to the segment length of beam to be designed.

---

**Design moment about y-y axis**  \( M_{yEd}' = 7 \text{ kNm} \)

**Design SF in z direction**  \( V_{zEd} = 2 \text{ kN} \)

**Design axial load (comp.positive)**  \( N_{Ed} = 621 \text{ kN} \)

**Length of member**  \( L = 3.264 \text{ m} \)

180 x 180 x 6.3 SHS - Hot finished.

**Properties (cm):**  
\[ A = 43.3 \text{ cm}^2, \quad i_y = 7.07 \text{ cm}, \quad W_{ely} = 241 \text{ cm}^3, \quad W_{ply} = 281 \text{ cm}^3, \quad I_y = 2170 \text{ cm}^4, \quad I_t = 3360 \text{ cm}^4, \quad W_t = 355 \text{ cm}^3 \]

**Radius of curvature of beam**  \( R = 55 \text{ m} \)

**Effective length for LTB**  \( L_T = 3.264 \text{ m} \)

**Effective length parameter**  \( k = 1 \)

**Correction factor**  \( k_c = 1 \)

**Length between restraints yy axis**  \( L_y = 3.264 \text{ m} \)

**Length between restraints zz axis**  \( L_z = 3.264 \text{ m} \)

**Maximum end moment**  \( M_{h(1)} = 22 \text{ kNm} \)

**Maximum span moment**  \( M_{s(1)} = 22 \text{ kNm} \)

**End moment**  \( pM_{h(1)} = 22 \text{ kNm} \)

**HOT FINISHED**

**SQUARE HOLLOW SECTION**

180 x 180 x 6.3 SHS  Grade S 355

**DESIGN SUMMARY**

Section is satisfactory for axial load, and overall buckling check.

**Design axial load**  \( 621 \text{ kN} \)

**Buckling resistance**  \( 1333.3 \text{ kN} \)

**Factored shear force**  \( 2 \text{ kN} \)

**Design shear resistance**  \( 443.74 \text{ kN} \)

**Design moment y-y axis**  \( 22.036 \text{ kNm} \)

**Buckling resist.moment**  \( 99.755 \text{ kNm} \)

**Interaction factors**

\[ 0.64026 \leq 1, \quad 0.73463 \leq 1, \quad 0.62709 \leq 1 \]
Location: Ex2 - Based on 'Design of Curved Steel' Example 1

![Diagram of structural hollow section]

**Structural hollow section design**

Section is curved in elevation and calculations are in accordance with BS EN 1993-1-1:2005. All loads and moments are factored.

In the calculations the length of member (L) refers to the segment length of beam to be designed.

- Design moment about y-y axis \( M_{Ed}' = 546 \text{ kNm} \)
- Design SF in z direction \( V_{zEd} = 155 \text{ kN} \)
- Design axial load (comp. positive) \( N_{Ed} = 0 \text{ kN} \)
- Length of member \( L = 5 \text{ m} \)
- 450 x 250 x 10 RHS - Hot finished.
- Properties (cm): \( A = 135 \text{ cm}^2 \), \( iz = 10.5 \text{ cm} \), \( W_{ely} = 1640 \text{ cm}^3 \), \( W_{ply} = 2000 \text{ cm}^3 \), \( I_y = 36900 \text{ cm}^4 \)
  \( I_t = 33300 \text{ cm}^4 \), \( W_{et} = 1990 \text{ cm}^3 \), \( W_{plz} = 1330 \text{ cm}^3 \), \( I_z = 14800 \text{ cm}^4 \), \( I_y = 16.5 \text{ cm} \)
- Radius of curvature of beam \( R = 17.05 \text{ m} \)
- Effective length for LTB \( LT = 5 \text{ m} \)
- Effective length parameter \( K = 1 \)
- Maximum moment on section \( M = 546 \text{ kNm} \)
- Far end moment \( psiM = 546 \text{ kNm} \)

**HOT FINISHED**

**RECTANGULAR HOLLOW SECTION**

**DESIGN SUMMARY**

Section is satisfactory for axial load, and overall buckling check.
- Factored shear force \( 155 \text{ kN} \)
- Design shear resistance \( 1778.8 \text{ kN} \)
- Design moment y-y axis \( 546 \text{ kNm} \)
- Buckling resist.moment \( 710 \text{ kNm} \)
- Interaction factors \( 0.90502 \leq 1 \)
  \( 0.90502 \leq 1 \)
Location: Example 3 'Design of Curved Steel'

Circular hollow section design

Section curved in elevation & calculations are in accordance with BS5950:Part 1:2000 and SCI publication entitled "Design of Curved Steel". All loads & moments are factored.

Factored moment axis z
Mz' = 7 kNm

Factored SF in y direction
Fv = 2 kN

Axial load (comp. is positive)
F = 621 kN

Length of member
L = 3.264 m

Diameter of section
D = 219.1 mm

Thickness
t = 6.3 mm

Radius of curvature of beam
Rc = 55 m

Young's Modulus
E = 205 kN/mm²

Length between restraints z axis
Lz = 3264 mm

Length between restraints y axis
Ly = 3264 mm

HOT FINISHED
In accordance with EN 10210
CIRCULAR HOLLOW SECTION
219.1 mm O.D. x 6.3 mm CHS Grade S 355
DESIGN SUMMARY
Section is satisfactory for axial load, and overall buckling check.
Axial load
621 kN
Compression resistance
1382.7 kN
Factored shear force
2 kN
Shear capacity
538.26 kN
Maximum moment z axis
22.036 kNm
Moment capacity
101.31 kNm
Local capacity check
0.63286 ≤ 1
Overall buckling checks
0.73411 ≤ 1
0.66665 ≤ 1
Location: Based on example 1

Circular hollow section design

Section curved in elevation & calculations are in accordance with BS5950:Part 1:2000 and SCI publication entitled "Design of Curved Steel". All loads & moments are factored.

Factored moment axis zz \( M_z' = 546 \text{ kNm} \)
Factored SF in y direction \( F_v = 155 \text{ kN} \)
Axial load (comp. is positive) \( F = 0 \text{ kN} \)
Length of member \( L = 5 \text{ m} \)
Diameter of section \( D = 406.4 \text{ mm} \)
Thickness \( t = 16 \text{ mm} \)
Radius of curvature of beam \( R_c = 17.05 \text{ m} \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

HOT FINISHED
CIRCULAR HOLLOW SECTION
406.4 mm O.D. x 16 mm CHS Grade S 355
DESIGN SUMMARY
Section is satisfactory for axial load, and overall buckling check.
Factored shear force 155 kN
Shear capacity 2507.9 kN
Maximum moment z axis 546 kNm
Moment capacity 866.19 kNm
Location: Ex1 - SCI publication 'Design of Curved Steel' Example 3

Circular hollow section (CHS)

Section is curved in elevation and design calculations are in accordance with BS EN 1993-1-1 2005. All loads and moments are factored.

In the calculations the length of member (L) refers to the segment length of beam to be designed.

Design moment about y-y axis \( M_{yEd} = 7 \text{ kNm} \)
Design SF in the z direction \( V_{zEd} = 2 \text{ kN} \)
Design axial load (comp.positive) \( N_{Ed} = 621 \text{ kN} \)
Length of member \( L = 3.264 \text{ m} \)
219.1 dia x 6.3 thick CHS - Hot finished.
Properties (cm): \( A = 42.117 \text{ cm} \)
\( i_y = 7.5269 \text{ cm} \)
\( W_{ely} = 217.81 \text{ cm}^3 \)
\( W_{ply} = 285.37 \text{ cm}^3 \)
\( I_y = 2386.1 \text{ cm}^4 \)
\( I_t = 4772.3 \text{ cm}^4 \)
Radius of curvature of beam \( R = 55 \text{ m} \)
Effective length for LTB \( L_T = 3.264 \text{ m} \)
Effective length parameter \( K = 1 \)
Correction factor \( k_c = 1 \)

Length between restraints yy axis \( L_y = 3.264 \text{ m} \)
Length between restraints zz axis \( L_z = 3.264 \text{ m} \)
Maximum end moment \( M_h(1) = 22 \text{ kNm} \)
Maximum span moment \( M_s(1) = 22 \text{ kNm} \)
End moment \( pM_h(1) = 22 \text{ kNm} \)

HOT FINISHED
CIRCULAR HOLLOW SECTION
DESIGN SUMMARY

In accordance with EN 10210
219.1 mm O.D. x 6.3 mm CHS Grade S 355
Section is satisfactory for axial load, and overall buckling check.
Design axial load \( 621 \text{ kN} \)
Buckling resistance \( 1348.5 \text{ kN} \)
Factored shear force \( 2 \text{ kN} \)
Design shear resistance \( 549.55 \text{ kN} \)
Design moment y-y axis \( 22.036 \text{ kNm} \)
Buckling resist.moment \( 101.31 \text{ kNm} \)
Interaction factors \( 0.63286 \leq 1 \)
\( 0.71485 \leq 1 \)
\( 0.61312 \leq 1 \)
Location: Ex2 - CHS without axial load

**Circular hollow section (CHS)**

Section is curved in elevation and design calculations are in accordance with BS EN 1993-1-1 2005. All loads and moments are factored.

In the calculations the length of member (L) refers to the segment length of beam to be designed.

Design moment about y-y axis \( M_{yEd}' = 546 \text{ kNm} \)
Design SF in the z direction \( V_{zEd} = 155 \text{ kN} \)
Design axial load (comp. positive) \( N_{Ed} = 0 \text{ kN} \)
Length of member \( L = 5 \text{ m} \)

406.4 dia x 16 thick CHS - Hot finished.

Properties (cm): \( A = 196.24 \) \( iy = 13.814 \) \( W_{ely} = 1843 \) \( W_{ply} = 2440 \) \( I_y = 37449 \) \( I_t = 74898 \)

Radius of curvature of beam \( R = 17.05 \text{ m} \)
Effective length for LTB \( L_T = 5 \text{ m} \)
Effective length parameter \( K = 1 \)
Correction factor \( k_c = 1 \)

**HOT FINISHED**

In accordance with EN 10210

406.4 mm O.D. x 16 mm CHS Grade S 355

**CIRCULAR HOLLOW SECTION**

**DESIGN SUMMARY**

Section is satisfactory for axial load, and overall buckling check.
Factored shear force \( 155 \text{ kN} \)
Design shear resistance \( 2560.5 \text{ kN} \)
Design moment y-y axis \( 546 \text{ kNm} \)
Buckling resist. moment \( 866.19 \text{ kNm} \)
Interaction factors \( 0.63035 \leq 1 \)
\( 0.63035 \leq 1 \)
**Location: Design Example 1 Curved Steel publication**

**I section beam curved on elevation**

Calculations are in accordance with BS5950-1:2000 and SCI publication entitled "Design of Curved Steel".

In the calculations the length of member (L) refers to the segment length of beam to be designed.

All loads and moments are factored.

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Design moment about z-z axis \( M_z = 321 \text{ kNm} \)
Design shear force \( F_v = 155 \text{ kN} \)
Design axial load (+ve comp) \( F = 0 \text{ kN} \)
Length of member \( L = 5 \text{ m} \)

533 x 210 x 122 UB.
Radius of curvature of beam \( R = 17.05 \text{ m} \)
Length of segment \( L_s = 5 \text{ m} \)
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Maximum moment \( ML_T = 321 \text{ kNm} \)
Quarter point \( m_2 = 80 \text{ kNm} \)
Mid-span \( m_3 = 181 \text{ kNm} \)
Three quarter point \( m_4 = 261 \text{ kNm} \)

**UNIVERSAL BEAM 533 x 210 x 122 UB Grade S 355**

**DESIGN**
Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.

**SUMMARY**
Factored shear force 155 kN
Shear capacity 1431.4 kN
Factored moment 321 kNm
Moment capacity 1104 kNm
LTB resistance 895.62 kNm
Location: Design Example 1 - compression convex side

I section beam curved on elevation

Calculations are in accordance with BS5950-1:2000 and SCI publication entitled "Design of Curved Steel".

In the calculations the length of member (L) refers to the segment length of beam to be designed.

All loads and moments are factored.

Design moment about z-z axis \( M_z = 546 \) kNm
Design shear force \( F_v = 155 \) kN
Design axial load (+ve comp) \( F = 0 \) kN
Length of member \( L = 5 \) m

533 x 210 x 122 UB.
Radius of curvature of beam \( R = 17.05 \) m
Length of segment \( L_s = 5 \) m
Young's Modulus \( E = 205 \) kN/mm²
Maximum moment \( M_{LT} = 546 \) kNm
Quarter point \( m_2 = 55 \) kNm
Mid-span \( m_3 = 198 \) kNm
Three quarter point \( m_4 = 362 \) kNm

UNIVERSAL BEAM

DESIGN

SUMMARY

Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.
Factored shear force 155 kN
Shear capacity 1431.4 kN
Factored moment 546 kNm
Moment capacity 1104 kNm
LTB resistance 971.92 kNm
Location: Based on example 2 load case 1 values

I section beam curved on elevation

Calculations are in accordance with BS5950-1:2000 and SCI publication entitled "Design of Curved Steel".

In the calculations the length of member (L) refers to the segment length of beam to be designed.

All loads and moments are factored.

- Design moment about z-z axis: Mz=172 kNm
- Design shear force: Fv=56 kN
- Design axial load (+ve comp): F=276 kN
- Length of member: L=3.348 m

457 x 191 x 98 UB.
- Radius of curvature of beam: R=18.6 m
- Length of segment: Ls=3.348 m
- Young's Modulus: E=205 kN/mm²
- Maximum moment on segment: MLT=172 kNm
- Far end BM: \( \beta M = 171 \) kNm
- Arch rise: h=11.282 m
- Arch span: l=34.2 m
- Quarter point: \( mz_2 = 148 \) kNm
- Mid-span: \( mz_3 = 134 \) kNm
- Three quarter point: \( mz_4 = 26.4 \) kNm
- Maximum moment on central half: \( M_{24} = 171 \) kNm

UNIVERSAL BEAM
- Design: 457 x 191 x 98 UB Grade S 275

SUMMARY
- Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.
- Factored shear force: 56 kN
- Shear capacity: 846.85 kN
- Factored moment: 172 kNm
- Moment capacity: 584.3 kNm
- LTB resistance: 451.35 kNm
- Axial load: 276 kN
- Compressive resistance: 1272.2 kN
- Local capacity check: 0.37864 ≤ 1
- Overall buckling check: 0.45108 ≤ 1
  0.50209 ≤ 1
Location: Ex1 - SCI publication 'Design of Curved Steel' (example 1)

I section beam curved in elevation

Beam is curved in elevation & design calculations are in accordance with BS EN 1993-1-1:2005.

In the calculations the length of member (L) refers to the segment length of beam to be designed.

All loads and moments are factored.

Design moment about major axis \( My_{Ed} = 321 \text{ kNm} \)
Design shear force \( V_{z Ed} = 155 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 0 \text{ kN} \)
Length of member \( L = 5 \text{ m} \)

533 x 210 x 122 UKB
Dimensions (mm): \( h=544.5 \text{ b=211.9} \text{ tw=12.7} \text{ tf=21.3} \text{ r=12.7} \)
Properties (cm): \( I_y=76000 \text{ Iz=3390} \text{ Wp_{ly}=3200} \text{ Wp_{lz}=500} \text{ It=178} \)
Radius of curvature of beam \( R=17.05 \text{ m} \)
Length of segment \( L_s=5 \text{ m} \)
Effective length for LTB \( LT=5 \text{ m} \)
Effective length parameter \( K=1 \)
Correction factor \( kc=1 \)

UNIVERSAL BEAM

533 x 210 x 122 UKB Grade S 355

DESIGN SUMMARY

Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.

Design shear force \( 155 \text{ kN} \)
Design shear resistance \( 1451 \text{ kN} \)
Design moment \( 321 \text{ kNm} \)
Moment resistance \( 1092.3 \text{ kNm} \)
LTB resistance \( 662.41 \text{ kNm} \)
Interaction factors \( 0.29386 \leq 1 \)
\( 0.48976 \leq 1 \)
Location: Ex2 - SCI Example 1 (compression convex side)

I section beam curved in elevation

Beam is curved in elevation & design calculations are in accordance with BS EN 1993-1-1:2005.

In the calculations the length of member (L) refers to the segment length of beam to be designed.

All loads and moments are factored.

Design moment about major axis \( MyEd = 546 \text{ kNm} \)
Design shear force \( VzEd = 155 \text{ kN} \)
Design axial load (+ve comp) \( NEd = 0 \text{ kN} \)
Length of member \( L = 5 \text{ m} \)

533 x 210 x 122 UKB
Dimensions (mm): \( h=544.5 \text{ b=211.9} \text{ tw=12.7} \text{ tf=21.3} \text{ r=12.7} \)
Properties (cm): \( Iy=76000 \text{ Iz=3390} \text{ Wply=3200} \text{ Wplz=500} \text{ It=178} \)
Radius of curvature of beam \( R=17.05 \text{ m} \)
Length of segment \( Ls=5 \text{ m} \)
Correction factor \( kc=1 \)

**UNIVERSAL BEAM**

533 x 210 x 122 UKB Grade S 355

**DESIGN SUMMARY**

Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.

Design shear force 155 kN
Design shear resistance 1451 kN
Design moment 546 kNm
Moment resistance 1084 kNm
LTB resistance 572.91 kNm
Interaction factors \( 0.50371 \leq 1 \text{ 0.97065 \leq 1} \)
Location: Ex3 - Based on SCI example 2 load case 1 values

I section beam curved in elevation

Beam is curved in elevation & design calculations are in accordance with BS EN 1993-1-1:2005.

In the calculations the length of member (L) refers to the segment length of beam to be designed.

All loads and moments are factored.

Design moment about major axis \( M_{yEd} = 172 \text{ kNm} \)
Design shear force \( V_{zEd} = 56 \text{ kN} \)
Design axial load (+ve comp) \( N_{Ed} = 276 \text{ kN} \)
Length of member \( L = 3.348 \text{ m} \)

457 x 191 x 98 UKB
Dimensions (mm): \( h = 467.2 \text{ b = 192.8 \ tw = 11.4 \ tf = 19.6 \ r = 10.2} \)
Properties (cm): \( I_y = 45700 \text{ Iz = 2350 \ W_{ply} = 2230 \ W_{plz} = 379 \ It = 121} \)
Radius of curvature of beam \( R = 18.6 \text{ m} \)
Length of segment \( L_s = 3.348 \text{ m} \)
Effective length for LTB \( L_T = 3.348 \text{ m} \)
Effective length parameter \( K = 1 \)
Correction factor \( k_c = 1 \)
Arch rise \( h' = 11.282 \text{ m} \)
Arch span \( l = 34.2 \text{ m} \)
Maximum end moment \( M_{h(1)} = 172 \text{ kNm} \)
Maximum span moment \( M_{s(1)} = 172 \text{ kNm} \)
End moment \( pM_{h(1)} = 172 \text{ kNm} \)
Maximum end moment \( M_{h(2)} = 172 \text{ kNm} \)
Maximum span moment \( M_{s(2)} = 172 \text{ kNm} \)
End moment \( pM_{h(2)} = 172 \text{ kNm} \)

UNIVERSAL BEAM
DESIGN SUMMARY

457 x 191 x 98 UKB Grade S 275
Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.

Design shear force \( 56 \text{ kN} \)
Design shear resistance \( 851.51 \text{ kN} \)
Design moment \( 172 \text{ kNm} \)
Moment resistance \( 584.3 \text{ kNm} \)
LTB resistance \( 590.95 \text{ kN姆} \)
Design axial load \( 276 \text{ kN} \)
Buckling resistance \( 1254.4 \text{ kN} \)
Interaction factors \( 0.37864 \leq 1 \)
\( 0.56622 \leq 1 \)
\( 0.41432 \leq 1 \)
Location: Ex1 - Sway check compliant

Portal frame (plastic design)

Procedure to determine the initial section sizes of pitched roof, haunched steel portal frames with partial base restraint.

Span of frame  \( L = 30 \text{ m} \)
Frame centres  \( L_2 = 6 \text{ m} \)
Height to eaves  \( h = 7 \text{ m} \)
Height, eaves to ridge  \( h_1 = 4 \text{ m} \)
Sheeting and insulation  \( w_{si} = 0.2 \text{ kN/m}^2 \)
Purlins  \( w_{p} = 0.07 \text{ kN/m}^2 \)
Frame  \( w_f = 0.16 \text{ kN/m}^2 \)
Imposed load  \( w_{i}' = 0.75 \text{ kN/m}^2 \)
Rafter to column moment ratio \( \beta = 0.95 \)
the bottom of the haunch \( a = 0.75 \text{ m} \)
533 x 210 x 82 UB.
457 x 191 x 82 UB.
Actual length considered \( L_h = 1.3 \text{ m} \)

**Column Details**

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<tr>
<th>Vertical reaction</th>
<th>( V = 162.18 \text{ kN} )</th>
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<td>Horizontal reaction</td>
<td>( H_r = 81.217 \text{ kN} )</td>
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<tr>
<td>Moment at base</td>
<td>( M_r = 56.4 \text{ kNm} )</td>
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**Plastic hinge**

<table>
<thead>
<tr>
<th>Bending moments in kNm</th>
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<tr>
<td>564.01 kNm</td>
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<td>56.4 kNm</td>
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SCALE 5.48 Office 1007 Proforma 460
Rafter details - increments of a 20th the length of the rafter

<table>
<thead>
<tr>
<th>Position</th>
<th>Distance on rafter</th>
<th>Distance on plan</th>
<th>Bending moment</th>
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<td>Column C.L</td>
<td></td>
<td>588.18 kNm</td>
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<td>12.75 m</td>
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<tr>
<td>Plastic hinge</td>
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<tr>
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<tr>
<td>19</td>
<td>14.748 m</td>
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<td>-279.76 kNm</td>
</tr>
<tr>
<td>20</td>
<td>15.524 m</td>
<td>15 m</td>
<td>-266.56 kNm</td>
</tr>
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</table>

Note: Negative sign indicates tension on bottom surface
Location: Ex2 - Nominal pin (alpha = 0.1)

Portal frame (plastic design)

Procedure to determine the initial section sizes of pitched roof, haunched steel portal frames with partial base restraint.

Span of frame: L=30 m
Frame centres: L2=6 m
Height to eaves: h=7 m
Height, eaves to ridge: h1=4 m
Sheeting and insulation: wsi=0.2 kN/m²
Purlins: wp=0.07 kN/m²
Frame: wf=0.16 kN/m²
Imposed load: wi'=0.75 kN/m²
Rafter to column moment ratio \[ \beta = 0.55 \]
the bottom of the haunch \[ a = 0.75 \text{ m} \]
533 x 210 x 82 UB.
406 x 178 x 60 UB.
Actual length considered \[ L_h = 2.8 \text{ m} \]

**Column Details**

Vertical reaction \( V = 162.18 \text{ kN} \)
Horizontal reaction \( H_r = 81.217 \text{ kN} \)
Moment at base \( M_r = 56.4 \text{ kNm} \)

Plastic hinge

Bending moments in kNm

56.4 kNm
## Rafter details - increments of a 20th the length of the rafter

<table>
<thead>
<tr>
<th>Position</th>
<th>Distance on rafter</th>
<th>Distance on plan</th>
<th>Bending moment</th>
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</thead>
<tbody>
<tr>
<td>0</td>
<td>Column C.L</td>
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<td>588.18 kNm</td>
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<tr>
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<td>0.75 m</td>
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<td>3 m</td>
<td>252.01 kNm</td>
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<tr>
<td>Point of contraflexure</td>
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<tr>
<td>20</td>
<td>15.524 m</td>
<td>15 m</td>
<td>-266.56 kNm</td>
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</tbody>
</table>

Note: Negative sign indicates tension on bottom surface
Portal frame (plastic design)

Procedure to determine the initial section sizes of pitched roof, haunched steel portal frames with partial base restraint.

Span of frame \( L = 30 \text{ m} \)
Frame centres \( L_2 = 6 \text{ m} \)
Height to eaves \( h = 7 \text{ m} \)
Height, eaves to ridge \( h_1 = 4 \text{ m} \)
Sheeting and insulation \( w_{si} = 0.2 \text{ kN/m}^2 \)
Purlins \( w_p = 0.07 \text{ kN/m}^2 \)
Frame \( w_f = 0.16 \text{ kN/m}^2 \)
Imposed load \( w_i' = 0.75 \text{ kN/m}^2 \)
Rafter to column moment ratio \( \beta = 0.5 \)
the bottom of the haunch \( a = 0.535 \text{ m} \)
533 x 210 x 82 UB.
406 x 178 x 54 UB.

**Column Details**

Vertical reaction \( V = 162.18 \text{ kN} \)
Horizontal reaction \( H_r = 87.24 \text{ kN} \)

<table>
<thead>
<tr>
<th>Position</th>
<th>Distance on rafter</th>
<th>Distance on plan</th>
<th>Bending moment</th>
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<td>0</td>
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<tr>
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SCALE 5.48          Office 1007          Proforma 460
Note: Negative sign indicates tension on bottom surface
Location: Ex4 - Ex1 from paper presented to The Structural Engineer

Portal frame (plastic design)

Procedure to determine the initial section sizes of pitched roof, haunched steel portal frames with partial base restraint.

\[ w \text{ kN/m} \]

\[ b_1 \]

\[ b_2 \]

\[ h_1 \]

\[ h_2 \]

Span of frame \( L = 25 \text{ m} \)
Frame centres \( L_2 = 6 \text{ m} \)
Height to eaves \( h = 7.6 \text{ m} \)
Height, eaves to ridge \( h_1 = 3.75 \text{ m} \)
Sheeting and insulation \( w_s i = 0.19 \text{ kN/m}^2 \)
Purlins \( w_p = 0.07 \text{ kN/m}^2 \)
Frame \( w_f = 0.12 \text{ kN/m}^2 \)
Imposed load \( w_i' = 0.65 \text{ kN/m}^2 \)
Rafter to column moment ratio  \(\beta = 0.3\)
the bottom of the haunch  \(a = 0.625\) m
457 x 191 x 67 UB.
356 x 127 x 33 UB.
* Tolerance (in range 0.1 to 1.0) tolst=0.1 %

Column self weight in kg/m 

Permanent loading (-ve is down) Gk=-6.15 kN/m

Imposed loading (-ve is down) Qki=-5 kN/m

Snow loading (-ve is down) Qks=-3.5 kN/m

Wind on left column Wk1=2.55 kN/m

Wind on left rafter Wk2=1.07 kN/m

Wind on right rafter Wk3=3.06 kN/m

Wind on right column Wk4=2.29 kN/m

Wind on left column Wk5=-2.33 kN/m

Wind on left rafter Wk6=-1.5 kN/m

Wind on right rafter Wk7=-1.5 kN/m

Wind on right column Wk8=2.33 kN/m
* Tolerance (in range 0.1 to 1.0) tolst=0.1 %

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<tr>
<th>Column self weight in kg/m</th>
<th>wtc=125 kg/m</th>
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<td>Permanent loading (-ve is down)</td>
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</tr>
<tr>
<td>Imposed loading (-ve is down)</td>
<td>Qki=-4.8 kN/m</td>
</tr>
<tr>
<td>Snow loading (-ve is down)</td>
<td>Qks=-3.2 kN/m</td>
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<tr>
<td>Wind on left column</td>
<td>Wk1=6.87 kN/m</td>
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<tr>
<td>Wind on left rafter</td>
<td>Wk2=3.2 kN/m</td>
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<tr>
<td>Wind on right rafter</td>
<td>Wk3=1.716 kN/m</td>
</tr>
<tr>
<td>Wind on right column</td>
<td>Wk4=0.67 kN/m</td>
</tr>
<tr>
<td>Wind on left column</td>
<td>Wk5=-4.33 kN/m</td>
</tr>
<tr>
<td>Wind on left rafter</td>
<td>Wk6=-1.92 kN/m</td>
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<td>Wind on right rafter</td>
<td>Wk7=-1.92 kN/m</td>
</tr>
<tr>
<td>Wind on right column</td>
<td>Wk8=4.33 kN/m</td>
</tr>
<tr>
<td>Tolerance (in range 0.1 to 1.0)</td>
<td>tolst=0.1 %</td>
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<tr>
<td>---------------------------------</td>
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</tr>
<tr>
<td>Column self weight in kg/m</td>
<td>wtc=113 kg/m</td>
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<td>Permanent loading (-ve is down)</td>
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<tr>
<td>Snow loading (-ve is down)</td>
<td>Qks=-3.2 kN/m</td>
</tr>
<tr>
<td>Wind on left column</td>
<td>Wk1=6.87 kN/m</td>
</tr>
<tr>
<td>Wind on left rafter</td>
<td>Wk2=3.2 kN/m</td>
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<td>Wind on right rafter</td>
<td>Wk3=1.716 kN/m</td>
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<tr>
<td>Wind on right column</td>
<td>Wk4=0.67 kN/m</td>
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<td>Wind on left column</td>
<td>Wk5=-4.33 kN/m</td>
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<tr>
<td>Wind on right column</td>
<td>Wk8=4.33 kN/m</td>
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</table>
* Tolerance (in range 0.1 to 1.0) tolst=0.1 %
Column self weight in kg/m wtc=113 kg/m
Permanent loading (-ve is down) Gk=-5.41 kN/m
Imposed loading (-ve is down) Qki=-4.8 kN/m
Snow loading (-ve is down) Qks=-3.2 kN/m
Wind on left column Wk1=6.87 kN/m
Wind on left rafter Wk2=3.2 kN/m
Wind on right rafter Wk3=1.716 kN/m
Wind on right column Wk4=0.67 kN/m
Wind on left column Wk5=-4.33 kN/m
Wind on left rafter Wk6=-1.92 kN/m
Wind on right rafter Wk7=-1.92 kN/m
Wind on right column Wk8=4.33 kN/m
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<td>Snow loading (-ve is down)</td>
<td>Qks=-3.2 kN/m</td>
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<tr>
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<tr>
<td>Wind on right rafter</td>
<td>Wk7=-1.92 kN/m</td>
</tr>
<tr>
<td>Wind on right column</td>
<td>Wk8=4.33 kN/m</td>
</tr>
</tbody>
</table>
Location: Example 1 from SCI P313

Span of portal frame \( L = 22 \) m
Eaves height \( E_c = 5.7 \) m
Frame centres \( S = 5 \) m
Rafter pitch \( \theta = 6^\circ \)
Length of haunch \( H = 1 \) m
457 x 152 x 52 UB.
457 x 152 x 52 UB.
cladding \( d_c = 0.08 \) kN/m²
purlins \( d_p = 0.03 \) kN/m²
rafter \( d_r = 0.1 \) kN/m²
Reduced cladding load \( r d_c = 0.07 \) kN/m²
Dead weight of wall panel \( w_p = 7 \) kN

SUMMARY
Overturning moment \( 53.847 \) kNm
Horizontal reaction \( 5.2109 \) kN
Vertical reaction \( 18 \) kN

Length of baseplate \( h_p = 650 \) mm
Breadth of baseplate \( b_p = 400 \) mm
Concrete strength \( f_{cu} = 25 \) N/mm²
Number of bolts \( N_b = 4 \)
Assumed diameter of bolt \( b_d = 20 \) mm
Spacing in line with frame axis \( s_p a c = 550 \) mm
Edge distance to tension bolts \( k = 50 \) mm
Spacing in longitudinal direct. \( s_b = 300 \) mm

Assumed weld size \( s_w = 8 \) mm
Selected base plate thickness \( t_p = 25 \) mm

Selected fillet weld size \( s_w = 8 \) mm
Selected fillet weld size \( s_f w = 8 \) mm

SUMMARY OF BASEPLATE
Size 650 mm x 400 mm x 25 mm
Grade S 275 steel
REQUIREMENTS
Number of H.D. bolts 4
Diameter of bolts M 20
Grade 8.8

WELDS
TENSION
Fillet weld 8 mm
Contact areas on the baseplate and column are machined to give a tight bearing contact
The specified tension weld will be used for both flanges

WEB
Fillet weld 8 mm
Longitudinal stability

Height of masonry wall \( H_w = 1 \text{ m} \)
Number of frames \( n_f = 10 \)
Load in fire on horizontal member \( H_R = 0.025 \times n_f \times V_r \times (E_c - H_w) / E_c = 3.7105 \text{ kN} \)

For a tensile stress of 0.065 times the design strength \( p_y = 275 \text{ N/mm}^2 \)
Cross-sectional area required \( A_{cl} = H_R \times 10^3 / (p_y \times 0.065) = 207.58 \text{ mm}^2 \)

The horizontal steel members providing longitudinal stability do not require fire protection.
Location: Ex2 - Single span steel frame with lattice rafter

Span of lattice frame                L=30 m
Eaves height                        Ec=6.7 m
Frame centres                       S=6 m
Rafter pitch                        theta=5°
457 x 191 x 89 UB.
cladding                            dc=0.13 kN/m²
purlins                             dp=0.03 kN/m²
Lattice rafter                      dr=0.1 kN/m²
Reduced cladding load               rdc=0.1 kN/m²
Dead weight of wall panel           wp=0 kN

SUMMARY                             Overturning moment 263.51 kNm
                                     Horizontal reaction 39.33 kN
                                     Vertical reaction 20.7 kN

Length of baseplate                 hp=650 mm
Breadth of baseplate                bp=400 mm
Concrete strength                   fcu=30 N/mm²
Number of bolts                     Nb=6
Assumed diameter of bolt            bd=30 mm
Spacing in line with frame axis     spac=550 mm
Edge distance to tension bolts      k=50 mm
Spacing in longitudinal direct.     sb=300 mm
Assumed weld size                   sw=13 mm
Selected base plate thickness       tp=45 mm
Selected fillet weld size           sw=13 mm
Selected fillet weld size           sfw=8 mm

SUMMARY OF BASEPLATE                Size 650 mm x 400 mm x 45 mm
GRADE S 275 steel                   Number of H.D. bolts 6
Diameter of bolts                   M 30
Grade 8.8

WELDS
TENSION                            Fillet weld 13 mm
Contact areas on the baseplate and
column are machined to give a tight
bearing contact
The specified tension weld will be
used for both flanges
WEB                                 Fillet weld 8 mm

Longitudinal stability
Height of masonry wall             Hw=0 m
Number of frames                   nf=10
Load in fire on horizontal member  HR=0.025*nf*Vr*(Ec-Hw)/Ec=5.175 kN

For a tensile stress of 0.065 times the design strength pys=265 N/mm²
Cross-sectional area required      Acl=HR*10^-3/(pys*0.065)=300.44 mm²

The horizontal steel members providing longitudinal stability do not
require fire protection.
Location: Ex3 - SCI Example 2 old publication

Span of portal frame L=30 m
Eaves height Ec=9 m
Frame centres S=6 m
Rafter pitch theta=7.5°
Length of haunch H=3 m
533 x 210 x 101 UB.
457 x 191 x 67 UB.
cladding dc=0.17 kN/m²
purlins dp=0.03 kN/m²
rafter dr=0.11 kN/m²
Reduced cladding load rdc=0.06 kN/m²
Dead weight of wall panel wp=14 kN

SUMMARY
Overturning moment 170.1 kNm
Horizontal reaction 10.136 kN
Vertical reaction 32 kN

Length of baseplate hp=700 mm
Breadth of baseplate bp=400 mm
Concrete strength fcu=25 N/mm²
Number of bolts Nb=6
Assumed diameter of bolt bd=24 mm
Spacing in line with frame axis spac=600 mm
Edge distance to tension bolts k=50 mm
Spacing in longitudinal direct. sb=300 mm
Assumed weld size sw=8 mm
Selected base plate thickness tp=35 mm
Selected fillet weld size sw=8 mm
Selected fillet weld size sfw=8 mm

SUMMARY OF BASEPLATE
Size 700 mm x 400 mm x 35 mm
Grade S 275 steel

REQUIREMENTS
Number of H.D. bolts 6
Diameter of bolts M 24
Grade 8.8

WELDS
TENSION
Fillet weld 8 mm
Contact areas on the baseplate and column are machined to give a tight bearing contact
The specified tension weld will be used for both flanges

WEB
Fillet weld 8 mm
**Longitudinal stability**

Height of masonry wall $H_w = 7$ m  

As the masonry wall is at least 75% of the height to the eaves provided it is connected to the column the requirements for longitudinal stability (6.2.3) will be satisfied.
Location: Ex4 - Long span example

Span of portal frame \( L = 38 \text{ m} \)
Eaves height \( E_c = 8.25 \text{ m} \)
Frame centres \( S = 6 \text{ m} \)
Rafter pitch \( \theta = 6^\circ \)
Length of haunch \( H = 3.8 \text{ m} \)
686 x 254 x 140 UB.
533 x 210 x 101 UB.
cladding \( d_c = 0.2 \text{ kN/m}^2 \)
purlins \( d_p = 0.08 \text{ kN/m}^2 \)
rafter \( d_r = 0.166 \text{ kN/m}^2 \)
Reduced cladding load \( r_d c = 0.02 \text{ kN/m}^2 \)
Dead weight of wall panel \( w_p = 10 \text{ kN} \)

**SUMMARY**

Overturning moment \( 350.03 \text{ kNm} \)
Horizontal reaction \( 23.281 \text{ kN} \)
Vertical reaction \( 40.324 \text{ kN} \)

Length of baseplate \( h_p = 875 \text{ mm} \)
Breadth of baseplate \( b_p = 450 \text{ mm} \)
Concrete strength \( f_{cu} = 20 \text{ N/mm}^2 \)
Offset distance \( o_d = 75 \text{ mm} \)
Number of bolts \( N_b = 6 \)
Assumed diameter of bolt \( b_d = 30 \text{ mm} \)
Spacing in line with frame axis \( s_{pac} = 650 \text{ mm} \)
Edge distance to tension bolts \( k = 50 \text{ mm} \)
Dimension between tension bolts \( a_1 = 120 \text{ mm} \)
Spacing in longitudinal direct. \( s_b = 350 \text{ mm} \)

Assumed weld size \( s_w = 12 \text{ mm} \)
Selected base plate thickness \( t_p = 40 \text{ mm} \)

Selected fillet weld size \( s_w = 12 \text{ mm} \)
Selected fillet weld size \( s_{wc} = 12 \text{ mm} \)
Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)

**SUMMARY OF BASEPLATE REQUIREMENTS**

Size 875 mm x 450 mm x 40 mm
Grade S 275 steel
Number of H.D. bolts 6
Diameter of bolts M 30
Grade 8.8

**WELDS**

TENSION
Fillet weld 12 mm

COMPRESSION
Fillet weld 12 mm

WEB
Fillet weld 8 mm
Longitudinal stability

Height of masonry wall \( H_w = 6.2 \text{ m} \)
As the masonry wall is at least 75% of the height to the eaves provided it is connected to the column the requirements for longitudinal stability (6.2.3) will be satisfied.
Location: Ex5 - Short span

Span of portal frame  L=10 m  
Eaves height  Ec=4.5 m  
Frame centres  S=5 m  
Rafter pitch  theta=10°  
Length of haunch  H=1 m  
178 x 102 x 19 UB.  
178 x 102 x 19 UB.  
cladding  dc=0.12 kN/m²  
purlins  dp=0.07 kN/m²  
rafter  dr=0.038 kN/m²  
Reduced cladding load  rdc=0.012 kN/m²  
Dead weight of wall panel  wp=8 kN  

SUMMARY  
Overturning moment  4.7025 kNm  
Horizontal reaction  1.0056 kN  
Vertical reaction  11 kN  

Length of baseplate  hp=340 mm  
Breadth of baseplate  bp=240 mm  
Concrete strength  fcu=20 N/mm²  

Offset distance  od=50 mm  
Number of bolts  Nb=4  
Assumed diameter of bolt  bd=20 mm  
Spacing in line with frame axis  spac=200 mm  
Edge distance to tension bolts  k=50 mm  
Spacing in longitudinal direct.  sb=120 mm  

Assumed weld size  sw=8 mm  
Selected base plate thickness  tp=15 mm  
Selected fillet weld size  sw=8 mm  
Selected fillet weld size  sfw=8 mm  

SUMMARY OF  
BASEPLATE  
Size 340 mm x 240 mm x 15 mm  
Grade S 275 steel  
NUMBER OF REQUIREMENTS  
Number of H.D. bolts  4  
Diameter of bolts  M 20  
Grade 4.6  

WELDS  
TENSION  
Fillet weld  8 mm  
Contact areas on the baseplate and column are machined to give a tight bearing contact  
The specified tension weld will be used for both flanges  
WEB  
Fillet weld  8 mm  

SCALE 5.48  
Office 1007  
Proforma 462
Longitudinal stability

Height of masonry wall \( H_w = 4.5 \text{ m} \)
As the masonry wall is at least 75\% of the height to the eaves provided it is connected to the column the requirements for longitudinal stability (6.2.3) will be satisfied.
Location: Example 1 from SCI P313

Design of column bases subject to boundary conditions using the recommended procedures set out in The Steel Construction Institute publication "Single Storey Steel Framed Buildings in Fire Boundary Conditions". Calculations are in accordance with EC3 Part 1-1.

Span of portal frame \( L = 22 \text{ m} \)
Eaves height \( H_e = 5.7 \text{ m} \)
Frame centres \( c_2 = 5 \text{ m} \)
Rafter pitch \( \theta = 6^\circ \)
Length of haunch \( L_h = 1 \text{ m} \)

Column section properties

457 x 152 x 52 UKB
Dimensions (mm): \( h_c = 449.8 \) \( b_c = 152.4 \) \( t_w_c = 7.6 \) \( t_f_c = 10.9 \) \( r_c = 10.2 \)
Properties (cm): \( I_{y_c} = 21400 \) \( I_{z_c} = 645 \) \( W_{p,l,y_c} = 1100 \) \( W_{p,l,z_c} = 133 \)
\( I_{t,c} = 21.4 \) \( A_c = 66.6 \) \( i_{y,c} = 17.925 \) \( i_{z,c} = 3.112 \)

Rafter section properties

457 x 152 x 52 UKB
Dimensions (mm): \( h_r = 449.8 \) \( b_r = 152.4 \) \( t_w_r = 7.6 \) \( t_f_r = 10.9 \) \( r_r = 10.2 \)
Properties (cm): \( I_{y,r} = 21400 \) \( I_{z,r} = 645 \) \( W_{p,l,y,r} = 1100 \) \( W_{p,l,z,r} = 133 \)
\( I_{t,r} = 21.4 \) \( A_r = 66.6 \) \( i_{y,r} = 17.925 \) \( i_{z,r} = 3.112 \)

Cladding loading \( d_c = 0.08 \text{ kN/m}^2 \)
Purlin loading \( d_p = 0.03 \text{ kN/m}^2 \)
Rafter loading \( d_r = 0.1 \text{ kN/m}^2 \)
Reduced cladding load \( r_d_c = 0.07 \text{ kN/m}^2 \)
Wall panel weight \( w_p = 7 \text{ kN} \)

SUMMARY

Overturning moment \( 53.847 \text{ kNm} \)
Horizontal reaction \( 5.2109 \text{ kN} \)
Vertical reaction \( 18 \text{ kN} \)

Length of baseplate \( h_p = 650 \text{ mm} \)
Breadth of baseplate \( b_p = 400 \text{ mm} \)
Number of bolts \( N_b = 4 \)
Assumed diameter of bolt \( b_d = 20 \text{ mm} \)
Spacing in line with frame axis \( s_{p,c} = 550 \text{ mm} \)
Edge distance to tension bolts \( k = 50 \text{ mm} \)
Spacing in longitudinal direct. \( s_b = 300 \text{ mm} \)
Selected baseplate thickness \( tp = 25 \text{ mm} \)

Tensile resistance per anchor \( FtRd = 0.9 \cdot fub \cdot As / (\gammaM2 \cdot 10^3) = 141.12 \text{ kN} \)

Selected fillet weld size \( sw = 8 \text{ mm} \)

Selected fillet weld size \( sfw = 8 \text{ mm} \)

**SUMMARY OF BASEPLATE**

Size 650 mm x 400 mm x 25 mm

Grade S 275 steel

**REQUIREMENTS**

Number of H.D. bolts 4

Diameter of bolts M 20

Grade 8.8

**WELD SUMMARY**

TENSION

Fillet weld 8 mm

Contact areas on the baseplate and column are machined to give a tight bearing contact.

The specified tension weld will be used for both flanges.

WEB

Fillet weld 8 mm

**Longitudinal stability to SCI P313 Section 2.8**

Height of masonry wall \( Hwall = 1 \text{ m} \)

Number of frames \( nf = 10 \)

Vertical reaction \( Vr = 18 \text{ kN} \)

Height of masonry wall \( Hwall = 1 \text{ m} \)

Height of unprotected area \( Hua = H - Hwall = 4.7 \text{ m} \)

Load in fire on horizontal member \( HR = 0.025 \cdot nf \cdot Vr \cdot Hua / He = 3.7105 \text{ kN} \)

For an unprotected member the tensile stress may be taken to be 0.065 of the design strength for normal temperature design.

Cross-sectional area required \( Acl = HR \cdot 10^3 / (fyc \cdot 0.065) = 207.58 \text{ mm}^2 \)

The horizontal steel members providing longitudinal stability do not require fire protection.
Location: Ex2 - Single span steel frame with lattice rafter

Design of column bases subject to boundary conditions using the recommended procedures set out in The Steel Construction Institute publication "Single Storey Steel Framed Buildings in Fire Boundary Conditions". Calculations are in accordance with EC3 Part 1-1.

Span of lattice frame \( L = 30 \text{ m} \)
Eaves height \( H_e = 6.7 \text{ m} \)
Frame centres \( c_2 = 6 \text{ m} \)
Rafter pitch \( \theta = 5^\circ \)

Column section properties

457 x 191 x 89 UKB
Dimensions (mm): \( h_c = 463.4 \text{ mm}, b_c = 191.9 \text{ mm}, t_w_c = 10.5 \text{ mm}, t_f_c = 17.7 \text{ mm}, r_c = 10.2 \text{ mm} \)
Properties (cm): \( I_{y_c} = 41000 \text{ cm}^4, I_{z_c} = 2090 \text{ cm}^4, W_{p_{y_c}} = 2010 \text{ cm}, W_{p_{z_c}} = 338 \text{ cm} \)
\( I_{t_c} = 90.7 \text{ cm}^4, A_c = 114 \text{ cm}^2, i_{y_c} = 18.964 \text{ cm}, i_{z_c} = 4.2817 \text{ cm} \)

Cladding loading \( d_c = 0.13 \text{ kN/m}^2 \)
Purlin loading \( d_p = 0.03 \text{ kN/m}^2 \)
Lattice rafter loading \( d_r = 0.1 \text{ kN/m}^2 \)
Reduced cladding load \( r_{d_c} = 0.1 \text{ kN/m}^2 \)
Wall panel weight \( w_p = 0 \text{ kN} \)

**SUMMARY**
- Overturning moment \( 263.51 \text{ kNm} \)
- Horizontal reaction \( 39.33 \text{ kN} \)
- Vertical reaction \( 20.7 \text{ kN} \)

Length of baseplate \( h_p = 650 \text{ mm} \)
Breadth of baseplate \( b_p = 400 \text{ mm} \)
Number of bolts \( N_b = 6 \)
Assumed diameter of bolt \( b_d = 30 \text{ mm} \)
Spacing in line with frame axis \( s_{p_{a_{c}}} = 550 \text{ mm} \)
Edge distance to tension bolts \( k = 50 \text{ mm} \)
Spacing in longitudinal direct. \( s_b = 300 \text{ mm} \)
Selected baseplate thickness \( t_p = 45 \text{ mm} \)
Tensile resistance per anchor \( F_{t_{Rd}} = 0.9 * f_{u_b} * A_s / (\gamma_{M_2} * 10^3) = 323.14 \text{ kN} \)

Selected fillet weld size \( s_{w} = 13 \text{ mm} \)
Selected fillet weld size \( s_{f_{w}} = 8 \text{ mm} \)

**SUMMARY OF BASEPLATE**
- Size \( 650 \text{ mm} \times 400 \text{ mm} \times 45 \text{ mm} \)
- Grade S 275 steel

**REQUIREMENTS**
- Number of H.D. bolts \( 6 \)
- Diameter of bolts \( M 30 \)
- Grade 8.8

**WELD SUMMARY**

**TENSION**
- Fillet weld \( 13 \text{ mm} \)
  Contact areas on the baseplate and column are machined to give a tight bearing contact.
  The specified tension weld will be used for both flanges.

**WEB**
- Fillet weld \( 8 \text{ mm} \)
Longitudinal stability to SCI P313 Section 2.8

Height of masonry wall  \( H_{\text{wall}} = 0 \text{ m} \)
Number of frames  \( n_f = 10 \)
Vertical reaction  \( V_r = 20.7 \text{ kN} \)
Height of masonry wall  \( H_{\text{wall}} = 0 \text{ m} \)
Height of unprotected area  \( H_{\text{ua}} = H_e - H_{\text{wall}} = 6.7 \text{ m} \)
Load in fire on horizontal member \( H_R = 0.025 \times n_f \times V_r \times H_{\text{ua}} / H_e = 5.175 \text{ kN} \)
For an unprotected member the tensile stress may be taken to be 0.065 of the design strength for normal temperature design.
Cross-sectional area required  \( A_{\text{cl}} = H_R \times 10^3 / (f_{\text{yc}} \times 0.065) = 300.44 \text{ mm}^2 \)

The horizontal steel members providing longitudinal stability do not require fire protection.
Location: Ex3 - SCI Example 2 old publication

Design of column bases subject to boundary conditions using the recommended procedures set out in The Steel Construction Institute publication "Single Storey Steel Framed Buildings in Fire Boundary Conditions". Calculations are in accordance with EC3 Part 1-1.

Span of portal frame       L=30 m  
Eaves height               He=9 m  
Frame centres              c2=6 m  
Rafter pitch               theta=7.5°  
Length of haunch           Lh=3 m  

Column section properties

533 x 210 x 101 UKB
Dimensions (mm): hc=536.7 bc=210 twc=10.8 tfe=17.4 rc=12.7
Properties (cm): Iyc=61500 Izc=2690 Wplyc=2610 Wplzc=399
                      Itc=101 Ac=129 iyc=21.834 izc=4.5665

Rafter section properties

457 x 191 x 67 UKB
Dimensions (mm): hr=453.4 br=189.9 twr=8.5 tfr=12.7 rr=10.2
Properties (cm): Iyr=29400 Izr=1450 Wplyr=1470 Wplzr=237
                      Itr=37.1 Ar=85.5 iyr=18.543 izr=4.1181

Cladding loading dc=0.17 kN/m²
Purlin loading dp=0.03 kN/m²
Rafter loading  dr=0.11 kN/m²
Reduced cladding load rdc=0.06 kN/m²
Wall panel weight wp=14 kN

SUMMARY

Overturning moment  170.1 kNm
Horizontal reaction 10.136 kN
Vertical reaction  32 kN

Length of baseplate hp=700 mm
Breadth of baseplate bp=400 mm
Number of bolts Nb=6
Assumed diameter of bolt bd=24 mm
Spacing in line with frame axis spac=600 mm
Edge distance to tension bolts k=50 mm
Spacing in longitudinal direct. sb=300 mm
Selected baseplate thickness \( tp = 35 \text{ mm} \)

Tensile resistance per anchor \( F_{Rd} = 0.9 \times f_{ub} \times A_s / (\gamma_{M2} \times 10^3) = 203.33 \text{ kN} \)

Selected fillet weld size \( sw = 8 \text{ mm} \)

Selected fillet weld size \( sfw = 8 \text{ mm} \)

**SUMMARY OF BASEPLATE**

<table>
<thead>
<tr>
<th>Size 700 mm x 400 mm x 35 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade S 275 steel</td>
</tr>
</tbody>
</table>

**REQUIREMENTS**

<table>
<thead>
<tr>
<th>Number of H.D. bolts</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bolts</td>
<td>M 24</td>
</tr>
<tr>
<td>Grade 8.8</td>
<td></td>
</tr>
</tbody>
</table>

**WELD SUMMARY**

**TENSION**

<table>
<thead>
<tr>
<th>Fillet weld</th>
<th>8 mm</th>
</tr>
</thead>
</table>

Contact areas on the baseplate and column are machined to give a tight bearing contact. The specified tension weld will be used for both flanges.

**WEB**

<table>
<thead>
<tr>
<th>Fillet weld</th>
<th>8 mm</th>
</tr>
</thead>
</table>

**Longitudinal stability to SCI P313 Section 2.8**

Height of masonry wall \( H_{wall} = 7 \text{ m} \)

As the masonry wall is at least 75% of the height to the eaves provided it is connected to the column the code requirements for longitudinal stability will be satisfied.
Location: Ex4 - Long span example

Design of column bases subject to boundary conditions using the recommended procedures set out in The Steel Construction Institute publication "Single Storey Steel Framed Buildings in Fire Boundary Conditions". Calculations are in accordance with EC3 Part 1-1.

Span of portal frame \( L = 38 \text{ m} \)
Eaves height \( H_e = 8.25 \text{ m} \)
Frame centres \( c_2 = 6 \text{ m} \)
Rafter pitch \( \theta = 6^\circ \)
Length of haunch \( L_h = 3.8 \text{ m} \)

**Column section properties**

686 x 254 x 140 UKB
Dimensions (mm): \( h_c = 683.5 \text{ mm}, b_c = 253.7 \text{ mm}, t_w_c = 12.4 \text{ mm}, t_f_c = 19 \text{ mm}, r_c = 15.2 \text{ mm} \)
Properties (cm): \( I_{y_c} = 136000 \text{ cm}^4, I_{z_c} = 5180 \text{ cm}^4, W_{p_{y_c}} = 4560 \text{ cm}^3, W_{p_{z_c}} = 638 \text{ cm}^3, I_{t_c} = 169 \text{ cm}^4, A_c = 178 \text{ cm}^2, i_{y_c} = 27.641 \text{ cm}, i_{z_c} = 5.3945 \text{ cm} \)

**Rafter section properties**

533 x 210 x 101 UKB
Dimensions (mm): \( h_r = 536.7 \text{ mm}, b_r = 210 \text{ mm}, t_w_r = 10.8 \text{ mm}, t_f_r = 17.4 \text{ mm}, r_r = 12.7 \text{ mm} \)
Properties (cm): \( I_{y_r} = 61500 \text{ cm}^4, I_{z_r} = 2690 \text{ cm}^4, W_{p_{y_r}} = 2610 \text{ cm}^3, W_{p_{z_r}} = 399 \text{ cm}^3, I_{t_r} = 101 \text{ cm}^4, A_r = 129 \text{ cm}^2, i_{y_r} = 21.834 \text{ cm}, i_{z_r} = 4.5665 \text{ cm} \)

Cladding loading \( d_c = 0.2 \text{ kN/m}^2 \)
Purlin loading \( d_p = 0.08 \text{ kN/m}^2 \)
Rafter loading \( d_r = 0.166 \text{ kN/m}^2 \)
Reduced cladding load \( r_d_c = 0.02 \text{ kN/m}^2 \)
Wall panel weight \( w_p = 10 \text{ kN} \)

**SUMMARY**

- Overturning moment \( 350.03 \text{ kNm} \)
- Horizontal reaction \( 23.281 \text{ kN} \)
- Vertical reaction \( 40.324 \text{ kN} \)

Length of baseplate \( h_p = 875 \text{ mm} \)
Breadth of baseplate \( b_p = 450 \text{ mm} \)
Number of bolts \( N_b = 6 \)
Assumed diameter of bolt \( b_d = 30 \text{ mm} \)
Spacing in line with frame axis \( s_p ac = 650 \text{ mm} \)
Edge distance to tension bolts \( k = 50 \text{ mm} \)
Spacing in longitudinal direct. \( s_b = 350 \text{ mm} \)
Sample output for SCALE Proforma 462. (ans=4)                       Page: 2
Steel design to BS5950-1:2000 and Eurocode 3                         Made by: IFB
Portal frame in boundary conditions (fire)                          Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.                            Ref No: SC462 EC

Selected baseplate thickness       tp=40 mm
Tensile resistance per anchor      FtRd=0.9*fub*As/(gamM2*10^3)=323.14 kN

Selected fillet weld size          sw=12 mm
Selected fillet weld size          swc=12 mm
Selected fillet weld size          swf=8 mm

SUMMARY OF
BASEPLATE
Size 875 mm x 450 mm x 40 mm
Grade S 275 steel

REQUIREMENTS
Number of H.D. bolts 6
Diameter of bolts M 30
Grade 8.8

WELD SUMMARY
TENSION
Fillet weld 12 mm
COMPRESSION
Fillet weld 12 mm
WEB
Fillet weld 8 mm

Longitudinal stability to SCI P313 Section 2.8
Height of masonry wall Hwall=6.2 m
As the masonry wall is at least 75% of the height to the eaves provided it is connected to the column the code requirements for longitudinal stability will be satisfied.
Location: Ex5 - Short span

Design of column bases subject to boundary conditions using the recommended procedures set out in The Steel Construction Institute publication "Single Storey Steel Framed Buildings in Fire Boundary Conditions". Calculations are in accordance with EC3 Part 1-1.

Span of portal frame \( L=10 \text{ m} \)
Eaves height \( H_e=4.5 \text{ m} \)
Frame centres \( c_2=5 \text{ m} \)
Rafter pitch \( \theta=10^\circ \)
Length of haunch \( L_h=1 \text{ m} \)

Column section properties

178 x 102 x 19 UKB
Dimensions (mm): \( h_c=177.8 \text{ mm} \) \( b_c=101.2 \text{ mm} \) \( t_w_c=4.8 \text{ mm} \) \( t_f_c=7.9 \text{ mm} \) \( r_c=7.6 \text{ mm} \)
Properties (cm): \( I_{y_c}=1360 \text{ cm}^4 \) \( I_{z_c}=137 \text{ cm}^4 \) \( W_{p_l y_c}=171 \text{ cm} \) \( W_{p_l z_c}=41.6 \text{ cm} \)
\( I_{t_c}=4.41 \text{ cm}^4 \) \( A_c=24.3 \text{ cm}^2 \) \( i_{y_c}=7.4811 \text{ cm} \) \( i_{z_c}=2.3744 \text{ cm} \)

Rafter section properties

178 x 102 x 19 UKB
Dimensions (mm): \( h_r=177.8 \text{ mm} \) \( b_r=101.2 \text{ mm} \) \( t_w_r=4.8 \text{ mm} \) \( t_f_r=7.9 \text{ mm} \) \( r_r=7.6 \text{ mm} \)
Properties (cm): \( I_{y_r}=1360 \text{ cm}^4 \) \( I_{z_r}=137 \text{ cm}^4 \) \( W_{p_l y_r}=171 \text{ cm} \) \( W_{p_l z_r}=41.6 \text{ cm} \)
\( I_{t_r}=4.41 \text{ cm}^4 \) \( A_r=24.3 \text{ cm}^2 \) \( i_{y_r}=7.4811 \text{ cm} \) \( i_{z_r}=2.3744 \text{ cm} \)

Cladding loading \( d_c=0.12 \text{ kN/m}^2 \)
Purlin loading \( d_p=0.07 \text{ kN/m}^2 \)
Rafter loading \( d_r=0.038 \text{ kN/m}^2 \)
Reduced cladding load \( r_{d_c}=0.012 \text{ kN/m}^2 \)
Wall panel weight \( w_p=8 \text{ kN} \)

SUMMARY

Overturning moment 4.7025 kNm
Horizontal reaction 1.0056 kN
Vertical reaction 11 kN

Length of baseplate \( h_p=340 \text{ mm} \)
Breadth of baseplate \( b_p=240 \text{ mm} \)
Number of bolts \( N_b=4 \)
Assumed diameter of bolt \( b_d=20 \text{ mm} \)
Spacing in line with frame axis \( \text{spac}=200 \text{ mm} \)
Edge distance to tension bolts \( k=50 \text{ mm} \)
Spacing in longitudinal direct. \( s_b=120 \text{ mm} \)
Selected baseplate thickness \( t_p = 15 \text{ mm} \)

Tensile resistance per anchor \( F_{tRd} = 0.9\cdot f_{ub}\cdot A_s / (\gamma_M 2\cdot 10^3) = 70.56 \text{ kN} \)

Selected fillet weld size \( s_w = 8 \text{ mm} \)

Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)

**SUMMARY OF BASEPLATE**
- Size \( 340 \text{ mm} \times 240 \text{ mm} \times 15 \text{ mm} \)
- Grade S 275 steel

**REQUIREMENTS**
- Number of H.D. bolts 4
- Diameter of bolts M 20
- Grade 4.6

**WELD SUMMARY**

**TENSION**
- Fillet weld 8 mm
- Contact areas on the baseplate and column are machined to give a tight bearing contact.
- The specified tension weld will be used for both flanges.

**WEB**
- Fillet weld 8 mm

**Longitudinal stability to SCI P313 Section 2.8**

Height of masonry wall \( H_{wall} = 4.5 \text{ m} \)

As the masonry wall is at least 75% of the height to the eaves provided it is connected to the column the code requirements for longitudinal stability will be satisfied.
Location: Example 9, 'Beams subject to bending and torsion'.

Simply supported steel channel unrestrained along its length

Calculations are in accordance with BS5950 and 'Beams Subject to Bending and Torsion' published by The Steel Construction Institute. The beam is considered as "torsion fixed, warping free".

Beam span \( L = 4 \text{ m} \)

Uniformly distributed load

\[
\begin{align*}
\text{w/metre} & \\
\text{vvvvvvvvvvvvvvvvvvvvvvvvvvvvvvvvvvvvv} & \\
\text{A} & \text{B} \\
\text{////} & \text{ooo} \\
\text{------------------------------} & \text{L} \end{align*}
\]

305 x 102 Tapered Channel.  
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Loading

Dead load \( wd = 2 \text{ kN/m} \)
Imposed load \( wi = 0 \text{ kN/m} \)

Distance between shear centre and centre of web \( eo = 35.869 \text{ mm} \)

Eccentricity \( ecc = decc - t/2 + eo = -19.231 \text{ mm} \)

Eccentricity value must be positive,
Eccentricity \( ecc = -ecc = 19.231 \text{ mm} \)
Shear force

Axial loading (inc. self weight) \( sw = 0.7 \text{ kN/m} \) (factored)
Shear force \( F_v = (sw+w) \frac{L}{2} = 7 \text{ kN} \)

Check section for combined bending and torsion

Torsional stress in flange \( fw = E \times 10^3 \times W_{no} \times 10^2 \times \phi'' = 2.9022 \text{ N/mm}^2 \)

(i) Buckling Check

Equation 2.21

\[
\frac{M}{Mb/mLT} + \frac{fbyt + fw}{py} \left[ 1 + \frac{0.5 M}{Mb/mLT} \right] \leq 1.0
\]

Equation 2.21 \( uf = mLT \frac{M}{Mb} + \frac{(fbyt + fw)}{py} \times (1 + 0.5 \frac{mLT \times M}{Mb}) = 0.082079 \)
Since \( uf \leq 1 \), Section is adequate for buckling.

(ii) Local "Capacity" Check

Equation 2.22

\( fbx + fbyt + fw \leq py \)

where:
bending normal stress \( fbx = \frac{M \times (D/2)}{Ix} \times 100 = 12.997 \text{ N/mm}^2 \)
combined stress \( fcb = fbx + fbyt + fw = 16.196 \text{ N/mm}^2 \)
Since \( fcb \leq py \) (16.196 N/mm² ≤ 275 N/mm²), section is suitable.

SECTION 305 x 102 Grade S 275
PROPERTIES Tapered Flange Channel
Applied shear force 7 kN
Shear capacity 512.98 kN
DESIGN Applied moment 7 kNm
SUMMARY Buckling resistance 92.449 kNm
Buckling check 0.082079 ≤ 1
Combined bending str. 16.196 N/mm²
Design strength 275 N/mm²
Midspan deflection 0 mm
Limiting deflection 11.111 mm
Location: Example using parallel flange channel

Simply supported steel channel unrestrained along its length

Calculations are in accordance with BS5950 and 'Beams Subject to Bending and Torsion' published by The Steel Construction Institute. The beam is considered as "torsion fixed, warping free".

Beam span \( L = 4 \text{ m} \)

Concentrated load along length

\[
\text{300 x 100 UKPFC}
\]

Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Loading

Dead load \( W_d = 0.5 \text{ kN} \)

Imposed load \( W_i = 20 \text{ kN} \)

Distance from support A to load \( xL = 2 \text{ m} \)

Eccentricity \( ecc = \text{decc} - \frac{t}{2} + eo = -16.898 \text{ mm} \)

Eccentricity value must be positive, \( ecc = -ecc = 16.898 \text{ mm} \)
Shear force

Axial loading (inc.self weight) \( sw = 0.7 \text{kN/m} \) (factored)
Max shear \( F_v = \frac{W}{2} + sw \cdot L/2 = 17.75 \text{kN} \)

Check section for combined bending and torsion

Torsional stress in flange \( fw = E \cdot 10^3 \cdot W_{no} \cdot 10^2 \cdot \phi'' = 20.923 \text{N/mm}^2 \)

(i) Buckling Check

Equation 2.21

\[
\frac{M}{Mb/mLT} + \left( \frac{fbyt + fw}{py} \right) \left[ \frac{0.5 M}{Mb/mLT} + 1 \right] \leq 1.0
\]

Equation 2.21 \( uf = \frac{mLT \cdot M}{Mb} + \left( \frac{fbyt + fw}{py} \right) \cdot (1 + 0.5 \cdot \frac{mLT \cdot M}{Mb}) = 0.30666 \)
Since \( uf \leq 1 \), Section is adequate for buckling.

(ii) Local "Capacity" Check

Equation 2.22

\[ fbx + fbyt + fw \leq py \]
where:
bending normal stress \( fbx = \frac{M \cdot (D/2)}{I_x} \cdot 100 = 62.151 \text{N/mm}^2 \)
combined stress \( fcb = fbx + fbyt + fw = 88.04 \text{N/mm}^2 \)
Since \( fcb \leq py \) (88.04 N/mm² \leq 345 N/mm²), section is suitable.

SECTION 300 x 100 Grade S 355
PROPERTIES Parallel Flange Channel
Applied shear force 17.75 kN
Shear capacity 558.9 kN

DESIGN
Applied moment 34.1 kN

SUMMARY
Buckling resistance 130.31 kNm
Buckling check 0.30666 ≤ 1
Combined bending str. 88.04 N/mm²
Design strength 345 N/mm²
Midspan deflection 1.5806 mm
Limiting deflection 11.111 mm
Simply supported steel channel unrestrained along its length

Calculations are in accordance with BS EN 1993-1-1: 2005 and 'Design of Steel Beams in Torsion' published by SCI. The beam is considered as "torsion fixed, warping free".

Beam span \( L = 5 \text{ m} \)

Uniformly distributed load

\[
	ext{w/metre} \\
\begin{array}{c}
A \\
\hline
/// \\
\hline
B
\end{array}
\]

300 x 100 UKPFC

Loading

Permanent load \( wd = 8.36 \text{ kN/m} \)
Variable (non-permanent) load \( wi = 0 \text{ kN/m} \)

Distance between shear centre and section centroid \( eo = 37.602 \text{ mm} \)

Eccentricity \( ecc = decc - tw/2 + eo = -21.898 \text{ mm} \)

Shear force

Axial loading (inc. self weight) \( sw = 0.63 \text{ kN/m (factored)} \)
Shear force \( Fv = (sw+w) \cdot L/2 = 29.79 \text{ kN} \)
Design value of torsion effects

Allowing for elastic interaction between St Venant torsion and warping torsion.

Torsional bending constant
\[ a = \left( \frac{E}{10^3 I_w} / (G I_t) \right)^{0.5} \times 10 \]
\[ = 756.81 \text{ mm} \]

Minor axis moment
\[ M_{zEd} = M \phi = 0.80095 \text{ kNm} \]

Warping moment at mid-span
\[ M_{wEd} = E I_w \phi'' \times 10^{-6} / (h-t_f) \]
\[ = 0.46264 \text{ kNm} \]

Bending resistance

Design bending moment
\[ M_{yEd} = 37.238 \text{ kNm} \]

Bending resistance moment (y-y)
\[ M_{yRd} = W_{ply} \phi_y / 10^3 = 169.87 \text{ kNm} \]

Bending resistance moment (z-z)
\[ M_{zRd} = W_{plz} \phi_z / 10^3 = 39.22 \text{ kNm} \]

Shear plastic resistance - Clause 6.2.6

Shear plastic resistance
\[ V_{plRd} = A_{vz} \phi_y / SQR(3) / 1000 = 443.08 \text{ kN} \]

Reduced shear resistance in the presence of torsion

at support \( z = 0 \) mm

Reduced shear plastic resistance
\[ V_{TRd} = (t_{r1} - t_{r2}) \times V_{plRd} = 428.53 \text{ kN} \]

As shear force less than shear plastic resistance, OK.

Lateral torsional buckling

Non-dimensional slenderness
\[ \lambda_{LT} = L \times 100 / (iz \times 96) = 1.6643 \]

Design Buckling resistance moment
\[ M_{bRd} = \chi_{LT} \times W_{ply} \phi_y / 10^3 = 49.476 \text{ kNm} \]

Unity factor
\[ \text{unitb} = M_{yEd} / M_{bRd} = 0.75264 \]

Section chosen is suitable for lateral torsional buckling.

Interaction of LTB with minor axis bending and torsion

Critical moment
\[ M_{cr} = W_{ply} \phi_y / (\lambda_{LT}^2 \times 1000) = 61.323 \text{ kNm} \]

Unity expression
\[ u_t = M_{yEd} / M_{bRd} + C_{mz} M_{zEd} / M_{zRd} + k_w k_z w \times k_a M_{wEd} / M_{wRd} \]
\[ = 0.84503 \]

Section chosen is suitable.

SECTION 300 x 100 Grade S 275
PROPERTIES Parallel Flange Channel
Shear force 29.79 kN
Shear resistance 428.53 kN

DESIGN

Applied Moment 37.238 kNm

SUMMARY
Buckling resistance 49.476 kNm
LTB interaction 0.84503 ≤ 1
Midspan deflection 4.1562 mm
Angle of twist 0.91288 deg
**Location:** Point load at mid-span

**Simply supported steel channel unrestrained along its length**

Calculations are in accordance with BS EN 1993-1-1: 2005 and 'Design of Steel Beams in Torsion' published by SCI.

The beam is considered as "torsion fixed, warping free".

Beam span $L=4 \text{ m}$

Concentrated load along length

\[
\begin{array}{c}
\text{A} \\
/// \\
\text{B} \\
\text{Load} \\
\text{Decc} \\
\text{eo} \\
\text{Section + shear centroid} \\
\text{Distance between shear centre and centre of web} \\
\text{eo} = 35.869 \text{ mm}
\end{array}
\]

305 x 102 Tapered Channel.

**Loading**

Variable load $W_d=30 \text{ kN}$

Variable (non-permanent) load $W_i=0 \text{ kN}$

Distance from support A to load $xL=2 \text{ m}$

\[
\begin{array}{c}
\text{load} \\
\text{Decc} \\
\text{eo} \\
\text{Distance from the back of channel} \\
\text{Decc}=-70 \text{ mm} \\
\text{Eccentricity} \\
ecc=\text{decc}-\text{tw}/2+\text{eo}=-39.231 \text{ mm}
\end{array}
\]

Eccentricity value must be positive,

Eccentricity $ecc=-ecc=39.231 \text{ mm}$
**Shear force**

Axial loading (inc. self weight) \( sw = 0.63 \text{ kN/m (factored)} \)

Max shear \( F_v = W/2 + sw \times L/2 = 21.51 \text{ kN} \)

**Design value of torsion effects**

Allowing for elastic interaction between St Venant torsion and warping torsion.

- Torsional bending constant \( a = \left( \frac{E}{10^3} \frac{I_w}{(G \times I_t)} \right)^{0.5} \times 10 \)
  - \( = 779.79 \text{ mm} \)
- Torsional moment \( T_d = W \times \text{ecc}/10^3 = 1.5889 \text{ kNm} \)
- Distance along member \( z = \alpha \times L \times 1000 = 2000 \text{ mm} \)
- Minor axis moment \( M_{zEd} = M \times \phi = 1.4026 \text{ kNm} \)
- Warping moment at mid-span \( M_{wEd} = E \times I_w \times \phi^{''} \times 10^{-6} / (h-t_f) = 2.111 \text{ kNm} \)

**Bending resistance**

- Design bending moment \( M_{yEd} = 41.76 \text{ kNm} \)
- Bending resistance moment (y-y) \( M_{yRd} = W_{ply} \times f_y / 10^3 = 175.45 \text{ kNm} \)
- Bending resistance moment (z-z) \( M_{zRd} = W_{plz} \times f_y / 10^3 = 35.2 \text{ kNm} \)

**Shear plastic resistance - Clause 6.2.6**

- Shear plastic resistance \( V_{plRd} = A_{vz} \times f_y / SQR(3) / 1000 = 517.37 \text{ kN} \)

**Reduced shear resistance in the presence of torsion**

at support \( z = 0 \text{ mm} \)

Reduced shear plastic resistance \( V_{TRd} = (tr_1 - tr_2) \times V_{plRd} = 479.05 \text{ kN} \)

As shear force less than shear plastic resistance, OK.

**Lateral torsional buckling**

- Non-dimensional slenderness \( \lambda_{LT} = L \times 100 / (iz \times 96) = 1.4315 \)
- Design Buckling resistance moment \( M_{bRd} = \chi_{LT} \times W_{ply} \times f_y / 10^3 = 63.453 \text{ kNm} \)
- Unity factor \( \text{unitb} = M_{yEd} / M_{bRd} = 0.65812 \)

Section chosen is suitable for lateral torsional buckling.

**Interaction of LTB with minor axis bending and torsion**

- Critical moment \( M_{cr} = W_{ply} \times f_y / (\lambda_{LT}^2 \times 1000) = 85.617 \text{ kNm} \)
- Unity expression \( \text{ut} = M_{yEd} / M_{bRd} + C_{mz} \times M_{zEd} / M_{zRd} + k_w \times k_{zw} \times k_a \times M_{wEd} / M_{wRd} = 0.94256 \)

Section chosen is suitable.

**SECTION**

**305 x 102 Grade S 275**

**PROPERTIES**

- Tapered Flange Channel
- Shear force 21.51 kN
- Shear resistance 479.05 kN

**DESIGN**

- Applied Moment 41.76 kNm

**SUMMARY**

- Buckling resistance 63.453 kNm
- LTB interaction 0.94256 ≤ 1
- Midspan deflection 2.4109 mm
- Angle of twist 1.4255 deg
Calculations are in accordance with BS5950 and 'Beams Subject to Bending and Torsion' published by The Steel Construction Institute. The beam is considered as "torsion fixed, warping free".

Beam span \( L = 4 \text{ m} \)

Concentrated load along length

Dead load \( W_d = 0 \text{ kN} \)
Imposed load \( W_i = 62.5 \text{ kN} \)
Distance from support A to load \( xL = 2 \text{ m} \)
Load Eccentricity \( \text{ecc} = 75 \text{ mm} \)
254 x 254 x 89 UC.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Factored axial loading \( sw = 1.22 \text{ kN/m} \)

**UNIVERSAL COLUMN**

254 x 254 x 89 UC Grade S 275

**DESIGN**

Shear force 52.44 kN

Applied Moment 102.44 kNm

Buckling resistance 298.01 kNm

Buckling check 0.76447 ≤ 1

**SUMMARY**

Combined bending str. 202.26 N/mm²

Design strength 265 N/mm²

Combined shear stress 49.454 N/mm²

Shear strength 159 N/mm²

Midspan deflection 2.8427 mm

Limiting deflection 11.111 mm
Calculations are in accordance with BS EN 1993-1-1: 2005 and 'Design of Steel Beams in Torsion' published by SCI. The beam is considered as "torsion fixed, warping free".

Beam span \( L = 4 \text{ m} \)

Concentrated load along length

\[ < \text{xL} \rightarrow \text{W} > \]

Permanent load \( W_d = 74 \text{ kN} \)
Variable (non-permanent) load \( W_i = 0 \text{ kN} \)
Distance from support A to load \( xL = 2 \text{ m} \)
Load Eccentricity \( \text{ecc} = 75 \text{ mm} \)

254 x 254 x 73 UKC

Dimensions (mm): \( h = 254.1 \text{ b=254.6 tw=8.6 tf=14.2 r=12.7} \)
Properties (cm): \( I_y = 11400 \text{ Iz=3910 Wply=992 Wplz=465 It=57.6} \)
Factored axial loading \( sw = 1 \text{ kN/m} \)
Eff.length between restraints \( LT = 4 \text{ m} \)
Coefficient C1 \( C1 = 1.35 \)

BEAM SECTION
254 x 254 x 73 UKC Grade S 275

Shear force \( 51.95 \text{ kN} \)
Shear resistance \( 378.68 \text{ kN} \)

DESIGN
Applied moment \( 101.9 \text{ kNm} \)
Buckling resistance \( 272.8 \text{ kNm} \)
Plastic interaction \( 0.51136 \leq 1 \)
LTB interaction \( 0.63316 \leq 1 \)

SUMMARY
Midspan deflection \( 4.2246 \text{ mm} \)
Angle of twist \( 2.2052 \text{ deg} \)
Location: Beam carrying udl

Calculations are in accordance with BS EN 1993-1-1: 2005 and 'Design of Steel Beams in Torsion' published by SCI. The beam is considered as "torsion fixed, warping free".

Beam span L=4 m

Uniformly distributed load

\[ w = \text{w/metre} \]

<table>
<thead>
<tr>
<th>Location: Beam carrying udl</th>
</tr>
</thead>
</table>

Permanent load \( wd = 14 \text{ kN/m} \)
Variable (non-permanent) load \( wi = 0 \text{ kN/m} \)
Load Eccentricity \( ecc = 75 \text{ mm} \)
254 x 254 x 73 UKC
Dimensions (mm): \( h=254.1 \text{ b}=254.6 \text{ tw}=8.6 \text{ tf}=14.2 \text{ r}=12.7 \)
Properties (cm): \( Iy=11400 \text{ Iz}=3910 \text{ Wply}=992 \text{ Wplz}=465 \text{ It}=57.6 \)
Factored axial loading \( sw = 1 \text{ kN/m} \)
Eff. length between restraints \( LT = 4.8 \text{ m} \)

**BEAM SECTION** 254 x 254 x 73 UKC Grade S 275
Shear force 39.8 kN
Shear resistance 392.51 kN

**DESIGN**
Applied moment 39.8 kNm

**SUMMARY**
Buckling resistance 272.8 kNm
Plastic interaction 0.13967 ≤ 1
LTB interaction 0.23241 ≤ 1
Midspan deflection 2.0525 mm
Angle of twist 1.0328 deg
**Location:** Example 2. 'Beams subject to bending and torsion'.

**Simply supported steel hollow section unrestrained along its length**

Calculations are in accordance with BS5950 and 'Beams Subject to Bending and Torsion' published by The Steel Construction Institute. The beam is considered as "torsion fixed, warping free".

Beam span \( L=4 \text{ m} \)

Concentrated load along length

\[
\begin{array}{c}
\text{A} \\
\text{B}
\end{array}
\]

Dead load \( W_d=0 \text{ kN} \)

Imposed load \( W_i=62.5 \text{ kN} \)

Distance from support A to load \( xL=2 \text{ m} \)

Eccentricity \( \text{ecc}=75 \text{ mm} \)

250 x 150 x 8 RHS - Hot finished.

Properties (cm): \( A=60.8 \), \( rx=9.17 \), \( Zx=409 \), \( Sx=501 \), \( Ix=5110 \)

\( J=5020 \), \( C=506 \), \( Zy=306 \), \( Sy=350 \), \( Iy=2300 \), \( ry=6.15 \)

Young's Modulus \( E=205 \text{ kN/mm}^2 \)

Factored axial load (inc. s.w.) \( sw=0.69 \text{ kN/m} \)

**HOT FINISHED**

**RECTANGULAR HOLLOW SECTION**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>250 x 150 x 8 RHS</td>
<td>Grade S 275</td>
</tr>
<tr>
<td>Shear force</td>
<td>51.38 kN</td>
</tr>
<tr>
<td>Factored moment</td>
<td>101.38 kNm</td>
</tr>
<tr>
<td>Buckling resistance</td>
<td>134.97 kNm</td>
</tr>
<tr>
<td>Combined bending str.</td>
<td>248.5 N/mm²</td>
</tr>
<tr>
<td>Design strength</td>
<td>275 N/mm²</td>
</tr>
<tr>
<td>Combined shear stress</td>
<td>23.153 N/mm²</td>
</tr>
<tr>
<td>Shear strength</td>
<td>165 N/mm²</td>
</tr>
<tr>
<td>Midspan deflection</td>
<td>7.9551 mm</td>
</tr>
<tr>
<td>Limiting deflection</td>
<td>11.111 mm</td>
</tr>
</tbody>
</table>
**Location:** Example 5 'Design of steel beams in torsion'

**Simply supported steel hollow section unrestrained along its length**

Calculations are in accordance with BS EN 1993-1-1: 2005 and 'Design of Steel Beams in Torsion' published by SCI.

The beam is considered as "torsion fixed, warping free".

**Beam span**  
$L=5$ m

**Uniformly distributed load**

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>///</td>
<td>oo</td>
</tr>
</tbody>
</table>

- **Permanent load**  
  $wd=5.1$ kN/m
- **Variable (non-permanent) load**  
  $wi=0$ kN/m
- **Eccentricity**  
  $ecc=180$ mm

300 x 100 x 8 RHS - Hot finished.

**Properties (cm):**  
$A=60.8 \text{ iz}=4.21 \ Wely=420 \ Wply=546 \ Iy=6310$  
$It=3070 \ Wt=387 \ Welz=216 \ Wplz=245 \ Iz=1080 \ iy=10.2$

- **Factored axial load (inc. s.w.)**  
  $sw=4.85$ kN/m
- **Eff.length between restraints**  
  $LT=5$ m

**HOT FINISHED**  
In accordance with EN 10210

**RECTANGULAR HOLLOW SECTION**  
300 x 100 x 8 RHS  Grade S 355

<table>
<thead>
<tr>
<th>HOT FINISHED</th>
<th>RECTANGULAR HOLLOW SECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>300 x 100 x 8 RHS  Grade S 355</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear force</td>
<td>29.338 kN</td>
</tr>
<tr>
<td>Shear resistance</td>
<td>890.11 kN</td>
</tr>
<tr>
<td>Applied moment</td>
<td>36.672 kNm</td>
</tr>
<tr>
<td>Moment resistance</td>
<td>193.83 kNm</td>
</tr>
<tr>
<td>Torsional moment</td>
<td>6.1965 kNm</td>
</tr>
<tr>
<td>Torsional resistance</td>
<td>79.319 kNm</td>
</tr>
<tr>
<td>Midspan deflection</td>
<td>5.3385 mm</td>
</tr>
<tr>
<td>Angle of twist</td>
<td>0.065687 deg</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.21318 ≤ 1</td>
</tr>
</tbody>
</table>

**SCALE 5.48 Office 1007 Proforma 465**
Sample output for SCALE Proforma 466. (ans=1)  
Steel design to BS5950-1:2000 and Eurocode 3  
Steel beam with openings in the web  
Copyright 1986-2019 Fitzroy Systems Ltd.  
Made by: IFB  
Date: 02/12/19  
Ref No: SC466 BS

Location: Ex1 - Example from CIRIA/SCI publication

Beam with rectangular web opening

Calculations in accordance with BS5950-1:2000 and SCI CIRIA publication entitled 'Design for openings in the web of composite beams'.

Section properties

457 x 152 x 60 UB.  
Young's Modulus E=205 kN/mm²

Opening details

Depth of opening Do=300 mm  
Length of opening l=500 mm  
Distance from top of comp. flange df=77.3 mm

Factored loading at web opening

Factored shear force V=37.4 kN  
Factored bending moment M=86.3 kNm  
Moment capacity Mc=py*Sx/10^3=457.95 kNm

Moment capacities

Vierendeel moment capacity Mv=0.9*(Mv1+Mv2)=31.998 kNm  
Ultimate vierendeel moment Mu=V*l/10^3=18.7 kNm

Stiffener details

Stiffener outstand bs=70 mm  
Stiffener thickness ts=10 mm  
Distance from opening edge to stiffener centre line d2=13 mm

Single stiffener to one side of beam web will be provided.
### Stiffener welding details

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fillet weld leg length</td>
<td>sw=6 mm</td>
</tr>
<tr>
<td>Overall length of stiffener</td>
<td>L=800 mm</td>
</tr>
</tbody>
</table>

#### UNIVERSAL BEAM
- **457 x 152 x 60 UB Grade S 355**
- Shear force: 37.4 kN

#### DESIGN
- Factored moment: 86.3 kNm

#### SUMMARY
- Vierendeel moment: 18.7 kNm
- Vierendeel mt. cap.: 71.338 kNm

#### SINGLE STIFFENER
- O/A length (2): 800 mm
- Width: 70 mm
- Thickness: 10 mm
- Distance from open.: 13 mm

#### STIFFENER WELDING
- Fillet weld size: 6 mm

#### DETAILS
- Electrode Grade: 35
Location: Ex1 - Stiffeners one side of web

Rectangular web opening


ho

Rectangular web opening

ho  Factored shear force VEd=37.4 kN
ho  Factored bending moment MyEd=86.3 kNm
ho  Moment resistance McRd=Wply*fy/(gamM0*10^3)=457.95 kNm
ho

Moment resistances

Vierendeel moment resistance MvRd=0.9*(Mv1+Mv2)=31.998 kNm
ho  Ultimate vierendeel moment MvEd=VED*lo/10^3=18.7 kNm

Stiffener details

ho  Stiffener thickness ts=10 mm
ho  Stiffener outstand bs=70 mm
ho  Distance from opening edge to stiffener centre line d2=13 mm
ho

Provide a single stiffener to one side of beam web as shown.
**Stiffener welding details**

- **Weld design strength**: \( fvwd = \frac{fu}{\sqrt{3} \times \beta_w \times \gamma_{M2}} \)
  \[ = 241.2 \text{ N/mm}^2 \]
- **Fillet weld leg length**: \( sw = 6 \text{ mm} \)
- **Resistance of weld per mm run**: \( FwR_d = 0.7 \times sw \times fvwd / 10^3 = 1.0131 \text{ kN/mm} \)
- **For fillet welds on both sides, take the projection of stiffener** beyond the opening to be \( lw = \frac{FaR_d}{2 \times FwR_d} = 122.65 \text{ mm} \)
- **Minimum length beyond opening**: \( lw = 20 \times ts = 200 \text{ mm} \)
- **Overall length of stiffener**: \( L = 900 \text{ mm} \)

**Shear buckling resistance - Clause 6.2.6(6)**

- **Clear web depth between flanges**: \( hw = h - 2 \times tf = 428 \text{ mm} \)
- **Buckling ratio**: \( hw'/tw = 52.84 \)
- Since \( hw/tw < 72e \ (58.58) \), no check for shear buckling is required.

**UNIVERSAL BEAM**

- **457 x 152 x 60 UKB Grade S 355**
  - **Shear force**: 37.4 kN
  - **Factored moment**: 86.3 kNm
  - **Vierendeel moment**: 18.7 kNm
  - **Vierendeel mt.resist.**: 71.338 kNm

**SUMMARY**

- **SINGLE STIFFENER**
  - **O/A length (2No)**: 900 mm
  - **Width**: 70 mm
  - **Thickness**: 10 mm
  - **Distance from open.**: 13 mm

**STIFFENER WELDING**

- **Fillet weld size**: 6 mm
Location: Ex2 - Stiffeners both sides of web

Rectangular web opening

- \( \text{ht} \): Calculations in accordance with BS EN 1993-1-1:2005, NCCI: Design rules for web openings in beams' reference "Access Steel" SN019b-EN-EU and SCI P355 entitled 'Design of Composite Beams With Large Web Openings'.
- \( \text{ho} \): Depth of opening \( \text{ho}=300 \text{ mm} \)
- \( \text{lo} \): Length of web opening \( \text{lo}=500 \text{ mm} \)
- \( \text{ht} \): Distance from top of comp.flange \( \text{ht}=78.5 \text{ mm} \)

Section properties

- \( 457 \times 191 \times 74 \text{ UKB} \)
- Young's Modulus \( E=210 \text{ kN/mm}^2 \)

Opening details

- Depth of opening \( \text{ho}=300 \text{ mm} \)
- Length of web opening \( \text{lo}=500 \text{ mm} \)
- Distance from top of comp.flange \( \text{ht}=78.5 \text{ mm} \)

Factored loading at web opening

- Factored shear force \( \text{VEd}=121 \text{ kN} \)
- Factored bending moment \( \text{MyEd}=310 \text{ kNm} \)
- Moment resistance \( \text{McRd}=\text{Wply*fy}/(\gamma\text{M0}*10^3)=585.75 \text{ kNm} \)

Moment resistances

- Vierendeel moment resistance \( \text{MvRd}=0.9*(\text{Mv1+Mv2})=25.819 \text{ kNm} \)
- Ultimate vierendeel moment \( \text{MvEd}=\text{VEd}*\text{lo}/10^3=60.5 \text{ kNm} \)

Stiffener details

- Stiffener thickness \( \text{ts}=10 \text{ mm} \)
- Stiffener outstand \( \text{bs}=80 \text{ mm} \)
- Distance from opening edge to stiffener centre line \( \text{d2}=13 \text{ mm} \)

Double stiffener will be provided as shown.
**Stiffener welding details**

Weld design strength  
\[ \text{fvwd} = \frac{\text{fu}}{\sqrt{3}} \left( \frac{1}{\text{betaw} \times \text{gamM2}} \right) \]  
\[ = 241.2 \text{ N/mm}^2 \]

Fillet weld leg length  
\[ \text{sw} = 6 \text{ mm} \]

Resistance of weld per mm run  
\[ \text{FwRd} = 0.7 \times \text{sw} \times \text{fvwd} / 10^3 = 1.0131 \text{ kN/mm} \]

For fillet welds on both sides, take the projection of stiffener beyond the opening to be  
\[ \text{lw} = \frac{\text{FaRd}}{4 \times \text{FwRd}} = 140.17 \text{ mm} \]

Minimum length beyond opening  
\[ \text{lw} = 20 \times \text{ts} = 200 \text{ mm} \]

Overall length of stiffener  
\[ \text{L} = 900 \text{ mm} \]

**Shear buckling resistance - Clause 6.2.6(6)**

Clear web depth between flanges  
\[ \text{hw} = \text{h} - 2 \times \text{tf} = 428 \text{ mm} \]

Buckling ratio  
\[ \text{hw'} = \frac{\text{hw}}{\text{tw}} = 47.556 \]

Since \( \text{hw/tw} < 72e \ (58.58) \), no check for shear buckling is required.

**UNIVERSAL BEAM**  
457 x 191 x 74 UKB Grade S 355

**DESIGN**  
Shear force 121 kN

**SUMMARY**  
Factored moment 310 kNm

Vierendeel moment 60.5 kNm

Vierendeel mt.resist. 66.218 kNm

**DOUBLE STIFFENER**  
O/A length (4No) 900 mm

Width 80 mm

Thickness 10 mm

Distance from open. 13 mm

**STIFFENER WELDING**  
Fillet weld size 6 mm
Location: Ex3 - Plate girder with stiffeners one side of web

Rectangular web opening

<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>ho</td>
<td></td>
</tr>
<tr>
<td>hb</td>
<td></td>
</tr>
</tbody>
</table>

Section properties

Depth of section                     h=460 mm
Width of section                     b=155 mm
Thickness of flange                  tf=15 mm
Thickness of web                     tw=15 mm
Young's Modulus                      E=210 kN/mm²

Opening details

Depth of opening                     ho=300 mm
Length of web opening                lo=500 mm
Distance from top of comp.flange     ht=80 mm

Factored loading at web opening

Factored shear force                 VEd=37.4 kN
Factored bending moment             MyEd=86.3 kNm
Moment resistance                   McRd=fyw*b*tf*(h-tf)/(gamM0*10^6) =367.29 kNm

Moment resistances

Vierendeel moment resistance        MvRd=0.9*(Mv1+Mv2)=30.339 kNm
Ultimate vierendeel moment          MvEd=VEd*lo/10^3=18.7 kNm

Stiffener details

Stiffener thickness                 ts=10 mm
Stiffener outstand                  bs=70 mm
Distance from opening edge          d2=13 mm

d2

Provide a single stiffener to one side of beam web as shown.
Stiffener welding details

Weld design strength  
fvwd = \( \frac{f_u}{\text{SQR}(3)/(\beta_{aw} \cdot \gamma_{M2})} \)  

= 241.2 \( \text{N/mm}^2 \)

Fillet weld leg length  
sw = 6 mm

Resistance of weld per mm run  
FwRd = 0.7 * sw * fvwd / 10^3 = 1.0131 \( \text{kN/mm} \)

For fillet welds on both sides, take the projection of stiffener beyond the opening to be  
\( lw = \frac{F_aRd}{(2 \cdot FwRd)} \) = 122.65 mm

Minimum length beyond opening  
\( lw = 20 \cdot ts = 200 \) mm

Overall length of stiffener  
L = 900 mm

Flange/web interface welds

Fillet weld leg length  
sww = 6 mm

Shear buckling resistance - Clause 6.2.6(6)

Clear web depth between flanges  
hw = h - 2 * tf = 430 mm

Buckling ratio  
\( hw' = \frac{hw}{tw} = 28.667 \)

Since hw/tw < 72e (58.58), no check for shear buckling is required.

| Shear force | 37.4 kN |
| Factored moment | 86.3 kNm |
| Vierendeel moment | 18.7 kNm |
| Vierendeel mt. resist. | 73.772 kNm |

DESIGN

SUMMARY

SINGLE STIFFENER

O/A length (2No)  
900 mm

Width  
70 mm

Thickness  
10 mm

Distance from open.  
13 mm

STIFFENER WELDING

Fillet weld size  
6 mm
Location: SCI Example

Simply supported steel beam

Calculations are in accordance with BS5950 and SCI publication SCI-P-100 entitled 'Design of Composite and Non-Composite Cellular Beams'.

Positions considered when determining shear, moment etc.

Beam span
L=10 m

457 x 191 x 67 UB.

Young's Modulus
E=205 kN/mm²

Diameter of openings
Do=400 mm

Spacing of openings
S=600 mm

w/metre
vvvvvvvvvvv

Uniformly distributed load

Unfactored dead load
Gku=7.14 kN/m

Unfactored imposed load
Qku=1.5 kN/m

Overall beam flexural capacity

Maximum moment
M=w*L²/8=154.95 kNm

Flexural capacity
Mp=677.57 kNm

Beam shear capacity

Shear at support
Vmax =61.98 kN

Shear capacity @ support
Pv=0.6*py*Avs/10³=1134.5 kN

Shear at opening centre line
Vmax =55.782 kN (Maximum)

Shear capacity of Tees
Pvy=0.6*py*Avt/10³=369.24 kN

Maximum horizontal shear
Vh=53.467 kN

Horizontal shear capacity
Pvh=0.6*py*Avwp/10³=325.89 kN

Compression flange is fully restrained.
Assumed angle for critical sect. \( \theta = 25^\circ \)

**Simplified deflection calculation**

<table>
<thead>
<tr>
<th>Total deflection</th>
<th>DEF = DEF1 + DEF2 = 12.182</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live load deflection</td>
<td>( 2.1149 \text{ mm} ) ( \leq L/360 = 27.778 \text{ mm} ), hence OK.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Original Section</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>UNIVERSAL BEAM</td>
<td>457 x 191 x 67 UB Grade S 355</td>
</tr>
<tr>
<td>CELLULAR</td>
<td>626.61 x 191 x 67</td>
</tr>
<tr>
<td>Opening size</td>
<td>400 mm</td>
</tr>
<tr>
<td>Opening spacing</td>
<td>600 mm</td>
</tr>
<tr>
<td>Shear force</td>
<td>61.98 kN</td>
</tr>
<tr>
<td>Shear resistance</td>
<td>1134.5 kN</td>
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<tr>
<td>DESIGN</td>
<td>Maximum moment</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Flexural capacity</td>
</tr>
<tr>
<td></td>
<td>154.95 kNm</td>
</tr>
<tr>
<td></td>
<td>677.57 kNm</td>
</tr>
</tbody>
</table>
Location: Ex1 - UDL loading and restrained compression flange

Simply supported steel beam

Calculations are in accordance with EC3 Part 1-1, SCI P355 entitled 'Design of Composite Beams With Large Web Openings' and SCI-P-100 (1990).
Openings are placed centrally in the web depth.

Positions 0, 2 and 3 are considered when determining shear, moment etc.

In the diagram position 0 is the beam support, position 1 is the end web post, positions 2 to 9 represent the centre line of web openings and position 10 is the centre of the beam under consideration.

Beam span L=10 m

Initial beam

457 x 191 x 67 UKB
Dimensions (mm): h=453.4 b=189.9 tw=8.5 tf=12.7 r=10.2
Properties (cm): Iy=29400 Iz=1450 Wply=1470 Wplz=237 It=37.1

Strength of steel

Design strength fy=355 N/mm²
Young's Modulus E=210 kN/mm²

Cellular beam details

Diameter of openings ho=400 mm
Spacing of web openings S=600 mm
Overall depth of beam h=h'+ho/2-e'=10=616.61 mm

Partial factors for actions

Factor for permanent actions gamG=1.35
Factor for variable actions gamQ=1.5

Properties of steel cellular beam at CL of opening

Second moment of area of Tee I=2*(Itee+Atee*z'^2)=540.88E6 mm⁴
Plastic modulus Wply=2*(Atee*z')=1.8592E6 mm³
Moment resistance (Cl.6.2.5(2)) Mrd=Wply*fy/(gamM0*10^6)=660.01 kNm
Loading details

<table>
<thead>
<tr>
<th>w/metre</th>
</tr>
</thead>
<tbody>
<tr>
<td>vvvvvv...</td>
</tr>
</tbody>
</table>

Loading diagram

Uniformly distributed load

Unfactored permanent load \( G_{ku} = 7.14 \text{ kN/m} \)
Unfactored variable load \( Q_{ku} = 1.5 \text{ kN/m} \)

The compression flange of the cellular beam is fully restrained.
Buckling resistance is \( M_{bRd} = M_{cRd} = 660.01 \text{ kNm} \)

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

The shear buckling resistance should be determined in accordance with BS EN 1993-1-5 Clause 5.2 and 5.3.
As the ratio \( h_w/t_w > 72e \) \( (69.554 > 58.58) \), allowance needs to be made for the influence of the web opening on the shear buckling resistance of the web. Assume transverse stiffeners at supports.
Modified slenderness \( \lambda_{mw} = h_w/(86.4*t_w*e) = 0.98943 \)
Shear resistance at support \( V_{plRd} = 1074.2 \text{ kN} \)
Reduced design shear resistance at support (web contribution only will be considered):
\[ V_{bwRd} = \chi_w f_y h_w / (3^{0.5} \gamma_{M1} 1000) = 864.01 \text{ kN} \]

Assumed angle for critical sect. \( \theta = 25^\circ \)

Simplified deflection calculation

Total deflection \( DEF = DEF_p + DEF_v = 12.381 \)

Original Section

UNIVERSAL BEAM
457 x 191 x 67 UKB Grade S 355
CELLULAR
616.61 x 191 x 67
Opening size 400 mm
Opening spacing 600 mm
Support shear force 59.445 kN
Reduced shear resistance 864.01 kN
Shear @ opening centre line 53.501 kN
Shear resistance 339.62 kN

DESIGN
Maximum moment 148.61 kNm

SUMMARY
Moment resistance 660.01 kNm
Buckling resistance 660.01 kNm

Assume non-rigid end posts to EN 1993-1-5, Figure 5.1 & Table 5.1
Location: Ex2 - With point loads on beam

Simply supported steel beam

Calculations are in accordance with EC3 Part 1-1, SCI P355 entitled 'Design of Composite Beams With Large Web Openings' and SCI-P-100 (1990). Openings are placed centrally in the web depth.

Positions 0, 2 and 3 are considered when determining shear, moment etc.

In the diagram position 0 is the beam support, position 1 is the end web post, positions 2 to 9 represent the centre line of web openings and position 10 is the centre of the beam under consideration.

Beam span \( L=10 \) m

Initial beam

457 x 191 x 67 UKB
Dimensions (mm): \( h=453.4 \) \( b=189.9 \) \( tw=8.5 \) \( tf=12.7 \) \( r=10.2 \)
Properties (cm): \( I_y=29400 \) \( I_z=1450 \) \( W_{ply}=1470 \) \( W_{plz}=237 \) \( I_t=37.1 \)

Strength of steel

Design strength \( f_y=355 \) N/mm\(^2\)
Young's Modulus \( E=210 \) kN/mm\(^2\)

Cellular beam details

Diameter of openings \( h_0=400 \) mm
Spacing of web openings \( S=600 \) mm
Overall depth of beam \( h=h'+h_0/2-e'-10=616.61 \) mm

Partial factors for actions

Factor for permanent actions \( \gamma_G=1.35 \)
Factor for variable actions \( \gamma_Q=1.5 \)

Properties of steel cellular beam at CL of opening

Second moment of area of Tee \( I=2*(I_{tee}+A_{tee}z'^2)=540.88E6 \) mm\(^4\)
Plastic modulus \( W_{ply}=2*(A_{tee}z')=1.8592E6 \) mm\(^3\)
Moment resistance (Cl.6.2.5(2)) \( Mc_{Rd}=W_{ply}*f_y/(\gamma_{M0}*10^6)=660.01 \) kNm
Loading details

Loads are symmetrical about the beam CL.

Number of point loads: \( N_c = 10 \)

Unfactored permanent load: \( G_{kc} = 7 \) kN

Unfactored variable load: \( Q_{kc} = 1.5 \) kN

The compression flange of the cellular beam is fully restrained.

Buckling resistance is: \( M_{BRd} = M_{CRd} = 660.01 \) kNm

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

The shear buckling resistance should be determined in accordance with BS EN 1993-1-5 Clause 5.2 and 5.3.

As the ratio \( h_w/t_w > 72 \) e.g. \( 69.554 > 58.58 \), allowance needs to be made for the influence of the web opening on the shear buckling resistance of the web. Assume transverse stiffeners at supports.

Modified slenderness: \( \lambda_{w} = h_w/(86.4*t_w*e) = 0.98943 \)

Shear resistance at support: \( V_{plRd} = 1074.2 \) kN

Reduced design shear resistance at support (web contribution only will be considered):

\[
V_{bwRd} = \chi_{w} * f_y * h_w * t_w / (3^{0.5} * \gamma_M1 * 1000) = 864.01 \text{ kN}
\]

Assumed angle for critical sect. \( \theta = 25^\circ \)

Simplified deflection calculation

Equivalent permanent UDL: \( G_{ku} = 7.14 \) kN/m

Equivalent variable UDL: \( Q_{ku} = 1.5 \) kN/m

Total deflection: \( \text{DEF} = \text{DEFp} + \text{DEFv} = 12.381 \)

Original Section

UNIVERSAL BEAM: 457 x 191 x 67 UKB Grade S 355
CELLULAR: 616.61 x 191 x 67

Opening size: 400 mm
Opening spacing: 600 mm
Support shear force: 58.5 kN
Reduced shear resistance: 864.01 kN
Shear @ opening centre line: 58.5 kN
Shear resistance: 339.62 kN

DESIGN:
Maximum moment: 159.55 kNm

SUMMARY:
Moment resistance: 660.01 kNm
Buckling resistance: 660.01 kNm

Assume non-rigid end posts to EN 1993-1-5, Figure 5.1 & Table 5.1
Location: Ex3 - UDL loading and unrestrained compression flange

Simply supported steel beam

Calculations are in accordance with EC3 Part 1-1, SCI P355 entitled 'Design of Composite Beams With Large Web Openings' and SCI-P-100 (1990).

Openings are placed centrally in the web depth.

Positions 0, 2 and 3 are considered when determining shear, moment etc.

In the diagram position 0 is the beam support, position 1 is the end web post, positions 2 to 9 represent the centre line of web openings and position 10 is the centre of the beam under consideration.

Beam span \( L = 9 \text{ m} \)

Initial beam

457 x 191 x 67 UKB

Dimensions (mm): \( h = 453.4 \) \( b = 189.9 \) \( tw = 8.5 \) \( tf = 12.7 \) \( r = 10.2 \)

Properties (cm): \( I_y = 29400 \) \( I_z = 1450 \) \( W_{ply} = 1470 \) \( W_{plz} = 237 \) \( I_t = 37.1 \)

Strength of steel

Design strength \( f_y = 355 \text{ N/mm}^2 \)

Young's Modulus \( E = 210 \text{ kN/mm}^2 \)

Cellular beam details

Diameter of openings \( h_o = 400 \text{ mm} \)

Spacing of web openings \( S = 600 \text{ mm} \)

Overall depth of beam \( h = h' + h_o/2 - e' - 10 = 616.61 \text{ mm} \)

Partial factors for actions

Factor for permanent actions \( \gamma_G = 1.25 \)

Factor for variable actions \( \gamma_Q = 1.5 \)

Properties of steel cellular beam at CL of opening

Second moment of area of Tee \( I = 2 \times (I_{tee} + A_{tee} \times z' \times 2) = 540.88 \times 10^6 \text{ mm}^4 \)

Plastic modulus \( W_{ply} = 2 \times (A_{tee} \times z') = 1.8592 \times 10^6 \text{ mm}^3 \)

Moment resistance (Cl.6.2.5(2)) \( M_{Rd} = W_{ply} \times f_y / (\gamma_M \times 10^6) = 660.01 \text{ kNm} \)
Loading details

w/metre

Unfactored permanent load \( G_{ku} = 5 \text{ kN/m} \)
Unfactored variable load \( Q_{ku} = 1.5 \text{ kN/m} \)
Eff. length between restraints \( L_T = 9 \text{ m} \)

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

The shear buckling resistance should be determined in accordance with BS EN 1993-1-5 Clause 5.2 and 5.3.
As the ratio \( h_w/t_w > 72e \) (69.554 > 58.58), allowance needs to be made for the influence of the web opening on the shear buckling resistance of the web. Assume transverse stiffeners at supports.
Modified slenderness \( \lambda_w = h_w/(86.4*t_w*e) = 0.98943 \)
Shear resistance at support \( V_{plRd} = 1074.2 \text{ kN} \)
Reduced design shear resistance at support (web contribution only will be considered):
\[
V_{bwRd} = \chi_w*f_y*h_w*t_w/(\sqrt{3}\cdot\gamma_{M1}*1000) = 864.01 \text{ kN}
\]
Assumed angle for critical sect. \( \theta = 25^\circ \)

Simplified deflection calculation

Total deflection \( \text{DEF} = \text{DEFp} + \text{DEFv} = 6.111 \)

Original Section

UNIVERSAL BEAM 457 x 191 x 67 UKB Grade S 355
CELLULAR 616.61 x 191 x 67

<table>
<thead>
<tr>
<th>Category</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Opening size</td>
<td>400 mm</td>
</tr>
<tr>
<td>Opening spacing</td>
<td>600 mm</td>
</tr>
<tr>
<td>Support shear force</td>
<td>38.25 kN</td>
</tr>
<tr>
<td>Reduced shear resistance</td>
<td>864.01 kN</td>
</tr>
<tr>
<td>Shear @ opening centre line</td>
<td>33.15 kN</td>
</tr>
<tr>
<td>Shear resistance</td>
<td>339.62 kN</td>
</tr>
</tbody>
</table>

Assume non-rigid end posts to EN 1993-1-5, Figure 5.1 & Table 5.1
Location: Ex1 - Equal angle simplified method

Length of beam \( L = 4 \text{ m} \)
200 x 200 x 16 mm Equal Angle.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Dead load factor \( \gamma_{md} = 1.4 \)
Imposed load factor \( \gamma_{mi} = 1.6 \)
Dist. from left support to start \( L_{au}(1) = 0 \text{ m} \)
Distance from left support to end \( L_{bu}(1) = 4 \text{ m} \)
Dead load (unfactored) \( G_{ku}(1) = 4 \text{ kN/m} \)
Imposed load (unfactored) \( Q_{ku}(1) = 3 \text{ kN/m} \)
Maximum span bending moment \( 20.8 \text{ kNm} \)
Design shear force \( F_v = 20.8 \text{ kN} \)
Length between restraint points \( L_v = 4 \text{ m} \)

EQUAL ANGLE

200 x 200 x 16 Grade S 275
Heel of angle in compression

DESIGN
Max. shear force 20.8 kN
Shear capacity 475.2 kN
Max. applied moment 20.8 kNm
Moment capacity 44.441 kNm
Buckling resistance 35.552 kNm
Moment factor (mLT) 0.925
Resistance (Mb/mLT) 38.435 kNm

SUMMARY
Unfactored DL defln 2.7795 mm
Unfactored LL defln 2.0846 mm
Limiting deflection 11.111 mm

Unfactored DL shear at LHE 8 kN
Unfactored LL shear at LHE 6 kN
DL shear at RHE 8 kN
LL shear at RHE 6 kN
Location: Ex2 - Equal angle (same example - basic method)

Length of beam $L=4 \text{ m}$
200 x 200 x 16 mm Equal Angle.
Young's Modulus $E=205 \text{ kN/mm}^2$
Dead load factor $\gamma_d=1.4$
Imposed load factor $\gamma_i=1.6$

Distance from left support to start $L_{au}(1)=0 \text{ m}$
Distance from left support to end $L_{bu}(1)=4 \text{ m}$
Dead load (unfactored) $G_{ku}(1)=4 \text{ kN/m}$
Imposed load (unfactored) $Q_{ku}(1)=3 \text{ kN/m}$
Maximum span bending moment $20.8 \text{ kNm}$
Design shear force $F_v=20.8 \text{ kN}$
Length between restraint points $L_v=4 \text{ m}$

**EQUAL ANGLE**
200 x 200 x 16 Grade S 275
Heel of angle in compression

**DESIGN**
Max. shear force $20.8 \text{ kN}$
Shear capacity $475.2 \text{ kN}$
Buckling resistance $65.633 \text{ kNm}$
Moment factor (mLT) $0.925$
Resistance (Mb/mLT) $70.955 \text{ kNm}$
Annex I.3 combined stresses < $p_y$ $275 \text{ N/mm}^2$
Compression $166.74 \text{ N/mm}^2$
Buckling $170.62 \text{ N/mm}^2$
Unfactored DL defln $2.7795 \text{ mm}$
Unfactored LL defln $2.0846 \text{ mm}$
Limiting deflection $11.111 \text{ mm}$

**SUMMARY**
Unfactored

<table>
<thead>
<tr>
<th>Unfactored end shears</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL shear at LHE</td>
</tr>
<tr>
<td>LL shear at LHE</td>
</tr>
<tr>
<td>DL shear at RHE</td>
</tr>
<tr>
<td>LL shear at RHE</td>
</tr>
</tbody>
</table>

Unfactored shear at LHE $8 \text{ kN}$
LL shear at LHE $6 \text{ kN}$
DL shear at RHE $8 \text{ kN}$
LL shear at RHE $6 \text{ kN}$
Location: Ex3 - Equal angle (slender section)

![Diagram of equal angle beam]

Heel of the equal angle is in compression.

Length of beam: L = 3 m

120 x 120 x 8 mm Equal Angle.

Young's Modulus: E = 205 kN/mm²

Dead load factor: \( \gamma_d = 1.4 \)

Imposed load factor: \( \gamma_i = 1.6 \)

Dist. from left support to start: \( L_u(1) = 0 \) m

Distance from left support to end: \( L_u(1) = 3 \) m

Dead load (unfactored): \( G_k(1) = 2 \) kN/m

Imposed load (unfactored): \( Q_k(1) = 1 \) kN/m

Maximum span bending moment: 4.95 kNm

Design shear force: \( F_v = 6.6 \) kN

Length between restraint points: \( L_v = 3 \) m

**EQUAL ANGLE**

120 x 120 x 8 Grade S 355

Heel of angle in compression

**DESIGN**

Max. shear force: 6.6 kN

Shear capacity: 184.03 kN

Buckling resistance: 11.626 kNm

Moment factor (mLT): 0.925

Resistance (Mb/mLT): 12.569 kNm

Annex I.3 combined stresses < \( \gamma_y \):

- Compression: 239.78 N/mm²
- Buckling: 260.64 N/mm²

Unfactored DL defln: 3.9728 mm

Unfactored LL defln: 1.9864 mm

Limiting deflection: 8.3333 mm

Unfactored DL shear at LHE: 3 kN

Unfactored LL shear at LHE: 1.5 kN

Unfactored DL shear at RHE: 3 kN

Unfactored LL shear at RHE: 1.5 kN
Location: Ex4 - Unequal angle short leg in compression

![Diagram of unequal angle beam with X and T labels]

The short leg of the unequal angle is in compression.

Length of beam: \( L = 4 \text{ m} \)
200 x 150 x 18 mm Unequal Angle.
Young's Modulus: \( E = 205 \text{ kN/mm}^2 \)
Dead load factor: \( \gamma_d = 1.4 \)
Imposed load factor: \( \gamma_i = 1.6 \)

Dist. from left support to start: \( L_{au} = 0 \text{ m} \)
Distance from left support to end: \( L_{bu} = 4 \text{ m} \)
Dead load (unfactored): \( G_{ku} = 3 \text{ kN/m} \)
Imposed load (unfactored): \( Q_{ku} = 2.5 \text{ kN/m} \)
Maximum span bending moment: \( 16.4 \text{ kNm} \)
Design shear force: \( F_v = 16.4 \text{ kN} \)
Length between restraint points: \( L_v = 4 \text{ m} \)

**UNEQUAL ANGLE** 200 x 150 x 18 Grade S 275

**DESIGN**
- Max. shear force: 16.4 kN
- Shear capacity: 515.16 kN
- Buckling resistance: 53.564 kNm
- Moment factor (mLT): 0.925
- Resistance (Mb/mLT): 57.907 kNm
- Annex I.3 combined stresses < \( \gamma_y \): 265 N/mm²
- Compression: 142.13 N/mm²
- Buckling: 143.23 N/mm²
- Tension: 108.53 N/mm²
- Unfactored DL defln: 2.041 mm
- Unfactored LL defln: 1.7009 mm
- Limiting deflection: 11.111 mm

**SUMMARY**
- Shear capacity: 515.16 kN
- Buckling resistance: 53.564 kNm
- Moment factor (mLT): 0.925
- Resistance (Mb/mLT): 57.907 kNm
- Annex I.3 combined stresses < \( \gamma_y \): 265 N/mm²
- Compression: 142.13 N/mm²
- Buckling: 143.23 N/mm²
- Tension: 108.53 N/mm²
- Unfactored DL defln: 2.041 mm
- Unfactored LL defln: 1.7009 mm
- Limiting deflection: 11.111 mm

Unfactored end shears:
- DL shear at LHE: 6 kN
- LL shear at LHE: 5 kN
- DL shear at RHE: 6 kN
- LL shear at RHE: 5 kN
Location: Ex5 - Unequal angle long leg in compression

The long leg of the unequal angle is in compression.

Length of beam \( L = 4 \text{ m} \)
200 x 150 x 18 mm Unequal Angle.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)
Dead load factor \( \gamma_d = 1.4 \)
Imposed load factor \( \gamma_i = 1.6 \)

Distance from left support to start \( L_{au(1)} = 0 \text{ m} \)
Distance from left support to end \( L_{bu(1)} = 4 \text{ m} \)
Dead load (unfactored) \( G_{ku(1)} = 3 \text{ kN/m} \)
Imposed load (unfactored) \( Q_{ku(1)} = 2.5 \text{ kN/m} \)
Maximum span bending moment \( 16.4 \text{ kNm} \)
Design shear force \( F_v = 16.4 \text{ kN} \)
Length between restraint points \( L_v = 4 \text{ m} \)

**UNEQUAL ANGLE**

200 x 150 x 18 Grade S 275
Long leg in compression

**DESIGN**
Max. shear force 16.4 kN
Shear capacity 515.16 kN
Buckling resistance 50.103 kNm
Moment factor (mLT) 0.925
Resistance (Mb/mLT) 54.166 kNm

**SUMMARY**
Annex I.3 combined stresses < \( \gamma_y \) 265 N/mm²
Compression 142.13 N/mm²
Buckling 147.77 N/mm²
Tension 138.81 N/mm²
Unfactored DL defln 2.041 mm
Unfactored LL defln 1.7009 mm
Limiting deflection 11.111 mm

Unfactored end shears
DL shear at LHE 6 kN
LL shear at LHE 5 kN
DL shear at RHE 6 kN
LL shear at RHE 5 kN
Location: Ex1 - Equal angle with unrestrained flange

Length of beam $L=4$ m
200 x 200 x 20 mm Equal Angle (Advance UKA)
Permanent UDL (including S.W) $w_d=4$ kN/m
Variable UDL $w_i=3$ kN/m
Length between restraint points $L_v=4$ m

**EQUAL ANGLE DESIGN SUMMARY**

200 x 200 x 20 Grade S 275
Heel of angle in compression
Design shear force 19.8 kN
Design shear resist. 550.79 kN
Design moment 19.8 kNm
Moment resistance 52.741 kNm
Buckling resistance 50.582 kNm
Unfactored deflection 1.6708 mm
Limiting deflection 11.111 mm
Interaction factor $0.53888 \leq 1$
Location: Ex2 - Unequal angle unrestrained, short leg in compression

Unequal angle short leg is in compression.

y-y = major axis

Length of beam \( L = 4 \text{ m} \)
200 x 150 x 18 mm Unequal Angle (Advance UKA)
Permanent UDL (including S.W) \( wd = 3 \text{ kN/m} \)
Variable UDL \( wi = 2.5 \text{ kN/m} \)
Length between restraint points \( Lv = 4 \text{ m} \)

UNEQUAL ANGLE
DESIGN SUMMARY

200 x 150 x 18 Grade S 275
Short leg in compression
Design shear force 15.6 kN
Design shear resist. 495.71 kN
Design moment 15.6 kNm
Moment resistance 46.138 kNm
Buckling resistance 44.495 kNm
Unfactored deflection 1.6673 mm
Limiting deflection 11.111 mm
Interaction factor 0.55161 ≤ 1
Location: Ex3 - Unequal angle unrestrained, long leg in compression

Unequal angle long leg is in compression.

\( y-y = \text{major axis} \)

Length of beam \( L = 4 \text{ m} \)
200 x 150 x 18 mm Unequal Angle (Advance UKA)
Permanent UDL (including S.W) \( w_d = 3 \text{ kN/m} \)
Variable UDL \( w_i = 2.5 \text{ kN/m} \)
Length between restraint points \( L_v = 4 \text{ m} \)

**UNEQUAL ANGLE DESIGN SUMMARY**

- 200 x 150 x 18 Grade S 275
- Long leg in compression
- Design shear force 15.6 \text{kN}
- Design shear resist. 495.71 \text{kN}
- Design moment 15.6 \text{kNm}
- Moment resistance 46.138 \text{kNm}
- Buckling resistance 41.36 \text{kNm}
- Unfactored deflection 1.6673 \text{mm}
- Limiting deflection 11.111 \text{mm}
- Interaction factor 0.57214 \( \leq 1 \)
Location: Ex3 - As Ex3 but restrained

Unequal angle long leg is in compression.

y - y = major axis

Length of beam: L = 4 m
200 x 150 x 18 mm Unequal Angle (Advance UKA)
Permanent UDL (including S.W): wd = 3 kN/m
Variable UDL: wi = 2.5 kN/m

UNEQUAL ANGLE
DESIGN SUMMARY

200 x 150 x 18 Grade S 275
Long leg in compression
Design shear force: 15.6 kN
Design shear resist.: 495.71 kN
Design moment: 15.6 kNm
Moment resistance: 46.138 kNm
Buckling resistance: 46.138 kNm
Unfactored deflection: 1.6673 mm
Limiting deflection: 11.111 mm
Location: Ex1 - Steelwork Design Guide twin end post (Ex6)

Stiffened plate girder

The girder is fully restrained along its length. The design method follows the procedure of section 4.4 & 4.5 of BS5950 Part 1: 2000.

Loads and moments are factored.

Span of girder \( \text{span} = 25 \text{ m} \)
Maximum factored bending moment \( \text{Mmax} = 14028 \text{ kNm} \)
Young's Modulus \( E = 210000 \text{ kN/mm}^2 \)
Depth of girder \( D = 2030 \text{ mm} \)
Flange thickness to be tried \( T = 40 \text{ mm} \)
Flange width to be tried \( B = 700 \text{ mm} \)
Web thickness to be tried \( t = 13 \text{ mm} \)
Stiffener corner snipe \( \text{snp} = 15 \text{ mm} \)

End stiffener spacing for end panel \( \text{ab} = 2500 \text{ mm} \)
Intermediate stiffener spacing \( \text{ai} = 3500 \text{ mm} \)
Accompanying shear force \( Fv_{\text{m}} = 0 \text{ kN} \)
End shear force \( Fv_{\text{e}} = 1910 \text{ kN} \)
Distance between end stiffeners \( \text{ae} = 400 \text{ mm} \)
Assumed weld size \( ss' = 6 \text{ mm} \)
ADOPT 2/250 mm x 20 mm plate stiffeners Grade S 275 connected by 4/6 mm fillet welds.

Panel length \( \text{ain}_1 = 2500 \text{ mm} \)
Thickness of stiffener \( \text{ti}_1 = 13 \text{ mm} \)
Shear force at stiffener \( V_{n1} = 1680 \text{ kN} \)
Outstand to be used in design \( \text{bi} = 100 \text{ mm} \)
Weld size selected \( \text{sw} = 6 \text{ mm} \)

Panel length \( \text{ain}_2 = 3500 \text{ mm} \)
Thickness of stiffener \( \text{ti}_2 = 13 \text{ mm} \)
Shear force at stiffener \( V_{n2} = 1082 \text{ kN} \)
External load on stiffener \( Fx_2 = 760 \text{ kN} \)
Outstand to be used in design \( \text{bi} = 100 \text{ mm} \)
Weld size selected \( \text{sw} = 6 \text{ mm} \)
UDL applied between stiffeners \( w' = 92 \text{ kN/m} \)
Largest panel length \( a' = 3000 \text{ mm} \)
Selected fillet weld size \( s = 8 \text{ mm} \)

**DESIGN SUMMARY**

Span of plate girder \( L = 25 \text{ m} \)
All steel grade to be \( S 275 \text{ steel} \)

End stiffeners:

<table>
<thead>
<tr>
<th>( ae )</th>
<th>( ab )</th>
<th>( ai )</th>
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</table>

Double stiffener required at end of girder:

- \( ae = 400 \text{ mm} \)
- \( ab = 2500 \text{ mm} \)

Double stiffener: end post 1 2No/250 mm x 20 mm flats
end post 2 2No/200 mm x 16 mm flats
Spacing \( ai = 3000 \text{ mm} \)

Intermediate stiffeners:

- Arrangement (1) 2No/100 mm x 13 mm flats
- Arrangement (2) 2No/100 mm x 13 mm flats

- Fillet welds to be used for connecting stiffeners to girder web & flanges
- Weld sizes as detailed in calculations
- Weld design strength to be 35 N/mm²
- Electrode Grade 35
Location: Ex2 - Same example with anchor panel

Stiffened plate girder

The girder is fully restrained along its length. The design method follows the procedure of section 4.4 & 4.5 of BS5950 Part 1: 2000.

Loads and moments are factored.

Span of girder \( \text{span}=25 \text{ m} \)
Maximum factored bending moment \( M_{\text{max}}=14028 \text{ kNm} \)
Young's Modulus \( E=210000 \text{ kN/mm}^2 \)
Depth of girder \( D=2030 \text{ mm} \)
Flange thickness to be tried \( T=40 \text{ mm} \)
Flange width to be tried \( B=700 \text{ mm} \)
Web thickness to be tried \( t=13 \text{ mm} \)
Stiffener corner snipe \( \text{snp}=15 \text{ mm} \)

End stiffener spc.for end panel \( \text{ab}=2500 \text{ mm} \)
Intermediate stiffener spacing \( \text{ai}=3500 \text{ mm} \)
Accompanying shear force \( F_{\text{vm}}=0 \text{ kN} \)
End shear force \( F_{\text{ve}}=1910 \text{ kN} \)
Distance between end stiffeners \( \text{ae}=400 \text{ mm} \)
Assumed weld size \( \text{ss}'=6 \text{ mm} \)
ADOPT 2/250 mm x 20 mm plate stiffeners Grade S 275 connected by 4/6 mm fillet welds.

Panel length \( \text{ain}=3000 \text{ mm} \)
Thickness of stiffener \( \text{ti}=13 \text{ mm} \)
Shear force at stiffener \( V_{\text{n}}=1680 \text{ kN} \)
Outstand to be used in design \( b_i=100 \text{ mm} \)
Weld size selected \( s_w=6 \text{ mm} \)

UDL applied between stiffeners \( w'=92 \text{ kN/m} \)
Largest panel length \( a'=3000 \text{ mm} \)
Selected fillet weld size \( s=8 \text{ mm} \)
DESIGN SUMMARY

Span of plate girder          L=25 m
All steel grade to be         S 275 steel

End stiffeners:

Anchor panel at each end of girder.

\[
\begin{array}{ccc}
\text{Anchor panel} & \text{End panel} \\
1 & 2 & \text{ae=400 mm} \\
\text{ab=2500 mm} & & \\
\end{array}
\]

Anchor panel: stiffener 1 2No/250 mm x 20 mm flats
stiffener 2 2No/200 mm x 16 mm flats
Spacing ai=3000 mm

Intermediate stiffeners:
Arrangement (1) 2No/100 mm x 13 mm flats

\[
\begin{array}{c}
\text{B=700 mm} \\
\text{T=40 mm} \\
\text{D=2030 mm} \\
\text{t=13 mm} \\
\text{T=40 mm} \\
\end{array}
\]

- Fillet welds to be used for connecting stiffeners to girder web & flanges
- Weld sizes as detailed in calculations
- Weld design strength to be 35 N/mm²
- Electrode Grade 35
Location: Ex3 - Same example with single stiffener end post

Stiffened plate girder

The girder is fully restrained along its length. The design method follows the procedure of section 4.4 & 4.5 of BS5950 Part 1: 2000.

Loads and moments are factored.

Span of girder span=25 m
Maximum factored bending moment Mmax=14028 kNm
Young's Modulus E=210000 kN/mm²
Depth of girder D=2030 mm
Flange thickness to be tried T=40 mm
Flange width to be tried B=700 mm
Web thickness to be tried t=13 mm
Stiffener corner snipe snp=15 mm

End stiffener spac.for end panel ab=2500 mm
Intermediate stiffener spacing ai=3500 mm
Accompanying shear force Fvm=0 kN
End shear force Fve=1910 kN
Assumed weld size ss'=6 mm
ADOPT 2/208 mm x 16 mm plate stiffeners Grade S 275 connected by 4/6 mm fillet welds.
Panel length ain1=3000 mm
Thickness of stiffener ti1=13 mm
Shear force at stiffener Vn1=1680 kN
Outstand to be used in design bi=100 mm
Weld size selected sw=6 mm

UDL applied between stiffeners w'=92 kN/m
Largest panel length a'=3000 mm
Selected fillet weld size s=8 mm
DESIGN SUMMARY

Span of plate girder          L=25 m
All steel grade to be         S 275 steel

End stiffener:  

Single stiffener required at end of girder.  

End panel  

ab=2500 mm

Single stiffener: end post 1  2No/208 mm x 16 mm flats
Spacing ai=3000 mm

Intermediate stiffeners:  
Arrangement (1)  2No/100 mm x 13 mm flats

B=700 mm

- Fillet welds to be used for connecting stiffeners to girder web & flanges
- Weld sizes as detailed in calculations
- Weld design strength to be 35 N/mm²
- Electrode Grade 35
Location: Ex1 - Rigid end posts and intermediate stiffeners

Stiffened plate girder - with rigid end post

The girder is fully restrained along its length. The design calculations are in accordance with BS EN 1993-1-1 and BS EN 1993-1-5. Loads and moments are factored.

Assume rigid end posts (two double-sided stiffeners at supports).

Span of girder \( L = 25 \text{ m} \)

Maximum design bending moment \( M_{yEd} = 14028 \text{ kNm} \)

Design bearing reaction \( N_{bEd} = 3500 \text{ kN} \)

Depth of girder \( h = 2030 \text{ mm} \)

Web thickness \( t_w = 18 \text{ mm} \)

Stiffener corner snipe \( s_{np} = 15 \text{ mm} \)

Dimensions of web and flanges

Stiffener spacing of first panel \( a_e = 3000 \text{ mm} \)

Intermediate stiffener spacing \( a_i = 3000 \text{ mm} \)

Accompanying design shear \( V_{zEd}' = 0 \text{ kN} \)

Bearing stiffeners double sided - BS EN 1993-1-5 section 9

Bearing stiffeners are assumed to be symmetric about the beam web and will be designed as a strut to BS EN 1993-1-5 Clause 9.4(2), resisting the bearing reaction together with any eccentricities.

compute \( k_t \) from Annex A

\[
k_t = 5.34 + 4 \left( \frac{h_w}{a} \right)^2 = 7.03
\]

Design shear resistance

\[
V_{bRd} = \frac{\chi w \cdot f_y \cdot h_w \cdot t_w}{(\sqrt{3}) \cdot \gamma_{M1} \cdot 1000} = 3955.2 \text{ kN}
\]

Design shear at the support \( V_{Ed(0)} = N_{bEd} = 3500 \text{ kN} \)

As \( V_{Ed(0)} \leq V_{bRd} \) (3500 kN \( \leq 3955.2 \text{ kN} \)), the design shear at the support is less than the design shear resistance. Hence OK.

Action as a bearing stiffener

Two double-sided stiffeners at support

\[
\text{ PLAN ON BEARING STIFFENERS}
\]

NOTE: Stiffeners to be equally spaced about the centreline of bearing.
Depth of bearing stiffener  
Thickness of bearing stiffener  
Distance  
Distance between stiffener CL's  
Eccentricity in each direction  
Worst stress in web plate: 
\[
\text{swp} = \frac{N_b E_d * 1E3}{A} + \frac{M_z E_d b * 1E6 * (15 * e_w * t_w + t_s b/2 + c_2/2)}{I_{zz}}
\]
= 115.88 N/mm²

Worst stress in stiffener: 
\[
\text{comp1} = \frac{N_b E_d * 1E3}{A} = 98.796 \text{ N/mm}²
\]
\[
\text{comp2} = \frac{M_z E_d b * 1E6 * (c_2/2 + t_s b/2)}{I_{zz}} = 6.6595 \text{ N/mm}²
\]
\[
\text{comp3} = \frac{M_y E_d b * 1E6 * (h_s b + t_w/2)}{I_{yy}} = 15.653 \text{ N/mm}²
\]
Stiffener stress  
\[
s_{st} = \text{comp1} + \text{comp2} + \text{comp3} = 121.11 \text{ N/mm}²
\]

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Ratio (h_s b/t_s b)  
\[p = \frac{h_s b}{t_s b} = 10\]
Limit to ratio  \[p = \frac{h_s b}{t_s b}\]  is 13r=12.242. 
Ratio (h_s b/t_s b)=10 is not greater than limit 13r=12.242. 
Stiffener complies with Clause 9.2.1(8).

Action as a rigid end post

A rigid end post comprise of two double-sided transverse stiffeners that form the flanges of a short beam of length hw. The effective section below will be taken to exclude the outer parts of web plate.

Minimum area  
\[A_{min} = 4 * h_w * t_w^2 / c_2 = 8424 \text{ mm}²\]
Actual area provided  
\[A_{sp} = 2 * h_s b * t_s b = 12500 \text{ mm}²\]
As \[A_{sp} \geq A_{min}\]  (12500 mm² ≥ 8424 mm²), stiffener area is adequate.
In-plane moment  
\[M_{zEdm} = N_h * h_w / (8 * 1E3) = 87.315 \text{ kN}m\]
End post 2nd moment of area  
\[I_{zze} = 2 * (A_{sp} * (c_2/2)^2) = 562.56E6 \text{ mm}^4\]
Worst stiffener stress from membrane action will be calculated below. 
Stress due to membrane action  
\[s_{stm} = \frac{M_z E_d b * 1E6}{I_{zze} / (c_2/2 + t_s b/2)} = 25.224 \text{ N/mm}²\]

Cross-section resistance check

The maximum stresses from membrane action and bearing reaction will be added together here for simplicity, even though they occur at different heights on the stiffener. 
Total stress  
\[s_{stT} = s_{st} + s_{stm} = 146.33 \text{ N/mm}²\]
As \[s_{stT} \leq f_y\]  (146.33 N/mm² ≤ 265 mm² ), stiffener is OK.
Buckling check under combined bending and axial load

The stiffener will be checked for buckling under combined bending and axial load in accordance with BS EN 1993-1-1, Clause 6.3.3.

Critical load
\[ N_{cry} = \pi^2 E I_{yy} / (L_{cry}^2 \times 1000) \]
\[ = 315666 \text{ kN} \]

Slenderness of the stiffener
\[ \lambda = (A \times f_y / (N_{cry} \times 1E3))^{0.5} = 0.17245 \]

Design axial load resistance
\[ N_{plRd} = A \times f_y / (\gamma_M 1 \times 1000) = 9388.1 \text{ kN} \]

Design moment resistance yy axis
\[ M_{yRd} = W_{ely} \times f_y / (\gamma_M 1 \times 10^6) = 592.55 \text{ kNm} \]

Design moment resistance zz axis
\[ M_{zRdb} = W_{elzb} \times f_y / (\gamma_M 1 \times 10^6) = 1392.8 \text{ kNm (bearing)} \]
\[ M_{zRdm} = W_{elzm} \times f_y / (\gamma_M 1 \times 10^6) = 917.31 \text{ kNm (membrane)} \]

Interaction equation:
\[ \text{Uni} = N_{Ed} / (\chi_{yy} \times N_{plRd}) + \text{ratio} \times (M_{yEd} / M_{yRd}) + (M_{zEdb} / M_{zRdb} + M_{zEdm} / M_{zRdm}) \]
\[ = 0.55286 \]

As \( \text{Uni} \leq 1.0 \) ( 0.55286 \leq 1.0 ), the interaction equation is OK.

Connection of stiffeners to girder web

Net stiffener length
\[ L_{sn} = h_w - 2 \times s_{np} = 1920 \text{ mm} \]

Assumed weld size
\[ s_s' = 6 \text{ mm} \]

Effective length of weld
\[ L_w = 8 \times (L_{sn} - 2 \times s_s') = 15264 \text{ mm} \]

Design load on weld
\[ F_{wEd} = N_{bEd} \times 10^3 / L_w = 229.3 \text{ N/mm} \]

Design weld resistance
\[ F_{wRd} = 0.7 \times s_s' \times f_{vwd} = 935.71 \text{ N/mm} \]

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

As \( h_w / t_w > 72 \times e_w \) ( 108.33 > 67.802 ), the web shear buckling resistance needs to be checked (EN 1993-1-5 Clause 5.2 and 5.3).

Buckling coefficient (Annex A)
\[ k_t = 5.34 + 4 \times (h_w / a)^2 = 7.03 \]

Modified slenderness \( \lambda \)
\[ \lambda_{mw} = h_w / (37.4 \times t_w \times e_w \times SQR(k_t)) = 1.1601 \]

Shear resistance of web
\[ V_{plRd} = 5370.2 \text{ kN} \]

Reduced design shear resistance at support (web contribution only will be considered):
\[ V_{bwRd} = c_l \times f_{yw} \times h_w / t_w / (3 \times 0.5 \times \gamma_M 1 \times 1000) = 3955.2 \text{ kN} \]

Design shear at support
\[ V_{Ed}(0) = N_{bEd} = 3500 \text{ kN} \]

As \( V_{Ed}(0) \leq V_{bwRd} \) ( 3500 kN \leq 3955.2 kN ), the shear force is less than the shear resistance. Hence section OK.

compute
\[ \text{comp} = \eta_1 + (1 - M_{fRd} / M_{plRd}) \times (2 \times \eta_3 - 1)^2 = 0.89626 \]

As \( \text{comp} \leq 1 \) ( 0.89626 \leq 1 ), interaction equation is satisfied.
Intermediate transverse web stiffeners - Arrangement (1)

Panel length 1 \(a(1) = 3000\) mm  
Length of stiffener \(l_s = 1950\) mm  

Stress at top of web plate \((\sigma_{\text{top}})\) \(\sigma_{\text{top}} = 251.76\) N/mm²  
Stress at btm of web plate \((\sigma_{\text{btm}})\) \(\sigma_{\text{btm}} = -251.76\) N/mm²  

Design parameters  
Stiffener outstand \(h_{s1} = 100\) mm  
Stiffener thickness \(t_{s1} = 15\) mm  

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Yield strength to be adopted \(f_y = 265\) N/mm²  
Ratio \((h_{s1}/t_{s1})\) \(p = h_{s1}/t_{s1} = 6.6667\)  
Limit to ratio \(p = (h_{s1}/t_{s1})\) is 13\(\varepsilon = 12.017\).  
Ratio \((h_{s1}/t_{s1}) = 6.6667\) is not greater than limit 13\(\varepsilon = 12.017\).  
Stiffener complies with Clause 9.2.1(8).
Design shear resistance (double sided transverse stiffener)

The effective cross section of the intermediate double-sided transverse stiffener is as shown below:

\[
\begin{align*}
&\text{tsi} \\
&15.6\text{tw} \\
&15.6\text{tw} \\
&\downarrow \\
&\text{hsi} \quad \text{PLAN ON INTERMEDIATE} \\
&\text{tw} \quad \text{STIFFENER} \\
&\downarrow \\
&\text{hsi} \\
&\uparrow y \\
&z
\end{align*}
\]

2nd moment of area yy axis:
\[
I_{yy} = tsi \times (2 \times hsi + tw)^3/12 + (30 \times e_w \times tw) \times tw^3/12 = 13.197E6 \text{ mm}^4
\]

Design shear resistance
\[
V_{bRd} = \chi \times A_e \times f_{yw}/(\gamma M1 \times 1000) = 2761.5 \text{ kN}
\]

Axial force in the stiffener due to shear - BS EN 1993-1-5

Shear buckling coefficient for web panel \( k_t \) is derived from Clause 5.2.
Buckling coefficient (Annex A) \( k_t = 5.34 + 4 \times (hw/a)^2 = 7.03 \)
Modified slenderness \( \lambda_w \) \( \lambda_w = 0.76 \times (f_y/t_{cr})^{0.5} = 1.1603 \)
Max design shear force in panel \( V_{Ed1} = 1700 \text{ kN} \)
Externally applied design load \( P_{ext1} = 1200 \text{ kN} \)

Option 1:
Longitudinal compressive stress \( \sigma_{xEd} = 0 \) for symmetrical sections.
Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):
\[
P_{Ed1}' = V_{Ed1} - hw \times tw \times 0.8 \times t_{cr}/1000 = -1492.5 \text{ kN}
\]

Option 2:
Vertical force generated in the stiffener as per BS EN 1993-1-5
(see NOTE, Clause 9.3.3(3)):
\[
P_{Ed1} = V_{Ed1} - f_{yw} \times hw \times tw/(\sqrt{3} \times \gamma M1 \times \lambda_w^2 \times 1000) = -2288.8 \text{ kN}
\]
Design axial load in stiffener \( P_{Ed} = P_{Ed1} + P_{ext1} = 1200 \text{ kN} \)
As \( P_{Ed} \leq V_{bRd} \) (1200 \text{ kN} \leq 2761.5 \text{ kN}) , the design shear force does not exceed the shear resistance. Hence OK.

Connection to the web

Design shear load on weld \( F_{wEd} = tw^2/(5 \times hsi) + P_{Ed}/(2 \times (hw-2 \times s_n)) \)
\[
= 0.9605 \text{ kN/mm}
\]
Weld size provided \( s_w1 = 6 \text{ mm} \)

Stiffness check - Clause 9.3.3(3)

2nd moment of area (effe. section) \( I_{pro} = 13.197E6 \text{ mm}^4 \)
2nd moment of area required \( I_{req} = 0.75 \times hw \times tw^3 = 8.5293E6 \text{ mm}^4 \)
As \( I_{req} \leq I_{pro} (8.5293E6 \text{ mm}^4 \leq 13.197E6 \text{ mm}^4) \), the stiffener is OK.
Web check between stiffeners

\[ w' \text{ kN/m} \]

Girder assumed to have rigid end posts.

UDL applied between stiffeners \( w' = 92 \text{ kN/m (factored load)} \)

Largest panel length \( a' = 3000 \text{ mm} \)

Compressive stress of web \( \text{sed} = w'/tw = 5.1111 \text{ N/mm}^2 \)

Compressive strength of web \( \text{fed} = (2.75 + 2/(a'/hw)^2) \times E/(C/tw)^2 \)

\[ = 65.94 \text{ N/mm}^2 \]

Since \( \text{sed} \leq \text{fed} \) \( (5.1111 \text{ N/mm}^2 \leq 65.94 \text{ N/mm}^2) \), the web is OK.

Selected fillet weld size \( s = 12 \text{ mm} \)

Intermediate transverse web stiffeners - Arrangement (2)

Panel length 1 \( a(3) = 2800 \text{ mm} \)

Stiffener outstand \( h_2 = 200 \text{ mm} \)

Stiffener thickness \( t_2 = 20 \text{ mm} \)

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Yield strength to be adopted \( f_y = 265 \text{ N/mm}^2 \)

Ratio \( (h_2/t_2) \)

\[ p = h_2/t_2 = 10 \]

Limit to ratio \( p = (h_2/t_2) \) is \( 13 \pi = 12.242 \).

Ratio \( (h_2/t_2) = 10 \) is not greater than limit \( 13 \pi = 12.242 \).

Stiffener complies with Clause 9.2.1(8).

Design shear resistance (double sided transverse stiffener)

The effective cross section of the intermediate double-sided transverse stiffener is as shown below:

2nd moment of area \( yy \) axis:

\[ I_{yy} = t_{si} \times (2 \times h_{si} + tw)^3/12 + (30 \times ew \times tw) \times tw^3/12 \]

\[ = 121.97E6 \text{ mm}^4 \]

Design shear resistance

\[ V_{bRd} = \chi \times A_e \times f_yw / (\gamma M_1 \times 1000) = 4641 \text{ kN} \]
Axial force in the stiffener due to shear

Max design shear force in panel $V_{Ed2}=3200 \text{ kN}$
Externally applied design load $P_{ext2}=1200 \text{ kN}$
Design axial load in stiffener $P_{Ed}'=P_{Ed2}+P_{ext2}=1200 \text{ kN}$

As $P_{Ed}' \leq V_{bRd}$ ($1200 \text{ kN} \leq 4641 \text{ kN}$), the design shear force does not exceed the shear resistance. Hence OK.

Connection to the web

Design shear load on weld $F_{wEd}=\frac{tw^2}{5*hs2}+\frac{P_{Ed}'}{2*(hw-2*snp)}=0.6365 \text{ kN/mm}$
Weld size provided $sw2=6 \text{ mm}$

Stiffness check - Clause 9.3.3(3)

2nd moment of area (eff. section) $I_{pro}=121.97E6 \text{ mm}^4$
2nd moment of area required $I_{req}=0.75*hw*tw^3=8.5293E6 \text{ mm}^4$

As $I_{req} \leq I_{pro}$ ($8.5293E6 \text{ mm}^4 \leq 121.97E6 \text{ mm}^4$), the stiffener is OK.

DESIGN SUMMARY

Span of plate girder $25 \text{ m}$
Design moment $14028 \text{ kNm}$
All steel to be Grade S 275

End stiffeners: $c_2||ae||ai$ $c_2=300 \text{ mm}$ $ae=3000 \text{ mm}$ $ai=3000 \text{ mm}$

Double-sided stiffeners: post 1 2No/250 mm x 25 mm flats
post 2 2No/250 mm x 25 mm flats

Intermediate stiffeners:
Arrangement (1) Double-sided stiffeners
2No/100 mm x 15 mm flats spaced at 3000 mm centres

Intermediate stiffeners:
Arrangement (2) Double-sided stiffeners
2No/200 mm x 20 mm flats spaced at 2800 mm centres
b=700 mm

h=2030 mm

tw=18 mm

tf=40 mm

• Fillet welds to be used for connecting stiffeners to girder web & flanges
• Weld sizes as detailed in calculations
• Weld design strength to be 222 N/mm²

Assume rigid end posts to BS EN 1993-1-5, Figure 5.1 & Table 5.1.
Location: Ex2 - Rigid end posts only

Stiffened plate girder - with rigid end post

The girder is fully restrained along its length. The design calculations are in accordance with BS EN 1993-1-1 and BS EN 1993-1-5. Loads and moments are factored.

Assume rigid end posts (two double-sided stiffeners at supports).
Span of girder \( L = 12 \, \text{m} \)
Maximum design bending moment \( M_{yEd} = 1800 \, \text{kNm} \)
Design bearing reaction \( N_{bEd} = 1845 \, \text{kN} \)
Depth of girder \( h = 1200 \, \text{mm} \)
Web thickness \( t_w = 12.5 \, \text{mm} \)
Stiffener corner snipe \( snp = 15 \, \text{mm} \)

Dimensions of web and flanges

Accompanying design shear \( V_{zEd}' = 0 \, \text{kN} \)

Bearing stiffeners double sided - BS EN 1993-1-5 section 9

Bearing stiffeners are assumed to be symmetric about the beam web and will be designed as a strut to BS EN 1993-1-5 Clause 9.4(2), resisting the bearing reaction together with any eccentricities.

Shear buckling coefficient for web panel \( k_t \) from BS EN 1993-1-5.
compute \( k_t \) from Annex A \( k_t = 5.34 \)
Design shear resistance \( V_{bRd} = \frac{\gamma_{w} \cdot f_y \cdot h_w \cdot t_w}{(\sqrt{2} \cdot \gamma_{M1}) \cdot 1000} = 1958.7 \, \text{kN} \)
Design shear at the support \( V_{Ed(0)} = N_{bEd} = 1845 \, \text{kN} \)
As \( V_{Ed(0)} \leq V_{bRd} \) (1845 kN \( \leq 1958.7 \, \text{kN} \)), the design shear at the support is less than the design shear resistance. Hence OK.

Action as a bearing stiffener

Two double-sided stiffeners at support

\[ \text{hsb} \]
\[ \text{tw} \]
\[ \text{hsb} \]

NOTE: Stiffeners to be equally spaced about the centreline of bearing.
Depth of bearing stiffener \( h_{sb} = 150 \, \text{mm} \)
Thickness of bearing stiffener: $tsb = 20$ mm
Distance: $c_1 = 325$ mm
Distance between stiffener CL's: $c_2 = 300$ mm
Eccentricity in each direction: $ecc = 10$ mm

Worst stress in web plate:
$$swp = \frac{NbEd \times 1E3}{A} + \frac{MzEdb \times 1E6 \times (15 \times ew \times tw + tsb/2 + c_2/2)}{Izz}$$
$$= 104.11 \text{ N/mm}^2$$

Worst stress in stiffener:
- $comp_1 = \frac{NbEd \times 1E3}{A} = 93.117 \text{ N/mm}^2$
- $comp_2 = \frac{MzEdb \times 1E6 \times (c_2/2 + tsb/2)}{Izz} = 5.6288 \text{ N/mm}^2$
- $comp_3 = \frac{MyEdb \times 1E6 \times (hsb + tw/2)}{Iyy} = 28.339 \text{ N/mm}^2$

Stiffener stress: $sst = comp_1 + comp_2 + comp_3 = 127.08 \text{ N/mm}^2$

Limiting outstanding to prevent torsional buckling - Clause 9.2.1(8)
Ratio $(hsb/tsb) = p = hsb/tsb = 7.5$
Limit to ratio $p = (hsb/tsb)$ is $13\pi = 10.729$.
Ratio $(hsb/tsb) = 7.5$ is not greater than limit $13\pi = 10.729$.
Stiffener complies with Clause 9.2.1(8).

Action as a rigid end post
A rigid end post comprise of two double-sided transverse stiffeners that form the flanges of a short beam of length $hw$. The effective section below will be taken to exclude the outer parts of web plate.

Minimum area: $A_{min} = 4 \times hw \times tw^2/c_2 = 2416.7 \text{ mm}^2$
Actual area provided: $A_{sp} = 2 \times hsb \times tsb = 6000 \text{ mm}^2$
As $A_{sp} \geq A_{min} (6000 \text{ mm}^2 \geq 2416.7 \text{ mm}^2)$, stiffener area is adequate.
In-plane moment: $MzEdm = NH \times hw/(8 \times 1E3) = 140.88 \text{ kNm}$
End post 2nd moment of area: $I_{zze} = 2 \times (Asp \times (c_2/2)^2) = 270E6 \text{ mm}^4$
Worst stiffener stress from membrane action will be calculated below.
Stress due to membrane action: $sst_m = \frac{MzEdm \times 1E6}{I_{zze}/(c_2/2 + tsb/2)} = 83.484 \text{ N/mm}^2$

Cross-section resistance check
The maximum stresses from membrane action and bearing reaction will be added together here for simplicity, even though they occur at different heights on the stiffener.
Total stress: $sst_T = sst + sstm = 210.57 \text{ N/mm}^2$
As $sst_T \leq fy (210.57 \text{ N/mm}^2 \leq 345 \text{ mm}^2)$, stiffener is OK.

Buckling check under combined bending and axial load
The stiffener will be checked for buckling under combined bending and axial load in accordance with BS EN 1993-1-1, Clause 6.3.3.
Critical load

\[ N_{cry} = \pi^2 E I_{yy} / (L_{cry}^2 \times 1000) \]
\[ = 156687 \text{ kN} \]

Slenderness of the stiffener

\[ \lambda = (A \times f_y / (N_{cry} \times E^3))^{0.5} = 0.5 = 0.20887 \]

Design axial load resistance

\[ N_{pl,Rd} = A \times f_y / (\gamma M_1 \times 1000) = 6835.8 \text{ kN} \]

Design moment resistance yy axis

\[ M_{y,Rd} = W_{ely} \times f_y / (\gamma M_1 \times 10^6) = 224.61 \text{ kNm} \]

Design moment resistance zz axis

\[ M_{z,Rdb} = W_{elzb} \times f_y / (\gamma M_1 \times 10^6) = 1130.8 \text{ kNm (bearing)} \]

Design moment resistance zz axis

\[ M_{z,Rdm} = W_{elzm} \times f_y / (\gamma M_1 \times 10^6) = 582.19 \text{ kNm (membrane)} \]

Interaction equation:

\[ \text{Uni} = N_{Ed} / (\chi_y \times N_{pl,Rd}) + \text{ratio} \times (M_{y,Edb} / M_{y,Rd}) + (M_{z,Edb} / M_{z,Rdb} + M_{z,Edm} / M_{z,Rdm}) \]
\[ = 0.61255 \]

As Uni \leq 1.0 (0.61255 \leq 1.0), the interaction equation is OK.

Connection of stiffeners to girder web

Net stiffener length

\[ L_{sn} = h_w - 2 \times s_{np} = 1130 \text{ mm} \]

Assumed weld size

\[ s_{w} = 6 \text{ mm} \]

Effective length of weld

\[ L_w = 8 \times (L_{sn} - 2 \times s_{w}) = 8944 \text{ mm} \]

Design load on weld

\[ F_{w,Ed} = N_{b,Ed} \times 10^3 / L_w = 206.28 \text{ N/mm} \]

Design weld resistance

\[ F_{w,Rd} = 0.7 \times s_{w} \times f_{wd} = 1013.1 \text{ N/mm} \]

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

As \( h_w / t_w > 72 \times w \) (92.8 > 58.58), the web shear buckling resistance needs to be checked (EN 1993-1-5 Clause 5.2 and 5.3).

Modified slenderness \( \lambda \)

\[ \lambda_{mw} = h_w / (86.4 \times t_w \times w) = 1.3201 \]

Shear resistance of web

\[ V_{pl,Rd} = 2888.2 \text{ kN} \]

Reduced design shear resistance at support (web contribution only will be considered):

\[ V_{bw,Rd} = \chi_{w} \times f_{yw} \times h_w / (3 \times 10^3 \times \gamma M_1) = 2015.5 \text{ kN} \]

Design shear at support

\[ V_{Ed}(0) = N_{b,Ed} = 1845 \text{ kN} \]

As \( V_{Ed}(0) < V_{bw,Rd} \) (1845 kN < 2015.5 kN), the shear force is less than the shear resistance. Hence section OK.

compute

\[ \text{comp} = \eta_1 + (1 - M_{f,Rd} / M_{pl,Rd}) \times (2 \times \eta_3 - 1)^2 = 0.58198 \]

As \( \text{comp} \leq 1 \) (0.58198 \leq 1), interaction equation is satisfied.

Selected fillet weld size

\[ s = 8 \text{ mm} \]
DESIGN SUMMARY

Span of plate girder: 12 m
Design moment: 1800 kNm
All steel to be: Grade S 355

End stiffeners:

Posts 1 & 2 are double-sided stiffeners at end of girder. c2=300 mm

Double-sided stiffeners: post 1 2No/150 mm x 20 mm flats
post 2 2No/150 mm x 20 mm flats

b=400 mm
htf=20 mm
h=1200 mm
tw=12.5 mm
tf=20 mm

- Fillet welds to be used for connecting stiffeners to girder web & flanges
- Weld sizes as detailed in calculations
- Weld design strength to be 241 N/mm²

Assume rigid end posts to BS EN 1993-1-5, Figure 5.1 & Table 5.1.
Location: Ex3 - Intermediate stiffeners and non-rigid end posts

Stiffened plate girder

The girder is fully restrained along its length. The design calculations are in accordance with BS EN 1993-1-1 and BS EN 1993-1-5. Loads and moments are factored.

Assume non-rigid end posts to BS EN 1993-1-5, Figure 5.1 & Table 5.1.

Span of girder                  L=12 m
Maximum design bending moment   MyEd=1800 kNm
Design bearing reaction        NbEd=1500 kN
Depth of girder                h=1200 mm
Web thickness                  tw=12.5 mm
Stiffener corner snipe         snp=15 mm

Dimensions of web and flanges

Stiffener spacing of first panel ae=1550 mm
Intermediate stiffener spacing ai=3000 mm
Accompanying design shear     VzEd'=0 kN

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

As hw/tw > 72*ew (92.8 > 66.558), the web shear buckling resistance needs to be checked (EN 1993-1-5 Clause 5.2 and 5.3).

Buckling coefficient (Annex A)       kt=5.34+4*(hw/a)^2=7.5803
Modified slenderness λ               lamw=hw/(37.4*tw*ew*SQR(kt))=0.97491
Shear resistance of web               VplRd=2218.5 kN
Reduced design shear resistance at support (web contribution only will be considered): VbwRd=chiw*fyw*hw*tw/(3^0.5*gamM1*1000)=1960 kN

Design shear at support VEd(0)=NbEd=1500 kN
As VEd(0) ≤ VbwRd (1500 kN ≤ 1960 kN), the shear force is less than the shear resistance. Hence section OK.
compute
comp=eta1+(1-MfRd/MplRd)*(2*eta3 -1)^2=0.62629
As comp ≤ 1 (0.62629 ≤ 1), interaction equation is satisfied.
**Intermediate transverse web stiffeners - Arrangement (1)**

![Diagram of web stiffeners]

---

**Panel length 1**

- \( a(1) = 3000 \text{ mm} \)
- \( ls = 1160 \text{ mm} \)

**Longitudinal stresses in web with transverse stiffeners**

- \( ls = 1160 \text{ mm} \)

**Compression stresses are positive**

**Stress at top of web plate (\( \sigma_{top} \))**

- \( \sigma_{top} = 190.68 \text{ N/mm}^2 \)

**Stress at btm of web plate (\( \sigma_{btm} \))**

- \( \sigma_{btm} = -190.68 \text{ N/mm}^2 \)

**Design parameters**

- Stiffener outstand \( hs_1 = 100 \text{ mm} \)
- Stiffener thickness \( ts_1 = 15 \text{ mm} \)

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

- Yield strength to be adopted \( fy = 275 \text{ N/mm}^2 \)
- Ratio \( p = \frac{hs_1}{ts_1} = 6.6667 \)
  - Limit to ratio \( p = \frac{hs_1}{ts_1} \) is \( 13\varepsilon = 10.577 \).
  - Ratio \( \frac{hs_1}{ts_1} = 6.6667 \) is not greater than limit \( 13\varepsilon = 10.577 \).
  - Stiffener complies with Clause 9.2.1(8).
Design shear resistance (double sided transverse stiffener)

The effective cross section of the intermediate double-sided transverse stiffener is as shown below:

\[
\begin{align*}
&\text{tsi} \\
&15.\text{t}w \\
&15.\text{t}w \\
&\text{hsi} \\
&\text{tw} \quad \text{PLAN ON INTERMEDIATE} \\
&\text{hsi} \quad \text{STIFFENER}
\end{align*}
\]

2nd moment of area yy axis:
\[
I_{yy} = tsi \times (2 \times hsi + tw)^3/12 + (30 \times ew \times tw) \times tw^3/12
\]
\[
= 12.051 \times 10^6 \text{ mm}^4
\]

Design shear resistance
\[
V_{bRd} = \frac{\chi A_e f_y w}{\gamma_{M1} \times 1000} = 2015.2 \text{ kN}
\]

Axial force in the stiffener due to shear - BS EN 1993-1-5

Shear buckling coefficient for web panel \(kt\) is derived from Clause 5.2.

Buckling coefficient (Annex A)
\[
kt = 5.34 + 4 \times (hw/a)^2 = 5.938
\]

Modified slenderness \(\lambda_w\)
\[
\lambda_w = 0.76 \times (fy/tcr)^0.5 = 1.1017
\]

Max design shear force in panel \(V_{Ed1} = 600 \text{ kN}\)

Externally applied design load \(P_{ext1} = 1000 \text{ kN}\)

Option 1:
Longitudinal compressive stress \(\sigma_{xEd} = 0\) for symmetrical sections.

Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):
\[
P_{Ed1}' = V_{Ed1} \times hw \times tw \times 0.8 \times tcr/1000 = -918.11 \text{ kN}
\]

Option 2:
Vertical force generated in the stiffener as per BS EN 1993-1-5 (see NOTE, Clause 9.3.3(3)):  
\[
P_{Ed1} = V_{Ed1} \times f_y w \times hw \times tw / (SQR(3) \times \gamma_{M1} \times \lambda_w^2 \times 1000) = -1296.8 \text{ kN}
\]

Design axial load in stiffener \(P_{Ed} = P_{Ed1} + P_{ext1} = 1000 \text{ kN}\)

As \(P_{Ed} \leq V_{bRd}\) (1000 kN \(\leq 2015.2 \text{ kN}\), the design shear force does not exceed the shear resistance. Hence OK.

Connection to the web

Design shear load on weld
\[
F_{wEd} = tw^2/(5 \times hsi) + P_{Ed}/(2 \times (hw-2 \times snp)) = 0.75498 \text{ kN/mm}
\]

Weld size provided \(sw1 = 6 \text{ mm}\)

Stiffness check - Clause 9.3.3(3)

2nd moment of area (effe. section) \(I_{pro} = 12.051 \times 10^6 \text{ mm}^4\)

2nd moment of area required \(I_{req} = 0.75 \times hw \times tw^3 = 1.6992 \times 10^6 \text{ mm}^4\)

As \(I_{req} \leq I_{pro}\) (1.6992 \text{ mm}^4 \(\leq 12.051 \times 10^6 \text{ mm}^4\)), the stiffener is OK.
Web check between stiffeners

Girder assumed to have non-rigid end posts.

UDL applied between stiffeners: $w' = 100 \text{ kN/m (factored load)}$
Largest panel length: $a' = 3000 \text{ mm}$
Compressive stress of web: $sed = w'/tw = 8 \text{ N/mm}^2$
Compressive strength of web: $fed = (2.75 + 2/(a'/hw)^2)*E/(C/tw)^2$

$$fed = 77.525 \text{ N/mm}^2$$

Since $sed \leq fed$ ($8 \text{ N/mm}^2 \leq 77.525 \text{ N/mm}^2$), the web is OK.

Selected fillet weld size: $s = 6 \text{ mm}$

Bearing stiffener (double sided non-rigid end post)

It is assumed that the support reaction acts at the centroid of the effective stiffener section, such that there is no bending moment induced. Bearing stiffener consists a single double-sided stiffener. The stiffener effective cross section is as shown below:

Depth of bearing stiffener: $hsb = 150 \text{ mm}$
Thickness of bearing stiffener: $tsb = 20 \text{ mm}$
Distance: $c_1 = 100 \text{ mm}$
Design buckling resistance: $NbRd = \chi*Ast*fysb/(\gamma M1*1000)$

$$NbRd = 2561.6 \text{ kN}$$

As $NbEd \leq NbRd$ (1500 kN $\leq 2561.6$ kN), the design bearing reaction at the support is less than the design buckling resistance. Hence satisfactory.

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Ratio ($hsb/tsb$): $p = hsb/tsb = 7.5$
Limit to ratio $p = (hsb/tsb)$ is $13\varepsilon = 12.242$.
Ratio ($hsb/tsb$) = 7.5 is not greater than limit $13\varepsilon = 12.242$.
Stiffener complies with Clause 9.2.1(8).
DESIGN SUMMARY

Span of plate girder: 12 m
Design moment: 1800 kNm
All steel to be: Grade S 275

End stiffener: c1---ai---ai---

Posts 1 is a double-sided stiffener at end of girder. c1=100 mm

Double-sided stiffeners: post 1 2No/150 mm x 20 mm flats

Intermediate stiffeners: ai=3000 mm

Intermediate stiffeners:
Arrangement (1) Double-sided stiffeners
2No/100 mm x 15 mm flats spaced at 3000 mm centres

b=400 mm
h=1200 mm
tf=20 mm
tw=12.5 mm

• Fillet welds to be used for connecting stiffeners to girder web & flanges
• Weld sizes as detailed in calculations
• Weld design strength to be 222 N/mm²

Assume non-rigid end posts to EN 1993-1-5, Figure 5.1 & Table 5.1.
Location: Ex4 - Intermediate stiffeners and non-rigid end posts

Stiffened plate girder

The girder is fully restrained along its length. The design calculations are in accordance with BS EN 1993-1-1 and BS EN 1993-1-5. Loads and moments are factored.

Assume non-rigid end posts to BS EN 1993-1-5, Figure 5.1 & Table 5.1.

Span of girder \( L = 12 \text{ m} \)
Maximum design bending moment \( M_{yEd} = 2800 \text{ kNm} \)
Design bearing reaction \( N_{bEd} = 1450 \text{ kN} \)
Depth of girder \( h = 1240 \text{ mm} \)
Web thickness \( t_w = 12 \text{ mm} \)
Stiffener corner snipe \( snp = 15 \text{ mm} \)

Dimensions of web and flanges

Stiffener spacing of first panel \( a_e = 1550 \text{ mm} \)
Intermediate stiffener spacing \( a_i = 5000 \text{ mm} \)
Accompanying design shear \( V_{zEd'} = 0 \text{ kN} \)

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

As \( hw/tw > 72*ew \) ( 99.167 > 66.558 ), the web shear buckling resistance needs to be checked (EN 1993-1-5 Clause 5.2 and 5.3).

Buckling coefficient (Annex A) \( k_t = 5.34 + 4*(hw/a)^2 = 7.6977 \)
Modified slenderness \( \lambda \) \( \lambda_{mw} = hw/(37.4*tw*ew*SQR(k_t)) = 1.0338 \)
Shear resistance of web \( V_{plRd} = 2184.8 \text{ kN} \)
Reduced design shear resistance at support (web contribution only will be considered):

\[ V_{bwRd} = \chi_{w}f_{yw}hw*tw/(3^0.5*\gamma_{M1}*1000) = 1820.3 \text{ kN} \]

Design shear at support \( V_{Ed(0)} = N_{bEd} = 1450 \text{ kN} \)
As \( V_{Ed(0)} \leq V_{bwRd} \) ( 1450 kN \( \leq 1820.3 \text{ kN} \) ), the shear force is less than the shear resistance. Hence section OK.

compute \( \text{comp} = \eta_1 + (1-MfRd/MplRd)*(2*\eta_3 -1)^2 = 0.78202 \)
As \( \text{comp} \leq 1 \) ( 0.78202 \( \leq 1 \) ), interaction equation is satisfied.
**Intermediate transverse web stiffeners - Arrangement (1)**

![Diagram](image)

Elevation on web without longitudinal stiffening

- Panel length 1: \(a(1)=5000\) mm
- Length of stiffener: \(l_s=1190\) mm

Longitudinal stresses in web with transverse stiffeners

- \(l_s=1190\) mm

Compression stresses are positive

Stress at top of web plate (\(\sigma_{\text{top}}\)) \(\sigma_{\text{top}}=230.45\) N/mm²
Stress at btm of web plate (\(\sigma_{\text{btm}}\)) \(\sigma_{\text{btm}}=-230.45\) N/mm²

**Design parameters**

- Stiffener outstand: \(h_{s1}=150\) mm
- Stiffener thickness: \(t_{s1}=15\) mm

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

- Yield strength to be adopted: \(f_y=275\) N/mm²
- Ratio \((h_{s1}/t_{s1})\): \(p=h_{s1}/t_{s1}=10\)
- Limit to ratio \(p=(h_{s1}/t_{s1})\) is \(13\pi=12.017\).
- Ratio \((h_{s1}/t_{s1})=10\) is not greater than limit \(13\pi=12.017\).
- Stiffener complies with Clause 9.2.1(8).
Design shear resistance (single-sided transverse stiffener)

The effective cross section of the intermediate single-sided transverse stiffener is as shown below:

![Diagram of the effective cross section]

2nd moment of area yy axis: \( I_{yy} = I_{pro} = 13.86E6 \text{ mm}^4 \)

Design shear resistance: \( V_{bRd} = \chi_i A_e f_{yw} / (\gamma M1 \times 1000) = 1747.2 \text{ kN} \)

Axial force in the stiffener due to shear - BS EN 1993-1-5

Shear buckling coefficient for web panel \( k_t \) is derived from Clause 5.2.
Buckling coefficient (Annex A) \( k_t = 5.34 + 4 \times (h_w / a)^2 = 5.5666 \)
Modified slenderness \( \lambda_w \):
\( \lambda_w = 0.76 \times (f_{yw} / t_{cr})^{0.5} = 1.2159 \)
Max design shear force in panel \( V_{Ed1} = 1200 \text{ kN} \)
Externally applied design load \( P_{ext1} = 975 \text{ kN} \)

Option 1:
Longitudinal compressive stress \( \sigma_{xEd} = 0 \) for symmetrical sections.
Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):
\( P_{Ed1}' = V_{Ed1} - h_w \times t_w \times 0.8 \times t_{cr} / 1000 = -27.358 \text{ kN} \)

Option 2:
Vertical force generated in the stiffener as per BS EN 1993-1-5 (see NOTE, Clause 9.3.3(3)):
\( P_{Ed1} = V_{Ed1} - f_{yw} \times h_w \times t_w \times ( \text{SQR}(3) \times \gamma M1 \times \lambda_w^2 \times 1000) = -333.53 \text{ kN} \)
Design axial load in stiffener \( P_{Ed} = P_{Ed1} + P_{ext1} = 975 \text{ kN} \)
As \( P_{Ed} \leq V_{bRd} \) (975 kN \( \leq 1747.2 \text{ kN} \)), the design shear force does not exceed the shear resistance. Hence OK.

Connection to the web

Design shear load on weld:
\( F_{wEd} = t_w^2 / (5 \times h_{s1}) + P_{Ed} / (h_w - 2 \times \text{snp}) = 1.0325 \text{ kN/mm} \)
Weld size provided:
\( s_{w1} = 6 \text{ mm} \)

Stiffness check - Clause 9.3.3(3)

2nd moment of area (effe. section):
\( I_{pro} = 13.86E6 \text{ mm}^4 \)
2nd moment of area required:
\( I_{req} = 0.75 \times h_w \times t_w^3 = 1.5422E6 \text{ mm}^4 \)
As \( I_{req} \leq I_{pro} \) (1.5422E6 mm\(^4\) \( \leq 13.86E6 \text{ mm}^4 \)), the stiffener is OK.
Web check between stiffeners

\[ w' \text{ kN/m} \]

Girder assumed to have non-rigid end posts.

UDL applied between stiffeners \( w'=100 \text{ kN/m} \) (factored load)
Largest panel length \( a'=5000 \text{ mm} \)
Compressive stress of web \( \text{sed}=w'/tw=8.3333 \text{ N/mm}^2 \)
Compressive strength of web \( \text{fed}=(2.75+2/(a'/hw)^2)*E/(C/tw)^2 \)
\[ =63.687 \text{ N/mm}^2 \]
Since \( \text{sed} \leq \text{fed} \) (8.3333 N/mm\(^2\) \leq 63.687 N/mm\(^2\)), the web is OK.
Selected fillet weld size \( s=6 \text{ mm} \)

Intermediate transverse web stiffeners - Arrangement (2)

Panel length 1 \( a(3)=4000 \text{ mm} \)
Stiffener outstand \( hs2=100 \text{ mm} \)
Stiffener thickness \( ts2=10 \text{ mm} \)

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Yield strength to be adopted \( fy=275 \text{ N/mm}^2 \)
Ratio \( (hs2/ts2) \)
\[ p=hs2/ts2=10 \]
Limit to ratio \( p=(hs2/ts2) \) is 13\( \pi=12.017 \).
Ratio \( (hs2/ts2)=10 \) is not greater than limit 13\( \pi=12.017 \).
Stiffener complies with Clause 9.2.1(8).

Design shear resistance (single-sided transverse stiffener)

The effective cross section of the intermediate single-sided transverse stiffener is as shown below:

\[ \text{ PLAN ON INTERMEDIATE STIFFENER } \]

2nd moment of area yy axis \( I_{yy}=I_{pro}=3.4054E6 \text{ mm}^4 \)
Design shear resistance \( V_{bd}=\chi A e fyw/(\gamma M1*1000)=1263 \text{ kN} \)
Axial force in the stiffener due to shear

Max design shear force in panel\( V_{Ed2} = 1500 \text{ kN} \)
Externally applied design load \( P_{ext2} = 800 \text{ kN} \)
Generated force to be adopted \( P_{Ed2} = 244.62 \text{ kN} \)
Design axial load in stiffener \( P_{Ed'} = P_{Ed2} + P_{ext2} = 1044.6 \text{ kN} \)
As \( P_{Ed'} \leq V_{bRd} \) (1044.6 kN \( \leq 1263 \text{ kN} \)), the design shear force does not exceed the shear resistance. Hence OK.

Connection to the web

Design shear load on weld \( F_{wEd} = \frac{t_w^2}{5*hs^2} + \frac{P_{Ed'}}{h_w - 2*snp} \)
= 1.1885 kN/mm
Weld size provided \( s_w^2 = 6 \text{ mm} \)

Stiffness check - Clause 9.3.3(3)

2nd moment of area (eff.section) \( I_{pro} = 3.4054E6 \text{ mm}^4 \)
2nd moment of area required \( I_{req} = 0.75*h_w*t_w^3 = 1.5422E6 \text{ mm}^4 \)
As \( I_{req} \leq I_{pro} \) (1.5422E6 mm\(^4\) \( \leq 3.4054E6 \text{ mm}^4 \)), the stiffener is OK.

Bearing stiffener (double sided non-rigid end post)

It is assumed that the support reaction acts at the centroid of the effective stiffener section, such that there is no bending moment induced. Bearing stiffener consists a single double-sided stiffener. The stiffener effective cross section is as shown below:

Depth of bearing stiffener \( h_{sb} = 150 \text{ mm} \)
Thickness of bearing stiffener \( t_{sb} = 20 \text{ mm} \)
Distance \( c_1 = 100 \text{ mm} \)
Design buckling resistance \( N_{bRd} = \frac{\chi * A_{st} * f_{ysb}}{(\gamma M_1 * 1000)} = 2500.7 \text{ kN} \)
As \( N_{bEd} \leq N_{bRd} \) (1450 kN \( \leq 2500.7 \text{ kN} \)), the design bearing reaction at the support is less than the design buckling resistance. Hence satisfactory.

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Ratio \( (h_{sb}/t_{sb}) \) \( p = h_{sb}/t_{sb} = 7.5 \)
Limit to ratio \( p = (h_{sb}/t_{sb}) \) is 13\( \varepsilon = 12.242 \).
Ratio \( (h_{sb}/t_{sb}) = 7.5 \) is not greater than limit 13\( \varepsilon = 12.242 \).
Stiffener complies with Clause 9.2.1(8).
DESIGN SUMMARY

Span of plate girder: 12 m
Design moment: 2800 kNm
All steel to be: Grade S 275

End stiffener:

```
  c1  ai  ai
  1
```
c1=100 mm

Double-sided stiffeners: post 1
2No/150 mm x 20 mm flats

Intermediate stiffeners:
ai=5000 mm

Intermediate stiffeners:
Arrangement (1) Single-sided stiffeners
150 mm x 15 mm flats
spaced at 5000 mm centres

Intermediate stiffeners:
Arrangement (2) Single-sided stiffeners
100 mm x 10 mm flats
spaced at 4000 mm centres

- Fillet welds to be used for connecting stiffeners to girder web & flanges
- Weld sizes as detailed in calculations
- Weld design strength to be 222 N/mm²

Assume non-rigid end posts to EN 1993-1-5, Figure 5.1 & Table 5.1.
Stiffened plate girder

The girder is fully restrained along its length. The design calculations are in accordance with BS EN 1993-1-1 and BS EN 1993-1-5.

Loads and moments are factored. Assume non-rigid end posts to BS EN 1993-1-5, Figure 5.1 & Table 5.1.

Span of girder: L = 23.38 m
Maximum design bending moment: MyEd = 29515 kNm
Design bearing reaction: NbEd = 6281 kN
Depth of girder: h = 2500 mm
Web thickness: tw = 35 mm
Stiffener corner snipe: snp = 15 mm

Dimensions of web and flanges

Stiffener spacing of first panel: ae = 2400 mm
Intermediate stiffener spacing: ai = 2400 mm
Accompanying design shear: VzEd' = 0 kN

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

As hw/tw > 72*ew (68.286 > 67.802), the web shear buckling resistance needs to be checked (EN 1993-1-5 Clause 5.2 and 5.3).

Buckling coefficient (Annex A): kt = 5.34 + 4*(hw/a)^2 = 9.3067
Modified slenderness λ: lamw = hw/(37.4*tw*ew*SQR(kt)) = 0.63555
Shear resistance of web: VplRd = 12315 kN
Reduced design shear resistance at support (web contribution only will be considered):

VbwRd = χiw*fyw*hw*tw/(3^0.5*γM1*1000) = 12798 kN

Design shear at support: VEd(0) = NbEd = 6281 kN
As VEd(0) ≤ VbwRd (6281 kN ≤ 12798 kN), the shear force is less than the shear resistance. Hence section OK.
Intermediate transverse web stiffeners - Arrangement (1)

Panel length 1 \( a(1) = 2400 \text{ mm} \)
Length of stiffener \( l_s = 2390 \text{ mm} \)

Longitudinal stresses in web with transverse stiffeners

Stress at top of web plate \( \sigma_{\text{top}} \) \( \sigma_{\text{top}} = 243.87 \text{ N/mm}^2 \)
Stress at btm of web plate \( \sigma_{\text{btm}} \) \( \sigma_{\text{btm}} = -243.87 \text{ N/mm}^2 \)

Design parameters

Stiffener outstand \( h_{s1} = 250 \text{ mm} \)
Stiffener thickness \( t_{s1} = 25 \text{ mm} \)

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Yield strength to be adopted \( f_y = 265 \text{ N/mm}^2 \)
Ratio \( \frac{h_{s1}}{t_{s1}} \) \( p = \frac{h_{s1}}{t_{s1}} = 10 \)
Limit to ratio \( p = \frac{h_{s1}}{t_{s1}} \) is \( 13p = 12.242 \).
Ratio \( \frac{h_{s1}}{t_{s1}} = 10 \) is not greater than limit \( 13p = 12.242 \).
Stiffener complies with Clause 9.2.1(8).
Design shear resistance (double sided transverse stiffener)

The effective cross section of the intermediate double-sided transverse stiffener is as shown below:

```
  tsi                           z
  _15.r.tw_                   _z_
  
  hsi                           y
  _15.r.tw_                   _hsi_

2nd moment of area yy axis:
\[ I_{yy} = tsi \times (2 \times hsi + tw)^3 / 12 + (30 \times ew \times tw) \times tw^3 / 12 \]
\[ = 322.55 \times 10^6 \text{ mm}^4 \]

Design shear resistance
\[ V_{bRd} = chi \times A_{e} \times f_{yw} / (\gamma_{M1} \times 1000) = 12410 \text{ kN} \]

Axial force in the stiffener due to shear - BS EN 1993-1-5

Shear buckling coefficient for web panel \( k_t \) is derived from Clause 5.2.
Buckling coefficient (Annex A)
\[ k_t = 5.34 + 4 \times (hw/a)^2 = 9.3067 \]
Modified slenderness \( \lambda_w \)
\[ \lambda_w = 0.76 \times (fy/t_{cr})^0.5 = 0.63565 \]
Max design shear force in panel \( V_{Ed1} = 5370 \text{ kN} \)
Externally applied design load \( P_{ext1} = 0 \text{ kN} \)

Option 1:
Longitudinal compressive stress \( ox_{Ed} = 0 \) for symmetrical sections.
Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):
\[ P_{Ed1} = V_{Ed1} - hw \times tw \times 0.8 \times t_{cr} / 1000 = -19981 \text{ kN} \]

Option 2:
Vertical force generated in the stiffener as per BS EN 1993-1-5 (see NOTE, Clause 9.3.3(3)):
\[ P_{Ed1} = V_{Ed1} - f_{yw} \times hw \times tw / (\text{SQR}(3) \times \gamma_{M1} \times \lambda_w^2 \times 1000) = -26305 \text{ kN} \]
Design axial load in stiffener \( P_{Ed} = P_{Ed1} + P_{ext1} = 0 \text{ kN} \)
As \( P_{Ed} \leq V_{bRd} \) (0 kN \( \leq 12410 \text{ kN} \)), the design shear force does not exceed the shear resistance. Hence OK.

Connection to the web

Design shear load on weld
\[ F_{wEd} = tw^2 / (5 \times hsi) = 0.98 \text{ kN/mm} \]
Weld size provided \( sw_1 = 6 \text{ mm} \)

Stiffness check - Clause 9.3.3(3)

2nd moment of area (effective section)
\[ I_{pro} = 322.55 \times 10^6 \text{ mm}^4 \]
2nd moment of area required
\[ I_{req} = 1.5 \times hw^3 \times tw^3 / a^2 = 152.43 \times 10^6 \text{ mm}^4 \]
As \( I_{req} \leq I_{pro} \) (152.43E6 mm^4 \( \leq 322.55E6 \text{ mm}^4 \)), the stiffener is OK.
Web check between stiffeners

\[ w' \text{ kN/m} \]

Girder assumed to have non-rigid end posts.

UDL applied between stiffeners \( w' = 432 \text{ kN/m} \) (factored load)

Largest panel length \( a' = 2400 \text{ mm} \)

Compressive stress of web \( \text{sed} = w'/tw = 12.343 \text{ N/mm}^2 \)

Compressive strength of web \( \text{fed} = (2.75 + 2/(a'/hw)^2) \times E/(C/tw)^2 = 217.52 \text{ N/mm}^2 \)

Since \( \text{sed} \leq \text{fed} \) (12.343 N/mm² ≤ 217.52 N/mm²), the web is OK.

Selected fillet weld size \( s = 13 \text{ mm} \)

Bearing stiffener (double sided non-rigid end post)

It is assumed that the support reaction acts at the centroid of the effective stiffener section, such that there is no bending moment induced. Bearing stiffener consists a single double-sided stiffener. The stiffener effective cross section is as shown below:

Depth of bearing stiffener \( h_{sb} = 300 \text{ mm} \)

Thickness of bearing stiffener \( t_{sb} = 30 \text{ mm} \)

Distance \( c_1 = 250 \text{ mm} \)

Design buckling resistance \( Nb_{Rd} = \chi * Ast * f_{yw} / (\gamma M_1 * 1000) = 11952 \text{ kN} \)

As \( Nb_{Ed} \leq Nb_{Rd} \) (6281 kN ≤ 11952 kN), the design bearing reaction at the support is less than the design buckling resistance. Hence satisfactory.

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Ratio \( (h_{sb}/t_{sb}) = 10 \)

Limit to ratio \( p = (h_{sb}/t_{sb}) \) is \( 13\pi = 12.242 \).

Ratio \( (h_{sb}/t_{sb}) = 10 \) is not greater than limit \( 13\pi = 12.242 \).

Stiffener complies with Clause 9.2.1(8).
DESIGN SUMMARY

Span of plate girder: 23.38 m
Design moment: 29515 kNm
All steel to be: Grade S 275

End stiffener: c1 = 250 mm
Posts 1 is a double-sided stiffener at end of girder.

Double-sided stiffeners: post 1 2No/300 mm x 30 mm flats
Intermediate stiffeners: ai = 2400 mm

Intermediate stiffeners:
Arrangement (1) Double-sided stiffeners
2No/250 mm x 25 mm flats
spaced at 2400 mm centres

Assume non-rigid end posts to EN 1993-1-5, Figure 5.1 & Table 5.1.
The girder is fully restrained along its length. The design calculations are in accordance with BS EN 1993-1-1 and BS EN 1993-1-5.

Loads and moments are factored.

Assume rigid end posts (two double-sided stiffeners at supports).

Span of girder \( L = 25 \text{ m} \)

Maximum design bending moment \( M_{yEd} = 14028 \text{ kNm} \)

Design bearing reaction \( N_{bEd} = 2200 \text{ kN} \)

Depth of girder \( h = 2030 \text{ mm} \)

Web thickness \( t_w = 13 \text{ mm} \)

Stiffener corner snipe \( s_{np} = 15 \text{ mm} \)

Determine the effective section properties - EN 1992-1-5, Cl. 4.4

The process is iterative as the amount of web not considered is a function of the stresses at the top and bottom of the web.

Iteration 1:

Gross area of section \( A = 2btf + hw'tw = 81350 \text{ mm}^2 \)

Gross 2nd moment of area \( I_y = (b'(hw+2tf)^3 - (b-tw)hw^3)/12 = 63.482E9 \text{ mm}^4 \)

Stress at top of web \( s_{1} = M_{yEd} * 10^6 * hw/2/I_y = 215.45 \text{ N/mm}^2 \)

Stress at bottom of web \( s_{2} = -M_{yEd} * 10^6 * hw/2/I_y = -215.45 \text{ N/mm}^2 \)

Iteration 10:

Position of effective centroid:

\[ z_{eff} = \left( A'h/2 - Aw\cdot(z_{effp} + be2 + lw/2) \right) / A_{eff} = 993.96 \text{ mm} \]

Effective 2nd moment of area:

\[ I_{y_{eff}} = I_y + A\cdot(h/2 - z_{eff})^2 - tw\cdot1w^3/12 - Aw\cdot(z_{effp} + be2 + lw/2 - z_{eff})^2 = 62.521E9 \text{ mm}^4 \]

Top of web stress \( s_{1(10)} = M_{yEd} \cdot 10^6 \cdot (h-tf - z_{eff})/I_{y_{eff}} = 223.48 \text{ N/mm}^2 \)

Bottom of web stress \( s_{2(10)} = -M_{yEd} \cdot 10^6 \cdot (z_{eff} - tf)/I_{y_{eff}} = -214.04 \text{ N/mm}^2 \)

Net loss area \( A_{w} = lw'tw = 2977.9 \text{ mm}^2 \)

Effective area of section \( A_{eff} = A - A_{w} = 78372 \text{ mm}^2 \)

Effective section modulus \( W_{effy} = I_{y_{eff}} / (1000 \cdot (h - z_{eff})) = 60346 \text{ cm}^3 \)
Dimensions of web and flanges

Stiffener spacing of first panel  $a_e=2500$ mm
Intermediate stiffener spacing  $a_i=2500$ mm
Accompanying design shear  $V_{zEd}'=0$ kN

**Bearing stiffeners double sided - BS EN 1993-1-5 section 9**

Bearing stiffeners are assumed to be symmetric about the beam web and will be designed as a strut to BS EN 1993-1-5 Clause 9.4(2), resisting the bearing reaction together with any eccentricities.

- Compute $k_t$ from Annex A: $k_t = 5.34 + 4 \times (h_w/a)^2 = 7.7736$
- Design shear resistance: $V_{bRd} = \chi_i w \times f_y \times h_w \times t_w / (SQR(3) \times \gamma_{M1} \times 1000) = 2355.2$ kN
- Design shear at the support: $V_{Ed(0)} = N_{bRd} = 2200$ kN

As $V_{Ed(0)} \leq V_{bRd}$ (2200 kN ≤ 2355.2 kN), the design shear at the support is less than the design shear resistance. Hence OK.

**Action as a bearing stiffener**

Compute $kt$ from Annex A: $k_t = 5.34 + 4 \times (h_w/a)^2 = 7.7736$

Design shear resistance: $V_{bRd} = \chi_i w \times f_y \times h_w \times t_w / (SQR(3) \times \gamma_{M1} \times 1000) = 2355.2$ kN

Design shear at the support: $V_{Ed(0)} = N_{bRd} = 2200$ kN

As $V_{Ed(0)} \leq V_{bRd}$ (2200 kN ≤ 2355.2 kN), the design shear at the support is less than the design shear resistance. Hence OK.

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

Ratio $(h_{sb}/t_{sb}) = h_{sb}/t_{sb} = 10$
Limit to ratio $p=(h_{sb}/t_{sb})$ is $13r=12.242$.
Ratio $(h_{sb}/t_{sb})=10$ is not greater than limit $13r=12.242$.
Stiffener complies with Clause 9.2.1(8).
**Action as a rigid end post**

A rigid end post comprise of two double-sided transverse stiffeners that form the flanges of a short beam of length $hw$. The effective section below will be taken to exclude the outer parts of web plate.

![Diagram of two double-sided stiffeners at support](image)

Minimum area: $A_{min} = 4 \times hw \times tw^2 / c^2 = 4394 \text{ mm}^2$

Actual area provided: $A_{sp} = 2 \times hsb \times tsb = 12500 \text{ mm}^2$

As $A_{sp} \geq A_{min}$ (12500 mm$^2 \geq 4394$ mm$^2$), stiffener area is adequate.

In-plane moment: $M_{zEdm} = N \times hw / (8 \times 1E3) = 513.98 \text{ kNm}$

End post 2nd moment of area: $I_{ze} = 2 \times (A_{sp} \times (c^2 / 2)^2) = 562.5E6 \text{ mm}^4$

Worst stiffener stress from membrane action will be calculated below.

Stress due to membrane action:

$$s_{stm} = M_{zEdm} \times 1E6 / (I_{ze} / (c^2 / 2 + tsb / 2))$$

$$= 148.48 \text{ N/mm}^2$$

**Cross-section resistance check**

The maximum stresses from membrane action and bearing reaction will be added together here for simplicity, even though they occur at different heights on the stiffener.

Total stress: $s_{stT} = s_{st} + s_{stm} = 233.3 \text{ N/mm}^2$

As $s_{stT} \leq f_y$ (233.3 N/mm$^2 \leq 265$ mm$^2$), stiffener is OK.

**Buckling check under combined bending and axial load**

The stiffener will be checked for buckling under combined bending and axial load in accordance with BS EN 1993-1-1, Clause 6.3.3.

Critical load: $N_{cry} = \frac{\pi^2 \times E \times I_{yy}}{L_{cry}^2 \times 1000}$

$$= 306613 \text{ kN}$$

Slenderness of the stiffener: $\lambda_{mb} = (A \times f_y / (N_{cry} \times 1E3))^{0.5} = 0.16518$

Design axial load resistance: $N_{plRd} = A \times f_y / (\gamma_{M1} \times 1000) = 8365.6 \text{ kN}$

Design moment resistance yy axis: $M_{yb} = W_{ely} \times f_y / (\gamma_{M1} \times 1E6) = 581.16 \text{ kNm}$

Design moment resistance zz axis: $M_{zbb} = W_{elzb} \times f_y / (\gamma_{M1} \times 1E6)$

$$= 1145.2 \text{ kNm (bearing)}$$

Design moment resistance zz axis:

$$M_{zRdm} = W_{elzm} \times f_y / (\gamma_{M1} \times 1E6) = 917.31 \text{ kNm (membrane)}$$

Interaction equation:

$$Uni = \frac{N_{Ed} / (\chi_{yy} \times N_{plRd}) + \text{ratio} \times (M_{yEd} / M_{yRd}) + (M_{zEdb} / M_{zRdb} + M_{Edm} / M_{Rdm})}{0.88064}$$

As $Uni \leq 1.0$ (0.88064 ≤ 1.0), the interaction equation is OK.
Connection of stiffeners to girder web

Net stiffener length \( L_{sn} = hw - 2snp = 1920 \text{ mm} \)
Assumed weld size \( ss' = 6 \text{ mm} \)
Effective length of weld \( L_w = 8(Lsn - 2ss') = 15264 \text{ mm} \)
Design load on weld \( F_{wEd} = NbEd 	imes 10^3 / Lw = 144.13 \text{ N/mm} \)
Design weld resistance \( F_{wRd} = 0.7ss'fvwd = 935.71 \text{ N/mm} \)

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

As \( hw/tw > 72\cdot ew \) (150 > 66.558), the web shear buckling resistance needs to be checked (EN 1993-1-5 Clause 5.2 and 5.3).
Buckling coefficient (Annex A) \( k_t = 5.34 + 4(hw/a)^2 = 7.7736 \)
Modified slenderness \( \lambda = \frac{hw}{(37.4\cdot tw\cdot ew\cdot SQR(kt))} = 1.5561 \)
Shear resistance of web \( V_{plRd} = 3878.5 \text{ kN} \)
Reduced design shear resistance at support (web contribution only will be considered):
\[ V_{bwRd} = \chi_{w} f_{yw} hw tw / (3^0.5 \cdot \gamma_{M1} \cdot 1000) = 2444 \text{ kN} \]

Design shear at support \( V_{Ed}(0) = NbEd = 2200 \text{ kN} \)
As \( V_{Ed}(0) \leq V_{bwRd} \) (2200 kN \( \leq 2444 \) kN), the shear force is less than the shear resistance. Hence section OK.
compute \( \text{comp} = \eta_1 + (1 - M_{Rd}/M_{plRd}) \cdot (2 \cdot \eta_3 - 1)^2 = 0.9263 \)
As \( \text{comp} \leq 1 \) (0.9263 \( \leq 1 \)), interaction equation is satisfied.

Intermediate transverse web stiffeners - Arrangement (1)

Intermediate transverse web stiffeners - Arrangement (1)

Elevation on web without longitudinal stiffening
Panel length 1 \( a(1) = 2500 \text{ mm} \)
Length of stiffener \( l_s = 1950 \text{ mm} \)

Longitudinal stresses in web with transverse stiffeners
\( l_s = 1950 \text{ mm} \)

Compression stresses are positive
Stress at top of web plate (\(\sigma_{\text{top}}\)) \(=251.76\) N/mm\(^2\)
Stress at btm of web plate (\(\sigma_{\text{btm}}\)) \(=251.76\) N/mm\(^2\)

**Design parameters**

Stiffener outstand \(h_{s1}=150\) mm
Stiffener thickness \(t_{s1}=20\) mm

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

Yield strength to be adopted \(f_y=275\) N/mm\(^2\)
Ratio \((h_{s1}/t_{s1})\) \(=h_{s1}/t_{s1}=7.5\)
Limit to ratio \(p=(h_{s1}/t_{s1})\) is \(13\pi=12.017\).
Ratio \(h_{s1}/t_{s1}=7.5\) is not greater than limit \(13\pi=12.017\).
Stiffener complies with Clause 9.2.1(8).

**Design shear resistance (double sided transverse stiffener)**

The effective cross section of the intermediate double-sided transverse stiffener is as shown below:

2nd moment of area \(y\) axis:
\[I_{yy}=t_{si}*(2*h_{si}+t_{w})^3/12+(30*e_{w}*t_{w})*t_{w}^3/12\]
\[=51.173E6\] mm\(^4\)

Design shear resistance \(V_{bRd}=\chi*AE*f_{ysi}/(\gamma M_1*1000)\)
\[=2839.1\] kN

**Axial force in the stiffener due to shear - BS EN 1993-1-5**

Shear buckling coefficient for web panel \(k_t\) is derived from Clause 5.2.
Buckling coefficient (Annex A) \(k_t=5.34+4*(h/w/a)^2=7.7736\)
Modified slenderness \(\lambda_w\) \(=0.76*(f_y/t_{cr})^0.5=1.5564\)
Max design shear force in panel \(V_{Ed1}=2000\) kN
Externally applied design load \(P_{ext1}=0\) kN

Option 1:
Longitudinal compressive stress \(\sigma_{xEd}=0\) for symmetrical sections.
Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):
\[P_{Ed1}'=V_{Ed1}-h*w*tw*0.8*t_{cr}/1000=670.15\] kN

Option 2:
Vertical force generated in the stiffener as per BS EN 1993-1-5
(see NOTE, Clause 9.3.3(3)):
\[P_{Ed1}=V_{Ed1}-f_y*w*h*w/(SQR(3)*\gamma M_1*\lambda_w^2*1000)=338.4\] kN
Generated force to be adopted \(P_{Ed1}=670.15\) kN
Design axial load in stiffener  \( P_{Ed} = P_{Ed1} + P_{ext1} = 670.15 \text{ kN} \)

As \( P_{Ed} \leq V_{bRd} \) (670.15 kN \( \leq 2839.1 \text{ kN} \)), the design shear force does not exceed the shear resistance. Hence OK.

**Connection to the web**

Design shear load on weld  \( F_{wEd} = \frac{t_w^2}{5 \times h_s1} + \frac{P_{Ed}}{2 \times (h_w - 2 \times s_{np})} \)

= 0.39985 kN/mm

Weld size provided  \( s_{w1} = 6 \text{ mm} \)

**Stiffness check - Clause 9.3.3(3)**

2nd moment of area (effe. section)  \( I_{pro} = 5.1.173E6 \text{ mm}^4 \)

2nd moment of area required  \( I_{req} = 1.5 \times h_w^3 \times t_w^3 / a^2 = 3.9097E6 \text{ mm}^4 \)

As \( I_{req} \leq I_{pro} \) (3.9097E6 mm\(^4\) \( \leq 5.1.173E6 \text{ mm}^4 \)), the stiffener is OK.

**Web check between stiffeners**

\( w' \) kN/m

\[ w' = 92 \text{ kN/m (factored load)} \]

\( a' = 2500 \text{ mm} \)

Compressive stress of web  \( s_{ed} = w' / t_w = 7.0769 \text{ N/mm}^2 \)

Compressive strength of web  \( s_{ed} = (2.75 + 2 / (a' / h_w)^2) \times E / (C / t_w)^2 \)

= 37.952 N/mm\(^2\)

Since \( s_{ed} \leq s_{ed} \) (7.0769 N/mm\(^2\) \( \leq 37.952 \text{ N/mm}^2 \)), the web is OK.

Selected fillet weld size  \( s = 12 \text{ mm} \)

**Intermediate transverse web stiffeners - Arrangement (2)**

Panel length 1  \( a(3) = 2800 \text{ mm} \)

Stiffener outstand  \( h_s2 = 200 \text{ mm} \)

Stiffener thickness  \( t_s2 = 20 \text{ mm} \)

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

Yield strength to be adopted  \( f_y = 265 \text{ N/mm}^2 \)

Ratio \( (h_s2 / t_s2) \)  \( p = h_s2 / t_s2 = 10 \)

Limit to ratio \( p = (h_s2 / t_s2) \) is 13\( r = 12.242 \).

Ratio \( (h_s2 / t_s2) = 10 \) is not greater than limit 13\( r = 12.242 \).

Stiffener complies with Clause 9.2.1(8).
Design shear resistance (double sided transverse stiffener)

The effective cross section of the intermediate double-sided transverse stiffener is as shown below:

```
  tsi                           z
  -----------------------------
  15.ε.tw                      15.ε.tw
  --------------------
  hsi                         15.ε.tw
  -----------------------------
  tw                         PLAN ON INTERMEDIATE
  hsi       STIFFENER
```

2nd moment of area yy axis:

\[ I_{yy} = tsi \times (2 \times hsi + tw)^3/12 + (30 \times ew \times tw) \times tw^3/12 \]

= \( 117.47E6 \) mm\(^4\)

Design shear resistance

\[ V_{bRd} = \frac{chi \times Ae \times fysi}{\gamma_M1 \times 1000} \]

= \( 3430.9 \) kN

Axial force in the stiffener due to shear

Max design shear force in panel \( V_{Ed2} = 2200 \) kN

Externally applied design load \( P_{ext2} = 0 \) kN

Generated force to be adopted \( P_{Ed2} = 954.56 \) kN

Design axial load in stiffener \( P_{Ed'} = P_{Ed2} + P_{ext2} = 954.56 \) kN

As \( P_{Ed'} \leq V_{bRd} \) (954.56 kN \( \leq 3430.9 \) kN), the design shear force does not exceed the shear resistance. Hence OK.

Connection to the web

Design shear load on weld \( F_{wEd} = tw^2/(5 \times hs2) + P_{Ed'}/(2 \times (hw-2 \times snp)) = 0.41758 \) kN/mm

Weld size provided \( sw2 = 6 \) mm

Stiffness check - Clause 9.3.3(3)

2nd moment of area (eff. section) \( I_{pro} = 117.47E6 \) mm\(^4\)

2nd moment of area required \( I_{req} = 0.75 \times hw \times tw^3 = 3.2131E6 \) mm\(^4\)

As \( I_{req} \leq I_{pro} \) (3.2131E6 mm\(^4\) \( \leq 117.47E6 \) mm\(^4\)), the stiffener is OK.
DESIGN SUMMARY

Span of plate girder: 25 m
Design moment: 14028 kNm
All steel to be: Grade S 275

End stiffeners:

- Posts 1 & 2 are double-sided stiffeners at end of girder.
  - Post 1: 2No/250 mm x 25 mm flats
  - Post 2: 2No/250 mm x 25 mm flats

Intermediate stiffeners:

- Arrangement (1): Double-sided stiffeners
  - 2No/150 mm x 20 mm flats
  - spaced at 2500 mm centres

- Arrangement (2): Double-sided stiffeners
  - 2No/200 mm x 20 mm flats
  - spaced at 2800 mm centres

- b=700 mm
- h=2030 mm
- tw=13 mm
- tf=40 mm

Fillet welds to be used for connecting stiffeners to girder web & flanges.
Weld sizes as detailed in calculations.
Weld design strength to be 222 N/mm².

Assume rigid end posts to BS EN 1993-1-5, Figure 5.1 & Table 5.1.
Stiffened plate girder

The girder is fully restrained along its length. The design calculations are in accordance with BS EN 1993-1-1 and BS EN 1993-1-5. Loads and moments are factored.

Assume non-rigid end posts to BS EN 1993-1-5, Figure 5.1 & Table 5.1.

Span of girder \( L = 30 \text{ m} \)

Maximum design bending moment \( M_y = 18909 \text{ kNm} \)

Design bearing reaction \( N_b = 2293 \text{ kN} \)

Depth of girder \( h = 2100 \text{ mm} \)

Web thickness \( t_w = 15 \text{ mm} \)

Stiffener corner snipe \( s_n = 15 \text{ mm} \)

Determine the effective section properties – EN 1992-1-5, Cl. 4.4

The process is iterative as the amount of web not considered is a function of the stresses at the top and bottom of the web.

Iteration 1:

Gross area of section \( A = 2b \times t_f + h_w \times t_w = 105000 \text{ mm}^2 \)

Gross 2nd moment of area \( I_y = \frac{b \times (h_w + 2 \times t_f - t_w)^3 - (b - t_w) \times h_w^3}{12} = 88.813E9 \text{ mm}^4 \)

Stress at top of web \( s_1 = \frac{M_y \times 10^6 \times h_w}{2 \times I_y} = 212.91 \text{ N/mm}^2 \)

Stress at bottom of web \( s_2 = \frac{-M_y \times 10^6 \times h_w}{2 \times I_y} = -212.91 \text{ N/mm}^2 \)

Iteration 10:

Position of effective centroid:
\[
zeff = \frac{(A \times h/2 - Aw \times (zeffp + be2 + lw/2))}{Aeff} = 1039.7 \text{ mm}
\]

Effective 2nd moment of area:
\[
I_{yeff} = I_y + A \times (h/2 - zeff)^2 - tw \times lw^3/12 - Aw \times (zeffp + be2 + lw/2 - zeff)^2 = 88.178E9 \text{ mm}^4
\]

Top of web stress \( s_1(10) = \frac{M_y \times 10^6 \times (h - tf - zeff)}{I_{yeff}} = 216.65 \text{ N/mm}^2 \)

Bottom of web stress \( s_2(10) = -\frac{M_y \times 10^6 \times zeff - tf}{I_{yeff}} = -212.23 \text{ N/mm}^2 \)

Net loss area \( Aw = lw \times tw = 1823.4 \text{ mm}^2 \)

Effective area of section \( A_{eff} = A - Aw = 103177 \text{ mm}^2 \)

Effective section modulus \( W_{yeff} = I_{yeff} / (1000 \times (h - zeff)) = 83162 \text{ cm}^3 \)
Dimensions of web and flanges

Stiffener spacing of first panel  \( ae = 1550 \text{ mm} \)
Intermediate stiffener spacing  \( ai = 3900 \text{ mm} \)
Accompanying design shear  \( V_{zEd'} = 0 \text{ kN} \)

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

As \( hw/tw > 72*ew \) (133.33 > 66.558), the web shear buckling resistance needs to be checked (EN 1993-1-5 Clause 5.2 and 5.3).
Buckling coefficient (Annex A)  \( kt = 4 + 5.34*(hw/a)^2 = 12.891 \)
Modified slenderness \( \lambda \)  \( lamw = hw/(37.4*tw*ew*SQR(kt)) = 1.0741 \)
Shear resistance of web  \( V_{plRd} = 4416.7 \text{ kN} \)
Reduced design shear resistance at support (web contribution only will be considered):
\[
V_{bwRd} = \chi_{iw} * f_{yw} * hw*tw/(3^0.5*\gamma_{M1}*1000) = 3680.5 \text{ kN}
\]

Design shear at support  \( V_{Ed(0)} = Nb_{Ed} = 2293 \text{ kN} \)
As \( V_{Ed(0)} \leq V_{bwRd} \) (2293 kN ≤ 3680.5 kN), the shear force is less than the shear resistance. Hence section OK.
compute  \( \text{comp} = \eta_1 + (1-M_{Rd}/M_{plRd})*(2*\eta_3 -1)^2 = 0.89624 \)
As \( \text{comp} \leq 1 \) (0.89624 ≤ 1), interaction equation is satisfied.

Intermediate transverse web stiffeners - Arrangement (1)

Elevation on web without longitudinal stiffening

Panel length 1  \( a(1) = 3900 \text{ mm} \)
Length of stiffener  \( ls = 2000 \text{ mm} \)

Longitudinal stresses in web with transverse stiffeners
Compression stresses are positive
Stress at top of web plate (σtop) stop=245.97 N/mm²
Stress at btm of web plate (σbtm) sbtm=-245.97 N/mm²

Design parameters

Stiffener outstand       hsl=80 mm
Stiffener thickness      tsl=15 mm

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Yield strength to be adopted      fy=275 N/mm²
Ratio (hsl/tsl)                   p=hsl/tsx=5.3333
Limit to ratio p=(hsl/tsx) is 13ε=12.017.
Ratio (hsl/tsx)=5.3333 is not greater than limit 13ε=12.017.
Stiffener complies with Clause 9.2.1(8).

Design shear resistance (double sided transverse stiffener)

The effective cross section of the intermediate double-sided transverse stiffener is as shown below:

\[
\begin{align*}
\text{tsi} & \quad 15.0.\text{tw} \quad 15.0.\text{tw} \\
\text{hsi} & \quad \text{tw} \quad \text{PLAN ON INTERMEDIATE STIFFENER} \\
\text{tw} & \quad \text{PLAN ON INTERMEDIATE STIFFENER}
\end{align*}
\]

2nd moment of area yy axis:
\[
I_{yy}=\text{tsi} \cdot (2 \times \text{hsi}+\text{tw})^3/12+(30 \times \text{ew} \times \text{tw}) \times \text{tw}^3/12 =6.8162E6 \text{ mm}^4
\]
Design shear resistance \( V_{bRd}=\chi \cdot A_{e} \cdot f_{yw}/(\gamma_{M1} \times 1000) =1880.9 \text{ kN} \)

Axial force in the stiffener due to shear - BS EN 1993-1-5

Shear buckling coefficient for web panel \( k_t \) is derived from Clause 5.2.
Buckling coefficient (Annex A) \( k_t=5.34+4 \times (h/w/a)^2=6.3919 \)
Modified slenderness \( \lambda w \) \( \lambda w=0.76 \times (f/y/tcr)^0.5=1.5256 \)
Max design shear force in panel \( V_{Ed1}=2152 \text{ kN} \)
Externally applied design load \( P_{ext1}=0 \text{ kN} \)

Option 1:
Longitudinal compressive stress \( \sigma_{xEd}=0 \) for symmetrical sections.
Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):
\[
P_{Ed1}'=V_{Ed1}-hw\times tw\times 0.8\times tcr/1000=514.19 \text{ kN}
\]

Option 2:
Vertical force generated in the stiffener as per BS EN 1993-1-5 (see NOTE, Clause 9.3.3(3)):
\[
P_{Ed1}=V_{Ed1}-f_{yw} \times hw\times tw/(\sqrt{3} \times \gamma_{M1} \times \lambda w^2 \times 1000)=105.63 \text{ kN}
\]
Generated force to be adopted \( P_{Ed1}=514.19 \text{ kN} \)
Design axial load in stiffener \( P_{Ed}=P_{Ed1}+P_{ext1}=514.19 \text{ kN} \)
As $P_{Ed} \leq V_{bRd}$ (514.19 kN $\leq$ 1880.9 kN), the design shear force does not exceed the shear resistance. Hence OK.

**Connection to the web**

Design shear load on weld

$$F_{wEd} = \frac{tw^2}{5*hs1} + \frac{P_{Ed}}{2*(hw-2*snp)} = 0.69301 \text{ kN/mm}$$

Weld size provided

$s_{w1} = 6 \text{ mm}$

**Stiffness check - Clause 9.3.3(3)**

2nd moment of area (effe. section) $I_{pro} = 6.8162E6 \text{ mm}^4$

2nd moment of area required

$I_{req} = 0.75*hw*tw^3 = 5.0625E6 \text{ mm}^4$

As $I_{req} \leq I_{pro}$ ($5.0625E6 \text{ mm}^4 \leq 6.8162E6 \text{ mm}^4$), the stiffener is OK.

**Web check between stiffeners**

Girder assumed to have non-rigid end posts.

UDL applied between stiffeners

$w' = 91.2 \text{ kN/m}$ (factored load)

Largest panel length

$a' = 3900 \text{ mm}$

Compressive stress of web

$sed = w'/tw = 6.08 \text{ N/mm}^2$

Compressive strength of web

$fed = \left(2.75 + \frac{2}{(a'/hw)^2}\right)E/(C/tw)^2 = 39.643 \text{ N/mm}^2$

Since $sed \leq fed$ ($6.08 \text{ N/mm}^2 \leq 39.643 \text{ N/mm}^2$), the web is OK.

Selected fillet weld size

$s = 8 \text{ mm}$

**Intermediate transverse web stiffeners - Arrangement (2)**

Panel length 1

$a(3) = 5650 \text{ mm}$

Stiffener outstand

$hs2 = 80 \text{ mm}$

Stiffener thickness

$ts2 = 15 \text{ mm}$

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

Yield strength to be adopted

$fy = 275 \text{ N/mm}^2$

Ratio ($hs2/ts2$)

$p = hs2/ts2 = 5.3333$

Limit to ratio $p = (hs2/ts2)$ is $13\varepsilon = 12.017$. Ratio ($hs2/ts2$) = 5.3333 is not greater than limit $13\varepsilon = 12.017$.

Stiffener complies with Clause 9.2.1(8).

**Axial force in the stiffener due to shear**

Max design shear force in panel

$V_{Ed2} = 1440 \text{ kN}$

Externally applied design load

$P_{ext2} = 925 \text{ kN}$

Design axial load in stiffener

$P_{Ed'} = P_{Ed2} + P_{ext2} = 925 \text{ kN}$
As $P_{Ed}' \leq V_{bRd}$ ($925 \text{ kN} \leq 1880.9 \text{ kN}$), the design shear force does not exceed the shear resistance. Hence OK.

**Connection to the web**

Design shear load on weld

$$F_{wEd} = \frac{tw^2}{5*(hs^2)} + \frac{P_{Ed}'}{2*(hw-2*snp)} = 0.79727 \text{ kN/mm}$$

Weld size provided

$$sw^2 = 6 \text{ mm}$$

**Stiffness check - Clause 9.3.3(3)**

2nd moment of area (eff. section) $I_{pro} = 6.8162E6 \text{ mm}^4$

2nd moment of area required $I_{req} = 0.75*hw*tw^3 = 5.0625E6 \text{ mm}^4$

As $I_{req} \leq I_{pro}$ ($5.0625E6 \text{ mm}^4 \leq 6.8162E6 \text{ mm}^4$), the stiffener is OK.

**Bearing stiffener (double sided non-rigid end post)**

It is assumed that the support reaction acts at the centroid of the effective stiffener section, such that there is no bending moment induced. Bearing stiffener consists a single double-sided stiffener. The stiffener effective cross section is as shown below:

![Diagram of bearing stiffener](image)

Depth of bearing stiffener $h_{sb} = 280 \text{ mm}$

Thickness of bearing stiffener $t_{sb} = 24 \text{ mm}$

Distance $c_1 = 0 \text{ mm}$

Design buckling resistance

$$N_{bRd} = \frac{\chi_1*A_{stf}*f_{ysb}}{\gamma_{M1}*1000} = 4483.8 \text{ kN}$$

As $N_{bEd} \leq N_{bRd}$ ($2293 \text{ kN} \leq 4483.8 \text{ kN}$), the design bearing reaction at the support is less than the design buckling resistance. Hence satisfactory.

**Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)**

Ratio $(h_{sb}/t_{sb}) = h_{sb}/t_{sb} = 11.667$

Limit to ratio $p = (h_{sb}/t_{sb})$ is $13\varepsilon = 12.242$.

Ratio $(h_{sb}/t_{sb}) = 11.667$ is not greater than limit $13\varepsilon = 12.242$.

Stiffener complies with Clause 9.2.1(8).
DESIGN SUMMARY

Span of plate girder              30 m
Design moment                     18909 kNm
All steel to be                   Grade S 275

End stiffener:

Posts 1 is a double-sided stiffener at end of girder.

Double-sided stiffeners: post 1 2No/280 mm x 24 mm flats

Intermediate stiffeners:

Intermediate stiffeners:
Arrangement (1) Double-sided stiffeners
2No/80 mm x 15 mm flats
spaced at 3900 mm centres

Intermediate stiffeners:
Arrangement (2) Double-sided stiffeners
2No/80 mm x 15 mm flats
spaced at 5650 mm centres

Assume non-rigid end posts to EN 1993-1-5, Figure 5.1 & Table 5.1.
Location: Ex8 - Single-sided intermediate stiffeners

Stiffened plate girder

The girder is fully restrained along its length. The design calculations are in accordance with BS EN 1993-1-1 and BS EN 1993-1-5. Loads and moments are factored.

Assume non-rigid end posts to BS EN 1993-1-5, Figure 5.1 & Table 5.1.
Span of girder L=12 m
Maximum design bending moment MyEd=2800 kNm
Design bearing reaction NbEd=1450 kN
Depth of girder h=1240 mm
Web thickness tw=12 mm
Stiffener corner snipe snp=15 mm

Dimensions of web and flanges

Stiffener spacing of first panel ae=1550 mm
Intermediate stiffener spacing ai=5000 mm
Accompanying design shear VzEd'=0 kN

Shear buckling resistance - EN 1993-1-1, Clause 6.2.6(6)

As hw/tw > 72*ew (99.167 > 66.558), the web shear buckling resistance needs to be checked (EN 1993-1-5 Clause 5.2 and 5.3).
Buckling coefficient (Annex A) kt=5.34+4*(hw/a)^2=7.6977
Modified slenderness λ lamw=hw/(37.4*tw*ew*SQR(kt))=1.0338
Shear resistance of web VplRd=2184.8 kN
Reduced design shear resistance at support (web contribution only will be considered):

VbwRd=chiw*fyw*hw*tw/(3^0.5*gamM1*1000)=1820.3 kN

Design shear at support VEd(0)=NbEd=1450 kN
As VEd(0) ≤ VbwRd (1450 kN ≤ 1820.3 kN), the shear force is less than the shear resistance. Hence section OK.
compute comp=eta1+(1-MfRd/MplRd)*(2*eta3 -1)^2=0.78202
As comp ≤ 1 (0.78202 ≤ 1), interaction equation is satisfied.
Intermediate transverse web stiffeners - Arrangement (1)

Panel length 1 \( a(1) = 5000 \text{ mm} \)
Length of stiffener \( ls = 1190 \text{ mm} \)

Stress at top of web plate \( \sigma_{\text{top}} = 230.45 \text{ N/mm}^2 \)
Stress at btm of web plate \( \sigma_{\text{btm}} = -230.45 \text{ N/mm}^2 \)

Design parameters
Stiffener outstand \( hs_1 = 150 \text{ mm} \)
Stiffener thickness \( ts_1 = 15 \text{ mm} \)

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)
Yield strength to be adopted \( f_y = 275 \text{ N/mm}^2 \)
Ratio \( p = \frac{hs_1}{ts_1} = 10 \)
Limit to ratio \( p = \frac{hs_1}{ts_1} = 13r = 12.017 \).
Ratio \( p = 10 = 13r = 12.017 \).
Stiffener complies with Clause 9.2.1(8).
Sample output for SCALE Proforma 470. (ans=8)  
Steel design to BS5950-1:2000 and Eurocode 3  
Stiffened plate girder  
Copyright 1986-2019 Fitzroy Systems Ltd.  

Steel design to BS5950-1:2000 and Eurocode 3  
Made by: IFB  
Date: 02/12/19  
Ref No: SC470 EC

**Design shear resistance (single-sided transverse stiffener)**

The effective cross section of the intermediate single-sided transverse stiffener is as shown below:

```
  tsi                  z
  15.ê.tw             15.ê.tw
    tw
    hsi

PLAN ON INTERMEDIATE STIFFENER
```

2nd moment of area yy axis \( I_{yy} = I_{pro} = 13.86 \times 10^6 \text{ mm}^4 \)

Design shear resistance \( V_{b,Rd} = \chi A_e f_{yw}/(\gamma_M 1 \times 1000) = 1747.2 \text{ kN} \)

**Axial force in the stiffener due to shear - BS EN 1993-1-5**

Shear buckling coefficient for web panel \( k_t \) is derived from Clause 5.2.

Buckling coefficient (Annex A) \( k_t = 5.34 + 4 \times (h_{w/a})^2 = 5.5666 \)

Modified slenderness \( \lambda_w \) \( \lambda_w = 0.76 \times (f_{yw}/t_{cr})^{0.5} = 1.2159 \)

Max design shear force in panel \( V_{Ed1} = 1200 \text{ kN} \)

Externally applied design load \( P_{ext1} = 975 \text{ kN} \)

Option 1:

Longitudinal compressive stress \( \sigma_{x,Ed} = 0 \) for symmetrical sections.

Vertical force generated in stiffener arising from the shear induced tension field (see older version of NA to BS EN 1993-1-5:2006):

\( P_{Ed1}' = V_{Ed1} - h_w \times t_w \times 0.8 \times t_{cr}/1000 = -27.358 \text{ kN} \)

Option 2:

Vertical force generated in the stiffener as per BS EN 1993-1-5 (see NOTE, Clause 9.3.3(3)):

\( P_{Ed} = V_{Ed1} - f_{yw} \times h_w \times t_w / (SQR(3) \times \gamma_M \times \lambda_w^2 \times 1000) = -333.53 \text{ kN} \)

Design axial load in stiffener \( P_{Ed} = P_{Ed1} + P_{ext1} = 975 \text{ kN} \)

As \( P_{Ed} \leq V_{b,Rd} \) (975 kN \( \leq 1747.2 \text{ kN} \)), the design shear force does not exceed the shear resistance. Hence OK.

**Connection to the web**

Design shear load on weld \( F_{w,Ed} = t_w^2/(5 \times h_{s1}) + P_{Ed}/(h_w - 2 \times s_{np}) = 1.0325 \text{ kN/mm} \)

Weld size provided \( s_{w1} = 6 \text{ mm} \)

**Stiffness check - Clause 9.3.3(3)**

2nd moment of area (effe. section) \( I_{pro} = 13.86 \times 10^6 \text{ mm}^4 \)

2nd moment of area required \( I_{req} = 0.75 \times h_w \times t_w^3 = 1.5422 \times 10^6 \text{ mm}^4 \)

As \( I_{req} \leq I_{pro} \) (1.5422 \times 10^6 \text{ mm}^4 \( \leq 13.86 \times 10^6 \text{ mm}^4 \)), the stiffener is OK.
Web check between stiffeners

\[ w' \text{ kN/m} \]

Girder assumed to have non-rigid end posts.

UDL applied between stiffeners \( w'=100 \text{ kN/m} \) (factored load)

Largest panel length \( a'=5000 \text{ mm} \)

Compressive stress of web \( \text{sed}=w'/tw=8.3333 \text{ N/mm}^2 \)

Compressive strength of web \( \text{fed}=(2.75+2/(a'/hw)^2)*E/(C/tw)^2 \)

\( =63.687 \text{ N/mm}^2 \)

Since \( \text{sed} \leq \text{fed} \) (8.3333 N/mm\(^2\) \leq 63.687 N/mm\(^2\)), the web is OK.

Selected fillet weld size \( s=6 \text{ mm} \)

Intermediate transverse web stiffeners - Arrangement (2)

Panel length 1 \( a(3)=4000 \text{ mm} \)

Stiffener outstand \( hs2=100 \text{ mm} \)

Stiffener thickness \( ts2=10 \text{ mm} \)

Limiting outstand to prevent torsional buckling - Clause 9.2.1(8)

Yield strength to be adopted \( f_y=275 \text{ N/mm}^2 \)

Ratio \( (hs2/ts2) \)

\( p=hs2/ts2=10 \)

Limit to ratio \( p=(hs2/ts2) \) is 13\( r=12.017 \).

Ratio \( (hs2/ts2)=10 \) is not greater than limit 13\( r=12.017 \).

Stiffener complies with Clause 9.2.1(8).

Design shear resistance (single-sided transverse stiffener)

The effective cross section of the intermediate single-sided transverse stiffener is as shown below:

2nd moment of area yy axis \( I_{yy}=I_{pro}=3.4054E6 \text{ mm}^4 \)

Design shear resistance \( V_{bRd}=\chi f_{yw}/(\gamma M1*1000)=1263 \text{ kN} \)
Axial force in the stiffener due to shear

Max design shear force in panel \( V_{Ed2} = 1500 \text{ kN} \)
Externally applied design load \( P_{ext2} = 800 \text{ kN} \)
Generated force to be adopted \( P_{Ed2} = 244.62 \text{ kN} \)
Design axial load in stiffener \( P_{Ed} = P_{Ed2} + P_{ext2} = 1044.6 \text{ kN} \)
As \( P_{Ed} \leq V_{bRd} (1044.6 \text{ kN} \leq 1263 \text{ kN}) \), the design shear force does not exceed the shear resistance. Hence OK.

Connection to the web

Design shear load on weld \( F_{wEd} = \frac{t_w^2}{5*hs} + \frac{P_{Ed}}{(hw-2*snp)} = 1.1885 \text{ kN/mm} \)
Weld size provided \( sw^2 = 6 \text{ mm} \)

Stiffness check - Clause 9.3.3(3)

2nd moment of area (eff.section) \( I_{pro} = 3.4054E6 \text{ mm}^4 \)
2nd moment of area required \( I_{req} = 0.75*hw*t_w^3 = 1.5422E6 \text{ mm}^4 \)
As \( I_{req} \leq I_{pro} (1.5422E6 \text{ mm}^4 \leq 3.4054E6 \text{ mm}^4) \), the stiffener is OK.

DESIGN SUMMARY

Span of plate girder \( 12 \text{ m} \)
Design moment \( 2800 \text{ kNm} \)
All steel to be \( \text{Grade S 275} \)

Intermediate stiffeners
\( ai = 5000 \text{ mm} \)

Intermediate stiffeners:
Arrangement (1) Single-sided stiffeners
150 mm x 15 mm flats
spaced at 5000 mm centres

Intermediate stiffeners:
Arrangement (2) Single-sided stiffeners
100 mm x 10 mm flats
spaced at 4000 mm centres

- Fillet welds to be used for connecting stiffeners to girder web & flanges
- Weld sizes as detailed in calculations
- Weld design strength to be 222 N/mm²

Assume non-rigid end posts to EN 1993-1-5, Figure 5.1 & Table 5.1.
Location: Ex1 - Based on Example 14 Guide to BS5950 - end bay

Design of Structural Tee

Calculations are in accordance with BS5950-1:2000.

All loads and moments are factored.

Factored moment about x-x axis \( M_x = 6 \, \text{kNm} \)
Factored axial load (+ comp) \( F_{ax} = 200 \, \text{kN} \)
Factored shear force \( F_v = 22 \, \text{kN} \)
Length of member \( L = 2.83 \, \text{m} \)
146 x 127 x 22 UKB.
Young's Modulus \( E = 205 \, \text{kN/mm}^2 \)

Length between restraints x-axis \( L_x = 2830 \, \text{mm} \)
Length between restraints y-axis \( L_y = 1780 \, \text{mm} \)
Length of beam between restraints \( L_T = 1.05 \, \text{m} \)
Quarter point \( m_{x2} = 0.5 \, \text{kNm} \)
Mid-span \( m_{x3} = 3.2 \, \text{kNm} \)
Three quarter point \( m_{x4} = 1.6 \, \text{kNm} \)
Maximum moment on central half \( M_{24} = 4 \, \text{kNm} \)

DESIGN SUMMARY

Factored axial load \( 200 \, \text{kN} \)
Compress. resistance \( 441.44 \, \text{kN} \)
Local capacity check \( 0.92394 \leq 1 \)
Maximum moment \( 6 \, \text{kNm} \)
Moment capacity \( 9.1171 \, \text{kNm} \)
Buckling resistance \( 9.1171 \, \text{kNm} \)
Overall buckling chk. \( 0.81831 \leq 1 \)
\( 0.99157 \leq 1 \)
Location: Ex2 - Simply supported beam

Design of Structural Tee

Calculations are in accordance with BS5950-1:2000.

All loads and moments are factored.

Length of member \( L = 4 \text{ m} \)

Dead UDL (including S.W) \( w_d = 8 \text{ kN/m} \)
Imposed UDL \( w_i = 6 \text{ kN/m} \)

229 x 305 x 70 UKB.
Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Length of beam between restraints \( L_T = 4 \text{ m} \)

DESIGN SUMMARY

- Maximum moment \( 41.6 \text{ kNm} \)
- Moment capacity \( 53.508 \text{ kNm} \)
- Buckling resistance \( 53.508 \text{ kNm} \)
Location: Ex1 - Continuous tee beam

Design of Structural Tee

Calculations are in accordance with BS EN 2003-1-1:2005.

All loads and moments are factored.

Design moment about y-y axis \[ M_{Ed} = 6 \text{ kNm} \]
Design shear force \[ V_{zEd} = 22 \text{ kN} \]
Design axial load (+ comp) \[ N_{Ed} = 200 \text{ kN} \]
Length of member \[ L = 2.83 \text{ m} \]
146 x 127 x 22 Advance UKT split from Advance UKB
Length between restraints \[ L_{T} = 1.05 \text{ m} \]
Far end bending moment \[ pM(1) = 0 \text{ kNm} \]
Far end bending moment \[ pM(2) = 0 \text{ kNm} \]

DESIGN SUMMARY

Design axial load \[ 200 \text{ kN} \]
Buckling resistance \[ 442.13 \text{ kN} \]
Maximum moment \[ 6 \text{ kNm} \]
Moment resistance \[ 9.1312 \text{ kNm} \]
Buckling resistance \[ 9.1312 \text{ kNm} \]
Interaction factors \[ 0.94519 \leq 1 \]
\[ 0.97251 \leq 1 \]
Location: Ex2 - Simply supported tee beam

**Design of Structural Tee**

Calculations are in accordance with BS EN 2003-1-1:2005.

All loads and moments are factored.

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum moment</td>
<td>39.6 kNm</td>
</tr>
<tr>
<td>Moment resistance</td>
<td>88.257 kNm</td>
</tr>
<tr>
<td>Buckling resistance</td>
<td>88.257 kNm</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.44869 ≤ 1</td>
</tr>
</tbody>
</table>

Length of member: L = 4 m

Permanent UDL (including S.W): wd = 8 kN/m

Variable UDL: wi = 6 kN/m

229 x 305 x 70 Advance UKT split from Advance UKB

Length between restraints: LT = 4 m
Location: Ex3 - Strut member

Design of Structural Tee

Calculations are in accordance with BS EN 2003-1-1:2005.

All loads and moments are factored.

Factored axial load (+ comp) \( N_{Ed} = 200 \text{ kN} \)
Length of member \( L = 2.83 \text{ m} \)

203 x 102 x 23 Advance UKT split from Advance UKC

**DESIGN SUMMARY**

- Design axial load \( 200 \text{ kN} \)
- Buckling resistance \( 304.79 \text{ kN} \)
- Interaction factor \( 0.65619 \leq 1 \)
Location: Ex4 - Tie member with no moment

Design of Structural Tee

Calculations are in accordance with BS EN 2003-1-1:2005.

All loads and moments are factored.

Factored axial load (+ comp)  \( N_{Ed} = -200 \) kN
Design moment about y-y axis  \( M_{yEd} = 0 \) kNm
Length of member  \( L = 2.83 \) m

146 x 127 x 22 Advance UKT split from Advance UKB
Estimated net area (tension)  \( A_e = 26 \) cm\(^2\)

**DESIGN SUMMARY**

Design axial load  -200 kN
Tension resistance  715 kN
Interaction factor  \( 0.27972 \leq 1 \)
Location: Ex4 - Tie member with moment

Design of Structural Tee

Calculations are in accordance with BS EN 2003-1-1:2005.

All loads and moments are factored.

Factored axial load (+ comp)  \( N_{Ed} = -200 \text{ kN} \)
Design moment about y-y axis  \( M_{yEd} = 5 \text{ kNm} \)
Length of member  \( L = 2.83 \text{ m} \)

146 x 127 x 22 Advance UKT split from Advance UKB
Estimated net area (tension)  \( A_e = 26 \text{ cm}^2 \)

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design axial load</td>
<td>-200 \text{ kN}</td>
</tr>
<tr>
<td>Tension resistance</td>
<td>715 \text{ kN}</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.8273 ( \leq 1 )</td>
</tr>
</tbody>
</table>
Location: Ex6 - Continuous tee beam cut from UC section

Design of Structural Tee

Calculations are in accordance with BS EN 2003-1-1:2005.

All loads and moments are factored.

Design moment about y-y axis \( \text{MyEd}=6 \text{ kNm} \)
Design shear force \( \text{VzEd}=22 \text{ kN} \)
Design axial load (+ comp) \( \text{NEd}=200 \text{ kN} \)
Length of member \( \text{L}=2.83 \text{ m} \)
254 x 127 x 37 Advance UKT split from Advance UKC
Length between restraints \( \text{LT}=1.05 \text{ m} \)
Far end bending moment \( \text{pM}(1)=0 \text{ kNm} \)
Far end bending moment \( \text{pM}(2)=0 \text{ kNm} \)

**DESIGN SUMMARY**

- **Design axial load**: 200 kN
- **Buckling resistance**: 627.06 kN
- **Maximum moment**: 6 kNm
- **Moment resistance**: 10.768 kNm
- **Buckling resistance**: 10.768 kNm
- **Interaction factors**: 0.71727 ≤ 1
  0.70969 ≤ 1
Simply Supported Crane Gantry Girder

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by The Steel Construction Institute. The beam is assumed to be a rolled universal section.

Crane capacity \( W_{\text{cap}} = 100 \text{ kN} \)
Weight of crab \( W_{\text{cb}} = 20 \text{ kN} \)
Weight of crane bridge \( W_{\text{c}} = 80 \text{ kN} \)
Self weight of gantry girder \( W_G = 15 \text{ kN} \)
Span of crane bridge \( L_c = 15 \text{ m} \)
Wheel spacing in end carriage \( \alpha_w = 4 \text{ m} \)
Minimum hook approach \( \alpha_h = 1 \text{ m} \)
Span of gantry girder \( L = 8 \text{ m} \)
Impact factor \( \text{Impfac} = 1.3 \)
610 x 229 x 125 UB.
Modulus of elasticity \( E = 205 \text{ kN/mm}^2 \)
Weld leg length \( s = 6 \text{ mm} \)
Assumed rail height \( H_r = 100 \text{ mm} \)
Force applied through flange \( F_{\text{bw}} = 247.62 \text{ kN} \)
Stiff bearing length \( b_1 = 75 \text{ mm} \)
Distance to nearer end \( b_e = 0 \text{ mm} \)
Distance to nearer end of member \( a_e = 35 \text{ mm} \)

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\[ 610 \times 229 \times 125 \text{ UB} \quad \text{Grade S 275} \]
with top plate 300 mm x 15 mm
connected together by 6 mm weld

DESIGN

Max. vertical moment \( 376.68 \text{ kNm} \)
Buckling resistance \( 658.77 \text{ kNm} \)
Max. horizontal moment \( 22.8 \text{ kNm} \)
Moment capacity flange \( 74.25 \text{ kNm} \)
Unity check \( 0.85462 \)
\( 0.68188 \)
Max. shear \( 247.62 \text{ kN} \)
Shear capacity \( 1158.3 \text{ kN} \)
Vertical deflection \( 4.0852 \text{ mm} \)
Limiting deflection \( 13.333 \text{ mm} \)
Surge deflection \( 6.3595 \text{ mm} \)
Limiting deflection \( 16 \text{ mm} \)
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied flange force</td>
<td>247.62 kN</td>
</tr>
<tr>
<td>Local web capacity</td>
<td>456.84 kN</td>
</tr>
<tr>
<td>Web buckling capacity</td>
<td>273.24 kN</td>
</tr>
</tbody>
</table>
Location: Example using parallel flange channel section

Simply Supported Crane Gantry Girder

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by The Steel Construction Institute. The beam is assumed to be a rolled universal section.

---

![Diagram of crane gantry girder]

Crane capacity $W_{cap}=100 \text{ kN}$
Weight of crab $W_{cb}=20 \text{ kN}$
Weight of crane bridge $W_c=80 \text{ kN}$
Self weight of gantry girder $W_G=15 \text{ kN}$
Span of crane bridge $L_c=15 \text{ m}$
Wheel spacing in end carriage $a_w=4 \text{ m}$
Minimum hook approach $a_h=1 \text{ m}$
Span of gantry girder $L=8 \text{ m}$
Impact factor $\text{Impfac}=1.3$

**610 x 229 x 125 UB.**

380 x 100 Parallel Flange Channel.

**SUMMARY**

Max. vertical moment $376.68 \text{ kNm}$
Buckling resistance $930.74 \text{ kNm}$
Max. horizontal moment $22.8 \text{ kNm}$
Moment capacity flange $247.25 \text{ kNm}$
Unity check $0.461$
$0.45842$
Max. shear $247.62 \text{ kN}$
Shear capacity $1158.3 \text{ kN}$
Vertical deflection $3.9133 \text{ mm}$
Limiting deflection $13.333 \text{ mm}$
Surge deflection $1.4309 \text{ mm}$
Limiting deflection $16 \text{ mm}$

---

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610 x 229 x 125 UB Grade S 275 with 380 x 100 Parallel Flange Channel connected together by 6 mm weld

---
Location: Example using UB only

Simply Supported Crane Gantry Girder

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by The Steel Construction Institute. The beam is assumed to be a rolled universal section.

Crane capacity \( W_{\text{cap}} = 80 \text{ kN} \)
Weight of crab \( W_{\text{cb}} = 16 \text{ kN} \)
Weight of crane bridge \( W_{\text{c}} = 70 \text{ kN} \)
Self weight of gantry girder \( W_{\text{G}} = 12 \text{ kN} \)
Span of crane bridge \( L_{\text{c}} = 12 \text{ m} \)
Wheel spacing in end carriage \( a_{\text{w}} = 3 \text{ m} \)
Minimum hook approach \( a_{\text{h}} = 1 \text{ m} \)
Span of gantry girder \( L = 8 \text{ m} \)
Impact factor \( \text{Impfac} = 1.1 \)

610 x 229 x 125 UB.
Modulus of elasticity \( E = 205 \text{ kN/mm}^2 \)
Assumed rail height \( H_r = 110 \text{ mm} \)
Force applied through flange \( F_{\text{bw}} = 184.29 \text{ kN} \)
Stiff bearing length \( b_1 = 80 \text{ mm} \)
Distance to nearer end \( b_e = 0 \text{ mm} \)
Distance to nearer end of member \( a_{\text{e}} = 30 \text{ mm} \)

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<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>610 x 229 x 125 UB Grade S 275</td>
<td>Max. vertical moment ( 302.62 \text{ kNm} )</td>
</tr>
<tr>
<td></td>
<td>Buckling resistance ( 361.82 \text{ kNm} )</td>
</tr>
<tr>
<td></td>
<td>Max. horizontal moment ( 5.07 \text{ kNm} )</td>
</tr>
<tr>
<td></td>
<td>Moment capacity flange ( 54.476 \text{ kNm} )</td>
</tr>
<tr>
<td></td>
<td>Unity check ( 0.876 )</td>
</tr>
<tr>
<td></td>
<td>( 0.42761 )</td>
</tr>
<tr>
<td></td>
<td>Max. shear ( 184.29 \text{ kN} )</td>
</tr>
<tr>
<td></td>
<td>Shear capacity ( 1158.3 \text{ kN} )</td>
</tr>
<tr>
<td></td>
<td>Vertical deflection ( 5.2928 \text{ mm} )</td>
</tr>
<tr>
<td></td>
<td>Limiting deflection ( 13.333 \text{ mm} )</td>
</tr>
<tr>
<td></td>
<td>Surge deflection ( 10.383 \text{ mm} )</td>
</tr>
<tr>
<td></td>
<td>Limiting deflection ( 16 \text{ mm} )</td>
</tr>
<tr>
<td></td>
<td>Applied flange force ( 184.29 \text{ kN} )</td>
</tr>
<tr>
<td></td>
<td>Local web capacity ( 473.2 \text{ kN} )</td>
</tr>
<tr>
<td></td>
<td>Web buckling capacity ( 192.33 \text{ kN} )</td>
</tr>
</tbody>
</table>
**Location: Ex1 - Using an I-beam section**

**Simply supported crane gantry girder**

Calculations are in accordance with EC3 Part 1-1 and Part 6. The beam is assumed to be a rolled universal section.

Crane capacity: \( W_{\text{cap}} = 100 \text{ kN} \)

Weight of crab: \( W_{\text{cb}} = 20 \text{ kN} \)

Weight of crane bridge: \( W_{c} = 80 \text{ kN} \)

Self weight of gantry girder: \( W_{G} = 15 \text{ kN} \)

Span of crane bridge: \( L_{c} = 15 \text{ m} \)

Wheel spacing in end carriage: \( a_{w} = 4 \text{ m} \)

Minimum hook approach: \( a_{h} = 1 \text{ m} \)

Span of gantry girder: \( L = 8 \text{ m} \)

Dynamic magnification factor: \( \text{impfac} = 1.3 \)

Partial factor (permanent action): \( \gamma_{G} = 1.35 \)

Partial factor (variable action): \( \gamma_{Q} = 1.5 \)

610 x 229 x 125 UKB

Correction factor: \( k_{c} = 1 \)

Weld leg length: \( s = 6 \text{ mm} \)

Assumed rail height: \( H_{r} = 100 \text{ mm} \)

Stiff bearing length: \( s_{s} = 75 \text{ mm} \)

Stiff bearing position: \( C = 0 \text{ mm} \)

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610 x 229 x 125 UB Grade S 275 with top plate 300 mm x 15 mm connected together by 6 mm FW

Design vertical moment: \( 353.7 \text{ kNm} \)

Buckling resistance: \( 674.48 \text{ kNm} \)

Design horiz. moment: \( 21.375 \text{ kNm} \)

Flange moment resist.: \( 61.875 \text{ kNm} \)

Design shear force: \( 232.43 \text{ kN} \)

Design shear resistance: \( 1171.1 \text{ kN} \)

Vertical deflection: \( 3.9879 \text{ mm} \)

Limiting deflection: \( 13.333 \text{ mm} \)

Surge deflection: \( 6.2081 \text{ mm} \)

Limiting deflection: \( 16 \text{ mm} \)

Support shear force: \( 232.43 \text{ kN} \)

Web bear. & buckl. res: \( 485.54 \text{ kN} \)
Interaction factors

\[ 0.86986 \leq 1 \]
\[ 0.72177 \leq 1 \]
\[ 0.42883 \leq 1 \]
Location: Ex2 - Using parallel flange channel section

Simply supported crane gantry girder

Calculations are in accordance with EC3 Part 1-1 and Part 6. The beam is assumed to be a rolled universal section.

Crane capacity, Wcap = 100 kN
Weight of crab, Wcb = 20 kN
Weight of crane bridge, Wc = 80 kN
Self weight of gantry girder, WG = 15 kN
Span of crane bridge, Lc = 15 m
Wheel spacing in end carriage, aw = 4 m
Minimum hook approach, ah = 1 m
Span of gantry girder, L = 8 m
Dynamic magnification factor, impfac = 1.3
Partial factor (permanent action), gamG = 1.35
Partial factor (variable action), gamQ = 1.5
610 x 229 x 125 UKB
380 x 100 UKPFC
Correction factor, kc = 1
Weld leg length, s = 6 mm
Assumed rail height, Hr = 100 mm

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DESIGN SUMMARY
610 x 229 x 125 UB  Grade S 275
with 380 x 100
Parallel Flange Channel
connected together by 6 mm FW
Design vertical moment, 353.7 kNm
Buckling resistance, 852.09 kNm
Design horiz. moment, 21.375 kNm
Flange moment resist., 236.62 kNm
Design shear force, 232.43 kN
Design shear resistance, 1171.1 kN
Vertical deflection, 3.8201 mm
Limiting deflection, 13.333 mm
Surge deflection, 1.3968 mm
Limiting deflection, 16 mm
Interaction factors, 0.50543 ≤ 1
0.46775 ≤ 1
0.17257 ≤ 1
Location: Ex3 - Using UB only

Simply supported crane gantry girder

Calculations are in accordance with EC3 Part 1-1 and Part 6. The beam is assumed to be a rolled universal section.

Crane capacity  
Weight of crab  
Weight of crane bridge  
Self weight of gantry girder  
Span of crane bridge  
Wheel spacing in end carriage  
Minimum hook approach  
Span of gantry girder  
Dynamic magnification factor  
Partial factor (permanent action)  
Partial factor (variable action)  
610 x 229 x 125 UKB  
Correction factor  
Assumed rail height  
Stiff bearing length  
Stiff bearing position

UNIVERSAL BEAM  
DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crane capacity</td>
<td>Wcap = 80 kN</td>
</tr>
<tr>
<td>Weight of crab</td>
<td>Wcb = 16 kN</td>
</tr>
<tr>
<td>Weight of crane bridge</td>
<td>Wc = 70 kN</td>
</tr>
<tr>
<td>Self weight of gantry girder</td>
<td>WG = 12 kN</td>
</tr>
<tr>
<td>Span of crane bridge</td>
<td>Lc = 12 m</td>
</tr>
<tr>
<td>Wheel spacing in end carriage</td>
<td>aw = 3 m</td>
</tr>
<tr>
<td>Minimum hook approach</td>
<td>ah = 1 m</td>
</tr>
<tr>
<td>Span of gantry girder</td>
<td>L = 8 m</td>
</tr>
<tr>
<td>Dynamic magnification factor</td>
<td>impfac = 1.1</td>
</tr>
<tr>
<td>Partial factor (permanent action)</td>
<td>gamG = 1.35</td>
</tr>
<tr>
<td>Partial factor (variable action)</td>
<td>gamQ = 1.5</td>
</tr>
<tr>
<td>610 x 229 x 125 UB</td>
<td>Grade S 275</td>
</tr>
<tr>
<td>Correction factor</td>
<td>kc = 1</td>
</tr>
<tr>
<td>Assumed rail height</td>
<td>Hr = 110 mm</td>
</tr>
<tr>
<td>Stiff bearing length</td>
<td>ss = 80 mm</td>
</tr>
<tr>
<td>Stiff bearing position</td>
<td>C = 0 mm</td>
</tr>
</tbody>
</table>
Wind-moment design of low rise frames

Calculations are in accordance with BS5950 and SCI publication P-263 entitled 'Wind-moment Design of Low Rise Frames'. The wind-moment design method will be adopted here which applies to low rise frames of four storeys or less and assumes that:

- under vertical loads the connections act as simple nominally pinned connections
- under wind loads the connections behave as nominally rigid joints. Points of contraflexure are assumed to occur at the mid-height of the columns and the mid-length of the beams
- with the wind-moment design method the end-plate connections should be sufficiently ductile to accommodate large rotations. Connections are required to resist the moments due to wind loading as well as the shear forces due to vertical loading
- typical connections with sufficient ductility are given in the BCSA/SCI publication 'Joints in steel construction: Moment connections'. Capacity tables have been reproduced in Appendix D of SCI-P-263
- second order effects due to frame sway (i.e. P-Δ effects) are allowed for by using effective lengths for the columns that are greater than the lengths between floor levels
- columns should be rigidly connected to the foundations, by bases that are designed to resist moments
- any recognised method could be used to determine the sway deflections (see design summary). Sway deflections in each storey should be within the limit of Height/300.

Frame dimensions - Table 1.1 SCI-P-263

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Number of storeys</td>
<td>storey=4</td>
</tr>
<tr>
<td>Number of bays</td>
<td>bay=4</td>
</tr>
<tr>
<td>Storey height (bottom storey)</td>
<td>Ht=5 m</td>
</tr>
<tr>
<td>Storey height (elsewhere)</td>
<td>ht=4 m</td>
</tr>
<tr>
<td>Parapet height</td>
<td>hp=1.55 m</td>
</tr>
<tr>
<td>Frame centres</td>
<td>crs=6 m</td>
</tr>
<tr>
<td>Number of frames</td>
<td>no=9</td>
</tr>
</tbody>
</table>

Bay widths

<table>
<thead>
<tr>
<th>Bay width</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of bay 1</td>
<td>L1=6 m</td>
</tr>
<tr>
<td>Width of bay 2</td>
<td>L2=6 m</td>
</tr>
<tr>
<td>Width of bay 3</td>
<td>L3=6 m</td>
</tr>
<tr>
<td>Width of bay 4</td>
<td>L4=6 m</td>
</tr>
</tbody>
</table>
Loading

Dead load on floors \( \text{deadf}=4.5 \text{ kN/m}^2 \)
Imposed load on floors \( \text{impf}=5 \text{ kN/m}^2 \)
Dead load on roof \( \text{deadr}=4 \text{ kN/m}^2 \)
Imposed load on roof \( \text{impr}=1.5 \text{ kN/m}^2 \)

Dynamic augmentation factor

Chosen site is the Town; built up areas with an average level of roof tops at least \( H_o=5.0 \text{ m} \) above ground level.
The building needs to consider the effects of obstruction height and other buildings proximity.

\[
\begin{align*}
\text{Ho} &= \text{Mean height of other buildings or trees.} \\
X &= \text{Distance to next building} \\
\text{Ho} &= \text{Average adjacent building height} \\
\text{Cr} &= \left( \frac{K_b(H/ho)^{0.75}}{800/lh} \right) = 0.0277 \\
\text{Cr} &< 0.25 \text{ then this structure is not dynamic.}
\end{align*}
\]

Standard wind loads

Basic wind speed \( \text{Vb}=20 \text{ m/s} \)
Site altitude above mean sea level \( \Delta S=112 \text{ m} \)
Directional factor \( \text{Sd}=1 \)
Probability factor \( \text{Sp}=1 \)
From Table 4 terrain & building factor \( \text{Sb}=1.55 \)
Site wind speed @ height \( \text{He} \)

\[
\begin{align*}
\text{Vs} &= \text{Vb} \times \text{Sa} \times \text{Sd} \times \text{Ss} \times \text{Sp} = 22.24 \text{ m/s} \\
\text{Effective height} &= \text{He} = 7.42 \text{ m} \\
\text{Altitude factor} &= \text{Sa} = 1.112 \\
\text{Direction factor} &= \text{Sd} = 1 \\
\text{Seasonal factor} &= \text{Ss} = 1 \\
\text{Probability factor} &= \text{Sp} = 1 \\
\text{Dynamic wind pressure} &= q_S = 0.7284 \text{ kN/m}^2
\end{align*}
\]
I-column design

Section suitability based on the following loading combinations:

- 1.4(DL) + 1.6(LL) + 1.0(NHF)
- 1.2(DL + LL + WL)
- 1.4(DL + WL)

The effects of patterned loading variations in beam spans and column S.W are ignored.

Loads and moments considered are: Beam loading, moment due to rigid column/beam connection, moment due to beam eccentricity, and moment due to NHF where applicable.

254 x 254 x 89 UC.  
Young's Modulus  E=205 kN/mm²

**UNIVERSAL COLUMN**  
254 x 254 x 89 UC Grade S 275  
Axial load  1548 kN  
Compressive resist.  1857 kN  
Design moment  52.61 kNm  
Moment capacity  314.7 kNm  
Unity factor (d+i)  0.9061 ≤ 1  
Unity factor (d+i+w)  0.843 ≤ 1  
Unity factor (d+w)  0.6699 ≤ 1

**Floor beam design**

Beam span  Lb=6 m

**Design dead plus imposed loading**

Distributed floor load  \( w'=(1.4\times{\text{deadf}}+1.6\times{\text{impf}})\times{\text{crs}}=85.8 \text{ kN/m} \)

Assume the end restraint moment due to gravity load is equal to 10% of the "free" moment.

Design moment  \( M=0.9\times{w'}\times{Lb}^2/8=347.5 \text{ kNm} \)

Design shear force  \( Fv=w'\times{Lb}/2=257.4 \text{ kN} \)

**Section properties**

406 x 178 x 74 UB.  
Dimensions (mm): D=412.8 B=179.5 t=9.5 T=16 r=10.2  
Properties (cm): Ix=27300 Iy=1550 Sx=1500 J=62.8 A=94.5
Strength of steel - Clause 3.1.1

For material thickness 16 mm and steel grade S 275.

Design strength \(py=275 \text{ N/mm}^2\)

Check section for moment and shear

Shear capacity \(P_v=0.6*py*Av/10^3=647.1 \text{ kN}\)
Design shear force \(F_v=257.4 \text{ kN}\)
Moment capacity for compact sec \(M_c=0.9*py*S_x/10^3=371.3 \text{ kNm}\)

Since \(M \leq M_c\) (347.5 kNm \leq 371.3 kNm), design moment within moment capacity.

Check for deflection

UDL for deflection calculation \(w_u=\text{impf*crs}=30 \text{ kN/m}\)
Total deflection \(D E L=(5*w_u*L_b^4/384)*10^5/(E*I_x) = 9.046 \text{ mm}\)
Limiting deflection \(D E L_{lim}=L_b*10^3/360=16.67 \text{ mm}\)

Since \(D E L \leq D E L_{lim}\) (9.046 mm \leq 16.67 mm), deflection is within the limiting value.

UNIVERSAL BEAM 406 x 178 x 74 UB Grade S 275
Design shear force 257.4 kN
Shear capacity 647.1 kN
Design moment 347.5 kNm
Moment capacity 371.3 kNm
Deflection 9.046 mm
Limiting deflection 16.67 mm

The frame should now be checked for sidesway using unfactored wind loads only and rigidly connected columns to the foundations. The calculated rigid frame deflections should then be increased by 50% (i.e. multiplied by a factor of 1.5) to allow for the flexibility of standard wind-moment connections (provided that the average bay width is at least 6.0 m). For an average bay width of 4.5 m, the deflections from the rigid frame analysis should be multiplied by a factor of 2.0. For bay widths between 4.5 m and 6.0 m, linear interpolation between 2.0 and 1.5 should be used. If the increased values are acceptable, then the design is considered complete.
Location: Ex2 - End column

Wind-moment design of low rise frames

Calculations are in accordance with BS5950 and SCI publication P-263 entitled 'Wind-moment Design of Low Rise Frames'. The wind-moment design method will be adopted here which applies to low rise frames of four storeys or less and assumes that:

- under vertical loads the connections act as simple nominally pinned connections
- under wind loads the connections behave as nominally rigid joints. Points of contraflexure are assumed to occur at the mid-height of the columns and the mid-length of the beams
- with the wind-moment design method the end-plate connections should be sufficiently ductile to accommodate large rotations. Connections are required to resist the moments due to wind loading as well as the shear forces due to vertical loading
- typical connections with sufficient ductility are given in the BCSA/SCI publication 'Joints in steel construction: Moment connections'. Capacity tables have been reproduced in Appendix D of SCI-P-263
- second order effects due to frame sway (i.e. P-Δ effects) are allowed for by using effective lengths for the columns that are greater than the lengths between floor levels
- columns should be rigidly connected to the foundations, by bases that are designed to resist moments
- any recognised method could be used to determine the sway deflections (see design summary). Sway deflections in each storey should be within the limit of Height/300.

Frame dimensions - Table 1.1 SCI-P-263

<table>
<thead>
<tr>
<th>Number of storeys</th>
<th>storey=4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bays</td>
<td>bay=4</td>
</tr>
<tr>
<td>Storey height (bottom storey)</td>
<td>Ht=5 m</td>
</tr>
<tr>
<td>Storey height (elsewhere)</td>
<td>ht=4 m</td>
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<td>crs=6 m</td>
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<td>Number of frames</td>
<td>no=9</td>
</tr>
</tbody>
</table>

Bay widths

<table>
<thead>
<tr>
<th>Width of bay 1</th>
<th>L1=6 m</th>
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</thead>
<tbody>
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Loading

Dead load on floors \( \text{deadf}=4.5 \text{ kN/m}^2 \)
Imposed load on floors \( \text{impf}=5 \text{ kN/m}^2 \)
Dead load on roof \( \text{deadr}=4 \text{ kN/m}^2 \)
Imposed load on roof \( \text{impr}=1.5 \text{ kN/m}^2 \)

Dynamic augmentation factor

Chosen site is the Town; built up areas with an average level of roof tops at least \( Ho=5.0 \text{ m} \) above ground level.
The building needs to consider the effects of obstruction height and other buildings proximity.

\[ Ho=\text{Mean height of other buildings or trees.} \]

Distance to next building \( X=8 \text{ m} \)
Average adjacent building height \( Ho=15 \text{ m} \)
Dynamic Augmentation Factor \( Cr=(Kb*(H/ho)^{0.75})/800/lh=0.0277 \)
Since Dynamic Augmentation Factor \( Cr \) is less than or equal to 0.25 then this structure is not dynamic.

Standard wind loads

Basic wind speed \( Vb=20 \text{ m/s} \)
Site altitude above mean sea level \( \delta S=112 \text{ m} \)
Directional factor \( Sd=1 \)
Probability factor \( Sp=1 \)
From Table 4 terrain & building factor \( Sb=1.55 \)
Site wind speed @ height \( He \)
\[ Vs=Vb*Sa*Sp*Sp=22.24 \text{ m/s} \]

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective height</td>
<td>He ( 7.42 \text{ m} )</td>
</tr>
<tr>
<td>Altitude factor</td>
<td>( Sa=1.112 )</td>
</tr>
<tr>
<td>Direction factor</td>
<td>( Sd=1 )</td>
</tr>
<tr>
<td>Seasonal factor</td>
<td>( Ss=1 )</td>
</tr>
<tr>
<td>Probability factor</td>
<td>( Sp=1 )</td>
</tr>
<tr>
<td>Dynamic wind pressure</td>
<td>qs ( 0.7284 \text{ kN/m}^2 )</td>
</tr>
</tbody>
</table>
I-column design

Section suitability based on the following loading combinations:

- $1.4(DL) + 1.6(LL) + 1.0(NHF)$
- $1.2(DL + LL + WL)$
- $1.4(DL + WL)$

The effects of patterned loading variations in beam spans and column S.W are ignored. Loads and moments considered are: Beam loading, moment due to rigid column/beam connection, moment due to beam eccentricity, and moment due to NHF where applicable.

Assumed depth of col. section $D_p=260$ mm
254 x 254 x 89 UC.
Young's Modulus $E=205$ kN/mm²

**UNIVERSAL COLUMN**

- **Axial load**: 773.8 kN
- **Compressive resist.**: 1857 kN
- **Design moment**: 65.29 kNm
- **Moment capacity**: 314.7 kNm
- **Unity factor (d+i)**: $0.6084 \leq 1$
- **Unity factor (d+i+w)**: $0.5608 \leq 1$
- **Unity factor (d+w)**: $0.4898 \leq 1$

**Floor beam design**

- **Beam span**: $L_b=6$ m

**Design dead plus imposed loading**

- **Distributed floor load**: $w'=(1.4*deadf+1.6*impf)*crs=85.8$ kN/m
- **Design moment**: $M=0.9*w'*L_b^2/8=347.5$ kNm
- **Design shear force**: $F_v=w'*L_b/2=257.4$ kN
Section properties

406 x 178 x 74 UB.
Dimensions (mm): D=412.8 B=179.5 t=9.5 T=16 r=10.2
Properties (cm): Ix=27300 Iy=1550 Sx=1500 J=62.8 A=94.5

Strength of steel - Clause 3.1.1

For material thickness 16 mm and steel grade S 275.
Design strength py=275 N/mm²

Check section for moment and shear

Shear capacity
Design shear force
Moment capacity for compact sec
Mc is within limiting value, therefore no reduction.
Since M ≤ Mc (347.5 kNm ≤ 371.3 kNm), design moment within moment capacity.

Check for deflection

UDL for deflection calculation
Total deflection
Limiting deflection
Since DEL ≤ DELlim (9.046 mm ≤ 16.67 mm), deflection is within the limiting value.

The frame should now be checked for sidesway using unfactored wind loads only and rigidly connected columns to the foundations. The calculated rigid frame deflections should then be increased by 50% (i.e. multiplied by a factor of 1.5) to allow for the flexibility of standard wind-moment connections (provided that the average bay width is at least 6.0 m). For an average bay width of 4.5 m, the deflections from the rigid frame analysis should be multiplied by a factor of 2.0. For bay widths between 4.5 m and 6.0 m, linear interpolation between 2.0 and 1.5 should be used. If the increased values are acceptable, then the design is considered complete.
Location: Ex1 - Internal column

Wind-moment design of low rise frames

Calculations are in accordance with EN 1993-1-1:2005 and publication SCI-P-263 entitled 'Wind-moment Design of Low Rise Frames'. The wind-moment design method will be adopted here which applies to low rise frames of four storeys or less and assumes that:

- under vertical loads the connections act as simple nominally pinned connections
- under wind loads the connections behave as nominally rigid joints. Points of contraflexure are assumed to occur at the mid-height of the columns and the mid-length of the beams
- with the wind-moment design method the end-plate connections should be sufficiently ductile to accommodate large rotations. Connections are required to resist the moments due to wind loading as well as the shear forces due to vertical loading
- typical connections with sufficient ductility are given in the BCSA/SCI publication 'Joints in steel construction: Moment connections'. Capacity tables have been reproduced in Appendix D of SCI-P-263
- second order effects due to frame sway (i.e. P-Δ effects) are allowed for by using effective lengths for the columns that are greater than the lengths between floor levels
- columns should be rigidly connected to the foundations, by bases that are designed to resist moments
- any recognised method could be used to determine the sway deflections (see design summary). Sway deflections in each storey should be within the limit of Height/300.

Frame dimensions - Table 1.1 SCI-P-263

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
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<td>bay=4</td>
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<td>Frame centres</td>
<td>crs=6 m</td>
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<tr>
<td>Number of frames</td>
<td>no=9</td>
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</tbody>
</table>

Bay widths

<table>
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<tr>
<th>Bay</th>
<th>Width</th>
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</thead>
<tbody>
<tr>
<td>Bay 1</td>
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<td>L2=6 m</td>
</tr>
<tr>
<td>Bay 3</td>
<td>L3=6 m</td>
</tr>
<tr>
<td>Bay 4</td>
<td>L4=6 m</td>
</tr>
</tbody>
</table>
Loading

Permanent load on floors \( G_{kf} = 4.5 \text{ kN/m}^2 \)
Imposed load on floors \( Q_{kf} = 5 \text{ kN/m}^2 \)
Permanent load on roof \( G_{kr} = 4 \text{ kN/m}^2 \)
Imposed load on roof \( Q_{kr} = 1.5 \text{ kN/m}^2 \)

Obstruction height and upwind spacing

Wind direction

\[ \begin{align*}
\text{Hav} &= \text{Obstruction height of other buildings} \\
\text{Xo} &= \text{building reference height}
\end{align*} \]

\[ \begin{align*}
\text{Hr} &= 18.55 \text{ m} \\
\text{Hd}(0) &= 3.8 \text{ m} \\
\text{Hd}(90) &= 3.8 \text{ m} \\
\text{H} &= 18.55 \text{ m}
\end{align*} \]

Standard wind loads

Basic wind velocity (Fig. NA.1) \( V_{bmap} = 21.5 \text{ m/s} \)
Altitude above mean sea level \( A_{site} = 25 \text{ m} \)
Directional factor (wind \( \theta = 0^\circ \)) \( C_{di}(0) = 1 \)
Directional factor (wind \( \theta = 90^\circ \)) \( C_{di}(90) = 0.73 \)
Distance upwind to shoreline \( D_{is}(0) = 2.5 \text{ km} \)
Distance inside town terrain \( D_{it}(0) = 2.5 \text{ km} \)
Distance upwind to shoreline \( D_{is}(90) = 2.5 \text{ km} \)
Distance inside town terrain \( D_{it}(90) = 2.5 \text{ km} \)
Probability factor \( C_p = 1 \)

| Effective height (long face) | He(0) | 14.75 m |
| Effective height (short face) | He(90) | 14.75 m |
| Altitude factor (long face) | Ca(0) | 1.025 |
| Altitude factor (short face) | Ca(90) | 1.025 |
| Direction factor (long face) | Cdi(0) | 1 |
| Direction factor (short face) | Cdi(90) | 0.73 |
| Seasonal factor | Cseason | 1 |
| Probability factor | C_p | 1 |
| Long face peak velocity pressure | q_p(0) | 0.7823 kN/m² |
| Short face peak velocity pressure | q_p(90) | 0.4169 kN/m² |
Partial factors for actions

Factor for permanent actions \( \gamma_G = 1.25 \)
Factor for imposed actions \( \gamma_Q = 1.5 \)

Column design

Section suitability will be based on the following three loading combinations:

- **Comb1**: \( 1.25(G_k) + 1.5(Q_k) + 1.0(EHF) \)
- **Comb2**: \( 1.25(G_k) + 1.5(Q_k) + 0.75(W_k) + 1.0(EHF) \)
- **Comb3**: \( 1.25(G_k) + 1.5(W_k) + 1.05(Q_k) + 1.0(EHF) \)

The effects of patterned loading and variations in beam spans and S.W of column will be ignored.

Loads/moments considered are:
- Beam loading and moment due to rigid column/beam connection
- Moment due to beam eccentricity and moment due to equivalent horizontal forces (EHF).

Factor \( \psi_0 \) for imposed \( \psi_0 = 0.7 \)
Factor \( \psi_0 \) for wind \( \psi_0 = 0.5 \)

254 x 254 x 89 UKC
Young's Modulus \( E = 210 \ kN/mm^2 \)

UNIVERSAL COLUMN
254 x 254 x 89 UKC Grade S 275
Design axial load \( 1412 \ kN \)
Buckling resist. \( 1866 \ kN \)

SUMMARY
Design moment \( 56.29 \ kNm \)
Moment resistance \( 288.4 \ kNm \)
Unity factor Comb1 \( 0.829 \leq 1 \)
Unity factor Comb2 \( 0.9514 \leq 1 \)
Unity factor Comb3 \( 0.9736 \leq 1 \)

Floor beam design
Beam span \( L_b = 6 \ m \)

Design permanent plus imposed loading
Distributed floor load \( w' = (\gamma_G \cdot G_k + \gamma_Q \cdot Q_k) \cdot c_r s = 78.75 \ kN/m \)
Assume the end restraint moment due to gravity load is equal to 10% of the "free" moment.
Design moment @ beam mid-span \( M_{Ed} = 0.9 \cdot w' \cdot L_b^2/2 = 318.9 \ kNm \)
Design shear force \( V_{zEd} = w' \cdot L_b/2 = 236.3 \ kN \)
Steel beam section properties

406 x 178 x 74 UKB
Dimensions (mm): h=412.8 b=179.5 tw=9.5 tf=16 r=10.2
Properties (cm): Iy=27300 Iz=1550 Wply=1500 It=62.8 A=94.5

Strength of steel beam

For material thickness 16 mm and steel grade S 275.
Design strength                  fy=275 N/mm²

Check section for moment and shear

Shear resistance                  VplRd=Avz*fy/SQR(3)/(gamM0*10^3)
=664.4 kN
Design shear force                VzEd=236.3 kN
Moment resistance                 McRd=0.9*fy*Wply/(gamM0*10^3)
=371.3 kNm
Since MEd ≤ McRd ( 318.9 kNm ≤ 371.3 kNm ), design moment within moment resistance.

Check for deflection

UDL for deflection calculation    wu=Qkf*crs=30 kN/m
Total deflection                  DEL=(5*wu*Lb^4/384)*10^5/(E*Iy)
=8.83 mm
Limiting deflection               DELlim=Lb*10^3/360=16.67 mm
As DEL ≤ DELlim ( 8.83 mm ≤ 16.67 mm ), deflection is OK.

UNIVERSAL BEAM                406 x 178 x 74 UKB Grade S 275
DESIGN                        Design shear force    236.3 kN
Shear resistance               664.4 kN
SUMMARY                       Design moment         318.9 kNm
Moment resistance              371.3 kNm
Deflection                     8.83 mm
Limiting deflection            16.67 mm

The frame should now be checked for sidesway using unfactored wind loads only and rigidly connected columns to the foundations. The calculated rigid frame deflections should then be increased by 50% (i.e. multiplied by a factor of 1.5) to allow for the flexibility of standard wind-moment connections (provided that the average bay width is at least 6.0 m). For an average bay width of 4.5 m, the deflections from the rigid frame analysis should be multiplied by a factor of 2.0. For bay widths between 4.5 m and 6.0 m, linear interpolation between 2.0 and 1.5 should be used. If the increased values are acceptable, then the design is considered complete.
Location: Ex2 - End column

Wind-moment design of low rise frames

Calculations are in accordance with EN 1993-1-1:2005 and publication SCI-P-263 entitled 'Wind-moment Design of Low Rise Frames'. The wind-moment design method will be adopted here which applies to low rise frames of four storeys or less and assumes that:

- under vertical loads the connections act as simple nominally pinned connections
- under wind loads the connections behave as nominally rigid joints. Points of contraflexure are assumed to occur at the mid-height of the columns and the mid-length of the beams
- with the wind-moment design method the end-plate connections should be sufficiently ductile to accommodate large rotations. Connections are required to resist the moments due to wind loading as well as the shear forces due to vertical loading
- typical connections with sufficient ductility are given in the BCSA/SCI publication 'Joints in steel construction: Moment connections'. Capacity tables have been reproduced in Appendix D of SCI-P-263
- second order effects due to frame sway (i.e. P-∆ effects) are allowed for by using effective lengths for the columns that are greater than the lengths between floor levels
- columns should be rigidly connected to the foundations, by bases that are designed to resist moments
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Loading

Permanent load on floors \(G_{kf} = 4.5 \text{ kN/m}^2\)
Imposed load on floors \(Q_{kf} = 5 \text{ kN/m}^2\)
Permanent load on roof \(G_{kr} = 4 \text{ kN/m}^2\)
Imposed load on roof \(Q_{kr} = 1.5 \text{ kN/m}^2\)

Obstruction height and upwind spacing

Wind direction

| \(z=0\) | \(H_{av}\) | \(2H_{av}\) | \(H_{e}\) | \(6H_{av}\)
|---|---|---|---|---|

| \(H_{av}\) | Obstruction height of other buildings

Building reference height \(H_r = 18.55 \text{ m}\)
buildings upwind of site \(H_{av} = 0.001 \text{ m}\)
Upwind spacing of building \(X_0 = 20 \text{ m}\)
Displacement height (wind \(\theta=0^\circ\)) \(H_{d}(0) = 3.8 \text{ m}\)
Displacement height (wind \(\theta=90^\circ\)) \(H_{d}(90) = 3.8 \text{ m}\)
Building height \(H = 18.55 \text{ m}\)

Standard wind loads

Basic wind velocity (Fig. NA.1) \(V_{b_{map}} = 21.5 \text{ m/s}\)
Altitude above mean sea level \(A_{site} = 25 \text{ m}\)
Directional factor (wind \(\theta=0^\circ\)) \(C_{d}(0) = 1\)
Directional factor (wind \(\theta=90^\circ\)) \(C_{d}(90) = 0.73\)
Distance upwind to shoreline \(D_{is}(0) = 2.5 \text{ km}\)
Distance inside town terrain \(D_{it}(0) = 2.5 \text{ km}\)
Distance upwind to shoreline \(D_{is}(90) = 2.5 \text{ km}\)
Distance inside town terrain \(D_{it}(90) = 2.5 \text{ km}\)
Probability factor \(C_p = 1\)

Effective height (long face) \(H_{e}(0) = 14.75 \text{ m}\)
Effective height (short face) \(H_{e}(90) = 14.75 \text{ m}\)
Altitude factor (long face) \(C_{a}(0) = 1.025\)
Altitude factor (short face) \(C_{a}(90) = 1.025\)
Direction factor (long face) \(C_{d}(0) = 1\)
Direction factor (short face) \(C_{d}(90) = 0.73\)
Seasonal factor \(C_{season} = 1\)
Probability factor \(C_p = 1\)
Long face peak velocity pressure \(q_{p}(0) = 0.7823 \text{ kN/m}^2\)
Short face peak velocity pressure \(q_{p}(90) = 0.4169 \text{ kN/m}^2\)
Partial factors for actions

Factor for permanent actions $\gamma_G = 1.35$
Factor for imposed actions $\gamma_Q = 1.5$

Column design

Section suitability will be based on the following three loading combinations:

Comb1: $1.35(G_k) + 1.5(Q_k) + 1.0(EHF)$
Comb2: $1.35(G_k) + 1.5(Q_k) + 0.75(W_k) + 1.0(EHF)$
Comb3: $1.35(G_k) + 1.5(W_k) + 1.05(Q_k) + 1.0(EHF)$

The effects of patterned loading and variations in beam spans and S.W of column will be ignored.

Loads/moments considered are:
- Beam loading and moment due to rigid column/beam connection
- Moment due to beam eccentricity and moment due to equivalent horizontal forces (EHF).

Factor $\psi_0$ for imposed $\psi_{0i} = 0.7$
Factor $\psi_0$ for wind $\psi_{0w} = 0.5$
Assumed depth of col. section $h_c = 260$ mm
254 x 254 x 89 UKC
Young's Modulus $E = 210$ kN/mm²

UNIVERSAL COLUMN
254 x 254 x 89 UKC Grade S 275
Design axial load $737.3$ kN
Buckling resist. $1866$ kN

SUMMARY
Design moment $74.97$ kNm
Moment resistance $288.4$ kNm
Unity factor Comb1 $0.5937 \leq 1$
Unity factor Comb2 $0.6652 \leq 1$
Unity factor Comb3 $0.7942 \leq 1$

Floor beam design

Beam span $L_b = 6$ m
Design permanent plus imposed loading

Distributed floor load \( w'=(\gamma G\cdot G_kf+\gamma Q\cdot Q_kf)\cdot c_{rs}=81.45 \text{ kN/m} \)
Assume the end restraint moment due to gravity load is equal to 10% of the "free" moment.
Design moment @ beam mid-span \( M_{Ed}=0.9\cdot w'\cdot L_b^2/8=329.9 \text{ kNm} \)
Design shear force \( V_{zEd}=w'\cdot L_b/2=244.4 \text{ kN} \)

Steel beam section properties

406 x 178 x 74 UKB
Dimensions (mm): \( h=412.8 \text{ b}=179.5 \text{ tw}=9.5 \text{ tf}=16 \text{ r}=10.2 \)
Properties (cm): \( I_y=27300 \text{ I}_z=1550 \text{ W}_{ply}=1500 \text{ I}_t=62.8 \text{ A}=94.5 \)

Strength of steel beam

For material thickness 16 mm and steel grade S 275.
Design strength \( f_y=275 \text{ N/mm}^2 \)

Check section for moment and shear

Shear resistance \( V_{plRd}=A_{pz}\cdot f_y/SQR(3)/(\gamma M_0\cdot 10^3) \)
\( =664.4 \text{ kN} \)
Design shear force \( V_{zEd}=244.4 \text{ kN} \)
Moment resistance \( M_{cRd}=0.9\cdot f_y\cdot W_{ply}/(\gamma M_0\cdot 10^3) \)
\( =371.3 \text{ kNm} \)
Since \( M_{Ed} \leq M_{cRd} \) (329.9 kNm \( \leq 371.3 \text{ kNm} \)), design moment within moment resistance.

Check for deflection

UDL for deflection calculation \( w_u=Q_kf\cdot c_{rs}=30 \text{ kN/m} \)
Total deflection \( \Delta L=(5\cdot w_u\cdot L_b^4/384)*10^5/(E\cdot I_y) \)
\( =8.83 \text{ mm} \)
Limiting deflection \( \Delta L_{lim}=L_b\cdot 10^3/360=16.67 \text{ mm} \)
As \( \Delta L \leq \Delta L_{lim} \) (8.83 mm \( \leq 16.67 \text{ mm} \)), deflection is OK.

UNIVERSAL BEAM

<table>
<thead>
<tr>
<th></th>
<th>406 x 178 x 74 UKB Grade S 275</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design</td>
<td>244.4 kN</td>
</tr>
<tr>
<td>DESIGN</td>
<td></td>
</tr>
<tr>
<td>Shear</td>
<td>664.4 kN</td>
</tr>
<tr>
<td>SUMMARY</td>
<td></td>
</tr>
<tr>
<td>Design</td>
<td>329.9 kNm</td>
</tr>
<tr>
<td>Moment</td>
<td>371.3 kNm</td>
</tr>
<tr>
<td>Deflection</td>
<td>8.83 mm</td>
</tr>
<tr>
<td>Limiting</td>
<td>16.67 mm</td>
</tr>
</tbody>
</table>

The frame should now be checked for sidesway using unfactored wind loads only and rigidly connected columns to the foundations. The calculated rigid frame deflections should then be increased by 50% (i.e. multiplied by a factor of 1.5) to allow for the flexibility of standard wind-moment connections (provided that the average bay width is at least 6.0 m). For an average bay width of 4.5 m, the deflections from the rigid frame analysis should be multiplied by a factor of 2.0. For bay widths between 4.5 m and 6.0 m, linear interpolation between 2.0 and 1.5 should be used. If the increased values are acceptable, then the design is considered complete.
**Location: Ex1 - British Steel example - K joint**

Chord details:
200 x 200 x 8 SHS - Hot finished.
Properties (cm): A=60.8 rx=7.81 Zx=371 Sx=436 Ix=3710 J=5780 C=545

Brace 1 (B1) details:
150 x 150 x 6.3 SHS - Hot finished.
Properties (cm): A=35.8 rx=5.85 Zx=163 Sx=192 Ix=1220 J=1910 C=240

Brace 2 (B2) details:
120 x 120 x 5 SHS - Hot finished.
Properties (cm): A=22.7 rx=4.68 Zx=83 Sx=97.6 Ix=498 J=777 C=122
Young's Modulus: E=205 kN/mm²
Angle of brace: theta1=35°
Angle of brace: theta2=35°
Gap between bracing elements: g=40 mm
Chord (load 1): Fc1=-550 kN
Chord (load 2): Fc2=-500 kN
Bracing: Fb1=-100 kN
Bracing: Fb2=50 kN

**CAPACITY SUMMARY**

**K & N Gap and Overlap joints**

<table>
<thead>
<tr>
<th>Brace loading (1)</th>
<th>Fb1</th>
<th>-100 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load within capacity</td>
<td>Pass</td>
<td></td>
</tr>
</tbody>
</table>

**Capacities:**

<table>
<thead>
<tr>
<th>Chord face deformation</th>
<th>N1</th>
<th>842.23 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chord shear between bracings</td>
<td>NS1</td>
<td>1241 kN</td>
</tr>
<tr>
<td>Brace effective width</td>
<td>NE1</td>
<td>1120.5 kN</td>
</tr>
<tr>
<td>Chord punching shear</td>
<td>NFS1</td>
<td>2095.5 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Brace loading (2)</th>
<th>Fb2</th>
<th>50 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load within capacity</td>
<td>Pass</td>
<td></td>
</tr>
</tbody>
</table>

**Capacities:**

<table>
<thead>
<tr>
<th>Chord face deformation</th>
<th>N2</th>
<th>842.23 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chord shear between bracings</td>
<td>NS2</td>
<td>1241 kN</td>
</tr>
<tr>
<td>Brace effective width</td>
<td>NE2</td>
<td>739.82 kN</td>
</tr>
<tr>
<td>Chord punching shear</td>
<td>NFS2</td>
<td>1676.4 kN</td>
</tr>
</tbody>
</table>
Location: Ex2 - K joint RHS chord CHS bracing

Chord details:
150 x 150 x 6.3 SHS - Hot finished.
Properties (cm): A=35.8 rx=5.85 Zx=163 Sx=192 Ix=1220 J=1910 C=240

Brace 1 (B1) details:
88.9 dia x 5 thick CHS - Hot finished.

Brace 2 (B2) details:
88.9 dia x 5 thick CHS - Hot finished.
Young's Modulus E=205 kN/mm²

Angle of brace theta1=45°
Angle of brace theta2=30°
Gap between bracing elements g=55 mm
Chord (load 1) Fc1=-200 kN
Chord (load 2) Fc2=-150 kN
Bracing Fb1=-50 kN
Bracing Fb2=50 kN

CAPACITY SUMMARY

K & N Gap and Overlap joints

Brace loading (1) Fb1 -50 kN
Load within capacity Pass

Capacities:-
Chord face deformation N1 285.13 kN
Chord shear between bracings NS1 430.26 kN
Brace effective width NE1 409.51 kN
Chord punching shear NFS1 541.68 kN

Brace loading (2) Fb2 50 kN
Load within capacity Pass

Capacities:-
Chord face deformation N2 403.23 kN
Chord shear between bracings NS2 608.48 kN
Brace effective width NE2 409.51 kN
Chord punching shear NFS2 977.3 kN
Location: Ex3 - K joint all CHS

Chord details:
193.7 dia x 10 thick CHS - Hot finished.
Properties (cm): A=57.711 r=6.5044 Z=252.1 S=337.79 I=2441.6

Brace 1 (B1) details:
114.3 dia x 5 thick CHS - Hot finished.
Properties (cm): A=17.169 r=3.8684 Z=44.955 S=59.774 I=256.92

Brace 2 (B2) details:
114.3 dia x 3.6 thick CHS - Hot finished.
Properties (cm): A=12.52 r=3.9159 Z=33.593 S=44.132 I=191.98

Young's Modulus
E=205 kN/mm²

Angle of brace
theta1=45°
Angle of brace
theta2=45°
Gap between bracing elements
g=32 mm
Chord (load 1)
Fc1=-50 kN
Chord (load 2)
Fc2=-80 kN
Bracing
Fb1=-500 kN
Bracing
Fb2=100 kN

CAPACITY SUMMARY

K & N Gap and Overlap joints

Brace loading (1)
Fb1 -500 kN
Load within capacity
Pass
Chord face deformation
N1 716.13 kN
Chord punching shear
NPS1 1256.4 kN

Brace loading (2)
Fb2 100 kN
Load within capacity
Pass
Chord face deformation
N2 716.13 kN
Chord punching shear
NPS2 1256.4 kN
**Location:** Ex4 - X joint

Chord details:
150 x 150 x 6.3 SHS - Hot finished.
Properties (cm): $A=35.8$, $rx=5.85$, $Zx=163$, $Sx=192$, $Ix=1220$, $J=1910$, $C=240$

Brace 1 (B1) details:
140 x 140 x 5 SHS - Hot finished.
Properties (cm): $A=26.7$, $rx=5.5$, $Zx=115$, $Sx=135$, $Ix=807$, $J=1250$, $C=170$

Brace 2 (B2) details:
140 x 140 x 5 SHS - Hot finished.
Properties (cm): $A=26.7$, $rx=5.5$, $Zx=115$, $Sx=135$, $Ix=807$, $J=1250$, $C=170$

Young's Modulus $E=205$ kN/mm²

Angle of brace $	heta_1=45°$
Angle of brace $	heta_2=45°$

Chord (load 1) $F_{c1}=-73$ kN
Chord (load 2) $F_{c2}=-70$ kN
Bracing $F_{b1}=50$ kN
Bracing $F_{b2}=50$ kN

**CAPACITY SUMMARY**

**T, Y and X joints**

| Brace loading | $F_{b1}$ 50 kN |
| Load within capacity | Pass |
| Capacities: | |
| Chord face deformation | $N_1$ - not within specified range for failure |
| Chord shear | $NS1$ 524.82 kN |
| Brace effective width | $NE1$ 724.51 kN |
| Chord punching shear | $NPS1$ - not within specified range for failure |
| Chord side wall buckling | $NSW1$ 466.87 kN |

| Brace loading (2) | $F_{b2}$ 50 kN |
| Load within capacity | Pass |
| Capacities: | |
| Chord face deformation | $N_2$ - not within specified range for failure |
| Chord shear | $NS2$ 524.82 kN |
| Brace effective width | $NE2$ 724.51 kN |
| Chord punching shear | $NPS2$ - not within specified range for failure |
| Chord side wall buckling | $NSW2$ 466.87 kN |
Location: Ex1 - British Steel example (K joint)

Details of chord:
200 x 200 x 8 SHS - Hot finished.
Properties (cm): A=60.8 iy=7.81 Weldy=371 Wply=436 Iy=3710 It=5780 Wt=545

Details of brace member 1:
150 x 150 x 6.3 SHS - Hot finished.
Properties (cm): A=35.8 iy=5.85 Weldy=163 Wply=192 Iy=1220 It=1910 Wt=240

Details of brace member 2:
120 x 120 x 5 SHS - Hot finished.
Properties (cm): A=22.7 iy=4.68 Weldy=83 Wply=97.6 Iy=498 It=777 Wt=122
Angle of brace 01 theta1=35°
Angle of brace 02 theta2=35°
Gap between bracing members g=40 mm
Chord (load 1) Nc1=-550 kN
Chord (load 2) Nc2=-500 kN
Bracing Nb1=-100 kN
Bracing Nb2=50 kN

DESIGN RESISTANCE SUMMARY

Welded K & N Gap and Overlap joints

Brace 1 loading Nb1 -100 kN
Load within design resistance Pass

Design resistances brace 1:
Chord face failure N1 841.33 kN
Chord design shear NS1 1241 kN
Brace effective width NE1 1120.5 kN
Chord punching shear NPS1 2095.5 kN

Brace 2 loading Nb2 50 kN
Load within design resistance Pass

Design resistances brace 2:
Chord face failure N2 841.33 kN
Chord design shear NS2 1241 kN
Brace effective width NE2 739.82 kN
Chord punching shear NPS2 1676.4 kN
Location: Ex2 - K joint RHS chord & CHS bracing

Details of chord:
150 x 150 x 6.3 SHS - Hot finished.
Properties (cm): A=35.8 iy=5.85 Wely=163 Wply=192 Iy=1220 It=1910
Wt=240

Details of brace member 1:
88.9 dia x 5 thick CHS - Hot finished.
It=232.75

Details of brace member 2:
88.9 dia x 5 thick CHS - Hot finished.
It=232.75

Angle of brace 01 theta1=45°
Angle of brace 02 theta2=30°
Gap between bracing members g=55 mm
Chord (load 1) Nc1=-200 kN
Chord (load 2) Nc2=-150 kN
Bracing Nb1=-50 kN
Bracing Nb2=50 kN

DESIGN RESISTANCE SUMMARY

Welded K & N Gap and Overlap joints

Brace 1 loading Nb1 -50 kN
Load within design resistance Pass

Design resistances brace 1:
Chord face failure N1 139276 kN
Chord design shear NS1 430.26 kN
Brace effective width NE1 409.51 kN
Chord punching shear NPS1 541.68 kN

Brace 2 loading Nb2 50 kN
Load within design resistance Pass

Design resistances brace 2:
Chord face failure N2 196966 kN
Chord design shear NS2 608.48 kN
Brace effective width NE2 409.51 kN
Chord punching shear NPS2 977.3 kN
Location: Ex3 - K joint all CHS

Details of chord:
193.7 dia x 10 thick CHS - Hot finished.
Properties (cm): A=57.711 iy=6.5044 Wely=252.1 Wply=337.79 Iy=2441.6
It=4883.2

Details of brace member 1:
114.3 dia x 5 thick CHS - Hot finished.
Properties (cm): A=17.169 iy=3.8684 Wely=44.955 Wply=59.774 Iy=256.92
It=513.84

Details of brace member 2:
114.3 dia x 3.6 thick CHS - Hot finished.
Properties (cm): A=12.52 iy=3.9159 Wely=33.593 Wply=44.132 Iy=191.98
It=383.97

Angle of brace 01 \( \theta \) = 45°
Angle of brace 02 \( \theta \) = 45°
Gap between bracing members \( g \) = 32 mm
Chord (load 1) \( N_c1 \) = -50 kN
Chord (load 2) \( N_c2 \) = -80 kN
Bracing \( N_b1 \) = -500 kN
Bracing \( N_b2 \) = 100 kN

DESIGN RESISTANCE SUMMARY

Welded K & N Gap and Overlap joints

Brace 1 loading
Load within design resistance Pass
Chord face failure resistance \( N_1 \) = 716.13 kN
Chord punching shear resistance \( N_{PS1} \) = 1256.4 kN

Brace 2 loading
Load within design resistance Pass
Chord face failure resistance \( N_2 \) = 716.13 kN
Chord punching shear resistance \( N_{PS2} \) = 1256.4 kN
**Location:** Ex4 - X joint

Details of chord:
150 x 150 x 6.3 SHS - Hot finished.
Properties (cm): A=35.8 iy=5.85 Wely=163 Wply=192 Iy=1220 It=1910
Wt=240

Details of brace member 1:
140 x 140 x 5 SHS - Hot finished.
Properties (cm): A=26.7 iy=5.5 Wely=115 Wply=135 Iy=807 It=1250
Wt=170

Details of brace member 2:
140 x 140 x 5 SHS - Hot finished.
Properties (cm): A=26.7 iy=5.5 Wely=115 Wply=135 Iy=807 It=1250
Wt=170

Angle of brace 01
\[ \theta_1=45^\circ \]
Angle of brace 02
\[ \theta_2=45^\circ \]

Chord (load 1)  \[ N_{c1}=-73 \text{ kN} \]
Chord (load 2)  \[ N_{c2}=-70 \text{ kN} \]
Bracing  \[ N_{b1}=50 \text{ kN} \]
Bracing  \[ N_{b2}=50 \text{ kN} \]

**DESIGN RESISTANCE SUMMARY**

**Welded T, X and Y joints**

<table>
<thead>
<tr>
<th>Brace 1 loading</th>
<th>Nb1</th>
<th>50 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load within design resistance</td>
<td>Pass</td>
<td></td>
</tr>
</tbody>
</table>

Design resistances brace 1:

<table>
<thead>
<tr>
<th>Chord face failure</th>
<th>N1</th>
<th>not within specified range for failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chord design shear</td>
<td>NS1</td>
<td>524.82 kN</td>
</tr>
<tr>
<td>Brace effective width</td>
<td>NE1</td>
<td>724.51 kN</td>
</tr>
<tr>
<td>Chord punching shear</td>
<td>NFS1</td>
<td>not within specified range for failure</td>
</tr>
<tr>
<td>Chord side wall buckling</td>
<td>NSW1</td>
<td>474.21 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Brace 2 loading</th>
<th>Nb2</th>
<th>50 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load within design resistance</td>
<td>Pass</td>
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</tr>
</tbody>
</table>

Design resistances brace 2:

<table>
<thead>
<tr>
<th>Chord face failure</th>
<th>N2</th>
<th>not within specified range for failure</th>
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<tbody>
<tr>
<td>Chord design shear</td>
<td>NS2</td>
<td>524.82 kN</td>
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<tr>
<td>Brace effective width</td>
<td>NE2</td>
<td>724.51 kN</td>
</tr>
<tr>
<td>Chord punching shear</td>
<td>NFS2</td>
<td>not within specified range for failure</td>
</tr>
<tr>
<td>Chord side wall buckling</td>
<td>NSW2</td>
<td>474.21 kN</td>
</tr>
</tbody>
</table>
Location: Ex1 - RHS section

Chord details:
200 x 200 x 8 SHS - Hot finished.
Properties (cm): A=60.8 rx=7.81 Zx=371 Sx=436 Ix=3710 J=5780 C=545

Brace member 1 details:
150 x 150 x 6.3 SHS - Hot finished.
Properties (cm): A=35.8 rx=5.85 Zx=163 Sx=192 Ix=1220 J=1910 C=240
Young's Modulus E=205 kN/mm²

Angle of brace 1 \( \theta_1 = 35^\circ \)
Chord axial load 1 \( F_{c1} = -550 \) kN
Chord axial load 2 \( F_{c2} = -500 \) kN
Brace axial load \( F_{b1} = -100 \) kN
Bracing in-plane moment \( M_x = 9.6 \) kNm
Bracing out-of-plane moment \( M_y = 0.7 \) kNm

LOAD SUMMARY - welded T, Y & X joints

<table>
<thead>
<tr>
<th>Brace loading</th>
<th>( F_{b1} = -100 ) kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load within capacity</td>
<td>Pass</td>
</tr>
</tbody>
</table>

Capacities:
Chord face deformation \( N_1 = 731.25 \) kN
Brace effective width \( N_{E1} \) not within specified range for failure
Chord punching shear \( N_{PS1} \) not within specified range for failure
Chord punching shear \( N_{SW1} \) not within specified range for failure

Welding
Prequalified weld throat thickness size:
Based on brace member 1 thickness and steel Grade S 355.
Minimum throat thickness \( a = 1.09 \times t_1 = 6.867 \) mm
Further details on welding can be obtained from British Steel Tubes and Pipes publication 'SHS Welding'.

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Brace 1 axial load</th>
<th>(-100 ) kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial capacity</td>
<td>731.25 kN</td>
</tr>
<tr>
<td>Brace 1 in-plane BM</td>
<td>9.6 kNm</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>24.424 kNm</td>
</tr>
<tr>
<td>Brace 1 out-of-plane BM</td>
<td>0.7 kNm</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>26.652 kNm</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>( 0.55607 \leq 1 )</td>
</tr>
</tbody>
</table>
Location: Ex2 - Welded Knee Joint (unreinforced)

Chord details:
200 x 100 x 8 RHS - Hot finished.
Properties (cm): A=44.8 rx=7.06 Zx=223 Sx=282 Ix=2230
J=1800 C=251 Zy=148 Sy=172 Iy=739 ry=4.06
Young's Modulus E=205 kN/mm²
Applied load Fc=-36 kN
Applied moment Mx=89 kNm
Angle of knee θ theta=150°

RHS KNEE JOINT
HOT FINISHED In accordance with EN 10210
RECTANGULAR HOLLOW SECTION 200 x 100 x 8 RHS Grade S 355
SECTION Connection is satisfactory for axial
SUMMARY load and interaction expression.
Axial load -36 kN
Axial tensile capacity 1590.4 kN
Applied moment 89 kNm
Moment capacity 100.11 kNm
Joint efficiency factor 0.91993
Interaction factor 0.91166 ≤ 1
Location: Ex3 - CHS section (N joint)

Chord details:
193.7 dia x 8 thick CHS - Hot finished.
Properties (cm): A=46.672 r=6.5716 Z=208.11 S=276.05 I=2015.5

Brace member 1 details:
114.3 dia x 5 thick CHS - Hot finished.
Properties (cm): A=17.169 r=3.8684 Z=44.955 S=59.774 I=256.92

Brace member 2 details:
76.1 dia x 4 thick CHS - Hot finished.
Properties (cm): A=9.0604 r=2.553 Z=15.52 S=20.815 I=59.055
Young's Modulus E=205 kN/mm²

Angle of brace 01 theta1=40°
Angle of brace 02 theta2=90°
Gap between bracing elements g=20 mm
Chord axial load 1 Fc1=-650 kN
Chord axial load 2 Fc2=-600 kN
Brace 1 axial load Fb1=-200 kN
Brace 2 axial load Fb2=150 kN
Bracing in-plane moment Mx=9.6 kNm
Bracing out-of-plane moment My=0.7 kNm

Bracing in-plane moment Mx2=2 kNm
Bracing out-of-plane moment My2=0.5 kNm

LOAD SUMMARY - welded K & N gap joints

Brace member 1 axial load Fb1 -200 kN
Load within capacity Pass
Chord face deformation N1 440.23 kN
Chord punching shear NPS1 906.72 kN

Brace member 2 axial load Fb2 150 kN
Load within capacity Pass
Chord punching shear NPS2 303.67 kN

Welding

Prequalified weld throat thickness size:
Based on brace member 1 thickness and steel Grade S 275.
Minimum throat thickness a=0.94*t1=4.7 mm
CHS bracing requires partial end profiling to achieve the weld fit up.
Based on brace member 2 thickness and steel Grade S 275.
Minimum throat thickness a2=0.94*t2=3.76 mm
CHS bracing requires partial end profiling to achieve the weld fit up.
Further details on welding can be obtained from British Steel Tubes and Pipes publication 'SHS Welding'.
<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Brace 1 axial load</th>
<th>-200 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bracing capacity</td>
<td>440.23 kN</td>
<td></td>
</tr>
<tr>
<td>Brace 1 in-plane BM</td>
<td>9.6 kNm</td>
<td></td>
</tr>
<tr>
<td>Moment capacity</td>
<td>25.956 kNm</td>
<td></td>
</tr>
<tr>
<td>Brace 1 out-of-plane BM</td>
<td>0.7 kNm</td>
<td></td>
</tr>
<tr>
<td>Moment capacity</td>
<td>14.039 kNm</td>
<td></td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.64096 ≤ 1</td>
<td></td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Brace 2 axial load</th>
<th>150 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial capacity</td>
<td>282.97 kN</td>
</tr>
<tr>
<td>Brace 2 in-plane BM</td>
<td>2 kNm</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>7.3558 kNm</td>
</tr>
<tr>
<td>Brace 2 out-of-plane BM</td>
<td>0.5 kNm</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>6.0081 kNm</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.68723 ≤ 1</td>
</tr>
</tbody>
</table>
Location: Ex1 - RHS section

Chord details:
200 x 200 x 8 SHS - Hot finished.
Properties (cm): A=60.8 iy=7.81 Wely=371 Wply=436 Iy=3710 It=5780
Wt=545

Brace member 1 details:
150 x 150 x 6.3 SHS - Hot finished.
Properties (cm): A=35.8 iy=5.85 Wely=163 Wply=192 Iy=1220 It=1910
Wt=240
Young's Modulus E=210 kN/mm²
Angle of brace θ1 theta1=35°
Chord axial load 1 Nc1=-550 kN
Chord axial load 2 Nc2=-500 kN
Brace member axial load Nb1=-100 kN
Bracing in-plane moment My1=9.6 kNm
Bracing out-of-plane moment Mz1=0.7 kNm

LOAD SUMMARY - welded T, Y & X joints

Brace loading Nb1 -100 kN
Load within resistance Pass

Resistances:
Chord face failure N1 731.25 kN
Brace effective width NE1 not within specified range for failure
Chord punching shear NFS1 not within specified range for failure
Chord punching shear NSW1 not within specified range for failure

Welding

Prequalified weld throat thickness size:
Based on brace member 1 thickness and steel Grade S 355.
Minimum throat thickness a=1.11*t1=6.993 mm
Further details on welding can be obtained from TATA Steel publication entitled 'Design of Welded Joints'.

DESIGN SUMMARY
Brace 1 axial load -100 kN
Axial resistance 731.25 kN
Brace 1 in-plane BM 9.6 kNm
Moment resistance 24.424 kNm
Brace 1 out-of-plane BM 0.7 kNm
Moment resistance 26.652 kNm
Interaction factor 0.55607 ≤ 1
Location: Ex2 - Welded Knee Joint (unreinforced)

Chord details:
200 x 100 x 8 RHS - Hot finished.
Properties (cm): A=44.8 iz=4.06 Wely=223 Wply=282 Iy=2230
It=1800 Wt=251 Welz=148 Wplz=172 Iz=739 iy=7.06
Young's Modulus E=210 kN/mm²
Applied axial load Nc=-36 kN
Applied moment My1=89 kNm
Angle of knee θ theta=150°

RHS KNEE JOINT
HOT FINISHED
RECTANGULAR HOLLOW SECTION 200 x 100 x 8 RHS Grade S 355
SECTION Connection is satisfactory for axial load and interaction expression.
SUMMARY Design axial load -36 kN
Axial tensile resistance 1590.4 kN
Applied moment 89 kNm
Moment resistance 100.11 kNm
Joint efficiency factor 0.91993
Interaction factor 0.91166 ≤ 1
Location: Ex3 - CHS section (N joint)

Chord details:
193.7 dia x 8 thick CHS - Hot finished.
Properties (cm): A=46.672 \( iy = 6.5716 \) Wely=208.11 Wply=276.05 Iy=2015.5 It=4031.1

Brace member 1 details:
114.3 dia x 5 thick CHS - Hot finished.
Properties (cm): A=17.169 \( iy = 3.8684 \) Wely=44.955 Wply=59.774 Iy=256.92 It=513.84

Brace member 2 details:
76.1 dia x 4 thick CHS - Hot finished.
Properties (cm): A=9.0604 \( iy = 2.553 \) Wely=15.52 Wply=20.815 Iy=59.055 It=118.11

Young's Modulus E=210 kN/mm²

Angle of brace 01 \( \theta_1 = 40^\circ \)
Angle of brace 02 \( \theta_2 = 90^\circ \)
Gap between bracing elements \( g = 20 \text{ mm} \)
Chord axial load 1 \( N_c1 = -650 \text{ kN} \)
Chord axial load 2 \( N_c2 = -600 \text{ kN} \)
Brace 1 axial load \( N_b1 = -200 \text{ kN} \)
Brace 2 axial load \( N_b2 = 150 \text{ kN} \)
Bracing in-plane moment \( M_y1 = 9.6 \text{ kNm} \)
Bracing out-of-plane moment \( M_z1 = 0.7 \text{ kNm} \)
Bracing in-plane moment \( M_y2 = 2 \text{ kNm} \)
Bracing out-of-plane moment \( M_z2 = 0.5 \text{ kNm} \)

LOAD SUMMARY - welded K & N gap joints

Brace member 1 axial load \( N_b1 = -200 \text{ kN} \)
Load within resistance Pass
Chord face failure \( N_1 = 568.3 \text{ kN} \)
Chord punching shear \( N_{PS1} = 1170.5 \text{ kN} \)

Brace member 2 axial load \( N_b2 = 150 \text{ kN} \)
Load within resistance Pass
Chord punching shear \( N_{PS2} = 392.01 \text{ kN} \)

Welding

Prequalified weld throat thickness size:
Based on brace member 1 thickness and steel Grade S 355.
Minimum throat thickness \( a = 1.11 \times t_1 = 5.55 \text{ mm} \)
CHS bracing requires partial end profiling to achieve the weld fit up.
Based on brace member 2 thickness and steel Grade S 355.
Minimum throat thickness \( a_2 = 1.11 \times t_2 = 4.44 \text{ mm} \)
CHS bracing requires partial end profiling to achieve the weld fit up.
Further details on welding can be obtained from TATA Steel publication entitled 'Design of Welded Joints'.
**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Brace 1 axial load</th>
<th>-200 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial resistance</td>
<td>568.3 kN</td>
</tr>
<tr>
<td>Brace 1 in-plane BM</td>
<td>9.6 kNm</td>
</tr>
<tr>
<td>Moment resistance</td>
<td>33.507 kNm</td>
</tr>
<tr>
<td>Brace 1 out-of-plane BM</td>
<td>0.7 kNm</td>
</tr>
<tr>
<td>Moment resistance</td>
<td>18.123 kNm</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.47264 ≤ 1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Brace 2 axial load</th>
<th>150 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial resistance</td>
<td>365.29 kN</td>
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<tr>
<td>Brace 2 in-plane BM</td>
<td>2 kNm</td>
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<tr>
<td>Moment resistance</td>
<td>9.4957 kNm</td>
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<tr>
<td>Brace 2 out-of-plane BM</td>
<td>0.5 kNm</td>
</tr>
<tr>
<td>Moment resistance</td>
<td>7.7559 kNm</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.51946 ≤ 1</td>
</tr>
</tbody>
</table>
Location: Ex4 - RHS section (X joint)

Chord details:
200 x 200 x 8 SHS - Hot finished.
Properties (cm): A=60.8 iy=7.81 Wely=371 Wply=436 Iy=3710 It=5780 Wt=545

Brace member 1 details:
180 x 180 x 6.3 SHS - Hot finished.
Properties (cm): A=43.3 iy=7.07 Wely=241 Wply=281 Iy=2170 It=3360 Wt=355

Brace member 2 details:
180 x 180 x 6.3 SHS - Hot finished.
Properties (cm): A=43.3 iy=7.07 Wely=241 Wply=281 Iy=2170 It=3360 Wt=355

Young's Modulus E=210 kN/mm²
Angle of brace 01 theta1=35°
Angle of brace 02 theta2=30°
Chord axial load 1 Nc1=-550 kN
Chord axial load 2 Nc2=-500 kN
Brace 1 axial load Nb1=-100 kN
Brace 2 axial load Nb2=-100 kN
Bracing in-plane moment My1=9.6 kNm
Bracing out-of-plane moment Mz1=0.7 kNm
Bracing in-plane moment My2=0 kNm
Bracing out-of-plane moment Mz2=0 kNm
Young's Modulus E=210 kN/mm²

LOAD SUMMARY - welded T, Y & X joints

Brace loading Nb1 -100 kN
Load within resistance Pass

Resistances:
Chord face failure N1 not within specified range for failure
Chord shear NS1 1097.7 kN
Brace effective width NE1 1157.7 kN
Chord punching shear NPS1 2205.9 kN
Chord side wall buckling NSW1 724.81 kN

Brace member 2 axial load Nb2 -100 kN
Load within resistance Pass

Resistances:
Chord face failure N2 not within specified range for failure
Chord shear NS2 1259.3 kN
Brace effective width NE2 1157.7 kN
Chord punching shear NPS2 2833.4 kN
Chord side wall buckling  NSW2  728.75 kN

Welding

Prequalified weld throat thickness size:
Based on brace member 1 thickness and steel Grade S 355.
Minimum throat thickness  \( a = 1.11 \times t_1 = 6.993 \text{ mm} \)
Based on brace member 2 thickness and steel Grade S 355.
Minimum throat thickness  \( a_2 = 1.11 \times t_2 = 6.993 \text{ mm} \)
Further details on welding can be obtained from TATA Steel publication entitled 'Design of Welded Joints'.

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Brace 1 axial load</th>
<th>-100 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial resistance</td>
<td>724.81 kN</td>
<td></td>
</tr>
<tr>
<td>Brace 1 in-plane BM</td>
<td>9.6 kNm</td>
<td></td>
</tr>
<tr>
<td>Moment resistance</td>
<td>54.982 kNm</td>
<td></td>
</tr>
<tr>
<td>Brace 1 out-of-plane BM</td>
<td>0.7 kNm</td>
<td></td>
</tr>
<tr>
<td>Moment resistance</td>
<td>72.441 kNm</td>
<td></td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.32223 ( \leq 1 )</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Brace 2 axial load</th>
<th>-100 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial resistance</td>
<td>728.75 kN</td>
</tr>
<tr>
<td>Brace 2 in-plane BM</td>
<td>0 kNm</td>
</tr>
<tr>
<td>Moment resistance</td>
<td>54.982 kNm</td>
</tr>
<tr>
<td>Interaction factor</td>
<td>0.13724 ( \leq 1 )</td>
</tr>
</tbody>
</table>
**Location: Ex1 - Transverse gusset**

193.7 dia x 8 thick CHS - Hot finished.
Properties (cm): A=46.672 r=6.5716 Z=208.11 S=276.05 I=2015.5
Width of plate \( b_1 = 180 \text{ mm} \)
Thickness of plate \( t_1 = 10 \text{ mm} \)

Chord (axial load 1) \( F_{c1} = -550 \text{ kN} \)
Chord (axial load 2) \( F_{c2} = -500 \text{ kN} \)
Bracing axial load \( F_b = 100 \text{ kN} \)
Out-of-plane moment \( M_o = 2 \text{ kNm} \)

### Capacity Summary

<table>
<thead>
<tr>
<th>Component</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse gusset plate</td>
<td>180 mm x 10 mm Grade S 355</td>
</tr>
<tr>
<td>Applied loading</td>
<td>( F_b ) 100 kN</td>
</tr>
<tr>
<td>Chord face deformation</td>
<td>( N_1 ) 459.38 kN</td>
</tr>
<tr>
<td>Load within capacity</td>
<td>Pass</td>
</tr>
<tr>
<td>Out-of-plane moment</td>
<td>( M_o ) 2 kNm</td>
</tr>
<tr>
<td>Out-of-plane moment capacity</td>
<td>( M_{o1} ) 41.344 kNm</td>
</tr>
<tr>
<td>Moment within capacity</td>
<td>Pass</td>
</tr>
<tr>
<td>Applied punching shear value</td>
<td>( N_{app} ) 166.67 kN</td>
</tr>
<tr>
<td>Punching shear capacity</td>
<td>( N_{PS} ) 590.28 kN</td>
</tr>
<tr>
<td>Load within capacity</td>
<td>Pass</td>
</tr>
</tbody>
</table>
Location: Ex2 - UB section brace to RHS chord

300 x 200 x 12.5 RHS - Hot finished.
Properties (cm): A=117 rx=11 Zx=952 Sx=1170 Ix=14300
J=15700 C=1220 Zy=754 Sy=877 Iy=7540 ry=8.02
254 x 146 x 31 UB.
Chord (axial load 1) Fc1=1100 kN
Chord (axial load 2) Fc2=900 kN
Bracing axial load Fb=-350 kN
In-plane moment Mi=67 kNm

Capacity Summary

Universal beam bracing 254 x 146 x 31 UB Grade S 275
Gusset loading Fb -350 kN
Load within capacity Pass

Capacities:
Brace effective width NE1 627.77 kN
Chord punching shear NPS1 793.16 kN
Chord side wall crushing NSW1 not within specified range for failure
In-plane moment Mi 67 kNm
In-plane moment capacity Mi1 152.42 kNm
Moment within capacity Pass
Location: Ex3 - Longitudinal gusset

150 x 150 x 8 SHS - Hot finished.
Properties (cm): A=44.8 r_x=5.77 Z_x=199 S_x=237 I_x=1490 J=2350 C=291
Plate length along chord line \( h_1=200 \text{ mm} \)
Thickness of plate \( t_1=10 \text{ mm} \)

Chord (axial load 1) \( F_{c1}=-550 \text{ kN} \)
Chord (axial load 2) \( F_{c2}=-500 \text{ kN} \)
Bracing axial load \( F_b=90 \text{ kN} \)
In-plane moment \( M_i=4 \text{ kNm} \)

Capacity Summary

<table>
<thead>
<tr>
<th>Longitudinal gusset plate</th>
<th>200 mm x 10 mm Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gusset loading</td>
<td>( F_b=90 \text{ kN} )</td>
</tr>
<tr>
<td>Chord face deformation</td>
<td>( N_1=135.2 \text{ kN} )</td>
</tr>
<tr>
<td>Load within capacity</td>
<td>Pass</td>
</tr>
<tr>
<td>In-plane moment</td>
<td>( M_i=4 \text{ kNm} )</td>
</tr>
<tr>
<td>In-plane moment capacity</td>
<td>( M_{i1}=13.52 \text{ kNm} )</td>
</tr>
<tr>
<td>Moment within capacity</td>
<td>Pass</td>
</tr>
</tbody>
</table>
Location: Exl - Transverse gusset to CHS chord

193.7 dia x 8 thick CHS - Hot finished.
Properties (cm): A=46.672 iy=6.5716 Wely=208.11 Wply=276.05 Iy=2015.5 It=4031.1

Width of plate                b1=180 mm
Thickness of plate            t1=10 mm
Chord (axial load 1)          Nc1=-550 kN
Chord (axial load 2)          Nc2=-500 kN
Bracing axial load            Nb=100 kN
Out-of-plane moment           Mo=2 kNm

DESIGN RESISTANCE SUMMARY

Transverse gusset plate       180 mm x 10 mm Grade S 355
Bracing axial loading         Nb 100 kN
Chord face failure            N1 459.38 kN
Load within design resistance Pass
Out-of plane design moment    Mo 2 kNm
Out-of plane BM resistance    Mo1 41.344 kNm
Moment within BM resistance   Pass
Applied punching shear value  Napp 166.67 kN
Punching shear resistance     NPS 590.28 kN
Load within shear resistance  Pass
### Location: Ex2 - UB section brace to RHS chord

300 x 200 x 12.5 RHS - Hot finished.
Properties (cm): A=117 iz=8.02 Wely=952 Wply=1170 Iy=14300  
It=15700 Wt=1220 Welz=754 Wplz=877 Iz=7540 iy=11

<table>
<thead>
<tr>
<th>254 x 146 x 37 UKB</th>
<th>Chord (axial load 1)</th>
<th>Nc1=1100 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chord (axial load 2)</td>
<td>Nc2=900 kN</td>
<td></td>
</tr>
<tr>
<td>Bracing axial load</td>
<td>Nb=-350 kN</td>
<td></td>
</tr>
<tr>
<td>In-plane moment</td>
<td>Mi=67 kNm</td>
<td></td>
</tr>
</tbody>
</table>

### DESIGN RESISTANCE SUMMARY

<table>
<thead>
<tr>
<th>Universal beam bracing</th>
<th>254 x 146 x 37 UKB Grade S 275</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gusset axial loading</td>
<td>Nb -350 kN</td>
</tr>
<tr>
<td>Load within resistance</td>
<td>Pass</td>
</tr>
</tbody>
</table>

Resistances:
- **Brace effective width**: NE1 629.06 kN
- **Chord punching shear**: NPS1 812.91 kN
- **Chord side wall crushing**: NSW1 not within failure range
- **In-plane design moment**: Mi 67 kNm
- **In-plane moment resistance**: Mi1 154.18 kNm
- **Moment within BM resistance**: Pass
Location: Ex3 - Longitudinal gusset to SHS chord

150 x 150 x 10 SHS - Hot finished.
Properties (cm): $A=54.9$ iy=$5.68$ Wely=$236$ Wply=$286$ Iy=$1770$ It=$2830$
Wt=$344$

Plate length along chord line $h_1=200$ mm
Thickness of plate $t_1=10$ mm
Chord (axial load 1) $N_c1=-550$ kN
Chord (axial load 2) $N_c2=-500$ kN
Bracing axial load $N_b=90$ kN
In-plane moment $M_i=4$ kNm

**DESIGN RESISTANCE SUMMARY**

<table>
<thead>
<tr>
<th>Longitudinal gusset plate</th>
<th>200 mm x 10 mm Grade S 355</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gusset axial loading</td>
<td>$N_b$ 90 kN</td>
</tr>
<tr>
<td>Chord face failure</td>
<td>$N_1$ 216.35 kN</td>
</tr>
<tr>
<td>Load within resistance</td>
<td>Pass</td>
</tr>
<tr>
<td>In-plane design moment</td>
<td>$M_i$ 4 kNm</td>
</tr>
<tr>
<td>In-plane moment resistance</td>
<td>$M_{ii}$ 21.635 kNm</td>
</tr>
<tr>
<td>Moment within BM resistance</td>
<td>Pass</td>
</tr>
</tbody>
</table>
Location: Example 17, Steelwork Design Guide to BS5950

Double angle web cleat connection

Analysis of the connection follows the treatment in the BCSA publication entitled 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with the requirements of BS5950-1:2000.

Universal section to universal section connection using standard bolts

Supporting member details

254 x 254 x 89 UC.
Thickness of connecting ply \( t_s = 17.3 \) mm

Supported beam details

533 x 210 x 92 UB.

Distance top of beam/top of cleat \( n = 40 \) mm

Thickness of angle cleats \( t_p = 10 \) mm
Length of angle cleat leg \( l_c = 90 \) mm
Backmark \( b_m = 60 \) mm
Diameter of bolts (double shear) \( d_b = 20 \) mm
Provide 1 line/s of fasteners.
Provide 8 rows of fasteners in each line.
Supported beam end clearance \( e_c = 10 \) mm
Diameter of bolts (single shear) \( d_{b1} = 20 \) mm
Pitch of bolts \( p' = 100 \) mm
No. of bolts in supporting memb \( b_n = 8 \)

To calculate the load on the outermost bolt due to moment, apply the unit area method. Function of inertia of bolt group:
WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

<table>
<thead>
<tr>
<th>35 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 mm</td>
</tr>
<tr>
<td>50 mm</td>
</tr>
<tr>
<td>50 mm</td>
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<tr>
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<td>50 mm</td>
</tr>
<tr>
<td>50 mm</td>
</tr>
<tr>
<td>50 mm</td>
</tr>
<tr>
<td>60 mm</td>
</tr>
</tbody>
</table>

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

<table>
<thead>
<tr>
<th>60 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 mm</td>
</tr>
<tr>
<td>100 mm</td>
</tr>
<tr>
<td>100 mm</td>
</tr>
<tr>
<td>30 mm</td>
</tr>
</tbody>
</table>

ANGLE DETAILS

<table>
<thead>
<tr>
<th>90 mm x 90 mm x 10 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade S 275</td>
</tr>
<tr>
<td>Length of cleat 420 mm</td>
</tr>
<tr>
<td>Number of bolts/leg 4</td>
</tr>
<tr>
<td>Diameter of bolts M 20 Grade 8.8</td>
</tr>
<tr>
<td>Pitch 100 mm</td>
</tr>
<tr>
<td>End distance 60 mm</td>
</tr>
<tr>
<td>Number of bolts 8</td>
</tr>
<tr>
<td>Diameter of bolts M 20 Grade 8.8</td>
</tr>
<tr>
<td>Pitch 50 mm</td>
</tr>
<tr>
<td>End distance 30 mm</td>
</tr>
</tbody>
</table>
Location: Example 1 - Joints publication - 2 lines of bolts

Double angle web cleat connection

Analysis of the connection follows the treatment in the BCSA publication entitled 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with the requirements of BS5950-1:2000.

Universal section to universal section connection using standard bolts

Supporting member details

610 x 229 x 140 UB.
Thickness of connecting ply ts=13.1 mm

Supported beam details

406 x 178 x 74 UB.

Length of notch to end face N=120 mm
Depth of notch n=50 mm
Thickness of angle cleats tp=10 mm
Length of angle cleat leg lc=150 mm
Length of angle cleat leg (short) lc'=90 mm
Backmark bmk=50 mm
Diameter of bolts (double shear) db=20 mm
Provide 2 line/s of fasteners.
Provide 4 rows of fasteners in each line.
Horizontal bolt spacing g=50 mm
Supported beam end clearance ec=10 mm
Diameter of bolts (single shear) db1=20 mm
Pitch of bolts p'=70 mm
No. of bolts in supporting memb bn1=8
To calculate the load on the outermost bolt due to moment, apply the unit area method. Function of inertia of bolt group:

Dimension: \( \ell = 50 \text{ mm} \)

Applied shear force: \( Q_a = 620 \text{ kN} \)

### WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

- **50 mm**
  - 40 mm
  - 70 mm
  - 70 mm
  - 70 mm
  - 70 mm
  - 40 mm

### WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

- **110 mm**
  - 40 mm
  - 40 mm

### ANGLE DETAILS

**SUMMARY**
- Grade S 275
- Length of cleat: 290 mm
- Number of bolts/leg: 4
- Diameter of bolts: M 20 Grade 8.8
- Pitch: 70 mm
- End distance: 40 mm

**TO SUPPORTING MEMBER**
- Number of bolts: 8

**TO BEAM WEB**
- Diameter of bolts: M 20 Grade 8.8
- Pitch: 70 mm
- End distance: 50 mm
Location: Example 2 - Joints publication - Tie force

Double angle web cleat connection

Analysis of the connection follows the treatment in the BCSA publication entitled 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with the requirements of BS5950-1:2000.

Universal section to universal section connection using standard bolts
Structural integrity tie force $T_f = 230$ kN

Supporting member details

305 x 305 x 137 UC.
Thickness of connecting ply $t_s = 13.8$ mm

Supported beam details

406 x 178 x 74 UB.

Distance top of beam/top of cleat $n = 50$ mm

Thickness of angle cleats $t_p = 10$ mm
Length of angle cleat leg $l_c = 150$ mm
Length of angle cleat leg (short) $l_c' = 90$ mm
Backmark $bmk = 50$ mm
Diameter of bolts (double shear) $d_b = 20$ mm
Provide 2 line/s of fasteners.
Provide 4 rows of fasteners in each line.
Horizontal bolt spacing $g = 50$ mm
Supported beam end clearance $e_c = 10$ mm
Diameter of bolts (single shear) $d_{b1} = 20$ mm

Pitch of bolts $p' = 70$ mm
No. of bolts in supporting memb $b_{n1} = 8$
WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

50 mm

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

ANGLE DETAILS
SUMMARY
LEGS CONNECTED TO SUPPORTING MEMBER
LEGS CONNECTED TO BEAM WEB

150 mm x 90 mm x 10 mm
Grade S 275
Length of cleat 290 mm
Number of bolts/leg 4
Diameter of bolts M 20 Grade 8.8
Pitch 70 mm
End distance 40 mm
Number of bolts 8
Diameter of bolts M 20 Grade 8.8
Pitch 70 mm
End distance 50 mm
Location: Example using non-standard section and tie force

Double angle web cleat connection

Analysis of the connection follows the treatment in the BCSA publication entitled 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with the requirements of BS5950-1:2000.

Universal section to universal section connection using standard bolts
Structural integrity tie force  $T_f=175 \text{ kN}$

Supporting member details

Thickness of connecting ply  $t_s=10 \text{ mm}$

Supported beam details

Distance top of beam/top of cleat  $n=50 \text{ mm}$

Thickness of angle cleats  $t_p=10 \text{ mm}$
Length of angle cleat leg  $l_c=90 \text{ mm}$
Backmark  $bmk=60 \text{ mm}$
Diameter of bolts (double shear)  $d_b=20 \text{ mm}$
Provide 1 line/s of fasteners.
Provide 8 rows of fasteners in each line.
Supported beam end clearance  $e_c=10 \text{ mm}$
Diameter of bolts (single shear)  $d_{b1}=20 \text{ mm}$

Pitch of bolts  $p'=100 \text{ mm}$
No. of bolts in supporting memb  $b_{n1}=8$
To calculate the load on the outermost bolt due to moment, apply the unit area method. Function of inertia of bolt group:
WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

<table>
<thead>
<tr>
<th>60 mm</th>
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</thead>
<tbody>
<tr>
<td>35 mm</td>
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<tr>
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<tr>
<td>50 mm</td>
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<td>35 mm</td>
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WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

<table>
<thead>
<tr>
<th>30 mm</th>
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</thead>
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<td>100 mm</td>
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<tr>
<td>100 mm</td>
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</tr>
</tbody>
</table>

ANGLE DETAILS
Summary: Grade S 275
Length of cleat: 420 mm
Number of bolts/leg: 4
Diameter of bolts: M 20 Grade 8.8
Pitch: 100 mm
End distance: 60 mm

LEGGS CONNECTED TO SUPPORTING MEMBER
Number of bolts: 8
Diameter of bolts: M 20 Grade 8.8
Pitch: 50 mm
End distance: 30 mm
Location: Example using Flowdrill bolts

Double angle web cleat connection

Analysis of the connection follows the treatment in the BCSA publication entitled 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with the requirements of BS5950-1:2000.

Universal section beam to RHS column connection using standard bolts connecting the beam to the web cleats. Flowdrill bolts will be utilised for connecting cleats to the column.

Supporting member details

250 x 250 x 10 SHS - Hot finished.
Properties (cm): A=94.9 rx=9.77 Zx=724 Sx=851 Ix=9060 J=14100 C=1070

Supported beam details

406 x 178 x 74 UB.

Distance top of beam/top of cleat n=50 mm

Thickness of angle cleats tp=10 mm
Length of angle cleat leg lc=150 mm
Length of angle cleat leg (short) lc'=90 mm
Backmark bmk=50 mm
Diameter of bolts (double shear) db=20 mm
Provide 2 line/s of fasteners.
Provide 4 rows of fasteners in each line.
Horizontal bolt spacing g=50 mm
Supported beam end clearance ec=10 mm
Diameter of Flowdrill bolts db1=20 mm

Pitch of bolts p'=70 mm
No. of bolts in supporting memb bn1=8
To calculate the load on the outermost bolt due to moment, apply the unit area method. Function of inertia of bolt group:
Distance top beam to plate \( n = 50 \text{ mm} \)

**WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR**

<table>
<thead>
<tr>
<th>50 mm</th>
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</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>40 mm</td>
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<tr>
<td></td>
</tr>
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<tr>
<td>40 mm</td>
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<td>50 mm</td>
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</table>

**WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR**

<table>
<thead>
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<th>40 mm</th>
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<tbody>
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<tr>
<td>70 mm</td>
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<tr>
<td>40 mm</td>
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</table>

**ANGLE DETAILS**

<table>
<thead>
<tr>
<th>Grade S 275</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of cleat 290 mm</td>
</tr>
<tr>
<td>Number of bolts/leg 4</td>
</tr>
<tr>
<td>Diameter of bolts M 20 Flowdrill</td>
</tr>
<tr>
<td>Pitch 70 mm</td>
</tr>
<tr>
<td>End distance 40 mm</td>
</tr>
<tr>
<td>Number of bolts 8</td>
</tr>
<tr>
<td>Diameter of bolts M 20 Grade 8.8</td>
</tr>
<tr>
<td>Pitch 70 mm</td>
</tr>
<tr>
<td>End distance 50 mm</td>
</tr>
</tbody>
</table>

**SCALE 5.48** Office 1007 Proforma 480
Location: Flowdrill bolts - Tie force

Double angle web cleat connection

Analysis of the connection follows the treatment in the BCSA publication entitled 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with the requirements of BS5950-1:2000.

Universal section beam to RHS column connection using standard bolts connecting the beam to the web cleats. Flowdrill bolts will be utilised for connecting cleats to the column. Structural integrity tie force \( T_f = 75 \) kN

Supporting member details

250 x 250 x 12.5 SHS - Hot finished. Properties (cm): \( A = 117 \), \( r_x = 9.66 \), \( Z_x = 873 \), \( S_x = 1040 \), \( I_x = 10900 \), \( J = 17200 \), \( C = 1280 \)

Supported beam details

406 x 178 x 74 UB.

Distance top of beam/top of cleat \( n = 50 \) mm

Thickness of angle cleats \( t_p = 10 \) mm
Length of angle cleat leg \( l_c = 90 \) mm
Backmark \( b_m = 50 \) mm
Diameter of bolts (double shear) \( d_b = 20 \) mm
Provide 1 line/s of fasteners.
Provide 4 rows of fasteners in each line.
Supported beam end clearance \( e_c = 10 \) mm
Diameter of Flowdrill bolts \( d_b = 1 = 20 \) mm

Pitch of bolts \( p' = 70 \) mm
No. of bolts in supporting memb \( b_n = 1 = 8 \)
As \( T_f \leq T_{rhs} \) (75 kN \( \leq 108.07 \) kN), tie force within the tying capacity of the RHS column wall.
WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

ANGLE DETAILS
SUMMARY
Length of cleat

LEGS CONNECTED
TO SUPPORTING MEMBER
Number of bolts/leg
Diameter of bolts
Pitch
End distance

LEGS CONNECTED
TO BEAM WEB
Number of bolts
Diameter of bolts
Pitch
End distance
Location: Hollo-Bolt - Tie force

Double angle web cleat connection

Analysis of the connection follows the treatment in the BCSA publication entitled 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with the requirements of BS5950-1:2000.

Universal section beam to RHS column connection using standard bolts connecting the beam to the web cleats. Hollo-Bolts will be utilised for connecting cleats to the column. Structural integrity tie force $T_f = 75$ kN

Supporting member details

250 x 250 x 12.5 SHS - Hot finished. Properties (cm): $A = 117 \text{ rx}=9.66 \text{ Zx}=873 \text{ Sx}=1040 \text{ Ix}=10900 \text{ J}=17200 \text{ C}=1280$

Supported beam details

406 x 178 x 74 UB.

Distance top of beam/top of cleat $n=50$ mm

Thickness of angle cleats $t_p=10$ mm
Length of angle cleat leg $l_c=90$ mm
Backmark $b_m=k=50$ mm
Diameter of bolts (double shear) $d_b=20$ mm
Provide 1 line/s of fasteners.
Provide 4 rows of fasteners in each line.
Supported beam end clearance $e_c=10$ mm
Diameter of Hollo-Bolts $d_{b1}=16$ mm

Pitch of bolts $p'=70$ mm
No. of bolts in supporting memb $b_{n1}=8$
As $T_f \leq \text{ Trhs ( } 75 \text{ kN } \leq 119.47 \text{ kN })$, tie force within the tying capacity of the RHS column wall.
WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

ANGLE DETAILS
SUMMARY
90 mm x 90 mm x 10 mm
Grade S 275
Length of cleat 290 mm

LEGS CONNECTED
Number of bolts/leg 4
TO SUPPORTING
Diameter of bolts M 16 Hollo-Bolt
MEMBER
Pitch 70 mm
End distance 40 mm

LEGS CONNECTED
Number of bolts 4
TO BEAM WEB
Diameter of bolts M 20 Grade 8.8
Pitch 70 mm
End distance 40 mm
Location: Ex8 - Double notch and with one line of bolts

Double angle web cleat connection

Analysis of the connection follows the treatment in the BCSA publication entitled 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with the requirements of BS5950-1:2000.

Universal section to universal section connection using standard bolts

Supporting member details

203 x 203 x 46 UC.
Thickness of connecting ply ts=7.2 mm

Supported beam details

203 x 203 x 52 UC.

- Length of notch to end face N=108 mm
- Depth of notch n=24 mm
- Thickness of angle cleats tp=8 mm
- Length of angle cleat leg lc=90 mm
- Backmark bmk=50 mm
- Diameter of bolts (double shear) db=16 mm
- Provide 1 line/s of fasteners.
- Provide 3 rows of fasteners in each line.
- Supported beam end clearance ec=3 mm
- Diameter of bolts (single shear) db1=16 mm
- Pitch of bolts p'=100 mm
- No. of bolts in supporting memb bn1=4
To calculate the load on the outermost bolt due to moment, apply the unit area method. Function of inertia of bolt group:
Dimension et=29.1 mm

WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

ANGLE DETAILS
SUMMARY 90 mm x 90 mm x 8 mm
Grade S 275
Length of cleat 156 mm
Number of bolts/leg 2
Diameter of bolts M 16 Grade 4.6
Pitch 100 mm
End distance 28 mm
Number of bolts 3
Diameter of bolts M 16 Grade 4.6
Pitch 50 mm
End distance 40 mm

Doubly notched beams - guidance should be sought on the effective length if unrestrained against lateral torsional buckling.
Double angle web cleat connection


Universal section to universal section connection using standard bolts

Supporting member details

254 x 254 x 89 UKC
Thickness of connecting ply $t_s = 17.3$ mm

Supported beam details

533 x 210 x 92 UKB

Distance top of beam/top of cleat $d_{nt} = 40$ mm

Thickness of angle cleats $t_p = 10$ mm
Length of angle cleat leg $l_c = 90$ mm
Distance to 1st line of bolts $z_p = 60$ mm
Diameter of bolts (double shear) $d_b = 20$ mm
Provide 1 vertical line of fasteners.
Provide 8 rows of fasteners in each line.
Pitch of bolts $p_1 = 50$ mm
End distance $e_1 = 35$ mm
Supported beam end clearance $g_n = 10$ mm
Diameter of bolts (single shear) $d_{b1} = 20$ mm

Pitch of bolts $p_1' = 100$ mm
No. of bolts in supporting memb $b_{n1} = 8$
To calculate the load on the outermost bolt due to moment, apply the unit area method. Function of inertia of bolt group:
Beam vertical edge distance $e_{lb}=75$ mm

WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR
<table>
<thead>
<tr>
<th><strong>ANGLE DETAILS</strong></th>
<th>90 mm x 90 mm x 10 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SUMMARY</strong></td>
<td>Grade S 275</td>
</tr>
<tr>
<td>Length of cleat</td>
<td>420 mm</td>
</tr>
<tr>
<td><strong>LEGS CONNECTED</strong></td>
<td>Number of bolts/leg 4</td>
</tr>
<tr>
<td>TO SUPPORTING</td>
<td>Diameter of bolts M 20 Grade 8.8</td>
</tr>
<tr>
<td>MEMBER</td>
<td>Pitch 100 mm</td>
</tr>
<tr>
<td>End distance</td>
<td>60 mm</td>
</tr>
<tr>
<td><strong>LEGS CONNECTED</strong></td>
<td>Number of bolts 8</td>
</tr>
<tr>
<td>TO BEAM WEB</td>
<td>Diameter of bolts M 20 Grade 8.8</td>
</tr>
<tr>
<td></td>
<td>Pitch 50 mm</td>
</tr>
<tr>
<td></td>
<td>End distance 30 mm</td>
</tr>
</tbody>
</table>
Location: Ex2 - Joints publication Example 1 (2 lines of bolts)

Double angle web cleat connection


Universal section to universal section connection using standard bolts

Supporting member details

610 x 229 x 140 UKB
Thickness of connecting ply \( t_s = 13.1 \text{ mm} \)

Supported beam details

406 x 178 x 74 UKB

\[
\begin{align*}
\text{gn} & \quad \text{ln} \\
\text{hp} & \quad \text{dnt} \\
\text{hn} & \quad \text{lc}
\end{align*}
\]

Length of notch to end face \( \text{ln} = 120 \text{ mm} \)
Depth of notch \( \text{dnt} = 50 \text{ mm} \)
Thickness of angle cleats \( \text{tp} = 10 \text{ mm} \)
Length of angle cleat long leg \( \text{lc} = 150 \text{ mm} \)
Length of angle cleat short leg \( \text{lc}' = 90 \text{ mm} \)
Distance to 1st line of bolts \( \text{zp} = 50 \text{ mm} \)
Diameter of bolts (double shear) \( \text{db} = 20 \text{ mm} \)
Provide 2 vertical lines of fasteners.
Provide 4 rows of fasteners in each line.
Pitch of bolts \( \text{p1} = 70 \text{ mm} \)
Horizontal bolt spacing \( \text{p2} = 50 \text{ mm} \)
End distance \( \text{e1} = 40 \text{ mm} \)
Supported beam end clearance \( \text{gn} = 10 \text{ mm} \)
Diameter of bolts (single shear) \( d_{bl} = 20 \text{ mm} \)

Pitch of bolts \( p_1' = 70 \text{ mm} \)

No. of bolts in supporting memb \( b_{n1} = 8 \)

To calculate the load on the outermost bolt due to moment, apply the unit area method. Function of inertia of bolt group:

Beam vertical edge distance \( e_{1b} = 90 \text{ mm} \)

WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

\[
\begin{array}{ccc}
  zp & p2 & e2 \\
  50 \text{ mm} & 50 \text{ mm} & 50 \text{ mm} \\
  \cdots & \cdots & \cdots \\
  e_1 = 40 \text{ mm} \\
  p_1 = 70 \text{ mm} \\
  p_1 = 70 \text{ mm} \\
  p_1 = 70 \text{ mm} \\
  e_1 = 40 \text{ mm} \\
\end{array}
\]

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

\[
\begin{array}{ccc}
  e_21 & p3 & e_22 \\
  40 \text{ mm} & 110 \text{ mm} & 40 \text{ mm} \\
  \cdots & \cdots & \cdots \\
  e_21 = 40 \text{ mm} \\
  p_1' = 70 \text{ mm} \\
  p_1' = 70 \text{ mm} \\
  p_1' = 70 \text{ mm} \\
  e_21 = 40 \text{ mm} \\
\end{array}
\]

ANGLE DETAILS

150 mm x 90 mm x 10 mm

SUMMARY

Grade S 275

LENGTH OF CLEAT 290 mm

NUMBER OF BOLTS/LEG 4

DIAMETER OF BOLTS M 20 Grade 8.8

PITCH 70 mm

END DISTANCE 40 mm
**Location: Ex3 - Joints publication Example 2 (Tie force)**

**Double angle web cleat connection**


Universal section to universal section connection using standard bolts

Structural integrity tie force \( F_{Ed} = 230 \, \text{kN} \)

**Supporting member details**

- 305 x 305 x 137 UKC
- Thickness of connecting ply \( t_s = 13.8 \, \text{mm} \)

**Supported beam details**

- 406 x 178 x 74 UKB
  
  ![Diagram of the connection](image)

- Distance top of beam/top of cleat \( d_{nt} = 50 \, \text{mm} \)

- Thickness of angle cleats \( t_p = 10 \, \text{mm} \)
- Length of angle cleat long leg \( l_c = 150 \, \text{mm} \)
- Length of angle cleat short leg \( l_c' = 90 \, \text{mm} \)
- Distance to 1st line of bolts \( z_p = 50 \, \text{mm} \)
- Diameter of bolts (double shear) \( d_b = 20 \, \text{mm} \)
- Provide 2 vertical lines of fasteners.
- Provide 4 rows of fasteners in each line.
- Pitch of bolts \( p_1 = 70 \, \text{mm} \)
- Horizontal bolt spacing \( p_2 = 50 \, \text{mm} \)
- End distance \( e_1 = 40 \, \text{mm} \)
- Supported beam end clearance \( g_n = 10 \, \text{mm} \)
- Diameter of bolts (single shear) \( d_{b1} = 20 \, \text{mm} \)
WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

ANGLE DETAILS
Summary
Length of cleat
Number of bolts/leg
Diameter of bolts
Pitch
End distance
Number of bolts
Diameter of bolts
Pitch
End distance
Location: Ex4 - Using non-standard section (Tie force)

Double angle web cleat connection


Universal section to universal section connection using standard bolts
Structural integrity tie force  $F_{Ed}=175$ kN

Supporting member details

Thickness of connecting ply $ts=10$ mm

Supported beam details

Distance top of beam/top of cleat $dnt=50$ mm

Thickness of angle cleats $tp=10$ mm
Length of angle cleat leg $lc=90$ mm
Distance to 1st line of bolts $zp=60$ mm
Diameter of bolts (double shear) $db=20$ mm
Provide 1 vertical line of fasteners.
Provide 8 rows of fasteners in each line.
Pitch of bolts $pl=50$ mm
End distance $e1=35$ mm
Supported beam end clearance $gn=10$ mm
Diameter of bolts (single shear) $db1=20$ mm

Pitch of bolts $pl'=100$ mm
No. of bolts in supporting memb $bn1=8$
To calculate the load on the outermost bolt due to moment, apply the unit area method. Function of inertia of bolt group:

- Beam vertical edge distance: \( e_1 \) = 85 mm
- Diameter of bolt washer: \( d_w \) = 33 mm

### WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

<table>
<thead>
<tr>
<th>( z_p )</th>
<th>( e_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( e_1 = 35 \text{ mm} )</td>
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<tr>
<td></td>
<td>( p_1 = 50 \text{ mm} )</td>
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<td></td>
<td>( p_1 = 50 \text{ mm} )</td>
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<td></td>
<td>( p_1 = 50 \text{ mm} )</td>
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<td>( p_1 = 50 \text{ mm} )</td>
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<td></td>
<td>( p_1 = 50 \text{ mm} )</td>
</tr>
<tr>
<td>60 mm</td>
<td>30 mm</td>
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### WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

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<tr>
<th></th>
<th>( e_21 = 60 \text{ mm} )</th>
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<tbody>
<tr>
<td></td>
<td>( p_1' = 100 \text{ mm} )</td>
</tr>
<tr>
<td></td>
<td>( p_1' = 100 \text{ mm} )</td>
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<tr>
<td></td>
<td>( p_1' = 100 \text{ mm} )</td>
</tr>
<tr>
<td>e22</td>
<td>( e_21 = 60 \text{ mm} )</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>( e_22 )</th>
<th>( p_3 )</th>
<th>( e_22 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 mm</td>
<td>130 mm</td>
<td>30 mm</td>
</tr>
<tr>
<td>ANGLE DETAILS</td>
<td>90 mm x 90 mm x 10 mm</td>
<td></td>
</tr>
<tr>
<td>-----------------------</td>
<td>-----------------------</td>
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</tr>
<tr>
<td>SUMMARY</td>
<td>Grade S 275</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Length of cleat 420 mm</td>
<td></td>
</tr>
<tr>
<td>LEGS CONNECTED</td>
<td>Number of bolts/leg 4</td>
<td></td>
</tr>
<tr>
<td>TO SUPPORTING</td>
<td>Diameter of bolts M 20 Grade 8.8</td>
<td></td>
</tr>
<tr>
<td>MEMBER</td>
<td>Pitch 100 mm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>End distance 60 mm</td>
<td></td>
</tr>
<tr>
<td>LEGS CONNECTED</td>
<td>Number of bolts 8</td>
<td></td>
</tr>
<tr>
<td>TO BEAM WEB</td>
<td>Diameter of bolts M 20 Grade 8.8</td>
<td></td>
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<tr>
<td></td>
<td>Pitch 50 mm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>End distance 30 mm</td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex5 - Using Flowdrill bolts

Double angle web cleat connection


Universal section beam to RHS column connection using standard bolts connecting the beam to the web cleats. Flowdrill bolts will be utilised for connecting cleats to the column.

Supporting member details

250 x 250 x 10 SHS - Hot finished.
Properties (cm): A=94.9 iy=9.77 Wely=724 Wply=851 Iy=9060 It=14100 Wt=1070

Supported beam details

406 x 178 x 74 UKB

Distance top of beam/top of cleat dnt=50 mm

Thickness of angle cleats tp=10 mm
Length of angle cleat long leg lc=150 mm
Length of angle cleat short leg lc'=90 mm
Distance to 1st line of bolts zp=50 mm
Diameter of bolts (double shear) db=20 mm
Provide 2 vertical lines of fasteners.
Provide 4 rows of fasteners in each line.
Pitch of bolts pl=70 mm
Horizontal bolt spacing p2=50 mm
End distance e1=40 mm
Supported beam end clearance gn=10 mm
Diameter of Flowdrill bolts \( d_{b1} = 20 \text{ mm} \)

Pitch of bolts \( p_1' = 70 \text{ mm} \)

No. of bolts in supporting memb \( b_{n1} = 8 \)

To calculate the load on the outermost bolt due to moment, apply the unit area method. Function of inertia of bolt group:

Beam vertical edge distance \( e_{1b} = 90 \text{ mm} \)

WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

<table>
<thead>
<tr>
<th>( z_p )</th>
<th>( p_2 )</th>
<th>( e_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( e_{1} = 40 \text{ mm} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( p_1 = 70 \text{ mm} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( p_1 = 70 \text{ mm} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( p_1 = 70 \text{ mm} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( e_{1} = 40 \text{ mm} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50 mm</td>
<td>50 mm</td>
<td>50 mm</td>
</tr>
</tbody>
</table>

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

| \( e_{21} = 40 \text{ mm} \) |
| \( p_1' = 70 \text{ mm} \) |
| \( p_1' = 70 \text{ mm} \) |
| \( p_1' = 70 \text{ mm} \) |
| \( e_{21} = 40 \text{ mm} \) |
| 40 mm | 110 mm | 40 mm |

ANGLE DETAILS

<table>
<thead>
<tr>
<th>SUMMARY</th>
<th>150 mm x 90 mm x 10 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade S 275</td>
<td></td>
</tr>
<tr>
<td>Length of cleat 290 mm</td>
<td></td>
</tr>
</tbody>
</table>

LEGGS CONNECTED TO SUPPORTING MEMBER

<table>
<thead>
<tr>
<th>Number of bolts/leg 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bolts M 20 Flowdrill</td>
</tr>
<tr>
<td>Pitch 70 mm</td>
</tr>
<tr>
<td>End distance 40 mm</td>
</tr>
</tbody>
</table>

LEGGS CONNECTED TO BEAM WEB

<table>
<thead>
<tr>
<th>Number of bolts 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bolts M 20 Grade 8.8</td>
</tr>
<tr>
<td>Pitch 70 mm</td>
</tr>
<tr>
<td>End distance 50 mm</td>
</tr>
</tbody>
</table>
Location: Ex6 - Using Flowdrill bolts (Tie force)

Double angle web cleat connection


Universal section beam to RHS column connection using standard bolts connecting the beam to the web cleats. Flowdrill bolts will be utilised for connecting cleats to the column. Structural integrity tie force $F_{Ed}=75 \text{ kN}$

Supporting member details

250 x 250 x 12.5 SHS - Hot finished.
Properties (cm): $A=117$ iy=9.66 Wely=873 Wply=1040 Iy=10900 It=17200 Wt=1280

Supported beam details

406 x 178 x 74 UKB

Distance top of beam/top of cleat $d_{nt}=50 \text{ mm}$

Thickness of angle cleats $t_{p}=10 \text{ mm}$
Length of angle cleat leg $l_{c}=90 \text{ mm}$
Distance to 1st line of bolts $z_{p}=50 \text{ mm}$
Diameter of bolts (double shear) $d_{b}=20 \text{ mm}$
Provide 1 vertical line of fasteners.
Provide 4 rows of fasteners in each line.
Pitch of bolts $p_{1}=70 \text{ mm}$
End distance $e_{1}=40 \text{ mm}$
Supported beam end clearance $g_{n}=10 \text{ mm}$
Diameter of Flowdrill bolts       db1=20 mm

Pitch of bolts                    p1'=70 mm
No. of bolts in supporting memb   bn1=8
As $F_{Ed} \leq F_{Rd}$ ( 75 kN $\leq$ 108.07 kN ), tie force is within the
tying resistance of the RHS column wall.
Diameter of bolt washer           dw=33 mm

WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

ANGLE DETAILS                 90 mm x 90 mm x 10 mm
SUMMARY                        Grade S 275
LENGTH OF CLEAT                290 mm
NUMBER OF BOLTS/LEG            4
DIAMETER OF BOLTS              M 20 Flowdrill
PITCH                         70 mm
END DISTANCE                  40 mm
NUMBER OF BOLTS                4
DIAMETER OF BOLTS              M 20 Grade 8.8
PITCH                         70 mm
END DISTANCE                  40 mm
Location: Ex7 - Using Hollo-Bolt (Tie force)

Double angle web cleat connection


Universal section beam to RHS column connection using standard bolts connecting the beam to the web cleats. Hollo-Bolts will be utilised for connecting cleats to the column. Structural integrity tie force \( F_{Ed} = 75 \) kN

Supporting member details

250 x 250 x 12.5 SHS - Hot finished.
Properties (cm): \( A = 117 \) iy=9.66 Wely=873 Wply=1040 Iy=10900 It=17200 Wt=1280

Supported beam details

406 x 178 x 74 UKB

Distance top of beam/top of cleat \( dnt = 50 \) mm

Thickness of angle cleats \( tp = 10 \) mm
Length of angle cleat leg \( lc = 90 \) mm
Distance to 1st line of bolts \( zp = 50 \) mm
Diameter of bolts (double shear) \( db = 20 \) mm
Provide 1 vertical line of fasteners.
Provide 4 rows of fasteners in each line.
Pitch of bolts \( pl = 70 \) mm
End distance \( el = 40 \) mm
Supported beam end clearance \( gn = 10 \) mm
Diameter of Hollo-Bolts $d_{b1} = 16$ mm

Pitch of bolts $p_{l1} = 70$ mm

No. of bolts in supporting memb $b_{n1} = 8$

As $F_{Ed} \leq FR_{du} (75$ kN $\leq 118.82$ kN $)$, tie force is within the tying resistance of the RHS column wall.

Diameter of bolt washer $d_{w} = 46$ mm

WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

ANGLE DETAILS

<table>
<thead>
<tr>
<th>SUMMARY</th>
<th>90 mm x 90 mm x 10 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade S 275</td>
<td></td>
</tr>
<tr>
<td>Length of cleat</td>
<td>290 mm</td>
</tr>
<tr>
<td>Number of bolts/leg</td>
<td>4</td>
</tr>
<tr>
<td>Diameter of bolts</td>
<td>M 16 Hollo-Bolt</td>
</tr>
<tr>
<td>Pitch</td>
<td>70 mm</td>
</tr>
<tr>
<td>End distance</td>
<td>40 mm</td>
</tr>
<tr>
<td>Number of bolts</td>
<td>4</td>
</tr>
<tr>
<td>Diameter of bolts</td>
<td>M 20 Grade 8.8</td>
</tr>
<tr>
<td>Pitch</td>
<td>70 mm</td>
</tr>
<tr>
<td>End distance</td>
<td>40 mm</td>
</tr>
</tbody>
</table>
Location: Ex8 - As Ex2 but with unrestrained flange for LTB

Double angle web cleat connection


Universal section to universal section connection using standard bolts

Supporting member details

610 x 229 x 140 UKB
Thickness of connecting ply \( t_s = 13.1 \text{ mm} \)

Supported beam details

406 x 178 x 74 UKB

\[
\begin{array}{c}
\text{gn} \\
\text{hp} \\
\text{ln} \\
\text{dnt} \\
\text{hn} \\
\text{lc}
\end{array}
\]

Length of notch to end face \( l_n = 120 \text{ mm} \)
Depth of notch \( d_{nt} = 50 \text{ mm} \)
Thickness of angle cleats \( t_p = 10 \text{ mm} \)
Length of angle cleat long leg \( l_c = 150 \text{ mm} \)
Length of angle cleat short leg \( l_c' = 90 \text{ mm} \)
Distance to 1st line of bolts \( z_p = 50 \text{ mm} \)
Diameter of bolts (double shear) \( d_b = 20 \text{ mm} \)

Provide 2 vertical lines of fasteners.
Provide 4 rows of fasteners in each line.
Pitch of bolts \( p_1 = 70 \text{ mm} \)
Horizontal bolt spacing \( p_2 = 50 \text{ mm} \)
End distance \( e_1 = 40 \text{ mm} \)
Supported beam end clearance \( g_{n} = 10 \text{ mm} \)
Diameter of bolts (single shear) $db_1=20$ mm

Pitch of bolts $p_1'=70$ mm

No. of bolts in supporting memb $bn_1=8$

To calculate the load on the outermost bolt due to moment, apply the unit area method. Function of inertia of bolt group:

Beam vertical edge distance $e_1b=90$ mm

Length between restraints $L_b=5000$ mm

WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

ANGLE DETAILS

SUMMARY

150 mm x 90 mm x 10 mm

Grade S 275

Length of cleat 290 mm

LEGGS CONNECTED

Number of bolts/leg 4

TO SUPPORTING

Diameter of bolts M 20 Grade 8.8

Pitch 70 mm

End distance 40 mm

LEGGS CONNECTED

Number of bolts 8

TO BEAM WEB

Diameter of bolts M 20 Grade 8.8

Pitch 70 mm

End distance 50 mm
Standard grade 8.8 bolts will be used with this fin plate connection.

Calculations are in accordance with BS 5950-1:2000 and the analysis follows the recommended design procedure checks given in the BCSA/SCI publication 'Joints in Simple Construction' 2nd Edition.

Grade 8.8 bolts used in clearance holes.

Beam to beam web connection.

Factored end reaction $Q=350$ kN

610 x 229 x 140 UKB available for shear and bearing $ts=13.1$ mm

533 x 210 x 92 UKB

Length of notch to end face $N=120$ mm

Depth of notch $n=50$ mm

Thickness of fin plate $tf=10$ mm

Diameter of bolts $db=20$ mm

Pitch of bolts vertically $p=70$ mm

Vertical edge distance $e1=40$ mm

Horizontal edge distance $e2=40$ mm

Cross-centres of bolts $g=50$ mm

Dimension $a=50$ mm

Supported beam end clearance $ec=10$ mm

Weld leg length $s=8$ mm
FIN PLATE SUMMARY

_plate size_ 360 mm x 140 mm x 10 mm
Grade S 275
Number of bolts 10
Diameter of bolts M 20 Grade 8.8
Fillet weld size 8 mm
Electrode strength Es 35
Location: Joints publication. Fin plates Example 2

Standard grade 8.8 bolts will be used with this fin plate connection.

**Fin plate connection**

Calculations are in accordance with BS 5950-1:2000 and the analysis follows the recommended design procedure checks given in the BCSA/SCI publication 'Joints in Simple Construction' 2nd Edition.

Grade 8.8 bolts used in clearance holes.

Beam to column flange connection.

- Structural integrity tie force \( T_f = 175 \) kN
- 305 x 305 x 137 UKC
- 406 x 178 x 74 UKB
- Thickness of fin plate \( t_f = 10 \) mm
- Diameter of bolts \( d_b = 20 \) mm
- Pitch of bolts vertically \( p = 70 \) mm
- Vertical edge distance \( e_1 = 40 \) mm
- Horizontal edge distance \( e_2 = 50 \) mm
- Eccentricity of bolt group \( a = 50 \) mm
- Supported beam end clearance \( e_c = 10 \) mm
- Weld leg length \( s = 8 \) mm

**FIN PLATE SUMMARY**

- Use 8 mm fillet weld
- Vertical distance: 40 mm, 70 mm, 70 mm, 70 mm
- Horizontal distance: 50 mm, 50 mm

---

SCALE 5.48  Office 1007  Proforma 481
<table>
<thead>
<tr>
<th>Specification</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate size</td>
<td>290 mm x 100 mm x 10 mm</td>
</tr>
<tr>
<td>Grade</td>
<td>S 275</td>
</tr>
<tr>
<td>Number of bolts</td>
<td>4</td>
</tr>
<tr>
<td>Diameter of bolts</td>
<td>M 20 Grade 8.8</td>
</tr>
<tr>
<td>Fillet weld size</td>
<td>8 mm</td>
</tr>
<tr>
<td>Electrode strength</td>
<td>Es 35</td>
</tr>
</tbody>
</table>
Location: Joints publication Ex 1 smaller beam connection

Standard grade 8.8 bolts will be used with this fin plate connection.

**Fin plate connection**

Calculations are in accordance with BS 5950-1:2000 and the analysis follows the recommended design procedure checks given in the BCSA/SCI publication 'Joints in Simple Construction' 2nd Edition.

Grade 8.8 bolts used in clearance holes.

Beam to beam web connection.

**Factored end reaction**

Q=200 kN

**610 x 229 x 140 UKB**

available for shear and bearing ts=13.1 mm

**406 x 178 x 74 UKB**

Length of notch to end face N=120 mm

Depth of notch n=50 mm

Thickness of fin plate tf=10 mm

Diameter of bolts db=20 mm

Pitch of bolts vertically p=70 mm

Vertical edge distance e1=40 mm

Horizontal edge distance e2=50 mm

Eccentricity of bolt group a=50 mm

Supported beam end clearance ec=10 mm

Weld leg length s=8 mm

Length of supported beam Lb=3500 mm

**Effective length of supported beam**

As supported notched beam is unrestrained it must be checked for torsional buckling in accordance with BS 5950 pt.1 Clause 4.3.

Length between restraints Lb=3500 mm

Rounding value, consider Le =1.29 Lb

Check supported beam for lateral torsional buckling using a value of 1.29 Lb as the effective length.
FIN PLATE SUMMARY

Plate size  290 mm x 100 mm x 10 mm
Grade S 275
Number of bolts  4
Diameter of bolts  M 20 Grade 8.8
Fillet weld size  8 mm
Electrode strength Es 35
Location: non-standard section example (extended fin plate)

Standard grade 8.8 bolts will be used with this fin plate connection.

Fin plate connection

Calculations are in accordance with BS 5950-1:2000 and the analysis follows the recommended design procedure checks given in the BCSA/SCI publication 'Joints in Simple Construction' 2nd Edition.

Grade 8.8 bolts used in clearance holes.

Beam to column web connection.

Structural integrity tie force available for shear and bearing
Tf=180 kN
ts=13 mm

Thickness of fin plate tf=10 mm
Diameter of bolts db=20 mm
Pitch of bolts vertically p=60 mm
Vertical edge distance e1=40 mm
Horizontal edge distance e2=50 mm
Eccentricity of bolt group a=160 mm
Supported beam end clearance ec=120 mm
Weld leg length s=8 mm
FIN PLATE SUMMARY

Use 8 mm fillet weld

Plate size 440 mm x 210 mm x 10 mm
Grade S 275
Number of bolts 7
Diameter of bolts M 20 Grade 8.8
Fillet weld size 8 mm
Electrode strength Es 35
Location: Beam to column web connection, 610UB section

Standard grade 8.8 bolts will be used with this fin plate connection.

Calculation of fin plate connection

Grades of bolts used in clearance holes.

Structural integrity tie force $T_f = 200$ kN

203 x 203 x 86 UKC available for shear and bearing $t_s = 12.7$ mm

610 x 305 x 238 UKB
Thickness of fin plate $t_f = 10$ mm
Diameter of bolts $d_b = 20$ mm
Pitch of bolts vertically $p = 70$ mm
Vertical edge distance $e_1 = 40$ mm
Horizontal edge distance $e_2 = 40$ mm
Eccentricity of bolt group $a = 50$ mm
Supported beam end clearance $e_c = 10$ mm
Weld leg length $s = 8$ mm
FIN PLATE SUMMARY

Plate size  430 mm x 90 mm x 10 mm
Grade S 275
Number of bolts  6
Diameter of bolts  M 20 Grade 8.8
Fillet weld size  8 mm
Electrode strength Es 35
Location: Ex1 - Fin plate supporting an I section beam

Standard grade 8.8 bolts will be used with this fin plate connection.

Fin plate connection

Calculations are in accordance with BS EN 1993-1:2005 and the analysis follows the recommended design procedure checks given in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3'. Grade 8.8 bolts used in clearance holes.

Beam to beam web connection.

Factored end reaction \( V_{Ed} = 350 \text{ kN} \)

available for shear and bearing \( t_s = 13.1 \text{ mm} \)

533 x 210 x 92 UKB
Length of notch to end face \( l_n = 120 \text{ mm} \)
Depth of notch \( d_{nt} = 50 \text{ mm} \)
Thickness of fin plate \( t_p = 10 \text{ mm} \)
Diameter of bolts \( d_b = 20 \text{ mm} \)
Vertical edge distance \( e_1 = 45 \text{ mm} \)
Horizontal edge distance \( e_2 = 50 \text{ mm} \)
Distance to 1st vert. bolt line \( z_p = 50 \text{ mm} \)
Bolt pitch vertically \( p_1 = 70 \text{ mm} \)
Cross-centres of bolts \( p_2 = 60 \text{ mm} \)
Supported beam end clearance \( g_n = 10 \text{ mm} \)
Beam vertical edge distance \( e_{1b} = 90 \text{ mm} \)
Length between restraints \( L_b = 5000 \text{ mm} \)

Check supported beam for lateral torsional buckling using a value of 1.21 \( L_b \) as the effective length.
Weld leg length \( s = 8 \text{ mm} \)
## FIN PLATE SUMMARY

<table>
<thead>
<tr>
<th></th>
<th>p2</th>
<th>e2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>o</td>
<td>o</td>
</tr>
<tr>
<td></td>
<td>o</td>
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<td>o</td>
<td>o</td>
</tr>
<tr>
<td></td>
<td>o</td>
<td></td>
</tr>
</tbody>
</table>
**Use 8 mm fillet weld**
|   |   |
|   |   |

- e1=45 mm
- p1=70 mm
- p1=70 mm
- p1=70 mm
- p1=70 mm
- e1=45 mm

50 mm 60 mm 50 mm

- Plate size 370 mm x 160 mm x 10 mm
- Grade S 275
- Number of bolts 10
- Diameter of bolts M20 Grade 8.8
- Fillet weld size 8 mm
Location: Ex2 - Fin plate supporting an I section beam

Standard grade 8.8 bolts will be used with this fin plate connection.

Calculations are in accordance with BS EN 1993-1:2005 and the analysis follows the recommended design procedure checks given in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3'. Grade 8.8 bolts used in clearance holes.

Beam to column web connection.

Structural integrity tie force
305 x 305 x 137 UKC
available for shear and bearing
406 x 178 x 74 UKB
Thickness of fin plate
Diameter of bolts
Vertical edge distance
Horizontal edge distance
Distance to 1st vert. bolt line
Bolt pitch vertically
Supported beam end clearance
Weld leg length

FIN PLATE SUMMARY

Use 8 mm fillet weld

SCALE 5.48 Office 1007 Proforma 481
Plate size    300 mm x 100 mm x 10 mm
Grade S 275
Number of bolts    4
Diameter of bolts    M 20 Grade 8.8
Fillet weld size    8 mm
Location: Ex3 - Supporting member is a SHS column

Standard grade 8.8 bolts will be used with this fin plate connection.

Fin plate connection

Calculations are in accordance with BS EN 1993-1:2005 and the analysis follows the recommended design procedure checks given in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3'. Grade 8.8 bolts used in clearance holes.

Beam to RHS column connection.

Factored end reaction $V_{Ed}=350$ kN

300 x 300 x 10 SHS - Hot finished.

Properties (cm): $A=115$ iy=11.8 Wely=1070 Wply=1250 Iy=16000 It=24800 Wt=1580

457 x 191 x 67 UKB

Thickness of fin plate $t_p=10$ mm

Diameter of bolts $d_b=20$ mm

Vertical edge distance $e_1=50$ mm

Horizontal edge distance $e_2=50$ mm

Distance to 1st vert. bolt line $z_p=50$ mm

Bolt pitch vertically $p_1=70$ mm

Supported beam end clearance $g_n=10$ mm

Beam vertical edge distance $e_{lb}=90$ mm

Weld leg length $s=10$ mm
FIN PLATE SUMMARY

Plate size: 380 mm x 100 mm x 10 mm
Grade: S 275
Number of bolts: 5
Diameter of bolts: M 20 Grade 8.8
Fillet weld size: 10 mm
Location: Example 8.3.1, 'Manual on Connections'

**Seating bracket**

The analysis of the connection is based on the procedure used in the BCSA publication 'Manual on connections'. Standard angle section is used for the seating bracket.

---

**Stability cleat**

Calculations are in accordance with BS5950-1:2000 Section 6.

Factored vertical load on bracket V=254 kN
150 x 90 x 10 mm Unequal Angle.

**Bolt group connecting the angle seat to supporting member**

<table>
<thead>
<tr>
<th>Diameter of bolts</th>
<th>db=20 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bolts to be used</td>
<td>bn=4</td>
</tr>
</tbody>
</table>

Pitch of bolts vertically pitch=55 mm
End distance e2=55 mm
Clearance hole diameter Dh=db+2=22 mm

**Column details**

254 x 254 x 73 UC.
Supported beam details

533 x 210 x 92 UB.

Bearing strength of the seating bracket

This is investigated at the root of the angle seat. A 45 degree load dispersion from the supported beam root is used.

Dispersion length at U/S of beam \( T_u = T_c + T + r = 37.6 \) mm
Dispersion length at seat root \( L_{eb} = 2T_u + t_c + 1.172r_c = 100.18 \) mm
Length of seating bracket \( l_s = 120 \) mm
Dispersion length less than seat length.

As \( V \leq P_{cr} \) (254 kN \leq 275.51 kN), shear force within bearing capacity at root of seat.

Hence the design of the seating bracket connection is OK.

<table>
<thead>
<tr>
<th>SEATING BRACKET</th>
<th>150 mm x 90 mm x 10 mm Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Grade S 275 x 120 mm long</td>
</tr>
<tr>
<td>LEG CONNECTED</td>
<td>Number of bolts 4</td>
</tr>
<tr>
<td>TO SUPPORTING MEMBER</td>
<td>Diameter of bolts M 20 Grade 8.8</td>
</tr>
<tr>
<td></td>
<td>Pitch vertically 55 mm</td>
</tr>
<tr>
<td></td>
<td>End distance ( e_2 ) 55 mm</td>
</tr>
<tr>
<td>SUPPORTED BEAM END DETAILS</td>
<td>533 x 210 x 92 UB</td>
</tr>
<tr>
<td></td>
<td>End clearance 6 mm</td>
</tr>
<tr>
<td></td>
<td>Bearing length 'b1' 23 mm</td>
</tr>
<tr>
<td></td>
<td>Applied flange force 254 kN</td>
</tr>
<tr>
<td></td>
<td>Local web capacity 221 kN</td>
</tr>
<tr>
<td></td>
<td>Web buckling capacity 148 kN</td>
</tr>
<tr>
<td></td>
<td>Bearing stiffeners are required.</td>
</tr>
</tbody>
</table>

SUPPORTING COLUMN

254 x 254 x 73 UC
**Location: Welded connection**

**Seating bracket**

The analysis of the connection is based on the procedure used in the BCSA publication 'Manual on connections'. Standard angle section is used for the seating bracket.

- Stability cleat
- Calculations are in accordance with BS5950-1:2000 Section 6.

Factored vertical load on bracket $V=156$ kN
Fillet weld leg length $s=6$ mm
150 x 90 x 10 mm Unequal Angle.

**Weld connecting seat to supporting member**

- Design strength of weld per mm $pw=0.7s*pw=924$ N/mm
- Total value of end plate weld $Pw=1e*pw/1000=266.11$ kN
Since $V \leq Pw$ (156 kN $\leq 266.11$ kN), shear on bracket is within allowable and the 6 mm fillet weld is suitable.

**Supported beam details**

457 x 152 x 67 UB.

**Bearing strength of the seating bracket**

This is investigated at the root of the angle seat. A 45 degree load dispersion from the supported beam root is used.
- Dispersion length at U/S of beam $Tus=Tc+T+r=37$ mm
- Dispersion length at seat root $Leb=2*Tus+tc+1.172*rc=94.954$ mm
- Length of seating bracket $ls=150$ mm
- Dispersion length less than seat length.
As $V \leq Pcr$ (156 kN $\leq 261.12$ kN), shear force within bearing capacity at root of seat.

Hence the design of the seating bracket connection is OK.
<table>
<thead>
<tr>
<th><strong>SEATING BRACKET</strong></th>
<th><strong>150 mm x 90 mm x 10 mm Angle</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SUMMARY</strong></td>
<td>Grade S 275 x 150 mm long</td>
</tr>
<tr>
<td><strong>LEG CONNECTED</strong></td>
<td>Fillet weld size 6 mm</td>
</tr>
<tr>
<td><strong>TO SUPPORTING</strong></td>
<td>Length of weld 288 mm</td>
</tr>
<tr>
<td><strong>MEMBER</strong></td>
<td></td>
</tr>
<tr>
<td><strong>SUPPORTED BEAM</strong></td>
<td>457 x 152 x 67 UB</td>
</tr>
<tr>
<td><strong>END DETAILS</strong></td>
<td>End clearance 6 mm</td>
</tr>
<tr>
<td></td>
<td>Bearing length 'b1' 23 mm</td>
</tr>
<tr>
<td></td>
<td>Applied flange force 156 kN</td>
</tr>
<tr>
<td></td>
<td>Local web capacity 182 kN</td>
</tr>
<tr>
<td></td>
<td>Web buckling capacity 123 kN</td>
</tr>
<tr>
<td></td>
<td>Bearing stiffeners are required.</td>
</tr>
</tbody>
</table>
Seating bracket

The analysis of the connection assumes that a standard angle section is used for the seating bracket.

Stability cleat not shown

Calculations are in accordance with EC3 Parts 1-5 and 1-8.

The stability cleat will provide torsional resistance against lateral buckling of the beam and will resist tie and erection forces. The seating cleat will resist the vertical load.

Design vertical load on bracket $V_{Ed}=254$ kN

150 x 90 x 10 mm Unequal Angle (Advance UKA)

Bolt group connecting the angle seat to supporting member

Diameter of bolts $d_b=20$ mm

Distance to first row of bolts $p=55$ mm

Pitch of bolts vertically $p_1=55$ mm

Supporting column details

254 x 254 x 73 UKC
Supported beam details

533 x 210 x 92 UKB
Length of seating bracket \( l_s = 120 \text{ mm} \)
Stiff bearing length \( s_s = 23 \text{ mm} \)
Stiff bearing position \( C = 0 \text{ mm} \)

Design resistance of the web is less than the design transverse force applied to the web (254 kN). Bearing stiffeners are required.

SEATING BRACKET
150 mm x 90 mm x 10 mm Angle
SUMMARY
Grade S 275 x 120 mm long
LEG CONNECTED
Number of bolts 4
TO SUPPORTING
Diameter of bolts M 20 Grade 8.8
MEMBER
Pitch vertically 55 mm
End distance \( p \) 55 mm
SUPPORTED BEAM
533 x 210 x 92 UKB
END DETAILS
End clearance 6 mm
Bearing length ‘\( s_s \)’ 23 mm
Applied flange force 254 kN
Web resistance 217 kN
Bearing stiffeners are required.

SUPPORTING COLUMN
254 x 254 x 73 UKC
Location: Ex2 - Welded connection

Seating bracket

The analysis of the connection assumes that a standard angle section is used for the seating bracket.

\[
\text{Stability cleat} \\
\text{not shown}
\]

\[
\text{VE}_d
\]

Calculations are in accordance with EC3 Parts 1-5 and 1-8.

The stability cleat will provide torsional resistance against lateral buckling of the beam and will resist tie and erection forces. The seating cleat will resist the vertical load.

Design vertical load on bracket \( \text{VE}_d = 156 \text{ kN} \)
150 x 90 x 10 mm Unequal Angle (Advance UKA)

Weld connecting seat to supporting member

Shelf angle thickness \( t = 10 \text{ mm} \)
Fillet weld leg length \( s = 6 \text{ mm} \)
Force in the weld \( \text{FW}_d = \text{VE}_d / lw = 0.54167 \text{ kN/mm} \)
Design strength of weld per mm \( \text{FW}_d = 0.7s\times \text{Fwvd}/1000 = 0.9324 \text{ kN/mm} \)
As \( \text{FW}_d \leq \text{FW}_d (0.54167 \text{ kN/mm} \leq 0.9324 \text{ kN/mm}) \), shear on bracket is within allowable and the 6 mm fillet weld is OK.

Supported beam details

457 x 152 x 67 UKB
Length of seating bracket \( l_s = 150 \text{ mm} \)
Stiff bearing length \( s_s = 23 \text{ mm} \)
Stiff bearing position \( C = 0 \text{ mm} \)

<table>
<thead>
<tr>
<th>SEATING BRACKET</th>
<th>150 mm x 90 mm x 10 mm Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Grade S 275 x 150 mm long</td>
</tr>
<tr>
<td>LEG CONNECTED</td>
<td>Fillet weld size 6 mm</td>
</tr>
<tr>
<td>TO SUPPORTING</td>
<td>Length of weld 288 mm</td>
</tr>
<tr>
<td>MEMBER</td>
<td></td>
</tr>
<tr>
<td>SUPPORTED BEAM</td>
<td>457 x 152 x 67 UKB</td>
</tr>
<tr>
<td>END DETAILS</td>
<td>End clearance 6 mm</td>
</tr>
<tr>
<td></td>
<td>Bearing length 'ss' 23 mm</td>
</tr>
<tr>
<td></td>
<td>Applied flange force 156 kN</td>
</tr>
<tr>
<td></td>
<td>Web resistance 179 kN</td>
</tr>
</tbody>
</table>
Location: Ex1 - Welded connection and equal angle

Shelf angle supporting slab

The analysis of the connection assumes that a standard angle section is used for the shelf angle. Distance ecc is from the face of the supporting beam to the point of application of the design vertical load V. The shelf angle vertical leg could be facing up or down. Calculations are in accordance with BS5950-1.

The shelf angle vertical leg is welded at points A and B to the web of the supporting beam. The latter is a UB or UC section. Hit and miss FW will be adopted.

Ultimate load on shelf angle V=60 kN/m
Grade of shelf angle S 275
Grade of supporting member S 275
Yield strength of steel fy=275 N/mm²

Shelf angle details

90 x 90 x 10 mm Equal Angle.

Supporting beam details

457 x 152 x 67 UB.

Weld connecting shelf angle to supporting beam

Supporting beam web thickness tsb=9 mm
Shelf angle thickness T=10 mm
Fillet weld leg length s=6 mm

Normally a hit and miss fillet weld length of hmFWl=100 mm along the length of the shelf angle at top and bottom is specified (i.e. 100 mm weld and then 100 mm miss along the length of the shelf angle). The distance below is from the face of the supporting beam to the point of application of the design vertical load V.

Distance to vertical load ecc=75 mm
Bottom weld shear force governs Fw=0.12 kN/mm
Design strength of weld per mm Pw=0.7*s*pw/1000=0.924 kN/mm

As Fw ≤ Pw (0.12 kN/mm ≤ 0.924 kN/mm), force on shelf angle is within allowable and the 6 mm fillet weld is OK.

Shelf angle bending check

Section modulus Z=1000*T^2/(6*1000)=16.667 cm³
Design moment M=V*ecc/1000=4.5 kNm/m
Moment capacity \( M_c = \frac{Z \cdot f_y}{1000} = 4.5833 \text{ kNm/m} \)

As \( M \leq M_c \) (4.5 kNm/m \( \leq 4.5833 \text{ kNm/m} \)), design moment is OK.

**Deflection check**

| Maximum allowable deflection: \( \Delta \) | 1.5 mm
| Defln at angle toe (support): \( \Delta \) | 1.0396 mm
| Defln at angle toe (midspan): \( \Delta \) | 1.0471 mm
| Maximum allowable deflection: \( \Delta \) | 1.5 mm

Deflection is OK.

The analysis is valid as the longitudinal bending deflection is small relative to the angle deflection (i.e. \( p = 0.6153 \leq 5 \text{ percent} \)).

**DESIGN SUMMARY**

| Shelf angle | 90 mm x 90 mm x 10 mm Angle
| Grade of steel | S 275
| Fillet weld size | 6 mm
| Hit and miss FW length | 100 mm hit, 100 mm miss
| Supporting beam | 457 x 152 x 67 UB
| Applied flange force | 60 kN/m
| Defln at angle toe (support): \( \Delta \) | 1.04 mm
| Defln at angle toe (midspan): \( \Delta \) | 1.05 mm
| Maximum allowable deflection: \( \Delta \) | 1.5 mm
Location: Ex2 - Welded connection and unequal angle

Shelf angle supporting slab

The analysis of the connection assumes that a standard angle section is used for the shelf angle. Distance ecc is from the face of the supporting beam to the point of application of the design vertical load V. The shelf angle vertical leg could be facing up or down. Calculations are in accordance with BS5950-1.

The shelf angle vertical leg is welded at points A and B to the web of the supporting beam. The latter is a UB or UC section. Hit and miss FW will be adopted.

Ultimate load on shelf angle \( V = 60 \) kN/m
Grade of shelf angle \( S \ 275 \)
Grade of supporting member \( S \ 275 \)
Yield strength of steel \( f_y = 275 \) N/mm²

Shelf angle details

150 x 90 x 10 mm Unequal Angle.

Supporting beam details

457 x 152 x 67 UB.

Weld connecting shelf angle to supporting beam

Supporting beam web thickness \( t_{sb} = 9 \) mm
Shelf angle thickness \( T = 10 \) mm
Fillet weld leg length \( s = 6 \) mm

Normally a hit and miss fillet weld length of \( h m F W l = 100 \) mm along the length of the shelf angle at top and bottom is specified (i.e. 100 mm weld and then 100 mm miss along the length of the shelf angle). The distance below is from the face of the supporting beam to the point of application of the design vertical load V.

Distance to vertical load \( e c c = 75 \) mm
Bottom weld shear force governs \( F_w = 0.12 \) kN/mm
Design strength of weld per mm \( P_w = 0.7 * s * p_w / 1000 = 0.924 \) kN/mm

As \( F_w \leq P_w \) (0.12 kN/mm ≤ 0.924 kN/mm), force on shelf angle is within allowable and the 6 mm fillet weld is OK.

Shelf angle bending check

Section modulus \( Z = 1000 * T^2 / (6 * 1000) = 16.667 \) cm³
Design moment \( M = V * e c c / 1000 = 4.5 \) kNm/m
Moment capacity

\[ Mc = \frac{Z \times fy}{1000} = 4.5833 \text{ kNm/m} \]

As \( M \leq Mc \) (4.5 kNm/m \leq 4.5833 kNm/m), design moment is OK.

**Deflection check**

<table>
<thead>
<tr>
<th>Maximum allowable deflection</th>
<th>delta=1.5 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Defln at angle toe (support)</td>
<td>0.98971 mm</td>
</tr>
<tr>
<td>Defln at angle toe (midspan)</td>
<td>1.0133 mm</td>
</tr>
<tr>
<td>Maximum allowable deflection</td>
<td>1.5 mm</td>
</tr>
</tbody>
</table>

Deflection is OK.

The analysis is valid as the longitudinal bending deflection is small relative to the angle deflection (i.e. \( p=0.11969 \leq 5 \text{ percent} \)).

**DESIGN SUMMARY**

| Shelf angle                   | 150 mm x 90 mm x 10 mm Angle |
| Grade of steel                | S 275                        |
| Fillet weld size              | 6 mm                         |
| Hit and miss FW length        | 100 mm hit, 100 mm miss      |
| Supporting beam               | 457 x 152 x 67 UB            |
| Applied flange force          | 60 kN/m                      |
| Defln at angle toe (support)  | 0.99 mm                      |
| Defln at angle toe (midspan)  | 1.01 mm                      |
| Maximum allowable deflection  | 1.5 mm                       |
Shelf angle supporting slab

The analysis of the connection assumes that a standard angle section is used for the shelf angle. Distance ecc is from the face of the supporting beam to the point of application of the design vertical load VEd. The shelf angle vertical leg could be facing up or down. Calculations are in accordance with EC3 Part 1-1 & EC3 Part 1-8.

The shelf angle vertical leg is welded at points A and B to the web of the supporting beam. The latter is a UB or UC section. Hit and miss FW will be adopted.

Design load on shelf angle VEd=60 kN/m
Grade of angle seat S 275
Grade of supporting member S 275
Yield strength of steel fy=275 N/mm^2

Shelf angle details

90 x 90 x 10 mm Equal Angle (Advance UKA)

Supporting beam details

457 x 152 x 67 UKB

Weld connecting shelf angle to supporting member

Supporting beam web thickness twsb=9 mm
Shelf angle thickness t=10 mm
Fillet weld leg length s=6 mm
The distance below is from the face of the supporting beam to the point of application of the design vertical load VEd.
Distance to vertical load ecc=75 mm
Bottom weld shear force governs FwEd=0.12 kN/mm
Design strength of weld per mm FwRd=0.7*s*fvwd/1000=0.9324 kN/mm
As FwEd ≤ FwRd ( 0.12 kN/mm ≤ 0.9324 kN/mm ), force on shelf angle is within allowable and the 6 mm fillet weld is OK.

Shelf angle bending check

Section modulus Wel=1000*t^2/(6*1000)=16.667 cm^3
Design moment MEEd=VEd*ecc/1000=4.5 kNm/m
Partial factor for resistance gamM0=1.0
Moment resistance per m McRd=Wel*fy/(gamM0*1000)=4.5833 kNm/m
As MEEd ≤ McRd ( 4.5 kNm/m ≤ 4.5833 kNm/m ), design moment is OK.
Deflection check

Maximum allowable deflection \( \delta = 1.5 \text{ mm} \)
Defln at angle toe (support) \( 1.0396 \text{ mm} \)
Defln at angle toe (midspan) \( 1.0471 \text{ mm} \)
Maximum allowable deflection \( 1.5 \text{ mm} \)

Deflection is OK.

The analysis is valid as the longitudinal bending deflection is small relative to the angle deflection \( (i.e., \ p = 0.6153 \leq 5 \text{ percent}) \).

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Shelf angle size</th>
<th>90 mm x 90 mm x 10 mm Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade of steel</td>
<td>S 275</td>
</tr>
<tr>
<td>Fillet weld size</td>
<td>6 mm</td>
</tr>
<tr>
<td>Hit and miss FW length</td>
<td>100 mm hit, 100 mm miss</td>
</tr>
<tr>
<td>Supporting beam</td>
<td>457 x 152 x 67 UKB</td>
</tr>
<tr>
<td>Design load on shelf angle</td>
<td>60 kN/m</td>
</tr>
<tr>
<td>Defln at angle toe (support)</td>
<td>1.04 mm</td>
</tr>
<tr>
<td>Defln at angle toe (midspan)</td>
<td>1.05 mm</td>
</tr>
<tr>
<td>Maximum allowable deflection</td>
<td>1.5 mm</td>
</tr>
</tbody>
</table>
Location: Modified Example 16, 'Steelwork Design Guide'.

Partial depth end plate

Analysis of the connection follows the treatment in the BCSA publication 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with BS5950-1:2000.

Universal section to universal section connection using standard bolts.

Factored end reaction \( Q = 490 \text{ kN} \)

Supporting member details

610 x 229 x 140 UKB

Thickness of connecting ply \( t_s = 13.1 \text{ mm} \)

Supported beam details

533 x 210 x 92 UKB

Length of notch to end face \( N = 120 \text{ mm} \)

Depth of notch \( n = 36 \text{ mm} \)

Reduced depth of sectn.at notch \( D_n = D - n = 497.1 \text{ mm} \)

Thickness of end plate \( t_p = 10 \text{ mm} \)

Diameter of bolts \( d_b = 20 \text{ mm} \)

Number of bolts in end plate \( b_n = 8 \)

Pitch of bolts vertically \( p = 100 \text{ mm} \)

Edge distance \( e_1 = 30 \text{ mm} \)

End distance \( e_2 = 50 \text{ mm} \)

Cross-centres of bolts \( c_r_s = 90 \text{ mm} \)

Weld leg length \( s = 6 \text{ mm} \)

Applied shear force \( Q_a = 1020 \text{ kN} \)
END PLATE SUMMARY

SUMMARY OF END PLATE REQUIREMENTS

- 400 mm x 150 mm x 10 mm plate
- Grade S 275
- Number of bolts: 8
- Diameter of bolts: M 20 Grade 8.8
- Fillet weld size: 6 mm
- Electrode strength: Es 35
**Partial depth end plate**

Analysis of the connection follows the treatment in the BCSA publication 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with BS5950-1:2000. Universal section to universal section connection using standard bolts. Factored end reaction \( Q = 300 \) kN

**Supporting member details**

610 x 229 x 140 UKB

Thickness of connecting ply \( t_s = 13.1 \) mm

**Supported beam details**

406 x 178 x 74 UKB

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of notch to end face</td>
<td>( N = 120 ) mm</td>
</tr>
<tr>
<td>Depth of notch</td>
<td>( n = 50 ) mm</td>
</tr>
<tr>
<td>Reduced depth of section at notch</td>
<td>( D_n = D - n = 362.8 ) mm</td>
</tr>
<tr>
<td>Thickness of end plate</td>
<td>( t_p = 10 ) mm</td>
</tr>
<tr>
<td>Diameter of bolts</td>
<td>( d_b = 20 ) mm</td>
</tr>
<tr>
<td>Number of bolts in end plate</td>
<td>( b_n = 8 )</td>
</tr>
<tr>
<td>Pitch of bolts vertically</td>
<td>( p = 70 ) mm</td>
</tr>
<tr>
<td>Edge distance</td>
<td>( e_1 = 30 ) mm</td>
</tr>
<tr>
<td>End distance</td>
<td>( e_2 = 40 ) mm</td>
</tr>
<tr>
<td>Cross-centres of bolts</td>
<td>( c_{rs} = 140 ) mm</td>
</tr>
<tr>
<td>Weld leg length</td>
<td>( s = 6 ) mm</td>
</tr>
<tr>
<td>Length between restraints</td>
<td>( L_b = 4500 ) mm</td>
</tr>
</tbody>
</table>
END PLATE SUMMARY

SUMMARY OF END PLATE REQUIREMENTS

- 290 mm x 200 mm x 10 mm plate
- Grade S 275
- Number of bolts: 8
- Diameter of bolts: M 20 Grade 8.8
- Fillet weld size: 6 mm
- Electrode strength: Es 35
Location: Example 2 - Joints publication

Partial depth end plate

Analysis of the connection follows the treatment in the BCSA publication 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with BS5950-1:2000.

Universal section to universal section connection using standard bolts.

Structural integrity tie force $T_f = 175$ kN

Supporting member details

305 x 305 x 137 UKC

Thickness of connecting ply $t_s = 13.8$ mm

Supported beam details

406 x 178 x 74 UKB

Thickness of end plate $t_p = 8$ mm

Diameter of bolts $d_b = 20$ mm

Number of bolts in end plate $b_n = 8$

Pitch of bolts vertically $p = 70$ mm

Edge distance $e_1 = 35$ mm

End distance $e_2 = 40$ mm

Cross-centres of bolts $c_{rs} = 90$ mm

Weld leg length $s = 6$ mm
END PLATE SUMMARY

290 mm x 160 mm x 8 mm plate
Grade S 275
Number of bolts 8
Diameter of bolts M 20 Grade 8.8
Fillet weld size 6 mm
Electrode strength Es 35
Location: Non-standard sections with tie force and vertical load

Partial depth end plate

Analysis of the connection follows the treatment in the BCSA publication 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with BS5950-1:2000.
Universal section to universal section connection using standard bolts.
Structural integrity tie force $T_f=160 \text{ kN}$

Supporting member details

Thickness of connecting ply $t_s=16 \text{ mm}$

Supported beam details

Thickness of end plate $t_p=8 \text{ mm}$
Diameter of bolts $d_b=20 \text{ mm}$
Number of bolts in end plate $b_n=8$
Pitch of bolts vertically $p=70 \text{ mm}$
Edge distance $e_1=35 \text{ mm}$
End distance $e_2=40 \text{ mm}$
Cross-centres of bolts $c_{rs}=90 \text{ mm}$
Weld leg length $s=6 \text{ mm}$
END PLATE SUMMARY

SUMMARY OF
END PLATE
REQUIREMENTS

290 mm x 160 mm x 8 mm plate
Grade S 275
Number of bolts 8
Diameter of bolts M 20 Grade 8.8
Fillet weld size 6 mm
Electrode strength Es 35
Location: Modified Example 16, 'Steelwork Design Guide' - RHS column

Partial depth end plate

Analysis of the connection follows the treatment in the BCSA publication 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with BS5950-1:2000. Universal section beam to RHS column connection using Flowdrill connectors.

Factored end reaction \( Q = 490 \text{ kN} \)

Supporting member details

350 x 350 x 12.5 SHS - Hot finished.
Properties (cm): \( A = 167 \), \( r_x = 13.7 \), \( z_x = 1800 \), \( S_x = 2110 \), \( I_x = 31500 \), \( J = 48900 \), \( C = 2650 \)

Supported beam details

533 x 210 x 92 UKB

Thickness of end plate \( t_p = 10 \text{ mm} \)
Diameter of bolts \( d_b = 20 \text{ mm} \)
Number of bolts in end plate \( b_n = 8 \)
Pitch of bolts vertically \( p = 100 \text{ mm} \)
Edge distance \( e_1 = 50 \text{ mm} \)
End distance \( e_2 = 50 \text{ mm} \)
Cross-centres of bolts \( c_r s = 90 \text{ mm} \)
Distance top beam to plate \( n = 50 \text{ mm} \)
Weld leg length \( s = 6 \text{ mm} \)
END PLATE SUMMARY

SUMMARY OF END PLATE REQUIREMENTS

400 mm x 190 mm x 10 mm plate
Grade S 275
Number of bolts 8
Diameter of bolts M 20 Flowdrill
Fillet weld size 6 mm
Electrode strength Es 35
**Location: Structural integrity - Flowdrill bolts**

**Partial depth end plate**

Analysis of the connection follows the treatment in the BCSA publication 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with BS5950-1:2000. Universal section beam to RHS column connection using Flowdrill connectors. Structural integrity tie force $T_f=75$ kN

**Supporting member details**

250 x 250 x 10 SHS - Hot finished.

Properties (cm): $A=94.9$ $r_x=9.77$ $Z_x=724$ $S_x=851$ $I_x=9060$ $J=14100$ $C=1070$

**Supported beam details**

406 x 178 x 74 UKB

Thickness of end plate $t_p=8$ mm

Diameter of bolts $d_b=20$ mm

Number of bolts in end plate $b_n=10$

Pitch of bolts vertically $p=70$ mm

Edge distance $e_1=35$ mm

End distance $e_2=40$ mm

Cross-centres of bolts $c_{rs}=90$ mm

Weld leg length $s=6$ mm

Since $T_f \leq T_{rhs}$ ($75$ kN $\leq 224.94$ kN) tie force within the tying capacity of the RHS column wall.
END PLATE SUMMARY

SUMMARY OF END PLATE REQUIREMENTS

360 mm x 160 mm x 8 mm plate
Grade S 275
Number of bolts 10
Diameter of bolts M 20 Flowdrill
Fillet weld size 6 mm
Electrode strength Es 35
Location: Similar to Flowdrill example using Hollo-Bolts

Partial depth end plate

Analysis of the connection follows the treatment in the BCSA publication 'Manual on Connections Volume 1 - Joints in simple construction' amended to comply with BS5950-1:2000. Universal section beam to RHS column connection using Hollo-Bolt connectors. Structural integrity tie force $T_f=75$ kN

Supporting member details

250 x 250 x 10 SHS - Hot finished. Properties (cm): $A=94.9$ $rx=9.77$ $Zx=724$ $Sx=851$ $Ix=9060$ $J=14100$ $C=1070$

Supported beam details

406 x 178 x 74 UKB

Thickness of end plate $tp=8$ mm
Diameter of bolts $db=16$ mm
Number of bolts in end plate $bn=10$
Pitch of bolts vertically $p=70$ mm
Edge distance $e1=35$ mm
End distance $e2=40$ mm
Cross-centres of bolts $crs=90$ mm
Weld leg length $s=6$ mm
Since $T_f \leq Trhs \ (75 \text{ kN} \leq 212.02 \text{ kN})$ tie force within the tying capacity of the RHS column wall.
END PLATE SUMMARY

360 mm x 160 mm x 8 mm plate
Grade S 275
Number of bolts 10
Diameter of bolts M16 Hollo-Bolt
Fillet weld size 6 mm
Electrode strength Es 35
Partial depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3' and BS EN 1993-1: 2005.

Universal section to universal section connection using standard bolts.

Factored end reaction \( V_{Ed} = 550 \) kN

Supporting member details

610 x 229 x 140 UKB

Thickness of connecting ply \( t_s = 13.1 \) mm

Supported beam details

533 x 210 x 92 UKB

Length of notch to end face \( l_n = 122 \) mm

Depth of notch \( d_{nt} = 50 \) mm

Reduced depth of section at notch \( h_n = h - d_{nt} = 483.1 \) mm

Thickness of end plate \( t_p = 12 \) mm

Diameter of bolts \( d_b = 20 \) mm

Number of bolts in end plate \( b_n = 12 \)

Pitch of bolts vertically \( p_1 = 70 \) mm

Vertical end distance \( e_1 = 40 \) mm

Horizontal edge distance \( e_2 = 30 \) mm

Cross-centres of bolts \( p_3 = 140 \) mm

Weld size (leg length) \( s = 6 \) mm
END PLATE SUMMARY

SUMMARY OF END PLATE REQUIREMENTS

430 mm x 200 mm x 12 mm plate
Grade S 275
Number of bolts 12
Diameter of bolts M 20 Grade 8.8
Fillet weld size 6 mm
Partial depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3' and BS EN 1993-1: 2005. Universal section to universal section connection using standard bolts. Structural integrity tie force $F_{Ed}=175$ kN

Supporting member details

305 x 305 x 137 UKC
Thickness of connecting ply $t_s=13.8$ mm

Supported beam details

406 x 178 x 74 UKB

<table>
<thead>
<tr>
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</tr>
</thead>
</table>

$ h $

Thickness of end plate $t_p=10$ mm
Diameter of bolts $d_b=20$ mm
Number of bolts in end plate $b_n=8$
Pitch of bolts vertically $p_1=70$ mm
Vertical end distance $e_1=40$ mm
Horizontal edge distance $e_2=35$ mm
Cross-centres of bolts $p_3=90$ mm
Weld size (leg length) $s=6$ mm
Diameter of bolt washer $d_w=33$ mm
END PLATE SUMMARY

<table>
<thead>
<tr>
<th></th>
<th>90 mm</th>
<th>40 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>70 mm</td>
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</table>

<table>
<thead>
<tr>
<th>SUMMARY OF END PLATE REQUIREMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>290 mm x 160 mm x 10 mm plate</td>
</tr>
<tr>
<td>Grade S 275</td>
</tr>
<tr>
<td>Number of bolts 8</td>
</tr>
<tr>
<td>Diameter of bolts M 20 Grade 8.8</td>
</tr>
<tr>
<td>Fillet weld size 6 mm</td>
</tr>
</tbody>
</table>
Location: Simple Joints to Eurocode 3, Example 3

Partial depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3' and BS EN 1993-1: 2005.

Universal section beam to RHS column connection using Flowdrill connectors.

Factored end reaction $V_{Ed}=550$ kN

Supporting member details

250 x 250 x 12.5 SHS - Hot finished.

Properties (cm): $A=117$ iy=9.66 Wely=873 Wply=1040 Iy=10900 It=17200 Wt=1280

Supported beam details

533 x 210 x 92 UKB

Thickening of end plate $t_p=12$ mm

Diameter of bolts $d_b=20$ mm

Number of bolts in end plate $b_n=12$

Pitch of bolts vertically $p_1=70$ mm

Vertical end distance $e_1=40$ mm

Horizontal edge distance $e_2=30$ mm

Cross-centres of bolts $p_3=140$ mm

Weld size (leg length) $s=6$ mm
END PLATE SUMMARY

SUMMARY OF
END PLATE
REQUIREMENTS

430 mm x 200 mm x 12 mm plate
Grade S 275
Number of bolts 12
Diameter of bolts M 20 Flowdrill
Fillet weld size 6 mm
Partial depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3' and BS EN 1993-1: 2005. Universal section beam to RHS column connection using Flowdrill connectors. Structural integrity tie force FEd=275 kN

Supporting member details

250 x 250 x 12.5 SHS - Hot finished. Properties (cm): A=117 iy=9.66 Wely=873 Wply=1040 Iy=10900 It=17200 Wt=1280

Supported beam details

533 x 210 x 92 UKB

Thickness of end plate tp=12 mm
Diameter of bolts db=20 mm
Number of bolts in end plate bn=12
Pitch of bolts vertically p1=70 mm
Vertical end distance e1=40 mm
Horizontal edge distance e2=35 mm
Cross-centres of bolts p3=140 mm
Weld size (leg length) s=6 mm
Diameter of bolt washer dw=33 mm
END PLATE SUMMARY

SUMMARY OF
END PLATE
REQUIREMENTS

430 mm x 210 mm x 12 mm plate
Grade S 275
Number of bolts 12
Diameter of bolts M 20 Flowdrill
Fillet weld size 6 mm
Partial depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3' and BS EN 1993-1: 2005.
Universal section beam to RHS column connection using Hollo-Bolt connectors.
Structural integrity tie force $F_{Ed}=275$ kN

Supporting member details

250 x 250 x 12.5 SHS - Hot finished.
Properties (cm): $A=117$ iy=9.66 Wely=873 Wply=1040 Iy=10900 It=17200 Wt=1280

Supported beam details

533 x 210 x 92 UKB

Thickness of end plate $t_p=12$ mm
Diameter of bolts $d_b=20$ mm
Number of bolts in end plate $b_n=10$
Pitch of bolts vertically $p_1=80$ mm
Vertical end distance $e_1=45$ mm
Horizontal edge distance $e_2=45$ mm
Cross-centres of bolts $p_3=110$ mm
Weld size (leg length) $s=6$ mm
Diameter of bolt washer $d_w=46$ mm
END PLATE SUMMARY

SUMMARY OF END PLATE REQUIREMENTS

- Number of bolts: 10
- Diameter of bolts: M 20 Hollo-Bolt
- Fillet weld size: 6 mm

410 mm x 200 mm x 12 mm plate
Grade S 275
Location: Simple Joints to Eurocode 3, Example 4

Partial depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3' and BS EN 1993-1: 2005.

Universal section beam to RHS column connection using Hollo-Bolt connectors.

Factored end reaction $V_{Ed}=550$ kN

Supporting member details

250 x 250 x 12.5 SHS - Hot finished.

Properties (cm): $A=117$ $iy=9.66$ $W_{ely}=873$ $W_{ply}=1040$ $Iy=10900$ $It=17200$

$W_t=1280$

Supported beam details

533 x 210 x 92 UKB

| / | / | / | / | h |

Thickness of end plate $tp=12$ mm

Diameter of bolts $db=20$ mm

Number of bolts in end plate $bn=10$

Pitch of bolts vertically $p1=80$ mm

Vertical end distance $e1=45$ mm

Horizontal edge distance $e2=45$ mm

Cross-centres of bolts $p3=110$ mm

Weld size (leg length) $s=6$ mm
END PLATE SUMMARY

SUMMARY OF END PLATE

REQUIREMENTS

410 mm x 200 mm x 12 mm plate
Grade S 275
Number of bolts 10
Diameter of bolts M 20 Hollo-Bolt
Fillet weld size 6 mm
Location: Simple Joints to Eurocode 3, Example 1

Full depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication entitled 'Joints in Steel Construction: Simple Joints to EC3' and BS EN 1993-1:2005.

Factored end reaction \( V_{Ed}=400 \text{ kN} \)

Supporting member details

305 x 305 x 137 UKC
Dimensions (mm): \( h_c=320.5 \) \( b_c=309.2 \) \( t_w=13.8 \) \( t_f=21.7 \) \( r_c=15.2 \)
Supported beam will be connected to the supporting member web.

Supported beam details

406 x 178 x 74 UKB
Dimensions (mm): \( h=412.8 \) \( b_f=179.5 \) \( t_w=9.5 \) \( t_f=16 \) \( r=10.2 \)
Properties (cm): \( A_r=94.5 \)

End plate details

Thickness of end plate \( t_p=10 \text{ mm} \)
Diameter of bolts \( d_b=20 \text{ mm} \)
Number of bolts in end plate \( b_n=8 \)
Distance from the top of the beam to the first row of bolts \( p=90 \text{ mm} \)
Pitch of bolts vertically \( p_1=70 \text{ mm} \)
Edge distance \( e_2=30 \text{ mm} \)
Cross-centres of bolts \( p_3=90 \text{ mm} \)
Length of end plate \( h_p=450 \text{ mm} \)
Weld size (leg length) \( s=6 \text{ mm} \)

END PLATE SUMMARY

![End plate diagram]

Scale 5.48 Office 1007 Proforma 485
| SUMMARY OF              |                             |
| END PLATE              | Grade S 275                 |
| REQUIREMENTS           |                             |
| Number of bolts        | 8                           |
| Diameter of bolts      | M 20 Grade 8.8              |
| Fillet weld size       | 6 mm                        |
Full depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication entitled 'Joints in Steel Construction: Simple Joints to EC3' and BS EN 1993-1:2005.

Structural integrity tie force $F_{Ed}=350$ kN

Supporting member details

305 x 305 x 137 UKC
Dimensions (mm): $h_c=320.5$ $b_c=309.2$ $t_w_c=13.8$ $t_f_c=21.7$ $r_c=15.2$
Supported beam will be connected to the supporting member web.

Supported beam details

406 x 178 x 74 UKB
Dimensions (mm): $h=412.8$ $b_f=179.5$ $t_w=9.5$ $t_f=16$ $r=10.2$
Properties (cm): $A_r=94.5$

End plate details

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of end plate</td>
<td>$t_p=10$ mm</td>
</tr>
<tr>
<td>Diameter of bolts</td>
<td>$d_b=20$ mm</td>
</tr>
<tr>
<td>Number of bolts in end plate</td>
<td>$b_n=8$</td>
</tr>
<tr>
<td>Distance from the top of the beam to the first row of bolts</td>
<td>$p=90$ mm</td>
</tr>
<tr>
<td>Pitch of bolts vertically</td>
<td>$p_1=70$ mm</td>
</tr>
<tr>
<td>Edge distance</td>
<td>$e_2=30$ mm</td>
</tr>
<tr>
<td>Cross-centres of bolts</td>
<td>$p_3=90$ mm</td>
</tr>
<tr>
<td>Length of end plate</td>
<td>$h_p=440$ mm</td>
</tr>
<tr>
<td>Weld size (leg length)</td>
<td>$s=6$ mm</td>
</tr>
<tr>
<td>Diameter of bolt washer</td>
<td>$d_w=33$ mm</td>
</tr>
</tbody>
</table>

END PLATE SUMMARY
| SUMMARY OF END PLATE REQUIREMENTS |
|-----------------------------------|-----------------|
| 440 mm x 150 mm x 10 mm plate     | Grade S 275     |
| Number of bolts: 8               | Diameter of bolts: M 20 Grade 8.8 |
| Fillet weld size: 6 mm           |
Location: Ex1 - Joints in Steel construction example 1

Bolted flush end plate eaves connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \[ M_a = 724 \text{ kNm} \]
Factored shear force \[ V_a = 304 \text{ kN} \]
Axial force (+ve compression) \[ N_a = 102 \text{ kN} \]

Supporting column details
610 x 229 x 101 UB.

Supported rafter section properties - parent section
457 x 191 x 74 UB.

Section properties - haunch cutting
533 x 210 x 82 UB.

Combined properties & restrictions
Pitch of rafter \[ \Theta = 10^\circ \]
Vertical overall depth \[ D_o = 900 \text{ mm} \]
Haunch angle to end plate \[ \phi' = 78^\circ \]
Sample output for SCALE Proforma 486. (ans=1)  Page: 2
Steel design to BS5950-1:2000 and Eurocode 3  Made by: IFB
Flush end plate haunched connection  Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.  Ref No: SC486 BS

Diameter of bolts  bd=24 mm
Thickness of plate  tp=20 mm
Total number of bolts  bn=12
Number of shear bolts  ns=4
Distance to first row of bolts  ex=80 mm
Distance to beam flange  x=20 mm
Pitch of tension bolts  p2=90 mm
Edge distance for shear bolts  pes=130 mm
Over-all length of end plate  Lc=965 mm
Bolt cross-centres  g=90 mm
Width of end plate  bp=200 mm
Assumed web fillet weld size  sww=8 mm
Assumed flange fillet weld size  swf=10 mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts  Pr1=303.75 kN
Row 2  Pr2=268.7 kN
Row 3  Pr3=209.99 kN
Row 4  Pr4=183.21 kN
Weld size  ss=8 mm
Selected fillet weld size  swt=10 mm
Selected fillet weld size  swf=8 mm
Factored moment  Mr=237 kNm
Factored shear force  Vr=43 kN
Axial force (+ve compression)  Nr=-56 kN
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts: 12
Bolt diameter: 24 mm
Bolt grade: 8.8
Welds:
- Tension flange fillet weld: 10 mm
- Web fillet weld: 8 mm
- Compression flange fillet weld: 8 mm

Supporting column section: 610 x 229 x 101 UB
Beam / Rafter section: 457 x 191 x 74 UB
Haunch section: 533 x 210 x 82 UB

Column stiffener details
Compression stiffeners: 2/90 mm x 20 mm Grade S 275
connected by 4/8 mm fillet welds.
Location: Ex2 - As example 1 with tension stiffeners

Bolted flush end plate eaves connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( Ma = 735 \, \text{kNm} \)
Factored shear force \( Va = 304 \, \text{kN} \)
Axial force (+ve compression) \( Na = 102 \, \text{kN} \)

Supporting column details
610 x 229 x 101 UB.

Supported rafter section properties - parent section
457 x 191 x 74 UB.

Section properties - haunch cutting
533 x 210 x 82 UB.

Combined properties & restrictions
Pitch of rafter \( \Theta = 10^\circ \)
Vertical overall depth \( Do = 925 \, \text{mm} \)
Haunch angle to end plate \( \phi' = 78^\circ \)
Diameter of bolts \( bd = 24 \text{ mm} \)
Thickness of plate \( tp = 20 \text{ mm} \)
Total number of bolts \( bn = 14 \)
Number of shear bolts \( ns = 4 \)
Distance to first row of bolts \( ex = 80 \text{ mm} \)
Distance to beam flange \( x = 20 \text{ mm} \)
Pitch of tension bolts \( p2 = 70 \text{ mm} \)
Edge distance for shear bolts \( pes = 90 \text{ mm} \)
Over-all length of end plate \( Lc = 975 \text{ mm} \)
Bolt cross-centres \( g = 90 \text{ mm} \)
Width of end plate \( bp = 200 \text{ mm} \)
Assumed web fillet weld size \( sww = 8 \text{ mm} \)
Assumed flange fillet weld size \( swf = 10 \text{ mm} \)

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

Top row of bolts \( Pr1 = 303.75 \text{ kN} \)
Row 2 \( Pr2 = 254.76 \text{ kN} \)
Row 3 \( Pr3 = 142.5 \text{ kN} \)
Row 4 \( Pr4 = 142.5 \text{ kN} \)
Row 5 \( Pr5 = 142.5 \text{ kN} \)
Weld size \( ss = 8 \text{ mm} \)

Applied moment exceeds moment capacity \( (665.06 \text{ kNm} > 655.26 \text{ kNm}) \).

Stiffener width \( bsgt = 90 \text{ mm} \)
Proposed rib thickness \( tsr = 15 \text{ mm} \)
Proposed weld size \( ssr = 11 \text{ mm} \)

Selected fillet weld size \( swt = 12 \text{ mm} \)
Selected fillet weld size \( sfw = 8 \text{ mm} \)
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 14
Bolt diameter 24 mm
Bolt grade 8.8
Welds
  Tension flange fillet weld 12 mm
  Web fillet weld 8 mm
  Compression flange fillet weld 10 mm

Supporting column section 610 x 229 x 101 UB
Beam / Rafter section 457 x 191 x 74 UB
Haunch section 533 x 210 x 82 UB

Column stiffener details

Compression stiffeners 2/90 mm x 30 mm Grade S 275
  connected by 4/8 mm fillet welds.
Tension stiffeners
  Full depth 15 mm thick.
  2/90 mm wide placed midway
  between the following bolt rows:
  1 and 2
  2 and 3
Location: Ex3 - Internal connection, tension rib stiffeners

Bolted flush end plate eaves connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment: $Ma=612 \text{ kNm}$
Factored shear force: $Va=214 \text{ kN}$
Axial force (+ve compression): $Na=95 \text{ kN}$

Supporting column details

610 x 229 x 101 UB.

Supported rafter section properties - parent section

406 x 178 x 74 UB.

Section properties - haunch cutting

533 x 210 x 82 UB.

Combined properties & restrictions

Pitch of rafter: $\Theta=15^\circ$
Vertical overall depth: $Do=850 \text{ mm}$
Haunch angle to end plate: $\phi'=67^\circ$
Diameter of bolts                    bd=24 mm
Thickness of plate                  tp=20 mm
Total number of bolts               bn=10
Number of shear bolts               ns=4
Distance to first row of bolts      ex=80 mm
Distance to beam flange             x=20 mm
Pitch of tension bolts              p2=90 mm
Edge distance for shear bolts       pes=130 mm
Over-all length of end plate        Lc=935 mm
Bolt cross-centres                  g=90 mm
Width of end plate                  bp=200 mm
Assumed web fillet weld size        sww=8 mm
Assumed flange fillet weld size     swf=10 mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts                    Pr1=315.52 kN
Row 2                                Pr2=280.47 kN
Row 3                                Pr3=186.45 kN
Weld size                            ss=8 mm
Applied moment exceeds moment capacity ( 552.31 kNm > 529.69 kNm ).
Stiffener width                      bsgt=90 mm
Proposed rib thickness               tsr=15 mm
Proposed weld size                   ssr=11 mm

Slip factor for preloaded bolts      mu=0.5
Coefficient for type of hole         Ks=1
Selected fillet weld size            swt=12 mm
Selected fillet weld size            sfw=8 mm
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts: 10
Bolt diameter: 24 mm
Bolt grade: HSFG
General grade (8.8) to BS4395 Part 1. Nuts grade 10.
Welds: Tension flange fillet weld 12 mm
Web fillet weld 8 mm
Compression flange fillet weld 10 mm

Supporting column section: 610 x 229 x 101 UB
Beam / Rafter section: 406 x 178 x 74 UB
Haunch section: 533 x 210 x 82 UB

Column stiffener details

Compression stiffeners: 2/90 mm x 20 mm Grade S 275
connected by 4/8 mm fillet welds.
Tension stiffeners: Full depth 15 mm thick.
2/90 mm wide placed midway
between the following bolt rows:
1 and 2
2 and 3
Location: Ex4 - Minimum number of bolts connection

Bolted flush end plate eaves connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( Ma = 165 \) kNm
Factored shear force \( Va = 94 \) kN
Axial force (+ve compression) \( Na = 56 \) kN

Supporting column details

457 x 191 x 98 UB.

Supported rafter section properties - parent section

305 x 165 x 46 UB.

Section properties - haunch cutting

356 x 171 x 51 UB.

Combined properties & restrictions

Pitch of rafter \( \Theta = 15^\circ \)
Vertical overall depth \( Do = 550 \) mm
Haunch angle to end plate \( \phi' = 63^\circ \)
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bolts</td>
<td>bd=20 mm</td>
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<tr>
<td>Thickness of plate</td>
<td>tp=16 mm</td>
</tr>
<tr>
<td>Total number of bolts</td>
<td>bn=6</td>
</tr>
<tr>
<td>Number of shear bolts</td>
<td>ns=2</td>
</tr>
<tr>
<td>Distance to first row of bolts</td>
<td>ex=80 mm</td>
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<tr>
<td>Distance to beam flange</td>
<td>x=20 mm</td>
</tr>
<tr>
<td>Pitch of tension bolts</td>
<td>p2=90 mm</td>
</tr>
<tr>
<td>Distance tension to shear bolts</td>
<td>p3=315 mm</td>
</tr>
<tr>
<td>Edge distance for shear bolts</td>
<td>pes=130 mm</td>
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<tr>
<td>Bolt cross-centres</td>
<td>g=90 mm</td>
</tr>
<tr>
<td>Width of end plate</td>
<td>bp=180 mm</td>
</tr>
<tr>
<td>Assumed web fillet weld size</td>
<td>sww=8 mm</td>
</tr>
<tr>
<td>Assumed flange fillet weld size</td>
<td>swf=8 mm</td>
</tr>
</tbody>
</table>

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

<table>
<thead>
<tr>
<th>Component</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top row of bolts</td>
<td>Pr1=243.59 kN</td>
</tr>
<tr>
<td>Row 2</td>
<td>Pr2=192.39 kN</td>
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<tr>
<td>Selected fillet weld size</td>
<td>swt=9 mm</td>
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<tr>
<td>Selected fillet weld size</td>
<td>swf=8 mm</td>
</tr>
<tr>
<td>Factored moment</td>
<td>Mr=98 kNm</td>
</tr>
<tr>
<td>Factored shear force</td>
<td>Vr=33 kN</td>
</tr>
<tr>
<td>Axial force (+ve compression)</td>
<td>Nr=-23 kN</td>
</tr>
</tbody>
</table>


**CONNECTION SUMMARY - all dimensions in diagram are in mm**

Bolt diameter increased for practical purposes to 24 mm.

<table>
<thead>
<tr>
<th></th>
<th>16</th>
<th>180</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>ex</td>
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</tr>
<tr>
<td>p2</td>
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<td>p3</td>
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<td></td>
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<td>pes</td>
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</tr>
</tbody>
</table>

Number of bolts 6
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 9 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm

Supporting column section 457 x 191 x 98 UB
Beam / Rafter section 305 x 165 x 46 UB
Haunch section 356 x 171 x 51 UB

Column stiffener details

No enhancement of the column is required.
Bolted flush end plate eaves connection - beam to column

The connection under consideration is assumed to be attached to the supporting column flange. The calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure in 'Joints in Steel Construction Moment Connections' by SCI.

The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment $\text{MyEd}'=724$ kNm
Factored shear force $\text{VED}'=304$ kN
Axial force (+ve compression) $\text{NEd}'=102$ kN

Supporting column details

$610 \times 229 \times 101$ UKB

Supported rafter section details - parent section

$457 \times 191 \times 74$ UKB

Section properties - haunch cutting

$533 \times 210 \times 82$ UKB

Combined properties and restrictions

Pitch of rafter $\theta = 10^\circ$
Vertical overall depth $h_o = 900$ mm
Haunch angle to end plate $\phi = 78^\circ$
Bolt details

Diameter of bolts db=24 mm
Tensile resistance (EC3 Part 1-8) \( F_{Rd} = k_2 f_{ub} A_t / (\gamma_M 2 \times 10^3) = 203.33 \text{ kN} \)

End plate details

Thickness of plate tp=20 mm
Total number of bolts bn=12
Number of shear bolts ns=4
Distance to beam flange x1=15 mm
Distance to first row of bolts e1=75 mm
Pitch of tension bolts p1=75 mm
Edge distance for shear bolts e1'=110 mm
Over-all length of end plate hp=965 mm
Bolt cross-centres p3=90 mm
Width of end plate bp=200 mm
Assumed web fillet weld size sww=8 mm
Assumed flange fillet weld size swf=10 mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts \( P_{r1} = 310.03 \text{ kN} \)
Row 2 \( P_{r2} = 258.66 \text{ kN} \)
Row 3 \( P_{r3} = 152.68 \text{ kN} \)
Row 4 \( P_{r4} = 152.68 \text{ kN} \)

Design axial load on column \( N_{Edc} = 500 \text{ kN} \)
Weld size (leg length) ss=8 mm

As \( M_{Ed} > M_{Rd} \) (656.61 kNm > 638.57 kNm), the applied moment exceeds the moment resistance. Tension stiffeners are required.

Stiffener width bsgt=90 mm
Proposed rib stiffener thickness tsr=15 mm
Proposed weld size ssr=12 mm

Selected fillet weld size swt=12 mm
Selected fillet weld size sfw=8 mm

REVERSED MOMENT CONDITION

Factored moment \( M_{rEd} = 237 \text{ kNm} \)
Factored shear force \( V_{rEd} = 43 \text{ kN} \)
Axial force (+ve compression) \( N_{rEd} = -56 \text{ kN} \)
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 12
Bolt diameter 24 mm
Bolt grade 8.8
Welds
  Tension flange fillet weld 12 mm
  Web fillet weld 8 mm
  Compression flange fillet weld 8 mm

Supporting column section 610 x 229 x 101 UKB
Beam / Rafter section 457 x 191 x 74 UKB
Haunch section 533 x 210 x 82 UKB

Column stiffener details

Compression stiffeners 2/90 mm x 20 mm Grade S 275
connected by 4/8 mm fillet welds.
Tension stiffeners Full depth 15 mm thick.
  2/90 mm wide placed midway
  between the following bolt rows:
  1 and 2
  2 and 3
  3 and 4
Location: Ex2 - With compression stiffeners only

Bolted flush end plate eaves connection - beam to column

The connection under consideration is assumed to be attached to the supporting column flange. The calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure in 'Joints in Steel Construction Moment Connections' by SCI.

The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment
MyEd' = 735 kNm
Factored shear force
VEd' = 304 kN
Axial force (+ve compression)
NEd' = 102 kN

Supporting column details
610 x 229 x 101 UKB

Supported rafter section details - parent section
457 x 191 x 74 UKB

Section properties - haunch cutting
533 x 210 x 82 UKB

Combined properties and restrictions
Pitch of rafter
Theta = 10°
Vertical overall depth
ho = 950 mm
Haunch angle to end plate
phi' = 78°
Bolt details

Diameter of bolts  
\[ db = 24 \text{ mm} \]

Tensile resistance (EC3 Part 1-8)  
\[ F_{Rd} = k_2 f_{ub} A_t / (\gamma_M 2 \times 10^3) = 203.33 \text{ kN} \]

End plate details

Thickness of plate  
\[ tp = 20 \text{ mm} \]

Total number of bolts  
\[ bn = 14 \]

Number of shear bolts  
\[ ns = 4 \]

Distance to beam flange  
\[ x_1 = 15 \text{ mm} \]

Distance to first row of bolts  
\[ e_1 = 75 \text{ mm} \]

Pitch of tension bolts  
\[ p_1 = 70 \text{ mm} \]

Edge distance for shear bolts  
\[ e_1' = 75 \text{ mm} \]

Over-all length of end plate  
\[ h_p = 1000 \text{ mm} \]

Bolt cross-centres  
\[ p_3 = 90 \text{ mm} \]

Width of end plate  
\[ b_p = 200 \text{ mm} \]

Assumed web fillet weld size  
\[ s_{ww} = 8 \text{ mm} \]

Assumed flange fillet weld size  
\[ s_{wf} = 10 \text{ mm} \]

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts  
\[ P_{r1} = 310.03 \text{ kN} \]

Row 2  
\[ P_{r2} = 248.49 \text{ kN} \]

Row 3  
\[ P_{r3} = 142.5 \text{ kN} \]

Row 4  
\[ P_{r4} = 142.5 \text{ kN} \]

Row 5  
\[ P_{r5} = 142.5 \text{ kN} \]

Design axial load on column  
\[ N_{Edc} = 500 \text{ kN} \]

Weld size (leg length)  
\[ s_{ss} = 8 \text{ mm} \]

Selected fillet weld size  
\[ s_{swt} = 12 \text{ mm} \]

Selected fillet weld size  
\[ s_{sfw} = 8 \text{ mm} \]
**CONNECTION SUMMARY** - all dimensions in diagram are in mm

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
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<tbody>
<tr>
<td>e1</td>
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<tr>
<td>Bolt diameter</td>
<td>24 mm</td>
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<tr>
<td>Bolt grade</td>
<td>8.8</td>
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<tr>
<td>Welds</td>
<td>Tension flange fillet weld</td>
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<tr>
<td></td>
<td>Web fillet weld</td>
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<tr>
<td></td>
<td>Compression flange fillet weld</td>
</tr>
<tr>
<td>Supporting column section</td>
<td>610 x 229 x 101 UKB</td>
</tr>
<tr>
<td>Beam / Rafter section</td>
<td>457 x 191 x 74 UKB</td>
</tr>
<tr>
<td>Haunch section</td>
<td>533 x 210 x 82 UKB</td>
</tr>
</tbody>
</table>

**Column stiffener details**

<table>
<thead>
<tr>
<th>Details</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression stiffeners</td>
<td>2/90 mm x 30 mm Grade S 275</td>
</tr>
<tr>
<td></td>
<td>connected by 4/8 mm fillet welds.</td>
</tr>
</tbody>
</table>
Location: Ex3 - With compression & full depth tension stiffeners

Bolted flush end plate eaves connection - beam to column

The connection under consideration is assumed to be attached to the supporting column flange. The calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure in 'Joints in Steel Construction Moment Connections' by SCI. Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment
\[ My_{Ed}' = 612 \text{ kNm} \]
Factored shear force
\[ V_{Ed}' = 214 \text{ kN} \]
Axial force (+ve compression)
\[ N_{Ed}' = 95 \text{ kN} \]

Supporting column details

610 x 229 x 101 UKB

Supported rafter section details - parent section

406 x 178 x 74 UKB

Section properties - haunch cutting

533 x 210 x 82 UKB
Combined properties and restrictions

Pitch of rafter \( \Theta = 15^\circ \)
Vertical overall depth \( h_o = 850 \text{ mm} \)
Haunch angle to end plate \( \phi' = 67^\circ \)

Bolt details

Diameter of bolts \( d_b = 24 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( F_{tRd} = k_2 f_{ub} A_t / (\gamma_M 2 \times 10^3) = 203.33 \text{ kN} \)

End plate details

Thickness of plate \( t_p = 20 \text{ mm} \)
Total number of bolts \( b_n = 10 \)
Number of shear bolts \( n_s = 4 \)
Distance to beam flange \( x_1 = 20 \text{ mm} \)
Distance to first row of bolts \( e_1 = 80 \text{ mm} \)
Pitch of tension bolts \( p_1 = 90 \text{ mm} \)
Edge distance for shear bolts \( e_1' = 130 \text{ mm} \)
Over-all length of end plate \( h_p = 935 \text{ mm} \)
Bolt cross-centres \( p_3 = 90 \text{ mm} \)
Width of end plate \( b_p = 200 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( s_{wf} = 10 \text{ mm} \)

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts \( P_{r1} = 310.03 \text{ kN} \)
Row 2 \( P_{r2} = 274.98 \text{ kN} \)
Row 3 \( P_{r3} = 197.44 \text{ kN} \)
Design axial load on column \( N_{Edc} = 500 \text{ kN} \)
Weld size (leg length) \( s = 8 \text{ mm} \)
As \( M_{Ed} > M_{Rd} \) (552.31 kNm > 542.87 kNm), the applied moment exceeds the moment resistance. Tension stiffeners are required.
Stiffener width \( b_{sgt} = 90 \text{ mm} \)
Proposed rib stiffener thickness \( t_{sr} = 15 \text{ mm} \)
Proposed weld size \( s_{sr} = 11 \text{ mm} \)
Selected fillet weld size \( s_{wt} = 12 \text{ mm} \)
Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)
### CONNECTION SUMMARY - all dimensions in diagram are in mm

<p>| | | | |</p>
<table>
<thead>
<tr>
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<tbody>
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</tbody>
</table>

Number of bolts: 10
Bolt diameter: 24 mm
Bolt grade: Preloaded HSFG bolts
General grade (8.8) with grade 10 nuts.
Welds:
- Tension flange fillet weld: 12 mm
- Web fillet weld: 8 mm
- Compression flange fillet weld: 10 mm

Supporting column section: 610 x 229 x 101 UKB
Beam / Rafter section: 406 x 178 x 74 UKB
Haunch section: 533 x 210 x 82 UKB

Column stiffener details

- Compression stiffeners: 2/90 mm x 20 mm Grade S 275 connected by 4/8 mm fillet welds.
- Tension stiffeners: Full depth 15 mm thick. 2/90 mm wide placed midway between the following bolt rows: 1 and 2, 2 and 3.
Location: Ex4 - Connection with no stiffeners

Bolted flush end plate eaves connection - beam to column

The connection under consideration is assumed to be attached to the supporting column flange. The calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure in 'Joints in Steel Construction Moment Connections' by SCI.

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( M_{Ed}' = 165 \text{ kNm} \)
Factored shear force \( V_{Ed}' = 94 \text{ kN} \)
Axial force (+ve compression) \( N_{Ed}' = 56 \text{ kN} \)

Supporting column details
457 x 191 x 98 UKB

Supported rafter section details - parent section
305 x 165 x 46 UKB

Section properties - haunch cutting
356 x 171 x 51 UKB
Combined properties and restrictions

- **Pitch of rafter**: $\Theta = 15^\circ$
- **Vertical overall depth**: $h_o = 550$ mm
- **Haunch angle to end plate**: $\phi' = 63^\circ$

Bolt details

- **Diameter of bolts**: $d_b = 20$ mm
- Tensile resistance (EC3 Part 1-8): $F_{Rd} = k_2 f_{ub} A_t / (\gamma_M 2 \times 10^3) = 141.12$ kN

End plate details

- **Thickness of plate**: $t_p = 16$ mm
- **Total number of bolts**: $b_n = 6$
- **Number of shear bolts**: $n_s = 2$
- **Distance to beam flange**: $x_1 = 20$ mm
- **Distance to first row of bolts**: $e_1 = 80$ mm
- **Pitch of tension bolts**: $p_1 = 90$ mm
- **Distance tension to shear bolts**: $e_1' = 130$ mm
- **Edge distance for shear bolts**: $p_3 = 90$ mm
- **Width of end plate**: $b_p = 180$ mm
- **Assumed web fillet weld size**: $s_{ww} = 8$ mm
- **Assumed flange fillet weld size**: $s_{wf} = 8$ mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

- **Top row of bolts**: $P_{r1} = 247.95$ kN
- **Row 2**: $P_{r2} = 196.74$ kN
- **Design axial load on column**: $N_{Edc} = 500$ kN
- **Selected fillet weld size**: $s_{wt} = 9$ mm
- **Selected fillet weld size**: $s_{fw} = 8$ mm

REVERSED MOMENT CONDITION

- **Factored moment**: $M_{Ed} = 98$ kNm
- **Factored shear force**: $V_{rEd} = 33$ kN
- **Axial force (+ve compression)**: $N_{Ed} = -23$ kN
CONNECTION SUMMARY - all dimensions in diagram are in mm

- Number of bolts: 6
- Bolt diameter: 20 mm
- Bolt grade: 8.8
- Welds:
  - Tension flange fillet weld: 9 mm
  - Web fillet weld: 8 mm
  - Compression flange fillet weld: 8 mm

Supporting column section: 457 x 191 x 98 UKB
Beam / Rafter section: 305 x 165 x 46 UKB
Haunch section: 356 x 171 x 51 UKB

Column stiffener details

No enhancement of the column is required.
Bolted flush end plate eaves connection - beam to column

The connection under consideration is assumed to be attached to the supporting column flange. The calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure in 'Joints in Steel Construction Moment Connections' by SCI.

The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment $M_{Ed'}=950$ kNm
Factored shear force $V_{Ed'}=300$ kN
Axial force (+ve compression) $N_{Ed'}=100$ kN

Supporting column details

610 x 229 x 101 UKB

Supported rafter section details - parent section

457 x 191 x 74 UKB

Section properties - haunch cutting

533 x 210 x 82 UKB

Combined properties and restrictions

Pitch of rafter $\Theta=10^\circ$
Vertical overall depth $h_o=950$ mm
Haunch angle to end plate $\phi'=78^\circ$
Bolt details

Diameter of bolts \( db = 30 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( FtRd = k_2 f_{ub} A_t / (\gamma_{M2} \times 10^3) = 323.14 \text{ kN} \)

End plate details

Thickness of plate \( tp = 20 \text{ mm} \)
Total number of bolts \( bn = 12 \)
Number of shear bolts \( ns = 2 \)
Distance to beam flange \( x_1 = 15 \text{ mm} \)
Distance to first row of bolts \( e_1 = 70 \text{ mm} \)
Pitch of tension bolts \( p_1 = 75 \text{ mm} \)
Edge distance for shear bolts \( e_1' = 70 \text{ mm} \)
Over-all length of end plate \( hp = 1035 \text{ mm} \)
Bolt cross-centres \( p_3 = 90 \text{ mm} \)
Width of end plate \( bp = 200 \text{ mm} \)
Assumed web fillet weld size \( sww = 8 \text{ mm} \)
Assumed flange fillet weld size \( swf = 10 \text{ mm} \)

SUMMARY OF BOLT POTENTIAL RESISTANCES

| Top row of bolts | \( Pr_1 = 378.47 \text{ kN} \) |
| Row 2 | \( Pr_2 = 190.22 \text{ kN} \) |
| Row 3 | \( Pr_3 = 152.68 \text{ kN} \) |
| Row 4 | \( Pr_4 = 152.68 \text{ kN} \) |
| Row 5 | \( Pr_5 = 152.68 \text{ kN} \) |

Design axial load on column \( N_{Edc} = 500 \text{ kN} \)
Weld size \( sc = 8 \text{ mm} \)
Weld size (leg length) \( ss = 8 \text{ mm} \)

As \( My_{Ed} > Mc_{Rd} \) (878.93 kNm > 760.9 kNm), the applied moment exceeds the moment resistance. Tension stiffeners are required.

Tension stiffener width \( bsgt = 150 \text{ mm} \)
Proposed rib stiffener thickness \( tsr = 20 \text{ mm} \)
Proposed weld size \( ssr = 12 \text{ mm} \)

Selected fillet weld size \( swt = 15 \text{ mm} \)
Selected fillet weld size \( sfw = 8 \text{ mm} \)
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 12
Bolt diameter 30 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 15 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm

Supporting column section  610 x 229 x 101 UKB
Beam / Rafter section  457 x 191 x 74 UKB
Haunch section  533 x 210 x 82 UKB

Column stiffener details

Diagonal shear stiffeners  2/100 mm x 15 mm Grade S 275
on line between the beam tension and compression surfaces connected by 4/8 mm fillet welds.

Compression stiffeners  2/108 mm x 20 mm Grade S 275
connected by 4/8 mm fillet welds.

Tension stiffeners
Provide 20 mm thick full depth tension stiffeners placed midway between bolt rows 1 and 2. Any remaining ribs are partial depth see code for curtailment length.
2/150 mm wide placed midway between the following bolt rows:
2 and 3
3 and 4
Location: Ex1 - Connection with compression and tension stiffener

Bolted extended end plate eaves connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( Ma = 824 \text{ kNm} \)
Factored shear force \( Va = 354 \text{ kN} \)
Axial force (+ve compression) \( Na = 121 \text{ kN} \)

Supporting column details

610 x 229 x 101 UB.

Supported rafter section properties - parent section

457 x 191 x 74 UB.

Section properties - haunch cutting

533 x 210 x 82 UB.

Combined properties & restrictions

Pitch of rafter (in degrees) \( \Theta = 10 \)
Vertical overall depth \( D_o = 925 \text{ mm} \)
Haunch angle to end plate \( \phi' = 78^\circ \)
Diameter of bolts     $bd=24$ mm
Thickness of plate   $tp=20$ mm
Total number of bolts $bn=14$
Number of shear bolts $ns=4$
Distance to first row of bolts $ex=50$ mm
Distance to beam flange $x=40$ mm
Pitch to second row of bolts $p_1=100$ mm
Pitch of tension bolts $p_2=90$ mm
Edge distance for shear bolts $pes=130$ mm
Over-all length of end plate $L_c=1060$ mm
Bolt cross-centres     $g=90$ mm
Width of end plate    $bp=200$ mm
Assumed web fillet weld size $sww=8$ mm
Assumed flange fillet weld size $swf=12$ mm

### SUMMARY OF BOLT POTENTIAL RESISTANCES

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<thead>
<tr>
<th>Bolt Row</th>
<th>Potential Resistance $P_r$ (kN)</th>
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</thead>
<tbody>
<tr>
<td>Top row</td>
<td>$297.13$</td>
</tr>
<tr>
<td>Row 2</td>
<td>$279.84$</td>
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<tr>
<td>Row 3</td>
<td>$225.82$</td>
</tr>
<tr>
<td>Row 4</td>
<td>$183.21$</td>
</tr>
<tr>
<td>Row 5</td>
<td>$183.21$</td>
</tr>
<tr>
<td>Weld size</td>
<td>$8$ mm</td>
</tr>
</tbody>
</table>

Applied moment exceeds moment capacity ($741.03$ kNm > $739.43$ kNm).
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 14
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 10 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Supporting column section 610 x 229 x 101 UB
Beam / Rafter section 457 x 191 x 74 UB
Haunch section 533 x 210 x 82 UB

Column stiffener details

Compression stiffeners 2/90 mm x 30 mm Grade S 275 connected by 4/8 mm fillet welds.
Tension stiffeners Full depth 15 mm thick.
2/100 mm wide placed midway between the following bolt rows:
1 and 2
2 and 3
3 and 4
4 and 5
Location: Ex2 - One sided connection with a compression stiffener

Bolted extended end plate eaves connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( Ma = 134 \text{ kNm} \)
Factored shear force \( Va = 56 \text{ kN} \)
Axial force (+ve compression) \( Na = 27 \text{ kN} \)

Supporting column details

406 x 178 x 54 UB.

Supported rafter section properties - parent section

254 x 146 x 37 UB.

Section properties - haunch cutting

305 x 165 x 46 UB.

Combined properties & restrictions

Pitch of rafter (in degrees) \( \Theta = 8 \)
Vertical overall depth \( Do = 450 \text{ mm} \)
Haunch angle to end plate \( \phi' = 65^\circ \)
Steel design to BS5950-1:2000 and Eurocode 3
Extended end plate haunched connection
Made by: IFB
Date: 02/12/19
Ref No: SC487 BS

Diameter of bolts        bd=24 mm
Thickness of plate       tp=20 mm
Total number of bolts    bn=8
Number of shear bolts    ns=4
Distance to first row of bolts  ex=50 mm
Distance to beam flange  x=40 mm
Pitch to second row of bolts  p1=100 mm
Pitch of shear bolts     p2=90 mm
Distance tension to shear bolts  p3=220 mm
Edge distance for shear bolts  pes=130 mm
Bolt cross-centres       g=90 mm
Width of end plate       bp=180 mm
Assumed web fillet weld size  sww=8 mm
Assumed flange fillet weld size  swf=8 mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts        Pr1=184.98 kN
Row 2                    Pr2=99.038 kN
Weld size                ss=8 mm
Selected fillet weld size swt=8
Selected fillet weld size sfw=8 mm
Factored moment          Mr=68 kNm
Factored shear force     Vr=37 kN
Axial force (+ve compression) Nr=-34 kN
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 8
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 8 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Supporting column section 406 x 178 x 54 UB
Beam / Rafter section 254 x 146 x 37 UB
Haunch section 305 x 165 x 46 UB

Column stiffener details

Compression stiffeners 2/80 mm x 10 mm Grade S 275
connected by 4/8 mm fillet welds.
Location: Ex3 - Double sided connection, HSFG bolts, heavy shear

Bolted extended end plate eaves connection - beam to column

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment \( Ma = 328 \text{ kNm} \)
Factored shear force \( Va = 487 \text{ kN} \)
Axial force (+ve compression) \( Na = -34 \text{ kN} \)

Supporting column details

254 x 254 x 89 UC.

Supported rafter section properties - parent section

356 x 127 x 33 UB.

Section properties - haunch cutting

406 x 140 x 39 UB.

Combined properties & restrictions

Pitch of rafter (in degrees) \( \Theta = 0 \)
Vertical overall depth \( Do = 710 \text{ mm} \)
Haunch angle to end plate \( \phi' = 50^\circ \)
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<tr>
<th>Parameter</th>
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<tr>
<td>Diameter of bolts</td>
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<tr>
<td>Thickness of plate</td>
<td>tp=20 mm</td>
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<tr>
<td>Total number of bolts</td>
<td>bn=14</td>
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<tr>
<td>Number of shear bolts</td>
<td>ns=6</td>
</tr>
<tr>
<td>Distance to first row of bolts</td>
<td>ex=50 mm</td>
</tr>
<tr>
<td>Distance to beam flange</td>
<td>x=40 mm</td>
</tr>
<tr>
<td>Pitch to second row of bolts</td>
<td>p1=100 mm</td>
</tr>
<tr>
<td>Pitch of tension bolts</td>
<td>p2=90 mm</td>
</tr>
<tr>
<td>Distance tension to shear bolts</td>
<td>p3=200 mm</td>
</tr>
<tr>
<td>Edge distance for shear bolts</td>
<td>pes=130 mm</td>
</tr>
<tr>
<td>Bolt cross-centres</td>
<td>g=90 mm</td>
</tr>
<tr>
<td>Width of end plate</td>
<td>bp=200 mm</td>
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<tr>
<td>Assumed web fillet weld size</td>
<td>sww=8 mm</td>
</tr>
<tr>
<td>Assumed flange fillet weld size</td>
<td>swf=12 mm</td>
</tr>
</tbody>
</table>

**SUMMARY OF BOLT POTENTIAL RESISTANCES**

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<thead>
<tr>
<th>Row</th>
<th>Potential Resistance (Pr)</th>
</tr>
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<tr>
<td>Top</td>
<td>Pr1=308.9 kN</td>
</tr>
<tr>
<td>Row 2</td>
<td>Pr2=342.15 kN</td>
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<tr>
<td>Row 3</td>
<td>Pr3=256.91 kN</td>
</tr>
<tr>
<td>Row 4</td>
<td>Pr4=148.5 kN</td>
</tr>
<tr>
<td>Weld size</td>
<td>ss=8 mm</td>
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<tr>
<td>Slip factor for preloaded bolts</td>
<td>mu=0.5</td>
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<td>Coefficient for type of hole</td>
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<tr>
<td>Selected fillet weld size</td>
<td>swt=8</td>
</tr>
<tr>
<td>Selected fillet weld size</td>
<td>sfw=10 mm</td>
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</tbody>
</table>

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CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts  14
Bolt diameter    24 mm
Bolt grade       HSFG
General grade (8.8) to BS4395 Part 1.    Nuts grade 10.
Welds          Tension flange fillet weld    8 mm
                Web fillet weld         10 mm
                Compression flange fillet weld    10 mm
Supporting column section 254 x 254 x 89 UC
Beam / Rafter section 356 x 127 x 33 UB
Haunch section 406 x 140 x 39 UB

Column stiffener details

Compression stiffeners  2/90 mm x 20 mm Grade S 275
connected by 4/8 mm fillet welds.
Location: Ex1 - Connection with compression and tension stiffeners

Bolted extended end plate eaves connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Column flange connection is assumed. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment $M_{Ed}' = 824$ kNm
Factored shear force $V_{Ed}' = 354$ kN
Axial force (+ve compression) $N_{Ed}' = 121$ kN

Supporting column details

610 x 229 x 101 UKB

Supported rafter section properties - parent section

457 x 191 x 74 UKB

Section properties - haunch cutting

533 x 210 x 82 UKB

Combined properties & restrictions

Pitch of rafter (in degrees) $\Theta = 10$
Vertical overall depth $h_o = 950$ mm
Haunch angle to end plate $\phi' = 78^\circ$
Bolt details

Diameter of bolts \( db=24 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( FtRd=k2*fub*At/(\gamma M2*10^3)=203.33 \text{ kN} \)

End plate details

Thickness of plate \( tp=20 \text{ mm} \)
Total number of bolts \( bn=14 \)
Number of shear bolts \( ns=4 \)
Distance to first row of bolts \( ex=50 \text{ mm} \)
Distance to beam flange \( x=40 \text{ mm} \)
Pitch to second row of bolts \( p2=100 \text{ mm} \)
Pitch of tension bolts \( p1=90 \text{ mm} \)
Edge distance for shear bolts \( e1'=130 \text{ mm} \)
Over-all length of end plate \( hp=1085 \text{ mm} \)
Bolt cross-centres \( p3=90 \text{ mm} \)
Width of end plate \( bp=200 \text{ mm} \)
Assumed web fillet weld size \( sww=8 \text{ mm} \)
Assumed flange fillet weld size \( swf=12 \text{ mm} \)

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts \( Pr1=303.41 \text{ kN} \)
Row 2 \( Pr2=286.12 \text{ kN} \)
Row 3 \( Pr3=213.27 \text{ kN} \)
Row 4 \( Pr4=183.21 \text{ kN} \)
Row 5 \( Pr5=183.21 \text{ kN} \)
Design axial load on column \( NEdc=500 \text{ kN} \)
Weld size (leg length) \( ss=8 \text{ mm} \)
As \( MyEd > McRd \) (738 kNm > 736.97 kNm), the applied moment exceeds the moment resistance. Tension stiffeners are required.

Stiffener width \( bsgt=100 \text{ mm} \)
Proposed rib stiffener thickness \( tsr=20 \text{ mm} \)
Proposed weld size \( ssr=15 \text{ mm} \)
Selected fillet weld size \( swt=12 \)
Selected fillet weld size \( sfw=8 \text{ mm} \)

REVERSED MOMENT CONDITION

Factored moment \( MrEd=385 \text{ kNm} \)
Factored shear force \( VrEd=178 \text{ kN} \)
Axial force (+ve compression) \( NrEd=-56 \text{ kN} \)
## CONNECTION SUMMARY - all dimensions in diagram are in mm

![Diagram with dimensions and notes]

**Number of bolts**: 14
**Bolt diameter**: 24 mm
**Bolt grade**: 8.8

**Welds**
- Tension flange fillet weld: 12 mm
- Web fillet weld: 8 mm
- Compression flange fillet weld: 8 mm

**Supporting column section**: 610 x 229 x 101 UKB
**Beam / Rafter section**: 457 x 191 x 74 UKB
**Haunch section**: 533 x 210 x 82 UKB

### Column stiffener details

**Compression stiffeners**: 2/90 mm x 30 mm Grade S 275 connected by 4/8 mm fillet welds.
**Tension stiffeners**: Full depth 20 mm thick.
2/100 mm wide placed midway between the following bolt rows:
- 1 and 2
- 2 and 3
- 3 and 4
- 4 and 5
Location: Ex2- One sided connection with compression stiffener

Bolted extended end plate eaves connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Column flange connection is assumed. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( MyEd' = 134 \text{ kNm} \)
Factored shear force \( VEd' = 56 \text{ kN} \)
Axial force (+ve compression) \( NEd' = 27 \text{ kN} \)

Supporting column details

406 x 178 x 54 UKB

Supported rafter section properties - parent section

254 x 146 x 37 UKB

Section properties - haunch cutting

305 x 165 x 46 UKB

Combined properties & restrictions

Pitch of rafter (in degrees) \( \Theta = 8 \)
Vertical overall depth  \( h_o = 450 \text{ mm} \)
Haunch angle to end plate \( \phi' = 65^\circ \)
Bolt details

Diameter of bolts $db=24\ mm$
Tensile resistance (EC3 Part 1-8) $FtRd=k2*fub*At/(\gamma_m^2*10^3)=203.33\ kN$

End plate details

Thickness of plate $tp=20\ mm$
Total number of bolts $bn=8$
Number of shear bolts $ns=4$
Distance to first row of bolts $ex=50\ mm$
Distance to beam flange $x=40\ mm$
Pitch to second row of bolts $p2=100\ mm$
Pitch of shear bolts $p1=90\ mm$
Distance tension to shear bolts $p1'=220\ mm$
Edge distance for shear bolts $e1'=130\ mm$
Bolt cross-centres $p3=90\ mm$
Width of end plate $bp=180\ mm$
Assumed web fillet weld size $sww=8\ mm$
Assumed flange fillet weld size $swf=8\ mm$

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts $Pr1=184.98\ kN$
Row 2 $Pr2=99.038\ kN$
Design axial load on column $NEdc=500\ kN$
Weld size (leg length) $ss=8\ mm$
Selected fillet weld size $swt=8$
Selected fillet weld size $sfw=8\ mm$

REVERSED MOMENT CONDITION

Factored moment $MrEd=68\ kNm$
Factored shear force $VrEd=37\ kN$
Axial force (+ve compression) $NrEd=-34\ kN$
CONNECTION SUMMARY - all dimensions in diagram are in mm

- Number of bolts: 8
- Bolt diameter: 24 mm
- Bolt grade: 8.8
- Welds:
  - Tension flange fillet weld: 8 mm
  - Web fillet weld: 8 mm
  - Compression flange fillet weld: 8 mm
- Supporting column section: 406 x 178 x 54 UKB
- Beam / Rafter section: 254 x 146 x 37 UKB
- Haunch section: 305 x 165 x 46 UKB

Column stiffener details:
- Compression stiffeners: 2/80 mm x 10 mm Grade S 275
  connected by 4/8 mm fillet welds.
Location: Ex3 - Double sided connection with compression stiffener

Bolted extended end plate eaves connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Column flange connection is assumed. Beams are connected to both column flanges. The web panel shear can be discounted in the determination of the compressive force as the connection is assumed to have balanced moments.

Factored moment  
MyEd' = 328 kNm

Factored shear force  
VEd' = 360 kN

Axial force (+ve compression)  
NEd' = -34 kN

Supporting column details

254 x 254 x 89 UKC

Supported rafter section properties - parent section

356 x 127 x 33 UKB

Section properties - haunch cutting

406 x 140 x 39 UKB

Combined properties & restrictions

Pitch of rafter (in degrees)  
Theta = 0

Vertical overall depth  
ho = 710 mm

Haunch angle to end plate  
phi' = 50°
Bolt details

<table>
<thead>
<tr>
<th>Diameter of bolts</th>
<th>db=24 mm</th>
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<tbody>
<tr>
<td>Tensile resistance (EC3 Part 1-8)</td>
<td>$F_{Rd} = k_2 \cdot f_{ub} \cdot A_t / (\gamma M_2 \cdot 10^3) = 203.33 \text{ kN}$</td>
</tr>
</tbody>
</table>

End plate details

<table>
<thead>
<tr>
<th>Thickness of plate</th>
<th>tp=20 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total number of bolts</td>
<td>bn=14</td>
</tr>
<tr>
<td>Number of shear bolts</td>
<td>ns=6</td>
</tr>
<tr>
<td>Distance to first row of bolts</td>
<td>ex=50 mm</td>
</tr>
<tr>
<td>Distance to beam flange</td>
<td>x=40 mm</td>
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<tr>
<td>Pitch to second row of bolts</td>
<td>p2=100 mm</td>
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<tr>
<td>Pitch of tension bolts</td>
<td>p1=90 mm</td>
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<td>Distance tension to shear bolts</td>
<td>p1'=200 mm</td>
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<td>Edge distance for shear bolts</td>
<td>e1'=130 mm</td>
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<td>Bolt cross-centres</td>
<td>p3=90 mm</td>
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<tr>
<td>Width of end plate</td>
<td>bp=200 mm</td>
</tr>
<tr>
<td>Assumed web fillet weld size</td>
<td>sww=8 mm</td>
</tr>
<tr>
<td>Assumed flange fillet weld size</td>
<td>swf=12 mm</td>
</tr>
</tbody>
</table>

SUMMARY OF BOLT POTENTIAL RESISTANCES

| Top row of bolts      | $P_{r1} = 303.41 \text{ kN}$ |
| Row 2                 | $P_{r2} = 336.66 \text{ kN}$ |
| Row 3                 | $P_{r3} = 256.91 \text{ kN}$ |
| Row 4                 | $P_{r4} = 148.5 \text{ kN}$  |
| Design axial load on column | $N_{Edc} = 500 \text{ kN}$ |
| Weld size (leg length) | ss=8 mm  |
| Selected fillet weld size | swt=8 mm |
| Selected fillet weld size | swf=10 mm |
CONNECTION SUMMARY - all dimensions in diagram are in mm

<table>
<thead>
<tr>
<th>ex</th>
<th>x1</th>
<th>50</th>
<th>20</th>
<th>200</th>
</tr>
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<tbody>
<tr>
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<td>100</td>
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<td></td>
<td></td>
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<tr>
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<td></td>
</tr>
<tr>
<td>p1</td>
<td>90</td>
<td>90</td>
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<tr>
<td>p1'</td>
<td>200</td>
<td>840</td>
<td></td>
<td></td>
</tr>
<tr>
<td>x2</td>
<td>55</td>
<td>90</td>
<td>55</td>
<td>40</td>
</tr>
</tbody>
</table>

Number of bolts 14  
Bolt diameter 24 mm  
Bolt grade Preloaded HSFG bolts  
General grade (8.8) with grade 10 nuts.  
Welds Tension flange fillet weld 8 mm  
Web fillet weld 10 mm  
Compression flange fillet weld 10 mm  
Supporting column section 254 x 254 x 89 UKC  
Beam / Rafter section 356 x 127 x 33 UKB  
Haunch section 406 x 140 x 39 UKB  

Column stiffener details  
Compression stiffeners 2/90 mm x 20 mm Grade S 275  
connected by 4/8 mm fillet welds.
Location: Ex4 - Connection with compression & tension stiffeners

Bolted extended end plate eaves connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Column flange connection is assumed. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment MyEd' = 835 kNm
Factored shear force VEd' = 354 kN
Axial force (+ve compression) NEd' = 121 kN

Supporting column details

610 x 229 x 101 UKB

Supported rafter section properties - parent section

457 x 191 x 74 UKB

Section properties - haunch cutting

533 x 210 x 82 UKB

Combined properties & restrictions

Pitch of rafter (in degrees) Theta = 10
Vertical overall depth ho = 950 mm
Haunch angle to end plate phi' = 78°
Bolt details

Diameter of bolts                      db=24 mm
Tensile resistance (EC3 Part 1-8)       FtRd=k2*fub*At/(gamM2*10^3)=203.33 kN

End plate details

Thickness of plate                  tp=20 mm
Total number of bolts                bn=14
Number of shear bolts                ns=4
Distance to first row of bolts       ex=50 mm
Distance to beam flange              x=40 mm
Pitch to second row of bolts         p2=100 mm
Pitch of tension bolts               p1=90 mm
Edge distance for shear bolts         e1'=130 mm
Over-all length of end plate         hp=1085 mm
Bolt cross-centres                   p3=90 mm
Width of end plate                   bp=200 mm
Assumed web fillet weld size          sww=8 mm
Assumed flange fillet weld size       swf=12 mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts                       Pr1=303.41 kN
Row 2                                 Pr2=286.12 kN
Row 3                                 Pr3=213.27 kN
Row 4                                 Pr4=183.21 kN
Row 5                                 Pr5=183.21 kN
Design axial load on column           NEdc=500 kN
Weld size (leg length)                ss=8 mm
As MyEd > McRd (749 kNm > 736.97 kNm), the applied moment exceeds the moment resistance. Tension stiffeners are required.
Stiffener width                       bsgt=100 mm
Proposed rib stiffener thickness      tsr=20 mm
Proposed weld size                    ssr=15 mm
Selected fillet weld size             swt=15 mm
Selected fillet weld size             swf=8 mm

REVERSED MOMENT CONDITION

Factored moment                      MrEd=385 kNm
Factored shear force                  VrEd=178 kN
Axial force (+ve compression)         NrEd=-56 kN
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 14
Bolt diameter 24 mm
Bolt grade 8.8
Welds Tension flange fillet weld 15 mm
                Web fillet weld 8 mm
                Compression flange fillet weld 8 mm
Supporting column section 610 x 229 x 101 UKB
Beam / Rafter section 457 x 191 x 74 UKB
Haunch section 533 x 210 x 82 UKB

Column stiffener details

Compression stiffeners 2/90 mm x 30 mm Grade S 275
connected by 4/8 mm fillet welds.
Tension stiffeners Full depth 20 mm thick.
2/100 mm wide placed midway
between the following bolt rows:
1 and 2
2 and 3
3 and 4
4 and 5
Location: Ex5 - Connection with shear and compression stiffeners

Bolted extended end plate eaves connection - beam to column

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. Column flange connection is assumed. The connection is one sided. The web panel shear cannot be discounted in the determination of the compressive force.

Factored moment \( M_{Ed}' = 950 \) kNm
Factored shear force \( V_{Ed}' = 300 \) kN
Axial force (+ve compression) \( N_{Ed}' = 75 \) kN

Supporting column details

610 x 229 x 101 UKB

Supported rafter section properties - parent section

457 x 191 x 74 UKB

Section properties - haunch cutting

533 x 210 x 82 UKB

Combined properties & restrictions

Pitch of rafter (in degrees) \( \Theta = 10^\circ \)
Vertical overall depth \( h_o = 960 \) mm
Haunch angle to end plate \( \phi' = 78^\circ \)
### Bolt details

- **Diameter of bolts**: $d_b = 30 \text{ mm}
- **Tensile resistance (EC3 Part 1-8)**: $F_{Rd} = k_2 \cdot f_{ub} \cdot A_t / (\gamma_M^2 \cdot 10^3) = 323.14 \text{ kN}$

### End plate details

- **Thickness of plate**: $t_p = 15 \text{ mm}$
- **Total number of bolts**: $b_n = 12$
- **Number of shear bolts**: $n_s = 2$
- **Distance to first row of bolts**: $e_x = 60 \text{ mm}$
- **Distance to beam flange**: $x = 30 \text{ mm}$
- **Pitch to second row of bolts**: $p_2 = 90 \text{ mm}$
- **Pitch of tension bolts**: $p_1 = 90 \text{ mm}$
- **Edge distance for shear bolts**: $e_1' = 110 \text{ mm}$
- **Over-all length of end plate**: $h_p = 1090 \text{ mm}$
- **Bolt cross-centres**: $p_3 = 90 \text{ mm}$
- **Width of end plate**: $b_p = 200 \text{ mm}$
- **Assumed web fillet weld size**: $s_{ww} = 8 \text{ mm}$
- **Assumed flange fillet weld size**: $s_{wf} = 12 \text{ mm}$

### SUMMARY OF BOLT POTENTIAL RESISTANCES

- **Top row of bolts**: $P_{r1} = 303.31 \text{ kN}$
- **Row 2**: $P_{r2} = 295.92 \text{ kN}$
- **Row 3**: $P_{r3} = 183.21 \text{ kN}$
- **Row 4**: $P_{r4} = 183.21 \text{ kN}$
- **Row 5**: $P_{r5} = 183.21 \text{ kN}$
- **Design axial load on column**: $N_{Edc} = 500 \text{ kN}$
- **Weld size**: $s_c = 8 \text{ mm}$
- **Weld size (leg length)**: $s_s = 8 \text{ mm}$
- **Selected fillet weld size**: $s_{wt} = 15 \text{ mm}$
- **Selected fillet weld size**: $s_{fw} = 8 \text{ mm}$
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 12
Bolt diameter 30 mm
Bolt grade 8.8
Welds
Tension flange fillet weld 15 mm
Web fillet weld 8 mm
Compression flange fillet weld 8 mm
Supporting column section 610 x 229 x 101 UKB
Beam / Rafter section 457 x 191 x 74 UKB
Haunch section 533 x 210 x 82 UKB

Column stiffener details

Diagonal shear stiffeners 2/100 mm x 15 mm Grade S 275
on line between the beam tension and compression surfaces
connected by 4/8 mm fillet welds.

Compression stiffeners 2/100 mm x 20 mm Grade S 275
connected by 4/8 mm fillet welds.
Location: Example 8 - Baseplate 1

Axial compressive load \( N = 1500 \text{ kN} \)
Shear on the base in Y direction \( F_y = 40 \text{ kN} \)
254 x 254 x 73 UKC
Length of baseplate \( h_p = 500 \text{ mm} \)
Breadth of baseplate \( b_p = 500 \text{ mm} \)
Edge distance to bolt centre line \( k = 50 \text{ mm} \)

Assumed fillet weld size \( s_w = 8 \text{ mm} \)
Bolt grade (4.6 or 8.8) \( b_{\text{grade}} = 4.6 \)
Grade of concrete \( g_{\text{dec}} = 30 \text{ N/mm}^2 \)

Selected baseplate thickness \( t_p = 20 \text{ mm} \)
Number of bolts to be used \( n = 4 \)
Bolt diameter \( d = 20 \text{ mm} \)

Column to baseplate welds

Compression flange welds

Surfaces of baseplate and end of column are machined square to comply with Clause 4.13.3 to give bearing contact, nominal welds only are required. Only a nominal weld size need be specified.
Selected fillet weld size \( s_{cw} = 8 \text{ mm} \)

Web welds

Web welds are assumed to carry the shear force.
Weld size required \( s_{w} = F_{yw}/(0.7*p_{w}) = 0.64838 \text{ mm} \)
Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)

SUMMARY OF BASEPLATE REQUIREMENTS

Size 500 mm x 20 mm x 500 mm
Grade S 275 steel
Edge distance 50 mm
Number of H.D. bolts 4
Diameter of bolts M 20
Grade 4.6
Concrete/Grout Grade 30
Web weld 8 mm
Contact areas on the baseplate and column are machined to give a tight bearing contact.
Location: Example 9 - Baseplate 2

Axial compressive load \( N = 3400 \text{ kN} \)
Shear on the base in Y direction \( F_y = 40 \text{ kN} \)
254 x 254 x 107 UKC
Length of baseplate \( h_p = 600 \text{ mm} \)
Breadth of baseplate \( b_p = 600 \text{ mm} \)
Edge distance to bolt centre line \( k = 50 \text{ mm} \)

Assumed fillet weld size \( s_w = 8 \text{ mm} \)
Bolt grade (4.6 or 8.8) \( b_{\text{grade}} = 4.6 \)
Grade of concrete \( g_{\text{dec}} = 20 \text{ N/mm}^2 \)

Selected baseplate thickness \( t_p = 50 \text{ mm} \)
Number of bolts to be used \( n = 4 \)
Bolt diameter \( b_d = 20 \text{ mm} \)

Column to baseplate welds

Compression flange welds

Surfaces of baseplate and end of column are machined square to comply with Clause 4.13.3 to give bearing contact, nominal welds only are required. Only a nominal weld size need be specified.

Selected fillet weld size \( s_{cw} = 8 \text{ mm} \)

Web welds

Web welds are assumed to carry the shear force.

Weld size required \( s_{wc} = F_{yw}/(0.7*p_w) = 0.64838 \text{ mm} \)
Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)

SUMMARY OF

<table>
<thead>
<tr>
<th>Requirements</th>
<th>Size 600 mm x 50 mm x 600 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>BASEPLATE</td>
<td>Grade S 275 steel</td>
</tr>
<tr>
<td>REQUIREMENTS</td>
<td>Edge distance 50 mm</td>
</tr>
<tr>
<td></td>
<td>Number of H.D. bolts 4</td>
</tr>
<tr>
<td></td>
<td>Diameter of bolts M 20</td>
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<td></td>
<td>Grade 4.6</td>
</tr>
<tr>
<td></td>
<td>Concrete/Grout Grade 20</td>
</tr>
<tr>
<td></td>
<td>Web weld 8 mm</td>
</tr>
</tbody>
</table>
Contact areas on the baseplate and column are machined to give a tight bearing contact.
Location: SCI Example 19

Calculations are also in accordance with EC3 Part 1-1, 1-5 and 1-8.

Design axial compressive load \( N_{Ed} = 2635 \text{ kN} \)
Accompanying shear force \( V_{cEd} = 100 \text{ kN} \)
305 x 305 x 137 UKC
Length of baseplate \( h_p = 600 \text{ mm} \)
Breadth of baseplate \( b_p = 600 \text{ mm} \)
Edge distance to bolt centre line \( k = 60 \text{ mm} \)

Assumed fillet weld size \( s_w = 8 \text{ mm} \)
Concrete strength class \( f_{ck} = 25 \text{ N/mm}^2 \)
Number of bolts to be used \( n = 4 \)
Bolt diameter \( d_b = 20 \text{ mm} \)
Length of web weld provided \( l_{wv} = 100 \text{ mm} \)

**SUMMARY OF BASEPLATE**

Size 600 mm x 35 mm x 600 mm
Grade S 275 steel

**REQUIREMENTS**

Edge distance 60 mm
Number of H.D. bolts 4
Diameter of bolts M 20
Grade of bolts 8.8
Concrete/Grout Grade C 25 /30

**COMPRESSION**

Design axial load 2635 kN
Required thickness 31.6 mm
Minimum plate thickness 35 mm
Weld size 8 mm

NOTE: Contact areas on the baseplate and column are assumed to be machined to give a tight bearing contact.
Location: Example including uplift

Calculations are also in accordance with EC3 Part 1-1, 1-5 and 1-8.

Design axial compressive load \( N_{jEd}=800 \text{kN} \)
Accompanying shear force \( V_{cEd}=60 \text{kN} \)
Design tensile force \( N_{tEd}=120 \text{kN} \)
Accompanying shear on the base \( V_{tEd}=40 \text{kN} \)

406 x 140 x 53 UKB
Length of baseplate \( hp=620 \text{ mm} \)
Breadth of baseplate \( bp=300 \text{ mm} \)
Edge distance to bolt centre line \( k=60 \text{ mm} \)
Assumed fillet weld size \( sw=8 \text{ mm} \)
Concrete strength class \( fck=25 \text{ N/mm}^2 \)
Total number of bolts to be used \( n=4 \)
Assumed diameter of bolt \( bd=24 \text{ mm} \)
Overall embedded depth \( l=400 \text{ mm} \)
Assumed cover to reinforcement \( cv=50 \text{ mm} \)
Length of web weld provided \( lwv=150 \text{ mm} \)

**SUMMARY OF BASEPLATE**

<table>
<thead>
<tr>
<th>Size 620 mm x 20 mm x 300 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade S 275 steel</td>
</tr>
</tbody>
</table>

**REQUIREMENTS**

- Edge distance: 60 mm
- Number of H.D. bolts: 4
- Diameter of bolts: M 24
- Grade of bolts: 8.8
- Concrete/Grout Grade: C 25 /30

**COMPRESSION**

- Design axial load: 800 kN
- Required thickness: 13.2 mm
- Minimum plate thickness: 15 mm

**TENSION**

- Tensile force: 120 kN
- Bolt tensile resistance: 49.8 kN
- Required thickness: 15 mm
- Minimum plate thickness: 20 mm
- Weld size: 8 mm

**NOTE:** Contact areas on the baseplate and column are assumed to be machined to give a tight bearing contact.
Location: Overlapping stubs

Calculations are also in accordance with EC3 Part 1-1, 1-5 and 1-8.

Design axial compressive load \( N_{jEd} = 6000 \text{ kN} \)
Accompanying shear force \( V_{cEd} = 200 \text{ kN} \)
305 x 305 x 158 UKC
Length of baseplate \( h_p = 620 \text{ mm} \)
Breadth of baseplate \( b_p = 620 \text{ mm} \)
Edge distance to bolt centre line \( k = 60 \text{ mm} \)

Assumed fillet weld size \( s_w = 10 \text{ mm} \)
Concrete strength class \( f_{ck} = 30 \text{ N/mm}^2 \)
Number of bolts to be used \( n = 4 \)
Bolt diameter \( b_d = 24 \text{ mm} \)
Length of web weld provided \( l_{wv} = 200 \text{ mm} \)

**SUMMARY OF BASEPLATE**

- Size 620 mm x 65 mm x 620 mm
- Grade S 275 steel

**REQUIREMENTS**

- Edge distance 60 mm
- Number of H.D. bolts 4
- Diameter of bolts M 24
- Grade of bolts 8.8
- Concrete/Grout Grade C 30 /37

**COMPRESSION**

- Design axial load 6000 kN
- Required thickness 61.5 mm
- Minimum plate thickness 65 mm
- Weld size 10 mm

**NOTE:** Contact areas on the baseplate and column are assumed to be machined to give a tight bearing contact.
Location: Minimal load

Calculations are also in accordance with EC3 Part 1-1, 1-5 and 1-8.

Design axial compressive load \( \text{NjEd} = 300 \, \text{kN} \)
Accompanying shear force \( \text{VcEd} = 60 \, \text{kN} \)
203 x 203 x 100 UKC
Length of baseplate \( \text{hp} = 300 \, \text{mm} \)
Breadth of baseplate \( \text{bp} = 300 \, \text{mm} \)
Edge distance to bolt centre line \( k = 60 \, \text{mm} \)

Assumed fillet weld size \( \text{sw} = 8 \, \text{mm} \)
Concrete strength class \( f_{ck} = 20 \, \text{N/mm}^2 \)
Revised base plate thickness \( \text{tp}' = 24 \, \text{mm} \)
Number of bolts to be used \( n = 2 \)
Bolt diameter \( \text{bd} = 20 \, \text{mm} \)
Length of web weld provided \( \text{lwv} = 100 \, \text{mm} \)

SUMMARY OF BASEPLATE

<table>
<thead>
<tr>
<th>Size</th>
<th>300 mm x 24 mm x 300 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade</td>
<td>S 275 steel</td>
</tr>
<tr>
<td>REQUIREMENTS</td>
<td></td>
</tr>
<tr>
<td>Edge distance</td>
<td>60 mm</td>
</tr>
<tr>
<td>Number of H.D. bolts</td>
<td>2</td>
</tr>
<tr>
<td>Diameter of bolts</td>
<td>M 20</td>
</tr>
<tr>
<td>Grade of bolts</td>
<td>8.8</td>
</tr>
<tr>
<td>Concrete/Grout Grade</td>
<td>C 20 /25</td>
</tr>
<tr>
<td>COMPRESSION</td>
<td></td>
</tr>
<tr>
<td>Design axial load</td>
<td>300 kN</td>
</tr>
<tr>
<td>Required thickness</td>
<td>24 mm</td>
</tr>
<tr>
<td>Minimum plate thickness</td>
<td>24 mm</td>
</tr>
<tr>
<td>Weld size</td>
<td>8 mm</td>
</tr>
</tbody>
</table>

NOTE: Contact areas on the baseplate and column are assumed to be machined to give a tight bearing contact.
Location: 300 x 300 x 10 SHS - lightly loaded

Axial compressive load \( N = 1500 \text{ kN} \)
Shear on the base in Y direction \( F_y = 300 \text{ kN} \)

300 x 300 x 10 SHS - Hot finished.

Properties (cm): \( A = 115 \), \( r_x = 11.8 \), \( Z_x = 1070 \), \( S_x = 1250 \), \( I_x = 16000 \), \( J = 24800 \), \( C = 1580 \)

Length of baseplate \( hp = 500 \text{ mm} \)
Breadth of baseplate \( bp = 500 \text{ mm} \)
Edge distance to bolt centre line \( k = 60 \text{ mm} \)

Assumed fillet weld size \( sw = 8 \text{ mm} \)
Grade of concrete \( \text{gradec} = 30 \text{ N/mm}^2 \)
Selected baseplate thickness \( tp = 15 \text{ mm} \)
Number of bolts to be used \( n = 4 \)
Bolt diameter \( bd = 30 \text{ mm} \)

### Column to baseplate welds

#### Compression flange welds

Surfaces of baseplate and end of column are machined square to comply with Clause 4.13.3 to give bearing contact, nominal welds only are required. Only a nominal weld size need be specified.

Selected fillet weld size \( scw = 8 \text{ mm} \)

#### Web welds

Web welds are assumed to carry the shear force coincide.

Weld size required \( sww = F_y/(0.7*pw) = 3.6075 \text{ mm} \)
Selected fillet weld size \( sfw = 8 \text{ mm} \)

### SUMMARY OF BASEPLATE REQUIREMENTS

<table>
<thead>
<tr>
<th>Size</th>
<th>Grade S 275 steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Edge distance</td>
<td>60 mm</td>
</tr>
<tr>
<td>Number of H.D. bolts</td>
<td>4</td>
</tr>
<tr>
<td>Diameter of bolts</td>
<td>M 30</td>
</tr>
<tr>
<td>Grade</td>
<td>4.6</td>
</tr>
<tr>
<td>Concrete/Grout Grade</td>
<td>30</td>
</tr>
<tr>
<td>Web weld</td>
<td>8 mm</td>
</tr>
</tbody>
</table>

NOTE: Contact areas on the baseplate and column are machined to give a tight bearing contact.
Location: circular section with uplift

Axial compressive load \( N = 2500 \text{ kN} \)
Shear on the base in Y direction \( F_y = 300 \text{ kN} \)
Axial uplift force \( T = 850 \text{ kN} \)
Accompanying shear on the base \( F_u = 149 \text{ kN} \)

273 dia x 8 thick CHS - Hot finished.
Properties (cm): \( A = 66.602 \), \( r = 9.3734 \), \( Z = 428.7 \), \( S = 561.97 \), \( I = 5851.7 \)
Length of baseplate \( h_p = 600 \text{ mm} \)
Breadth of baseplate \( b_p = 600 \text{ mm} \)
Edge distance to bolt centre line \( k = 60 \text{ mm} \)
Assumed fillet weld size \( s_w = 8 \text{ mm} \)
Grade of concrete \( \text{grade} = 40 \text{ N/mm}^2 \)
Selected baseplate thickness \( t_p = 35 \text{ mm} \)
Total number of bolts to be used \( n = 6 \)
Assumed diameter of bolt \( b_d = 30 \text{ mm} \)
Overall embedded depth \( L_o = 450 \text{ mm} \)
Assumed cover to reinforcement \( c_v = 50 \text{ mm} \)
tension reinforcement \( A_s = 0.15 \% \)
Location: Example 3 200 x 100 x 10 RHS - 900kN load

Calculations are also in accordance with EC3 Part 1-1, 1-5 and 1-8.

Axial compressive load NEd=900 kN
Accompanying shear force VEd=100 kN
200 x 100 x 10 RHS - Hot finished.
Properties (cm): A=54.9 iz=3.98 Wely=266 Wply=341 Iy=2660
It=2160 Wt=295 Welz=174 Wplz=206 Iz=869 iy=6.96
Length of baseplate hp=400 mm
Breadth of baseplate bp=300 mm
Edge distance to bolt centre line k=50 mm
Assumed fillet weld size s=6 mm
Concrete cylinder strength fck=30 N/mm²
Number of bolts to be used n=4
Bolt diameter bd=20 mm
Bolt grade bgrade=8.8 mm

SUMMARY OF BASEPLATE
REQUIREMENTS
Size 400 mm x 20 mm x 300 mm
Grade S 275 steel
Edge distance 50 mm
Number of H.D. bolts 4
Diameter of bolts M 20
Grade of bolts 8.8
Concrete/Grout Grade C 30 /37
Design axial load 900 kN
Minimum plate thickness 20 mm
Weld size 6 mm

NOTE: Contact areas on the baseplate and column are machined to
give a tight bearing contact.
Location: Example 4 - Column base CHS

Calculations are also in accordance with EC3 Part 1-1, 1-5 and 1-8.

Axial compressive load \( N_{Ed} = 1400 \text{ kN} \)
Accompanying shear force \( V_{Ed} = 100 \text{ kN} \)
273 dia x 10 thick CHS - Hot finished.
Properties (cm): \( A = 82.624 \), \( iy = 9.3052 \), \( W_{ely} = 524.11 \), \( W_{ply} = 692.02 \), \( I_y = 7154.1 \), \( I_t = 14308 \)
Length of baseplate \( h_p = 400 \text{ mm} \)
Breadth of baseplate \( b_p = 400 \text{ mm} \)
Edge distance to bolt centre line \( k = 50 \text{ mm} \)
Assumed fillet weld size \( s = 6 \text{ mm} \)
Concrete cylinder strength \( f_{ck} = 30 \text{ N/mm}^2 \)
Number of bolts to be used \( n = 4 \)
Bolt diameter \( b_d = 20 \text{ mm} \)
Bolt grade \( b_{\text{grade}} = 8.8 \)

**SUMMARY OF BASEPLATE**
Size 400 mm x 20 mm x 400 mm
Grade S 275 steel

**REQUIREMENTS**
Edge distance 50 mm
Number of H.D. bolts 4
Diameter of bolts M 20
Grade of bolts 8.8
Concrete/Grout Grade C 30 /37

**COMPRESSION**
Design axial load 1400 kN
Minimum plate thickness 20 mm
Weld size 6 mm

NOTE: Contact areas on the baseplate and column are machined to give a tight bearing contact.
Location: Ex1 - Joints in simple construction (Ex 2)

Bolted column splice

Bearing connection

Calculations are in accordance with BS5950-1:2000 Clause 6.1.8.2.

Design assumptions:

- the column splice is about 500 mm above the floor level and hence the moment due to strut action is considered to be insignificant
- the centroidal axis of the splice is coincident with the centroidal axis of the members.

Dead                              Fcd=760 kN
Imposed                           Fci=870 kN
Bending moment                    M=110 kNm
Shear force                       Fv=60 kN
Load from floor below splice      Fst=340 kN

Upper column

254 x 254 x 73 UC.

Lower column

305 x 305 x 118 UC.

Thickness of plate adopted        tdp=25 mm
Thickness of plate                tcp=12 mm
Width of cover plate              Bcp=250 mm
Bolt diameter                     bd=20 mm
Total no. of bolts in cover plate nt=8
Pitch of bolts (vertically)       pitch=160 mm
Spacing of bolts across columns   spac=125 mm
End distance                      e=40 mm
Bolt spacing (horizontally)       crs=150 mm
Thickness of packing plate (30 mm), cannot exceed 26.667 mm, see Clause 6.3.2.2. If the packs are fillet welded to the column flange this check can be omitted.
Bolt diameter                     bd=20 mm
Thickness of packing plate (30 mm), cannot exceed 26.667 mm, see Clause 6.3.2.2. If the packs are fillet welded to the column flange this check can be omitted.
Total no. of bolts in cover plate nt=8
Pitch of bolts (vertically)       pitch=160 mm
End distance                      e=40 mm
SPLICE SUMMARY - all dimensions in diagram are in mm

Flange cover plates:

```
250

40

160

125

525

Division plate
thickness 25 mm

Flange cover plates 2/ 250 mm x 12 mm x 525 mm
Number of bolts 4/ 4 (Total No of bolts=16)
Pulls 2/ 250 mm x 30 mm x 250 mm
Bolt diameter 20 mm
Bolt grade 8.8
Web angles Provide web angles, each with four bolts.
```
Location: Ex2 - Joints in simple construction (Ex 3)

Bolted column splice

Non-bearing connection

Calculations are in accordance with BS5950-1:2000 Clause 6.1.8.2.

Design assumptions:

- the column splice is about 500 mm above the floor level and hence the moment due to strut action is considered to be insignificant
- the centroidal axis of the splice is coincident with the centroidal axis of the members.

Dead Fcd=825 kN
Imposed Fci=942 kN
Bending moment M=15 kNm
Shear force Fv=8 kN
Load from floor below splice Fst=405 kN

Upper column

254 x 254 x 73 UC.

Lower column

305 x 305 x 118 UC.

Thickness of plate tcp=12 mm
Width of cover plate Bcp=250 mm
Bolt diameter bd=24 mm
Total no. of bolts in cover plate nt=16
Pitch of bolts (vertically) pitch=80 mm
Spacing of bolts across columns spac=110 mm
End distance e=50 mm
Bolt spacing (horizontally) crs=150 mm
Allowance for gap gap=10 mm
Bolt diameter bd=24 mm
Bolt spacing (horizontally) crs=150 mm
Total no. of bolts in cover plate nt=16
Pitch of bolts (vertically) pitch=80 mm
End distance e=50 mm
Thickness of plate tcw=8 mm
Width of cover plate Bcw=150 mm
Bolt cross-centres crw=80 mm
Total no. of bolts in web cleat ntw=8
SPLICE SUMMARY - all dimensions in diagram are in mm

Flange cover plates:

250

50

80

80

80

110

690

50

150

50

Allowance for gap 10 mm

Flange cover plates 2/ 250 mm x 12 mm x 690 mm
Number of bolts 4/ 8 ( Total No of bolts=32 )
Packs 2/ 250 mm x 30 mm x 340 mm
Bolt diameter 24 mm
Bolt grade 8.8
Web cover plates:

Web cover plates: 2/ 150 mm x 8 mm x 370 mm
Number of bolts: 2/ 4 (Total No of bolts=8)
Location: Ex1 - Bearing column splice

Bolted column splice

Bearing connection

Calculations are in accordance with EC3 Parts 1-1, 1-5 and 1-8.

Design assumptions:

- the column splice is about 500 mm above the floor level and hence the moment due to strut action is considered to be insignificant
- the centroidal axis of the splice is coincident with the centroidal axis of the members.

Factored loads

Permanent compressive axial load \( N_{EdG} = 760 \) kN
Variable compressive axial load \( N_{EdQ} = 870 \) kN
Design bending moment \( M_{yEd} = 110 \) kNm
Design shear force \( V_{Ed} = 60 \) kN
Load from floor below splice \( Fst_{Ed} = 340 \) kN

Upper column

254 x 254 x 73 UKC

Lower column

305 x 305 x 118 UKC
Division plate thickness \( tdp = 25 \) mm
Thickne$$

ness of flange cover plate \( tfp = 12 \) mm
Width of flange cover plate \( bfp = 250 \) mm
Bolt diameter \( db = 20 \) mm
Total no. of bolts in cover plate \( nt = 8 \)
Pitch of bolts (vertically) \( plfp = 160 \) mm
Spacing of bolts across columns \( spac = 125 \) mm
End distance \( elfp = 40 \) mm
Bolt spacing (horizontally) \( p2fp = 150 \) mm
As the thickness of packing plate (30 mm) exceeds 26.667 mm, the packs need to be fillet welded to the column flange.
Coefficient of friction \( cmu = 0.2 \)
SPLICE SUMMARY - all dimensions in diagram are in mm

Flange cover plates:

\[
\begin{array}{c}
250 \\
40 \\
160 \\
125 \\
160 \\
40 \\
\end{array}
\]

Division plate
thickness 25 mm

<table>
<thead>
<tr>
<th>Flange cover plates</th>
<th>2/ 250 mm x 12 mm x 525 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bolts</td>
<td>4/ 4 (Total No of bolts=16)</td>
</tr>
<tr>
<td>Packs</td>
<td>2/ 250 mm x 30 mm x 250 mm</td>
</tr>
<tr>
<td>Bolt diameter</td>
<td>20 mm</td>
</tr>
<tr>
<td>Bolt grade</td>
<td>8.8</td>
</tr>
</tbody>
</table>

Web angles
Provide web angles, each with four bolts.

NOTE: Packs to be fillet welded to the column flange.
Location: Ex2 - Non-bearing column splice, web/flange cover plates

Bolted column splice

Non-bearing connection

Calculations are in accordance with EC3 Parts 1-1, 1-5 and 1-8.

Design assumptions:

- the column splice is about 500 mm above the floor level and hence the moment due to strut action is considered to be insignificant
- the centroidal axis of the splice is coincident with the centroidal axis of the members.

Factored loads

Permanent compressive axial load $N_{EdG}=825$ kN
Variable compressive axial load $N_{EdQ}=942$ kN
Design bending moment $M_{Ed}=15$ kNm
Design shear force $V_{Ed}=8$ kN
Load from floor below splice $F_{stEd}=405$ kN

Upper column

254 x 254 x 73 UKC

Lower column

305 x 305 x 118 UKC
Thickness of flange cover plate $t_{fp}=12$ mm
Width of flange cover plate $b_{fp}=250$ mm
Bolt diameter $d_{b}=24$ mm
Total no. of bolts in cover plate $n_{t}=16$
Pitch of bolts (vertically) $p_{1fp}=80$ mm
Spacing of bolts across columns $s_{pac}=110$ mm
End distance $e_{1fp}=50$ mm
Bolt spacing (horizontally) $p_{2fp}=150$ mm
Allowance for gap $g_{ap}=10$ mm

Web cover plates

Thickness of web plate $t_{wp}=10$ mm
Width of web cover plate $b_{wp}=150$ mm
Bolt cross-centres $p_{2wp}=80$ mm
Total no. of bolts in web cleat $n_{tw}=8$
SPLICE SUMMARY - all dimensions in diagram are in mm

Flange cover plates:

250

2

690 Allowance for gap 10 mm

50 150 50

2/ 250 mm x 12 mm x 690 mm

4/ 8 (Total No of bolts=32)

2/ 250 mm x 30 mm x 340 mm

24 mm

8.8
Web cover plates:

- Width: 150 mm
- Height: 370 mm
- Allowance for gap: 10 mm

Number of bolts: 2/ 4 (Total No of bolts=8)
Location: Ex3 - Bearing column splice with HSFG bolts

Bolted column splice

Bearing connection

Calculations are in accordance with EC3 Parts 1-1, 1-5 and 1-8.

Design assumptions:

- the column splice is about 500 mm above the floor level and hence the moment due to strut action is considered to be insignificant
- the centroidal axis of the splice is coincident with the centroidal axis of the members.

Factored loads

Permanent compressive axial load  \( N_{EdG} = 700 \text{ kN} \)
Variable compressive axial load  \( N_{EdQ} = 1000 \text{ kN} \)
Design bending moment  \( M_{yEd} = 150 \text{ kNm} \)
Design shear force  \( V_{Ed} = 60 \text{ kN} \)
Load from floor below splice  \( F_{stEd} = 340 \text{ kN} \)

Upper column

254 x 254 x 73 UKC

Lower column

305 x 305 x 118 UKC
Division plate thickness  \( t_{dp} = 25 \text{ mm} \)
Thickness of flange cover plate  \( t_{fp} = 12 \text{ mm} \)
Width of flange cover plate  \( b_{fp} = 250 \text{ mm} \)
Bolt diameter  \( d_b = 20 \text{ mm} \)
Total no. of bolts in cover plate  \( n_t = 12 \)
Pitch of bolts (vertically)  \( p_{1fp} = 160 \text{ mm} \)
Spacing of bolts across columns  \( s_pac = 125 \text{ mm} \)
End distance  \( e_{1fp} = 40 \text{ mm} \)
Bolt spacing (horizontally)  \( p_{2fp} = 150 \text{ mm} \)

As the thickness of packing plate (30 mm) exceeds 26.667 mm, the packs need to be fillet welded to the column flange.

Slip resistance  \( F_{sRd} = k_s \cdot n_f_s \cdot \mu \cdot F_{pc} / 1.25 = 54.88 \text{ kN} \)
Coefficient of friction  \( \mu = 0.2 \)
SPLICE SUMMARY - all dimensions in diagram are in mm

Flange cover plates:

```
+---------+---------+---------+---------+---------+---------+
| 50      | 150     | 50      |
+---------+---------+---------+---------+---------+---------+
| 40      | o       | o       | 40      | o       | o       |
| 160     | o       | o       | 160     | o       | o       |
| 160     | o       | o       | 160     | o       | o       |
| 125     | =========|         |         |         |         |
| 125     | o       | o       |         |         |         |
| 160     | o       | o       | 160     | o       | o       |
| 160     | o       | o       | 160     | o       | o       |
| 40      |         |         | 40      |         |         |
+---------+---------+---------+---------+---------+---------+
```

Flange cover plates: 2/ 250 mm x 12 mm x 845 mm
Number of bolts: 4/ 6 (Total No of bolts=24)
Packs: 2/ 250 mm x 30 mm x 410 mm
Bolt diameter: 20 mm
Bolt grade: Preloaded HSFG bolts
General grade (8.8), with grade 10 nuts.
Web angles: Provide web angles, each with four bolts.

NOTE: Packs to be fillet welded to the column flange.
Location: Ex4 - Non-bearing splice, HSFG bolts & web/flange plates

Bolted column splice

Non-bearing connection

Calculations are in accordance with EC3 Parts 1-1, 1-5 and 1-8.

Design assumptions:

- the column splice is about 500 mm above the floor level and hence the moment due to strut action is considered to be insignificant
- the centroidal axis of the splice is coincident with the centroidal axis of the members.

Factored loads

Permanent compressive axial load \( N_{EdG} = 650 \text{ kN} \)
Variable compressive axial load \( N_{EdQ} = 700 \text{ kN} \)
Design bending moment \( M_{yEd} = 65 \text{ kNm} \)
Design shear force \( V_{Ed} = 8 \text{ kN} \)
Load from floor below splice \( F_{stEd} = 405 \text{ kN} \)

Upper column

254 x 254 x 73 UKC

Lower column

305 x 305 x 118 UKC
Thickness of flange cover plate \( t_{fp} = 15 \text{ mm} \)
Width of flange cover plate \( b_{fp} = 250 \text{ mm} \)
Bolt diameter \( d_b = 24 \text{ mm} \)
Total no. of bolts in cover plate \( n_{t} = 20 \)
Pitch of bolts (vertically) \( p_{1fp} = 80 \text{ mm} \)
Spacing of bolts across columns \( s_{pac} = 110 \text{ mm} \)
End distance \( e_{1fp} = 50 \text{ mm} \)
Bolt spacing (horizontally) \( p_{2fp} = 150 \text{ mm} \)
Allowance for gap \( g_{ap} = 10 \text{ mm} \)

Web cover plates

Thickness of web plate \( t_{wp} = 12 \text{ mm} \)
Width of web cover plate \( b_{wp} = 190 \text{ mm} \)
Bolt cross-centres \( p_{2wp} = 100 \text{ mm} \)
Total no. of bolts in web cleat \( n_{tw} = 8 \)
SPLICE SUMMARY - all dimensions in diagram are in mm

Flange cover plates:

250

50

80

80

80

80

80

80

80

110

850

Allowance for gap 10 mm

Flange cover plates

Number of bolts

Packs

Bolt diameter

Bolt grade

250 mm x 15 mm x 850 mm

4/ 10 (Total No of bolts=40)

2/ 250 mm x 30 mm x 420 mm

24 mm

Preloaded HSFG bolts

General grade (8.8), with grade 10 nuts.
Web cover plates:

<table>
<thead>
<tr>
<th>Web cover plates</th>
<th>2/ 190 mm x 12 mm x 370 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bolts</td>
<td>2/ 4 (Total No of bolts=8)</td>
</tr>
</tbody>
</table>
**Location: Moment connection within middle third**

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (x-x axis) of column
- any applied moment is reversible and is carried by half the bolts
- the connection consists of two columns having the same serial size and end plate arrangement.

Factored moment $M=200 \text{ kNm}$
Factored shear force $V=75 \text{ kN}$
Axial force (+ve compression) $N=2000 \text{ kN}$
Load from floor below splice $Fst=770 \text{ kN}$

305 x 305 x 97 UC.
Diameter of bolts $bd=24 \text{ mm}$
Thickness of plate $tp=25 \text{ mm}$
Total number of bolts $bn=8$
Length of end plate $hp=600 \text{ mm}$
Distance to first row of bolts $ex=75 \text{ mm}$
Width of end plate $bp=600 \text{ mm}$
CONNECTION SUMMARY

Number of bolts  8
Bolt diameter    24 mm  75  75
Bolt grade       8.8
Welds            Tension flange fillet weld  8 mm
                 Web fillet weld         8 mm
                 Compression flange fillet weld  8 mm
Column section   305 x 305 x 97 UC
Location: Moment connection outside middle third

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (x-x axis) of column
- any applied moment is reversible and is carried by half the bolts
- the connection consists of two columns having the same serial size and end plate arrangement.

Factored moment $M=300 \text{ kNm}$
Factored shear force $V=75 \text{ kN}$
Axial force (+ve compression) $N=2000 \text{ kN}$
Load from floor below splice $Fst=700 \text{ kN}$

305 x 305 x 97 UC.
Diameter of bolts $bd=24 \text{ mm}$
Thickness of plate $tp=25 \text{ mm}$
Total number of bolts $bn=8$
Length of end plate $hp=600 \text{ mm}$
Distance to first row of bolts $ex=75 \text{ mm}$
Width of end plate $bp=600 \text{ mm}$

Selected fillet weld size $swt=9 \text{ mm}$
**CONNECTION SUMMARY**

Number of bolts: 8
Bolt diameter: 24 mm
Bolt grade: 8.8
Welds:
- Tension flange fillet weld: 9 mm
- Web fillet weld: 8 mm
- Compression flange fillet weld: 9 mm
Column section: 305 x 305 x 97 UC
Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

**Design assumptions:**
- the bolt arrangement is symmetrical about the centre line (x-x axis)
- moment is reversible
- moment is carried by half the bolts
- shear is carried by the other half of the bolts.
- the connection consists of two columns having the same serial size and end plate arrangements.

Factored moment  \( M = 400 \text{ kNm} \)
Factored shear force  \( V = 75 \text{ kN} \)
Axial force (+ve compression)  \( N = 300 \text{ kN} \)
Load from floor below splice  \( F_{st} = 210 \text{ kN} \)

305 x 305 x 97 UC.
Diameter of bolts  \( b_d = 30 \text{ mm} \)
Thickness of plate  \( t_p = 25 \text{ mm} \)
Total number of bolts  \( b_n = 12 \)
Length of end plate  \( h_p = 650 \text{ mm} \)
Distance to first row of bolts  \( e_x = 90 \text{ mm} \)
Pitch to second row of bolts  \( p_1 = 100 \text{ mm} \)
Pitch of tension bolts  \( p_2 = 90 \text{ mm} \)
Width of end plate  \( b_p = 320 \text{ mm} \)
Bolt cross-centres  \( g = 200 \text{ mm} \)

Penetration butt weld size  \( s_{bt} = 6 \text{ mm} \)
Superimposed fillet weld size  \( s_{wt} = 9 \text{ mm} \)
Full strength weld used
CONNECTION SUMMARY

Number of bolts 12
Bolt diameter 30 mm
Bolt grade 8.8
Tension Partial penetration butt welds 6 mm
Flange Welds Superimposed fillet welds 9 mm
Web fillet weld 8 mm
Compression Partial penetration butt welds 6 mm
Flange Welds Superimposed fillet welds 9 mm
Column section 305 x 305 x 97 UC
Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (x-x axis) of column
- any applied moment is reversible and is carried by half the bolts
- the connection consists of two columns having the same serial size and end plate arrangement.

Factored moment $M = 400$ kNm
Factored shear force $V = 75$ kN
Axial force (+ve compression) $N = 2500$ kN
Load from floor below splice $Fst = 770$ kN

300 x 300 x 10 SHS - Hot finished.
Properties (cm): $A = 115$ $rx = 11.8$ $Zx = 1070$ $Sx = 1250$ $Ix = 16000$ $J = 24800$ $C = 1580$

Diameter of bolts $bd = 24$ mm
Thickness of plate $tp = 30$ mm
Total number of bolts $bn = 8$
Length of end plate $hp = 550$ mm
Distance to first row of bolts $ex = 75$ mm
Width of end plate $bp = 550$ mm

Selected fillet weld size $swt = 16$ mm

Full strength weld used
## CONNECTION SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bolts</td>
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<tr>
<td>Bolt diameter</td>
<td>24 mm</td>
</tr>
<tr>
<td>Bolt grade</td>
<td>8.8</td>
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<tr>
<td>Welds</td>
<td></td>
</tr>
<tr>
<td>Tension flange fillet weld</td>
<td>16 mm</td>
</tr>
<tr>
<td>Web fillet weld</td>
<td>8 mm</td>
</tr>
<tr>
<td>Compression flange fillet weld</td>
<td>16 mm</td>
</tr>
<tr>
<td>Column section</td>
<td>300 x 300 x 10 Hot Finished SHS</td>
</tr>
</tbody>
</table>

- Bolt diameter: 24 mm
- Bolt grade: 8.8
- Welds:
  - Tension flange fillet weld: 16 mm
  - Web fillet weld: 8 mm
  - Compression flange fillet weld: 16 mm
- Column section: 300 x 300 x 10 Hot Finished SHS
Location: Moment connection outside middle third - RHS

Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (x-x axis) of column
- any applied moment is reversible and is carried by half the bolts
- the connection consists of two columns having the same serial size and end plate arrangement.

Factored moment \( M = 190 \text{ kNm} \)
Factored shear force \( V = 75 \text{ kN} \)
Axial force (+ve compression) \( N = 700 \text{ kN} \)
Load from floor below splice \( F_{st} = 250 \text{ kN} \)

300 x 300 x 10 SHS - Hot finished.
Properties (cm): \( A = 115 \) \( r_x = 11.8 \) \( Z_x = 1070 \) \( S_x = 1250 \) \( I_x = 16000 \) \( J = 24800 \) \( C = 1580 \)
Diameter of bolts \( b_d = 20 \text{ mm} \)
Thickness of plate \( t_p = 20 \text{ mm} \)
Total number of bolts \( b_n = 4 \)
Length of end plate \( h_p = 550 \text{ mm} \)
Distance to first row of bolts \( e_x = 75 \text{ mm} \)
Width of end plate \( b_p = 550 \text{ mm} \)
Selected fillet weld size \( s_wt = 12 \text{ mm} \)
<table>
<thead>
<tr>
<th>Number of bolts</th>
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<tbody>
<tr>
<td>Bolt diameter</td>
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</tr>
<tr>
<td>Bolt grade</td>
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<td>Welds</td>
<td>Tension flange fillet weld 12 mm</td>
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<td>Web fillet weld 8 mm</td>
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<td></td>
<td>Compression flange fillet weld 12 mm</td>
</tr>
<tr>
<td>Column section</td>
<td>300 x 300 x 10 Hot Finished SHS</td>
</tr>
</tbody>
</table>
Calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

**Design assumptions:**
- The bolt arrangement is symmetrical about the centre line (x-x axis) of column
- Any applied moment is reversible and is carried by half the bolts
- The connection consists of two columns having the same serial size and end plate arrangement.

**Factored moment** \( M = 0 \text{ kNm} \)
**Factored shear force** \( V = 75 \text{ kN} \)
**Axial force (+ve compression)** \( N = 2000 \text{ kN} \)
**Load from floor below splice** \( F_{st} = 540 \text{ kN} \)

305 x 305 x 97 UC.
**Diameter of bolts** \( b_d = 20 \text{ mm} \)
**Thickness of plate** \( t_p = 24 \text{ mm} \)
**Total number of bolts** \( b_n = 4 \)
**Length of end plate** \( h_p = 600 \text{ mm} \)
**Distance to first row of bolts** \( e_x = 75 \text{ mm} \)
**Width of end plate** \( b_p = 600 \text{ mm} \)
CONNECTION SUMMARY

Number of bolts 4
Bolt diameter 20 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 8 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Column section 305 x 305 x 97 UC
Location: Ex1- Moment connection within middle third

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (y-y axis) of column
- any applied moment is reversible and is carried by half the bolts
- the connection consists of two columns having the same serial size and end plate arrangement.

Factored moment \( M_{Ed} = 200 \text{ kNm} \)
Factored shear force \( V_{Ed} = 75 \text{ kN} \)
Axial force (positive compression) \( N_{Ed} = 2000 \text{ kN} \)
Load from floor below splice \( F_{stEd} = 770 \text{ kN} \)

Grade details

Supported column details

- 305 x 305 x 97 UKC

Bolt details

- Diameter of bolts \( d_b = 24 \text{ mm} \)
- Tensile resistance (EC3 Part 1-8) \( F_{tRd} = k_2 f_{ub} A_t / (\gamma_M 2 \times 10^3) = 203.33 \text{ kN} \)

End plate details

- Thickness of plate \( t_p = 25 \text{ mm} \)
- Total number of bolts \( b_n = 8 \)
- Length of end plate \( h_p = 600 \text{ mm} \)
- Distance to first row of bolts \( e_x = 75 \text{ mm} \)
- Width of end plate \( b_p = 600 \text{ mm} \)
- Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
- Assumed flange fillet weld size \( s_{wf} = 12 \text{ mm} \)
Column tension flange welds

Weld design strength \[ fvwd = \frac{fu}{\sqrt{3}} \times (\beta_{aw} \times \gamma_{M2}) \]
\[ = 222.79 \text{ N/mm}^2 \]

Selected fillet weld size \[ swt = 8 \text{ mm} \]

Column web welds

Selected fillet weld size \[ sfw = 8 \text{ mm} \]

Resistance of weld for shear \[ F_{wRds} = 2 \times 0.7 \times sfw \times fvwd \times Lws / 10^3 \]
\[ = 615.58 \text{ kN} \]

Compression welds

CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 8
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 8 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Column section 305 x 305 x 97 UKC
**Location: Ex2 - Moment connection outside middle third**

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

---

**Design assumptions:**
- the bolt arrangement is symmetrical about the centre line (y-y axis) of column
- any applied moment is reversible and is carried by half the bolts
- the connection consists of two columns having the same serial size and end plate arrangement.

---

**Factored moment**

\[ M_{Ed} = 300 \text{ kNm} \]

**Factored shear force**

\[ V_{Ed} = 75 \text{ kN} \]

**Axial force (positive compression)**

\[ N_{Ed} = 2000 \text{ kN} \]

**Load from floor below splice**

\[ F_{stEd} = 700 \text{ kN} \]

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**Grade details**

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**Supported column details**

305 x 305 x 97 UKC

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**Bolt details**

Diameter of bolts\(d_b=24 \text{ mm}\)

Tensile resistance (EC3 Part 1-8)\(F_{Rd}=k_2f_{ub}A_t/(\gamma_{M2}10^3)=203.33 \text{ kN}\)

---

**End plate details**

Thickness of plate\(t_p=25 \text{ mm}\)

Total number of bolts\(b_n=8\)

Length of end plate\(h_p=600 \text{ mm}\)

Distance to first row of bolts\(e_x=75 \text{ mm}\)

Width of end plate\(b_p=600 \text{ mm}\)

Assumed web fillet weld size\(s_{ww}=8 \text{ mm}\)

Assumed flange fillet weld size\(s_{wf}=12 \text{ mm}\)
**Column tension flange welds**

Weld design strength \( \text{fvwd} = \frac{\text{fu}}{\sqrt{3}} \) (betaw*gamM2) = 222.79 N/mm²

Weld size required \( \text{swfr} = \frac{FwEdf \times 10^3}{0.7 \times \text{fvwd}} \) = 2.7877 mm

Selected fillet weld size \( \text{swt} = 9 \) mm

**Column web welds**

Selected fillet weld size \( \text{sfw} = 8 \) mm

Resistance of weld for shear \( FwRds = 2 \times 0.7 \times \text{sfw} \times \text{fvwd} \times Lws/10^3 \) = 615.58 kN

**Compression welds**

**CONNECTION SUMMARY - all dimensions in diagram are in mm**

Number of bolts 8
Bolt diameter 24 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 9 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 9 mm
Column section 305 x 305 x 97 UKC
Location: Ex3 - Bolts along side of base

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (y-y axis)
- moment is reversible
- moment is carried by half the bolts
- shear is carried by the other half of the bolts.
- the connection consists of two columns having the same serial size and end plate arrangements.

Factored moment \( M_{Ed} = 400 \text{ kNm} \)
Factored shear force \( V_{Ed} = 75 \text{ kN} \)
Axial force (+ve compression) \( N_{Ed} = 300 \text{ kN} \)
Load from floor below splice \( F_{st Ed} = 210 \text{ kN} \)

Grade details

Supported column details
305 x 305 x 97 UKC

Bolt details
Diameter of bolts \( d_b = 30 \text{ mm} \)
Tensile resistance (EC3 Part 1-8) \( F_{t Rd} = k_2 \times f_{ub} \times A_t / (\gamma_M 2 \times 10^3) = 323.14 \text{ kN} \)

End plate details
Thickness of plate \( t_p = 25 \text{ mm} \)
Total number of bolts \( b_n = 12 \)
Length of end plate \( h_p = 650 \text{ mm} \)
Distance to first row of bolts \( e_x = 90 \text{ mm} \)
Pitch to second row of bolts \( p_2 = 100 \text{ mm} \)
Pitch of tension bolts \( p_1 = 90 \text{ mm} \)
Width of end plate \( b_p = 320 \text{ mm} \)
Bolt cross-centres \( p_3 = 200 \text{ mm} \)
Assumed web fillet weld size \( s_{ww} = 8 \text{ mm} \)
Assumed flange fillet weld size \( swf=12 \text{ mm} \)

**Column tension flange welds**

Weld design strength \( fvwd=\frac{fu}{\sqrt{3}/(\betaaw \cdot \gamma M2)} \)

\[ 222.79 \text{ N/mm}^2 \]

Weld size required \( swfr=FwEdf \times 10^3/(0.7 \cdot fvwd) = 13.376 \text{ mm} \)

Penetration butt weld size \( sbt=6 \text{ mm} \)

Superimposed fillet weld size \( swt=9 \text{ mm} \)

Design material strength \( fyw=265 \text{ N/mm}^2 \)

**Column web welds**

Full strength weld used

Selected fillet weld size \( sfw=8 \text{ mm} \)

**Compression welds**

**CONNECTION SUMMARY** - all dimensions in diagram are in mm

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<tr>
<td>p2</td>
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<td></td>
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</tbody>
</table>

Number of bolts 12

Bolt diameter 30 mm

Bolt grade 8.8

Tension Partial penetration butt welds 6 mm

Flange Welds Superimposed fillet welds 9 mm

Web fillet weld 8 mm

Compression Partial penetration butt welds 6 mm

Flange Welds Superimposed fillet welds 9 mm

Column section 305 x 305 x 97 UKC
Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (y-y axis) of column
- any applied moment is reversible and is carried by half the bolts
- the connection consists of two columns having the same serial size and end plate arrangement.

Factored moment: $M_{Ed} = 400 \text{ kNm}$
Factored shear force: $V_{Ed} = 75 \text{ kN}$
Axial force (+ve compression): $N_{Ed} = 2500 \text{ kN}$
Load from floor below splice: $F_{stEd} = 770 \text{ kN}$

Grade details

Supported column details

300 x 300 x 10 SHS - Hot finished.
Properties (cm): $A=115$ $iy=11.8$ $W_{ely}=1070$ $W_{ply}=1250$ $Iy=16000$ $It=24800$ $Wt=1580$

Bolt details

Diameter of bolts: $d_b = 24 \text{ mm}$
Tensile resistance (EC3 Part 1-8): $F_{tRd} = k_2*f_{ub}*A_t/(\gamma_M^2*10^3) = 203.33 \text{ kN}$

End plate details

Thickness of plate: $t_p = 30 \text{ mm}$
Total number of bolts: $b_n = 8$
Length of end plate: $h_p = 550 \text{ mm}$
Distance to first row of bolts: $e_x = 75 \text{ mm}$
Width of end plate: $b_p = 550 \text{ mm}$
Assumed web fillet weld size: $s_{ww} = 8 \text{ mm}$
Assumed flange fillet weld size: $s_{wf} = 12 \text{ mm}$
Column tension flange welds

Weld design strength \( fvwd = \frac{fu}{\sqrt{3}} \cdot \text{(betaw*gamM2)} \)

=222.79 N/mm²

Weld size required \( swfr = \frac{FwEdf \cdot 10^3}{0.7 \cdot fvwd} = 15.542 \text{ mm} \)

Selected fillet weld size \( swt = 16 \text{ mm} \)

Column web welds

Full strength weld used

Selected fillet weld size \( sfw = 8 \text{ mm} \)

Compression welds

### CONNECTION SUMMARY - all dimensions in diagram are in mm

- Number of bolts: 8
- Bolt diameter: 24 mm
- Bolt grade: 8.8
- Welds:
  - Tension flange fillet weld: 16 mm
  - Web fillet weld: 8 mm
  - Compression flange fillet weld: 16 mm
- Column section: 300 x 300 x 10 Hot Finished SHS
Location: Ex5 - Moment connection outside middle third - RHS

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

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Design assumptions:
- the bolt arrangement is symmetrical about the centre line (y-y axis) of column
- any applied moment is reversible and is carried by half the bolts
- the connection consists of two columns having the same serial size and end plate arrangement.

Factored moment $M_{Ed}=190$ kNm
Factored shear force $V_{Ed}=75$ kN
Axial force (+ve compression) $N_{Ed}=700$ kN
Load from floor below splice $F_{stEd}=250$ kN

Grade details

Supported column details

300 x 300 x 10 SHS - Hot finished.
Properties (cm): $A=115$ iy=11.8 Wely=1070 Wply=1250 Iy=16000 It=24800 Wt=1580

Bolt details

Diameter of bolts $d_b=20$ mm
Tensile resistance (EC3 Part 1-8) $F_{tRd}=k_2*f_{ub}*A_t/(\gamma_m M_2*10^3)=141.12$ kN

End plate details

Thickness of plate $t_p=20$ mm
Total number of bolts $b_n=4$
Length of end plate $h_p=550$ mm
Distance to first row of bolts $e_x=75$ mm
Width of end plate $b_p=550$ mm
Assumed web fillet weld size $s_{ww}=8$ mm
Assumed flange fillet weld size $s_{wf}=12$ mm
Column tension flange welds

Weld design strength \[ \text{fvwd} = \frac{\text{fu}}{\sqrt{3}} \left( \frac{1}{\text{betaw}} \right) \text{gamM2} \]
= 222.79 kN/mm²

Weld size required \[ \text{swfr} = \frac{\text{FwEdf} \times 10^3}{0.7 \times \text{fvwd}} = 10.101 \text{ mm} \]
Selected fillet weld size \[ \text{swt} = 12 \text{ mm} \]

Column web welds

Selected fillet weld size \[ \text{sfw} = 8 \text{ mm} \]
Resistance of weld for shear \[ \text{FwRds} = 2 \times 0.7 \times \text{sfw} \times \text{fvwd} \times \text{Lws} / 10^3 \]
= 673.71 kN

Compression welds

CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 4
Bolt diameter 20 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 12 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 12 mm
Column section 300 x 300 x 10 Hot Finished SHS
Location: Ex6 - Axial load only

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'.

Design assumptions:
- the bolt arrangement is symmetrical about the centre line (y-y axis) of column
- any applied moment is reversible and is carried by half the bolts
- the connection consists of two columns having the same serial size and end plate arrangement.

Factored moment \( M_{Ed} = 0 \) kNm
Factored shear force \( V_{Ed} = 75 \) kN
Axial force (+ve compression) \( N_{Ed} = 2000 \) kN
Load from floor below splice \( F_{stEd} = 540 \) kN

Grade details

Supported column details
305 x 305 x 97 UKC

Bolt details
Diameter of bolts \( d_b = 20 \) mm
Tensile resistance (EC3 Part 1-8) \( F_{tRd} = k_2 f_{ub} A_t / (\gamma M_2 10^3) = 141.12 \) kN

End plate details
Thickness of plate \( t_p = 24 \) mm
Total number of bolts \( b_n = 4 \)
Length of end plate \( h_p = 600 \) mm
Distance to first row of bolts \( e_x = 75 \) mm
Width of end plate \( b_p = 600 \) mm
Assumed web fillet weld size \( s_{ww} = 8 \) mm
Assumed flange fillet weld size \( s_{wf} = 12 \) mm
Column tension flange welds

Weld design strength \( f_{vwd} = \frac{f_u}{\sqrt{3}} \left( \frac{\beta \omega \gamma}{M_2} \right) \)
\[
= 222.79 \text{ N/mm}^2
\]

Selected fillet weld size \( s_{wt} = 8 \text{ mm} \)

Column web welds

Selected fillet weld size \( s_{fw} = 8 \text{ mm} \)

Resistance of weld for shear \( F_{wRds} = 2 \times 0.7 \times s_{fw} \times f_{vwd} \times L_{ws} / 10^3 \)
\[
= 615.58 \text{ kN}
\]

Compression welds

Connection summary - all dimensions in diagram are in mm

Number of bolts 4
Bolt diameter 20 mm
Bolt grade 8.8
Welds
- Tension flange fillet weld 8 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 8 mm
Column section 305 x 305 x 97 UKC
Bolted beam splice

Design assumptions:

- the flange plates resist the full bending moment & axial loading
- the web splice resists the vertical shear and torsional moment induced by the eccentricity of this loading on the bolt groups on each side of the joint
- the splice uses HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

Design of splice plate subjected to an applied moment, shear force and axial load to BS5950.

All loads and moment are factored.

Shear force \( F_v = 250 \text{ kN} \)
Bending moment \( M = 150 \text{ kNm} \)
Axial force (+ve compression) \( N = 0 \text{ kN} \)

457 x 152 x 60 UB.
Total number of bolts \( n = 8 \)
Diameter of web bolts \( b_d = 20 \text{ mm} \)
Bolt pitch \( p = 90 \text{ mm} \)
Vertical edge distance \( e_d 1 = 35 \text{ mm} \)
Horizontal edge distance \( e_d 2 = 35 \text{ mm} \)
Distance between bolt centres \( g = 70 \text{ mm} \)
Thickness of web plate \( t_{wp} = 8 \text{ mm} \)
Slip factor for preloaded bolts \( \mu = 0.5 \)
Coefficient for type of hole \( K_s = 1 \)
Allowance for clearance between ends \( c_l = 6 \text{ mm} \)
WEB COVER PLATE SUMMARY - all dimensions in diagram are in mm

Web cover plates: 2No/140 mm x 8 mm x 340 mm
Number of bolts: 8
Bolt diameter: 20 mm
Bolt grade: HSFG
Nut grade: 10
General grade (8.8) to BS4395 Part 1

FLANGE SPLICE

Plate details

Thickness of flange plate: tcp=15 mm
Width of flange cover plate: Bfp=150 mm
Projection of flange cover plate: P=220 mm

Bolt details

Bolt spacing (across the beam): crs=90 mm
Total no. of bolts in cover plate: np=12
Pitch of bolts (along the beam): pit=70 mm
Spacing of bolts across beams: spac=70 mm
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plates: 2No/150 mm x 15 mm x 440 mm
Number of bolts: 12 per flange cover plate
Bolt diameter: 20 mm
Bolt grade: HSFG
Nut grade: 10

General grade (8.8) to BS4395 Part 1
Location: Ex2 - Moment connections example

Bolted beam splice

Design assumptions:

- the flange plates resist the full bending moment & axial loading
- the web splice resists the vertical shear and torsional moment induced by the eccentricity of this loading on the bolt groups on each side of the joint
- the splice uses HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

Design of splice plate subjected to an applied moment, shear force and axial load to BS5950.

All loads and moment are factored.

Shear force \( F_v = 150 \text{ kN} \)

Bending moment \( M = 190 \text{ kNm} \)

Axial force (+ve compression) \( N = 150 \text{ kN} \)

457 x 191 x 67 UB.

Total number of bolts \( n = 6 \)

Diameter of web bolts \( b_d = 20 \text{ mm} \)

Bolt pitch \( p = 100 \text{ mm} \)

Vertical edge distance \( e_d1 = 50 \text{ mm} \)

Horizontal edge distance \( e_d2 = 50 \text{ mm} \)

Distance between bolt centres \( g = 100 \text{ mm} \)

Thickness of web plate \( t_w = 10 \text{ mm} \)

Slip factor for preloaded bolts \( \mu = 0.5 \)

Coefficient for type of hole \( K_s = 1 \)

Allowance for clearance btwn ends \( cl = 10 \text{ mm} \)
WEB COVER PLATE SUMMARY - all dimensions in diagram are in mm

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<tr>
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<td>0</td>
<td>50</td>
</tr>
</tbody>
</table>

Web cover plates 2No/200 mm x 10 mm x 300 mm
Number of bolts 6
Bolt diameter 20 mm
Bolt grade HSFG
  General grade (8.8) to BS4395 Part 1
Nut grade 10

FLANGE SPLICE

Plate details

| Thickness of flange plate | tcp=12 mm |
| Width of flange cover plate | Bfp=180 mm |
| Projection of flange cover plate | P=360 mm |

Bolt details

| Bolt spacing (across the beam) | crs=90 mm |
| Total no. of bolts in cover plate | np=16 |
| Pitch of bolts (along the beam) | pit=75 mm |
| Spacing of bolts across beams | spac=150 mm |

Effective length of beam Le=5.1 m
Distance between pts.of restraint L=6 m
Point of restraint Lx=2 m
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

180

60

75

75

75

150

45 90 45

720

Flange cover plates  2No/180 mm x 12 mm x 720 mm
Number of bolts  16 per flange cover plate
Bolt diameter  20 mm
Bolt grade  HSFG
  General grade (8.8) to BS4395 Part 1
Nut grade  10
**Location:** Ex3 - Web cleat 4 rows of bolts, double flange plates

**Bolted beam splice**

**Design assumptions:**

- the flange plates resist the full bending moment & axial loading
- the web splice resists the vertical shear and torsional moment induced by the eccentricity of this loading on the bolt groups on each side of the joint
- the splice uses HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

---

**Design of splice plate subjected to an applied moment, shear force and axial load to BS5950.**

All loads and moment are factored.

Shear force \( F_v = 390 \text{ kN} \)

Bending moment \( M = 286 \text{ kNm} \)

Axial force (+ve compression) \( N = 225 \text{ kN} \)

533 x 210 x 92 UB.

Total number of bolts \( n = 16 \)

Diameter of web bolts \( b_d = 20 \text{ mm} \)

Bolt pitch \( p = 100 \text{ mm} \)

Vertical edge distance \( e_d = 50 \text{ mm} \)

Horizontal edge distance \( e_d = 50 \text{ mm} \)

Distance between bolt centres \( S_1 = 100 \text{ mm} \)

Distance between bolt centres \( S_2 = 90 \text{ mm} \)

Thickness of web plate \( t_w = 10 \text{ mm} \)

Slip factor for preloaded bolts \( \mu = 0.5 \)

Coefficient for type of hole \( K_s = 1 \)

Allowance for clearance between ends \( c_l = 10 \text{ mm} \)
WEB COVER PLATE SUMMARY - all dimensions in diagram are in mm

<table>
<thead>
<tr>
<th>50</th>
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</tbody>
</table>

Web cover plates          2No/380 mm x 10 mm x 400 mm
Number of bolts           16
Bolt diameter             20 mm
Bolt grade                HSFG
Nut grade                 General grade (8.8) to BS4395 Part 1
Number of bolts in cover plate          np=12
Pitch of bolts (along the beam)          pit=75 mm
Spacing of bolts across beams          spac=75 mm
Slip factor for preloaded bolts          mu=0.5
Coefficient for type of hole           Ks=1
Moment about y-y axis                  Myy=12 kNm
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plates:
outer (as illustrated) 2No/210 mm x 16 mm x 500 mm
inner (not illustrated) 4No/95 mm x 16 mm x 500 mm
same holing arrangement

Number of bolts 12 per flange cover plate
Bolt diameter 24 mm
Bolt grade HSFG
Nut grade General grade (8.8) to BS4395 Part 1
Location: Ex1 - Using preloaded HSFG bolts

Bolted beam splice

Design assumptions:

- the flange plates resist a proportion of the BM and axial load
- the web plates resist all the vertical shear, the moment due to the eccentricity of the bolt group resisting shear from the centreline of the splice and a proportion of the bending moment
- the splice uses preloaded HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

Design of splice plate subjected to an applied moment, shear force and axial load to EC3 Parts 1-1, 1-5, 1-8 and SCI publication P398. Loads and moment are factored.

Shear force \( V_{Ed} = 250 \text{ kN} \)
Bending moment \( My_{Ed} = 150 \text{ kNm} \)
Axial force (+ve compression) \( N_{Ed} = 0 \text{ kN} \)

Grade details

457 x 152 x 60 UKB
Total number of bolts \( nb = 8 \)
Diameter of web bolts \( dbw = 20 \text{ mm} \)
Bolt pitch \( p1 = 90 \text{ mm} \)
Vertical edge distance \( e1 = 35 \text{ mm} \)
Horizontal edge distance \( e2 = 35 \text{ mm} \)
Thickness of web plate \( twp = 8 \text{ mm} \)
Distance between bolt centres \( p3 = 70 \text{ mm} \)
WEB COVER PLATE SUMMARY - all dimensions in diagram are in mm

35
90
90
90
35

35 70 35

Web cover plates 2No/140 mm x 8 mm x 340 mm
Number of bolts 8
Bolt diameter 20 mm
Bolt grade Preloaded HSFG bolts
General grade (8.8) with grade 10 nuts

FLANGE SPLICE

Plate details

Thickness of flange plate tfp=15 mm
Width of flange cover plate bfp=150 mm
Projection of flange cover plate P=220 mm

Bolt details

Bolt spacing (across the beam) p3fp=90 mm
Total no. of bolts in cover plate np=12
Pitch of bolts (along the beam) plfp=70 mm
Spacing of bolts across beams spac=70 mm
Edge distance e2fp=(bfp-p3fp)/2=30 mm
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plates: 2No/150 mm x 15 mm x 440 mm
Number of bolts: 12 per flange cover plate
Bolt diameter: 20 mm
Bolt grade: Preloaded HSFG bolts
            General grade (8.8) with grade 10 nuts
Location: Ex2 - Moment connections example

Bolted beam splice

Design assumptions:

- the flange plates resist a proportion of the BM and axial load
- the web plates resist all the vertical shear, the moment due to the eccentricity of the bolt group resisting shear from the centreline of the splice and a proportion of the bending moment
- the splice uses preloaded HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

| Design of splice plate subjected to an applied moment, shear force and axial load to EC3 Parts 1-1, 1-5, 1-8 and SCI publication P398. Loads and moment are factored. |
|---|---|
| Shear force | VEd=150 kN |
| Bending moment | MyEd=160 kNm |
| Axial force (+ve compression) | NEd=150 kN |

Grade details

457 x 191 x 67 UKB
Total number of bolts nb=6
Diameter of web bolts dbw=20 mm
Bolt pitch p1=100 mm
Vertical edge distance e1=50 mm
Horizontal edge distance e2=50 mm
Thickness of web plate twp=10 mm
Distance between bolt centres p3=100 mm
WEB COVER PLATE SUMMARY - all dimensions in diagram are in mm

Web cover plates 2No/200 mm x 10 mm x 300 mm
Number of bolts 6
Bolt diameter 20 mm
Bolt grade Preloaded HSFG bolts
General grade (8.8) with grade 10 nuts

FLANGE SPLICE

Plate details
Thickmess of flange plate tfp=12 mm
Width of flange cover plate bfp=180 mm
Projection of flange cover plate P=360 mm

Bolt details
Bolt spacing (across the beam) p3fp=90 mm
Total no. of bolts in cover plate np=16
Pitch of bolts (along the beam) plfp=75 mm
Spacing of bolts across beams spac=150 mm
Edge distance e2fp=(bfp-p3fp)/2=45 mm
Moment about z-z axis Mzz=10 kNm
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plates: 2No/180 mm x 12 mm x 720 mm
Number of bolts: 16 per flange cover plate
Bolt diameter: 20 mm
Bolt grade: Preloaded HSFG bolts
General grade (8.8) with grade 10 nuts
Location: Ex3 - Web cleat 4 rows of bolts, double flange plates

Bolted beam splice

Design assumptions:

- the flange plates resist a proportion of the BM and axial load
- the web plates resist all the vertical shear, the moment due to the eccentricity of the bolt group resisting shear from the centreline of the splice and a proportion of the bending moment
- the splice uses preloaded HSFG bolts to avoid producing bolt slip and consequent additional beam deflection.

Shear force $V_{Ed}=390$ kN
Bending moment $M_{Ed}=286$ kNm
Axial force (+ve compression) $N_{Ed}=225$ kN

Grade details

533 x 210 x 92 UKB
Total number of bolts $nb=16$
Diameter of web bolts $dbw=20$ mm
Bolt pitch $p1=100$ mm
Vertical edge distance $e1=50$ mm
Horizontal edge distance $e2=50$ mm
Thickness of web plate $twp=10$ mm
Distance between bolt centres $p5=100$ mm
Distance between bolt centres $p3=90$ mm
WEB COVER PLATE SUMMARY - all dimensions in diagram are in mm

Web cover plates 2No/380 mm x 10 mm x 400 mm
Number of bolts 16
Bolt diameter 20 mm
Bolt grade Preloaded HSFG bolts
  General grade (8.8) with grade 10 nuts

FLANGE SPLICE

Plate details

<table>
<thead>
<tr>
<th>Thickness of flange plate</th>
<th>tfp=16 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of outer flange cover plate</td>
<td>bfp=210 mm</td>
</tr>
<tr>
<td>Width of inner splice plate</td>
<td>bif=95 mm</td>
</tr>
<tr>
<td>Projection of flange cover plate</td>
<td>P=270 mm</td>
</tr>
</tbody>
</table>

Bolt details

| Bolt diameter in flange plate | dbfp=24 mm |
| Total no. of bolts in cover plate | np=12 |
| Pitch of bolts (along the beam) | plfp=75 mm |
| Spacing of bolts across beams | spac=75 mm |
| Edge distance | e2fp=(bfp-p3fp)/2=35 mm |
| Moment about z-z axis | Mzz=12 kNm |
FLANGE COVER PLATE SUMMARY - all dimensions in diagram are in mm

Flange cover plates:
outer (as illustrated) 2No/210 mm x 16 mm x 540 mm
inner (not illustrated) 4No/95 mm x 16 mm x 540 mm
same holing arrangement
Number of bolts 12 per flange cover plate
Bolt diameter 24 mm
Bolt grade Preloaded HSFG bolts
General grade (8.8) with grade 10 nuts
Location: Ex1 - X joint subject to moment

Calculations are in accordance with BS5950 and ENV 1993-1-1-ANNEX K using the procedure of Section 5.4 of 'Design of SHS Welded Joints' published by British Steel Tubes & Pipes.

Steel section properties

Chord details:
305 x 305 x 118 UC.

Brace 1 details:
120 x 120 x 10 SHS - Hot finished.
Properties (cm): A=42.9 rx=4.46 Zx=142 Sx=175 Ix=852 J=1380 C=206

Brace 2 details:
100 x 100 x 8 SHS - Hot finished.
Properties (cm): A=28.8 rx=3.73 Zx=79.9 Sx=98.2 Ix=400 J=646 C=116

Geometry

Angle of brace θ1 = 45°
Angle of brace θ2 = 45°

Factored axial load details - compression is negative

Chord (axial load 1)  Fc1 = -1200 kN
Chord (axial load 2)  Fc2 = -1500 kN
Brace member 1 (axial load)  Fb1 = -450 kN
Brace member 1 (in-plane moment)  Mi1 = 20 kNm
Brace member 2 (axial load)  Fb2 = -350 kN
Brace member 2 (in-plane moment)  Mi2 = 10 kNm

CAPACITY SUMMARY

Brace member 1:
Applied brace 1 axial loading  Fb1 = -450 kN
Load within capacity  Pass
Load capacities:
Chord web yielding  N1 = 1986 kN
Bracing effective width  NE1 = 1204.3 kN
In-plane moment  Mi1 = 20 kNm
Moment within capacity  Pass
In-plane moment capacities:
Chord web yielding  Mic1 = 77.237 kNm
Bracing effective width  Mib1 = 66.234 kNm
The load and moment interaction formula for universal section chord joints is not specified in the 'Design of SHS Welded Joints' publication. The formula below has been assumed based on the interaction equation for RHS chord joints.

\[
\frac{F_{b1}}{N_{crit}} + \frac{M_{i1}}{M_{c1}} \leq 1.0
\]

Critical load capacity \( N_{crit} = 1204.3 \text{ kN} \)
Critical moment capacity \( M_{c1} = 66.234 \text{ kNm} \)
Absolute value of brace force \( F_{b1} = -F_{b1} = 450 \text{ kN} \)
Interaction \( \text{unity} = F_{b1}/N_{crit} + M_{i1}/M_{c1} = 0.67564 \)

The interaction formula is satisfied, joint arrangement suitable.

Brace member 2:
Applied brace 2 axial loading \( F_{b2} = -350 \text{ kN} \)
Load within capacity Pass
Load capacities:
Chord web yielding \( N_{2} = 1820.4 \text{ kN} \)
Bracing effective width \( N_{E2} = 963.4 \text{ kN} \)
In-plane moment \( M_{i2} = 10 \text{ kNm} \)
Moment within capacity Pass
In-plane moment capacities:
Chord web yielding \( M_{ic2} = 59.212 \text{ kNm} \)
Bracing effective width \( M_{ib2} = 44.316 \text{ kNm} \)

\[
\frac{F_{b2}}{N_{crit2}} + \frac{M_{i2}}{M_{c2}} \leq 1.0
\]

Critical load capacity \( N_{crit2} = 963.4 \text{ kN} \)
Critical moment capacity \( M_{c2} = 44.316 \text{ kNm} \)
Absolute value of brace force \( F_{b2} = -F_{b2} = 350 \text{ kN} \)
Interaction \( \text{unity} = F_{b2}/N_{crit2} + M_{i2}/M_{c2} = 0.58895 \)

The interaction formula is satisfied, joint arrangement suitable.
Location: Ex2 - K gap joint with CHS braces

Calculations are in accordance with BS5950 and ENV 1993-1-1-ANNEX K using the procedure of Section 5.4 of 'Design of SHS Welded Joints' published by British Steel Tubes & Pipes.

Steel section properties

Chord details:
254 x 254 x 107 UC.

Brace 1 details:
168.3 dia x 8 thick CHS - Hot finished.
Properties (cm): A=40.288 r=5.6745 Z=154.16 S=205.74 I=1297.3

Brace 2 details:
139.7 dia x 6.3 thick CHS - Hot finished.
Properties (cm): A=26.403 r=4.7217 Z=84.269 S=112.2 I=588.62

Geometry

Angle of brace \(\theta_1\) \(\theta_1=45^\circ\)
Angle of brace \(\theta_2\) \(\theta_2=45^\circ\)
Gap between bracing elements \(g=80\) mm

Factored axial load details - compression is negative

Chord (axial load 1) \(F_{c1}=-1200\) kN
Chord (axial load 2) \(F_{c2}=-1500\) kN
Brace member 1 (axial load) \(F_{b1}=-650\) kN
Brace member 2 (axial load) \(F_{b2}=450\) kN

CAPACITY SUMMARY

Brace member 1:
Applied brace 1 axial loading \(F_{b1}=-650\) kN
Load within capacity Pass

Load capacities brace 1:
Chord web yielding capacity \(N_1=1706.9\) kN
Chord shear capacity \(N_{S1}=1062.6\) kN
Bracing effective width \(N_{W1}=792.54\) kN

Brace member 2:
Applied brace 2 loading \(F_{b2}=450\) kN
Load within capacity Pass

Load capacities:
Chord web yielding capacity \(N_2=1706.9\) kN
Chord shear capacity \(N_{S2}=1062.6\) kN
Bracing effective width \(N_{W2}=624.13\) kN
**Location:** Ex3 - K overlap joint with RHS braces

Calculations are in accordance with BS5950 and ENV 1993-1-1-ANNEX K using the procedure of Section 5.4 of 'Design of SHS Welded Joints' published by British Steel Tubes & Pipes.

**Steel section properties**

Chord details:
203 x 203 x 71 UC.

Brace 1 details:
100 x 200 x 8 RHS - Hot finished.
Properties (cm): $A=44.8$, $r_x=4.06$, $Z_x=148$, $S_x=172$, $I_x=739$

\[
J=1800 \quad C=251 \quad Z_y=223 \quad S_y=282 \quad I_y=2230 \quad r_y=7.06
\]

Brace 2 details:
100 x 150 x 6.3 RHS - Hot finished.
Properties (cm): $A=29.5$, $r_x=4.01$, $Z_x=94.8$, $S_x=110$, $I_x=474$

\[
J=986 \quad C=153 \quad Z_y=120 \quad S_y=147 \quad I_y=898 \quad r_y=5.52
\]

**Geometry**

Angle of brace 01 $\theta_1=60^\circ$
Angle of brace 02 $\theta_2=60^\circ$
Gap between bracing elements $g=-50$ mm

**Factored axial load details - compression is negative**

<table>
<thead>
<tr>
<th></th>
<th>Chord (axial load 1)</th>
<th>Chord (axial load 2)</th>
<th>Brace member 1 (axial load)</th>
<th>Brace member 2 (axial load)</th>
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<tbody>
<tr>
<td>Fc1</td>
<td>-500 kN</td>
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<tr>
<td>Fc2</td>
<td>-600 kN</td>
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<tr>
<td>Fb1</td>
<td>-450 kN</td>
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<tr>
<td>Fb2</td>
<td>300 kN</td>
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</table>

**CAPACITY SUMMARY**

**Brace member 1:**
- Applied brace 1 axial loading Fb1 -450 kN
- Bracing effective width NO1 1264.4 kN
- Load within capacity Pass

**Brace member 2:**
- Applied brace loading (2) Fb2 300 kN
- Bracing effective width NO2 832.61 kN
- Load within capacity Pass
Location: Ex1 - X joint subject to moment

Calculations are in accordance with EC3 Part 1-1 and TATA Steel publication entitled 'Design of Welded Joints'.

Steel section properties

Chord details:
305 x 305 x 118 UKC

Brace 1 details:
120 x 120 x 10 SHS - Hot finished.
Properties (cm): A=42.9 iy=4.46 Wely=142 Wply=175 Iy=852 It=1380 Wt=206

Brace 2 details:
100 x 100 x 8 SHS - Hot finished.
Properties (cm): A=28.8 iy=3.73 Wely=79.9 Wply=98.2 Iy=400 It=646 Wt=116

Geometry

Angle of brace 1: $\theta_1 = 45^\circ$
Angle of brace 2: $\theta_2 = 45^\circ$

Factored axial load details - compression is negative

Chord (axial load 1) $N_{c1} = -1200$ kN
Chord (axial load 2) $N_{c2} = -1500$ kN
Brace member 1 (axial load) $N_{b1} = -450$ kN
Brace member 1 (in-plane moment) $M_{i1} = 20$ kNm
Brace member 2 (axial load) $N_{b2} = -350$ kN
Brace member 2 (in-plane moment) $M_{i2} = 10$ kNm

DESIGN RESISTANCE SUMMARY

Brace member 1:
Applied brace 1 axial loading $N_{b1} = -450$ kN
Load within resistance Pass
Load resistances:
Chord web yielding $N_1 = 1986$ kN
Bracing effective width $N_{E1} = 1204.3$ kN
In-plane moment $M_{i1} = 20$ kNm
Moment within resistance Pass
In-plane moment resistances:
Chord web failure $M_{ic1} = 77.237$ kNm
Bracing effective width $M_{ib1} = 66.234$ kNm
The load and moment interaction formula for universal section chord joints is not specified in the 'Design of Welded Joints' publication. The formula below has been assumed based on the interaction equation for RHS chord joints.

\[
\frac{Nb1}{N_{crit1}} + \frac{Mi1}{Mc1} \leq 1.0
\]

Critical load resistance \( N_{crit1} = 1204.3 \) kN
Critical moment resistance \( Mc1 = 66.234 \) kNm
Interaction \( unity = \text{ABS}(Nb1)/N_{crit1} + Mi1/Mc1 = 0.67564 \)

The interaction formula is satisfied, joint arrangement suitable.

Brace member 2:
Applied brace 2 axial loading \( Nb2 = -350 \) kN
Load within resistance Pass
Load resistances:
- Chord web yielding \( N2 = 1820.4 \) kN
- Bracing effective width \( NE2 = 963.4 \) kN
- In-plane moment \( Mi2 = 10 \) kNm

Moment within resistance Pass
In-plane moment resistances:
- Chord web failure \( Mic2 = 59.212 \) kNm
- Bracing effective width \( Mib2 = 44.316 \) kNm

\[
\frac{Nb2}{N_{crit2}} + \frac{Mi2}{Mc2} \leq 1.0
\]

Critical load resistance \( N_{crit2} = 963.4 \) kN
Critical moment resistance \( Mc2 = 44.316 \) kNm
Interaction \( unity = \text{ABS}(Nb2)/N_{crit2} + Mi2/Mc2 = 0.58895 \)

The interaction formula is satisfied, joint arrangement suitable.
Location: Ex2 - K gap joint with CHS braces

Calculations are in accordance with EC3 Part 1-1 and TATA Steel publication entitled 'Design of Welded Joints'.

Steel section properties

Chord details:
254 x 254 x 107 UKC

Brace 1 details:
168.3 dia x 8 thick CHS - Hot finished.
Properties (cm): A=40.288 iy=5.6745 Wely=154.16 Wply=205.74 Iy=1297.3
It=2594.5

Brace 2 details:
139.7 dia x 6.3 thick CHS - Hot finished.
Properties (cm): A=26.403 iy=4.7217 Wely=84.269 Wply=112.2 Iy=588.62
It=1177.2

Geometry

Angle of brace θ1 = 45°
Angle of brace θ2 = 45°
Gap between bracing elements = g=80 mm

Factored axial load details - compression is negative

Chord (axial load 1) Nc1=-1200 kN
Chord (axial load 2) Nc2=-1500 kN
Brace member 1 (axial load) Nb1=-650 kN
Brace member 2 (axial load) Nb2=450 kN

DESIGN RESISTANCE SUMMARY

Brace member 1:
Applied brace 1 axial loading Nb1 -650 kN
Load within resistance Pass

Load resistances brace 1:
Chord web yielding resistance N1 1706.9 kN
Chord shear resistance NS1 1062.6 kN
Bracing effective width NW1 792.54 kN

Brace member 2:
Applied brace 2 loading Nb2 450 kN
Load within resistance Pass

Load resistances:
Chord web yielding resistance N2 1706.9 kN
Chord shear resistance NS2 1062.6 kN
Bracing effective width NW2 624.13 kN
Location: Ex3 - K overlap joint with RHS braces

Calculations are in accordance with EC3 Part 1-1 and TATA Steel publication entitled 'Design of Welded Joints'

Steel section properties

Chord details:
203 x 203 x 71 UKC

Brace 1 details:
100 x 200 x 8 RHS - Hot finished.
Properties (cm): A=44.8 iz=7.06 Wely=148 Wply=172 Iy=739
Iz=1800 Wt=251 Welz=223 Wplz=282 Iy=739 iy=4.06

Brace 2 details:
100 x 150 x 6.3 RHS - Hot finished.
Properties (cm): A=29.5 iz=5.52 Wely=94.8 Wply=110 Iy=474
It=986 Wt=153 Welz=120 Wplz=147 Iy=986 iy=4.01

Geometry

Angle of brace O1 theta1=60°
Angle of brace O2 theta2=60°
Gap between bracing elements g=-50 mm

Factored axial load details - compression is negative

<table>
<thead>
<tr>
<th>Chord (axial load 1)</th>
<th>Nc1=-500 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chord (axial load 2)</td>
<td>Nc2=-600 kN</td>
</tr>
<tr>
<td>Brace member 1 (axial load)</td>
<td>Nb1=-450 kN</td>
</tr>
<tr>
<td>Brace member 2 (axial load)</td>
<td>Nb2=300 kN</td>
</tr>
</tbody>
</table>

DESIGN RESISTANCE SUMMARY

<table>
<thead>
<tr>
<th>Brace member 1:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied brace 1 axial loading</td>
</tr>
<tr>
<td>Bracing effective width</td>
</tr>
<tr>
<td>Load within resistance</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Brace member 2:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied brace loading (2)</td>
</tr>
<tr>
<td>Bracing effective width</td>
</tr>
<tr>
<td>Load within resistance</td>
</tr>
</tbody>
</table>
Location: Ex1 - using 4 bolts in haunch

**Portal frame apex connection (haunched)**

The calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The factored moment and forces at the end of the rafter act on a cross-section normal to the axis of the basic rafter (unhaunched) member.

Factored moment \( Ma = 387 \text{ kNm} \)
Factored shear force \( Va = 256 \text{ kN} \)
Axial force (+ve compression) \( Na = 150 \text{ kN} \)
Pitch of rafter (in degrees) \( \theta = 10 \)

**Grade details**

**Rafter details - parent section**

457 x 191 x 74 UB.

**Haunch plate details**

Flange width \( Bh = 200 \text{ mm} \)
Flange thickness \( Th = 15 \text{ mm} \)
Web thickness \( th = 10 \text{ mm} \)

**Bolt details**

Diameter of bolts \( bd = 24 \text{ mm} \)
End plate details

Thickness of plate          tp=20 mm
Number of bolts in haunch  bnf=4
Total number of bolts      bn=12
Number of shear bolts      ns=4
Distance to first row of bolts  pe=100 mm
Distance between tension bolts  p1=130 mm
Distance to haunch bolts   x=70 mm
Pitch of bolts             p2=90 mm
Distance to flange from edge(s)  ex=15 mm
Bolt cross-centres         g=90 mm
Width of end plate         bp=200 mm
Assumed web fillet weld size  swww=8 mm
Assumed flange fillet weld size  swf=12 mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts          Pr1=363.73 kN
Row 2                      Pr2=284.41 kN
Row 3                      Pr3=363.55 kN
Row 4                      Pr4=281.58 kN
Selected fillet weld size  swt=12 mm

Selected fillet weld size  sfw=8 mm

REVERSED MOMENT CONDITION

Factored moment           Mr=123 kNm
Factored shear force      Vr=103 kN
Axial force (+ve compression)  Nr=45 kN
### CONNECTION SUMMARY - all dimensions in diagram are in mm

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
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<tbody>
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<table>
<thead>
<tr>
<th>Number of bolts</th>
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<tbody>
<tr>
<td>Bolt diameter</td>
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<tr>
<td>Bolt grade</td>
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<td>Welds</td>
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<tr>
<td>Tension flange fillet weld</td>
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<td>Web fillet weld</td>
<td>8 mm</td>
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<tr>
<td>Compression flange fillet weld</td>
<td>12 mm</td>
</tr>
<tr>
<td>Beam / Rafter section</td>
<td>457 x 191 x 74 UB</td>
</tr>
<tr>
<td>Haunch Flange plate</td>
<td>200 mm x 15 mm</td>
</tr>
<tr>
<td>Web plate</td>
<td>230 mm x 10 mm</td>
</tr>
<tr>
<td>Grade</td>
<td>S 275</td>
</tr>
</tbody>
</table>
Location: Ex2 - using 2 bolts in haunch

Portal frame apex connection (haunched)

The calculations are in accordance with BS5950-1:2000 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The factored moment and forces at the end of the rafter act on a cross-section normal to the axis of the basic rafter (unhaunched) member.

Factored moment \( M_a = 283 \text{ kNm} \)
Factored shear force \( V_a = 150 \text{ kN} \)
Axial force (+ve compression) \( N_a = 110 \text{ kN} \)
Pitch of rafter (in degrees) \( \Theta = 13 \)

Grade details

Rafter details - parent section

406 x 178 x 60 UB.

Haunch plate details

Flange width \( B_h = 200 \text{ mm} \)
Flange thickness \( T_h = 15 \text{ mm} \)
Web thickness \( t_h = 10 \text{ mm} \)

Bolt details

Diameter of bolts \( b_d = 24 \text{ mm} \)
End plate details

Thickness of plate  tp=20 mm
Number of bolts in haunch  bnf=2
Total number of bolts  bn=8
Number of shear bolts  ns=2
Distance to first row of bolts  pe=100 mm
Distance between tension bolts  pl=130 mm
Distance to haunch bolts  x=70 mm
Pitch of bolts  p2=90 mm
Distance to flange from edge(s)  ex=15 mm
Bolt cross-centres  g=90 mm
Width of end plate  bp=200 mm
Assumed web fillet weld size  sww=8 mm
Assumed flange fillet weld size  swf=12 mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

Top row of bolts  Pr1=367.65 kN
Row 2  Pr2=362.14 kN
Row 3  Pr3=280.32 kN
Selected fillet weld size  swt=13 mm

Selected fillet weld size  sfw=8 mm

CONNECTION SUMMARY - all dimensions in diagram are in mm
Steel design to BS5950-1:2000 and Eurocode 3
Portal apex connection

Number of bolts 8
Bolt diameter 24 mm
Bolt grade 8.8
Welds
Tension flange fillet weld 13 mm
Web fillet weld 8 mm
Compression flange fillet weld 12 mm
Beam / Rafter section 406 x 178 x 60 UB
Haunch Flange plate 200 mm x 15 mm
Web plate 140 mm x 10 mm
Grade S 275
**Location:** Ex1 - using 4 bolts in haunch

**Portal frame apex connection (haunched)**

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The factored moment and forces at the end of the rafter act on a cross-section normal to the axis of the basic rafter (unhaunched) member.

![Diagram of portal frame apex connection](image)

Factored moment $MyEd'=387$ kNm
Factored shear force $VEd'=256$ kN
Axial force (+ve compression) $NEd'=150$ kN
Pitch of rafter $\theta=10^\circ$

**Grade details**

**Rafter section properties details - parent section**

457 x 191 x 74 UKB

**Haunch plate details**

- Flange width $bh=200$ mm
- Flange thickness $tfh=15$ mm
- Web thickness $twh=10$ mm

**Bolt details**

- Diameter of bolts $db=24$ mm
- Tensile resistance (EC3 Part 1-8) $FtRd=k2*fub*At/(gamM2*10^3)=203.33$ kN
End plate details

- Thickness of plate: $tp = 20 \text{ mm}$
- Number of bolts in haunch: $bnf = 4$
- Total number of bolts: $bn = 12$
- Number of shear bolts: $ns = 4$
- Distance to first row of bolts: $e1 = 100 \text{ mm}$
- Distance between tension bolts: $p1 = 130 \text{ mm}$
- Distance to haunch bolts: $x = 70 \text{ mm}$
- Pitch of bolts: $p2 = 90 \text{ mm}$
- Bolt cross-centres: $p3 = 90 \text{ mm}$
- Width of end plate: $bp = 200 \text{ mm}$
- Assumed web fillet weld size: $sww = 8 \text{ mm}$
- Assumed flange fillet weld size: $swf = 12 \text{ mm}$

SUMMARY OF BOLT POTENTIAL RESISTANCES

- Top row of bolts: $Pr1 = 370.01 \text{ kN}$
- Row 2: $Pr2 = 290.68 \text{ kN}$
- Row 3: $Pr3 = 369.79 \text{ kN}$
- Row 4: $Pr4 = 287.81 \text{ kN}$
- Selected fillet weld size: $swt = 12 \text{ mm}$
- Selected fillet weld size: $sfw = 8 \text{ mm}$

REVERSED MOMENT CONDITION

- Factored moment: $MrEd = 123 \text{ kNm}$
- Factored shear force: $VrEd = 103 \text{ kN}$
- Axial force (+ve compression): $NrEd = 45 \text{ kN}$
CONNECTION SUMMARY - all dimensions in diagram are in mm

Number of bolts 12 mm
Bolt diameter 24 mm
Bolt grade 8.8
Welds  
- Tension flange fillet weld 12 mm
- Web fillet weld 8 mm
- Compression flange fillet weld 12 mm
Beam / Rafter section 457 x 191 x 74 UKB
Haunch Flange plate 200 mm x 15 mm
Web plate 230 mm x 10 mm
Grade S 275
Location: Ex2 - using 2 bolts in haunch

Portal frame apex connection (haunched)

Calculations are in accordance with EC3 Parts 1-1, 1-5, 1-8 and follow the procedure given in the SCI publication entitled 'Joints in Steel Construction Moment Connections'. The factored moment and forces at the end of the rafter act on a cross-section normal to the axis of the basic rafter (unhaunched) member.

Factored moment $MyEd' = 283$ kNm
Factored shear force $VEd' = 150$ kN
Axial force (+ve compression) $NEd' = 110$ kN
Pitch of rafter $\Theta = 13^\circ$

Grade details

Rafter section properties details - parent section

406 x 178 x 60 UKB

Haunch plate details

Flange width $bh = 200$ mm
Flange thickness $tfh = 15$ mm
Web thickness $twh = 10$ mm

Bolt details

Diameter of bolts $db = 24$ mm
Tensile resistance (EC3 Part 1-8) $FtRd = k2 * fub * At / (\gamma M2 * 10^3) = 203.33$ kN
End plate details

- Thickness of plate: $tp=20$ mm
- Number of bolts in haunch: $bnf=2$
- Total number of bolts: $bn=8$
- Number of shear bolts: $ns=2$
- Distance to first row of bolts: $e1=100$ mm
- Distance between tension bolts: $p1=130$ mm
- Distance to haunch bolts: $x=70$ mm
- Pitch of bolts: $p2=90$ mm
- Bolt cross-centres: $p3=90$ mm
- Width of end plate: $bp=200$ mm
- Assumed web fillet weld size: $sww=8$ mm
- Assumed flange fillet weld size: $swf=12$ mm

SUMMARY OF BOLT POTENTIAL RESISTANCES

- Top row of bolts: $Pr1=373.92$ kN
- Row 2: $Pr2=368.33$ kN
- Row 3: $Pr3=286.51$ kN
- Selected fillet weld size: $swt=13$ mm
- Selected fillet weld size: $sfw=8$ mm

CONNECTION SUMMARY - all dimensions in diagram are in mm

- Number of bolts: 8 mm
- Bolt diameter: 24 mm
- Bolt grade: 8.8
- Welds:
  - Tension flange fillet weld: 13 mm
  - Web fillet weld: 8 mm
  - Compression flange fillet weld: 12 mm

Beam / Rafter section: 406 x 178 x 60 UKB
Haunch Flange plate: 200 mm x 15 mm
Web plate: 140 mm x 10 mm
Grade S 275
General design information


The shear stud resistance check will be as per EC4 Part 1-1. The shear force is transferred to the fin plate by bolts. The cast-in plate and its welded connection to the fin plate will resist the eccentricity moment due to the offset of the bolt group from the face of the plate. The standard arrangement for fin plates in the Green Book (i.e., SCI publication P358) is 10 mm thick fin plate using S275 material, with two 8 mm FW. This will eliminate the possibility of weld failure. The face of the cast-in plate is normally inset to allow for the sliding form.

Both the cast-in plate and fin plate will need site-applied corrosion protection system if corrosion protection of steelwork is specified. Furthermore parts of the connection plate (e.g., the fin plate) will also require fire protection as for the connecting beam.

Design shear (+ve value) \( V_{Ed}=570 \) kN
Design tying force (+ve value) \( F_{tEd}=500 \) kN

Fin plate arrangement

<table>
<thead>
<tr>
<th>No. of rows of M20 bolts</th>
<th>nbr=7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fin plate depth</td>
<td>hfp=500 mm</td>
</tr>
<tr>
<td>Fin plate width</td>
<td>bfp=140 mm</td>
</tr>
</tbody>
</table>

Choose initial shear stud arrangement

<table>
<thead>
<tr>
<th>Stud diameter</th>
<th>ds=25 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stud height</td>
<td>hs=100 mm</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>fck=40 N/mm²</td>
</tr>
<tr>
<td>Spacing of studs (vertically)</td>
<td>d1=125 mm</td>
</tr>
<tr>
<td>Spacing of studs (horizontally)</td>
<td>d2=130 mm</td>
</tr>
<tr>
<td>Edge distance (vertically)</td>
<td>e1=155 mm</td>
</tr>
<tr>
<td>Edge distance (horizontally)</td>
<td>e2=60 mm</td>
</tr>
</tbody>
</table>
Initial reinforcement arrangement

Reinforcement welded to cast-in plate for bending and tying resistance will need to be provided.
Bar diameter (top & bottom bars)  dr=25 mm
Total number of bars              nb=4
Tension resistance (bar group)    PtRd=TABLE 3 for nb=4, dr=25 =982 kN
Design tying force                FtEd=500 kN
As PtRd ≥ FtEd ( 982 kN ≥ 500 kN ), the tension resistance exceeds the design tying force.

Confirm the shear stud arrangement - EC4 Part 1-1 Cl. 6.6.3.1

Governing group shear resistance  PRd=PRd1=848 kN
As PRd ≥ VEd ( 848 kN ≥ 570 kN ), the shear resistance exceeds the design shear and hence studs provided are satisfactory.
Horizontal deviation              ecc=35 mm
Angle β (rotational deviation)    beta=5.71°
Maximum shear in corner stud      Vmax2=SQR((VEd/ns+V2)^2+V1^2)=116 kN
Shear resistance per stud         PRd1=PRd/ns=141 kN
As Vmax2 ≤ PRd1 ( 116 kN ≤ 141 kN ), corner stud shear OK.

Reinforcement arrangement and bar size

(A) Eccentricity moment on the cast-in plate
Maximum eccentricity              ep=150 mm
Design moment due to eccentricity Md=VEd*ep/1000=85.5 kNm

(B) Vertical lever arm for tension - compression couple
Design tension force              T=Md/l=202 kN
Design compression force          C=T2=202 kN

(C) Confirm the bar size
Top bar spacing (laterally)       bsp=155 mm
As T ≤ FtRd ( 202 kN ≤ 427 kN ), bars provided are OK.
As FtEd' ≤ FtRd1 ( 147 kN ≤ 213 kN ), maximum tensile force due to eccentricity of loading is less than the bar resistance.

Bending of the cast-in plate

The following calculations will check the resistance of the cast-in plate in bending for the eccentricity moment tension. The resistance model for end-plate from BS EN 1993-1-8 Clause 6.2.4 will be used.

(A) Check Mode 2 failure: bolt failure with yielding of the flange. In this case the bars are substituted for bolts.
Steel design to Eurocode 3  
Made by: IFB  
Date: 02/12/19  
Ref No: SC495 EC

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Plan on cast-in plate  
ecc is the eccentricity to the centre line (CL) of the cast-in plate.  
This was defined earlier as 35 mm.  
fin = fin plate

**Equation 1:**  
\[ M_c + R_a((n+(m+ecc))) - aF_t(m+ecc) = 0 \]

**Equation 2:**  
\[ M_c + R_b((n+(m-ecc))) - F_t(m-ecc) = 0 \]

**Bar end distance**  
ex = 70 mm

**Cast-in plate thickness**  
\( tp = 25 \text{ mm} \)

**Force**  
\( F_t = \frac{F_tR_d}{n_t} = 213 \text{ kN} \)

**Moment**  
\( M_c = M_{plR_d} = 12.5 \text{ kNm} \)

**From equation 2**  
\[ R_b = \frac{(F_t \times (m-ecc)-M_c \times 1E3)}{(n+(m-ecc))} = -74.6 \text{ kN} \]

**Design tension force**  
\( T = \frac{M_d}{l} = 202 \text{ kN} \)

As \( T \leq F_T \ (202 \text{ kN} \leq 285 \text{ kN}) \), Mode 2 failure check OK.

**Bending moment at fin plate face**  
\( M = F_t \times (m-ecc)/1E3 = 6.64 \text{ kNm} \)

As \( M \leq M_{plR_d} \ (6.64 \text{ kNm} \leq 12.5 \text{ kNm}) \), bending moment at the face of the fin plate is less than the cast-in plate moment resistance.

(B) Check Mode 1 failure: consider a vertical centre line through the cast-in plate and fin plate.

**Total resistance force**  
\( F_{RdT1} = F_{RdT1A} + F_{RdT1B} = 1197 \text{ kN} \)

As \( T \leq F_{RdT1} \ (202 \text{ kN} \leq 1197 \text{ kN}) \), Mode 1 failure check OK.
CAST-IN PLATE      Depth     560 mm  
DETAILS            Breadth   250 mm     
Thickness 25 mm

(C) Check the cast-in plate under the tying load case
Tension resistance    FT=Ft*(a*n+(n+bsp))/com1=328 kN
As FT ≥ 2*FtEd/nb ( 328 kN ≥ 250 kN ), tension resistance exceeds
the design tying force.
As M ≤ MplRd ( 7.63 kNm ≤ 15.5 kNm ), bending moment in cast-in
plate is less than the cast-in plate moment resistance. Hence the
25 mm thick cast-in plate is acceptable for the tying load case.

Resistance of the combined cast-in plate and fin plate

The resistance of the combined cast-in plate and fin plate to bending
in the vertical plane will be considered. The fin plate will need to
be checked in bending at the face of the cast-in plate. The nominal
eccentricity moment at the back of the plate calculated previously for
determining the bar sizes is Md=85.5 kNm. The latter will be reduced
by the assumed plate thickness.
As M ≤ MRd ( 71.7 kNm ≤ 115 kNm ), the reduced bending moment is
less than the fin plate moment resistance.

(A) Tying load case
The fin plate and cast-in plate acting as a Tee section spans between
the top and bottom tie bars to resist tie forces. The span of the Tee
section is the distance

Consider the cast-in plate and fin plate
acting as a Tee section.

Assumed Tee section flange width b=100 mm
Length of Tee stem ls=100 mm
Span of Tee section L=0.44 m

(B) Plastic modulus
Plastic modulus of Tee section Z=z1+z2+z3=75625 mm³
Moment resistance MRd=fufp*Z/(gammu*1E6)=28.2 kNm
As M ≤ MRd ( 27.5 kNm ≤ 28.2 kNm ), the bending moment is less
than the Tee section moment resistance.

Welding reinforcement to cast-in plate

Bar strength          Ft=245 kN
Partial factor (cast-in plate) gamm=1.1
Weld coefficient betaw=0.9
Design strength of weld

\[ fvwd = \frac{f_{up}}{\beta \omega \gamma(3)^{0.5}} \]

\[ = 274 \text{ N/mm}^2 \]

Two welds round the bar will need to be provided as shown in diagram.

Throat length required

\[ a = \frac{F_t \cdot 1E3}{2 \cdot fvwd \cdot PI \cdot d_r} = 5.7 \text{ mm} \]

Weld leg length (reinf'ment)

\[ sr = \frac{a}{0.7071} = 8.06 \text{ mm} \]

Weld leg length to be adopted

\[ sr = 10 \text{ mm} \]

**PUNCHING SHEAR**

The shear stress expression below is as per Clause 6.4.4(2).

\[ v_{Ed} = \frac{V_{Ed}}{u_i \cdot d} + \frac{k \cdot M_{Ed}}{W_i \cdot d} \]

Consider the bending component only for the eccentricity moment and the direct component only for the tie force as these are separate load cases.

Definitions:

- loaded area refers to the cross-sectional area of the stiff bearing
- load perimeter refers to the stiff bearing plate perimeter
- load face refers to the stiff bearing plate face
- face of the load refers to the face of the stiff bearing plate
- loaded area dimensions refers to stiff bearing area dimensions
(A) **Shear stress on plate perimeter due to eccentricity moment**

First link perimeter at ≤ 0.5d from face of loaded area.

Link perimeters at ≤ 0.75d apart.

\[
\text{cx} \text{ and } \text{cy} \text{ are the loaded area dimensions.}
\]

Basic control perimeter is at 2d from the face of the loaded area.

\[
\text{Sr} = \text{spacing of links in the radial direction.}
\]

Average effective depth \( d = 170 \text{ mm} \)
Dimension of loaded area \( \text{cx} = 250 \text{ mm} \)
Dimension of loaded area \( \text{cy} = 560 \text{ mm} \)

As \( v_{Ed} \leq v_{Rdmax} \) (1.227 N/mm² ≤ 5.712 N/mm²), shear stress at load perimeter is within the maximum punching shear resistance.

Shear stress on the basic control perimeter \( u_1 \) due to bending

Determine the value of the shear resistance assuming there is no shear reinforcement as per EC2 Clause 6.4.4(1).

Reinforcement ratio in x-dir \( p_{1x} = 0.004 \)
Reinforcement ratio in y-dir \( p_{1y} = 0.0023 \)
Design punching shear resistance \( v_{Rdc} = (0.18/\gamma_{mc}) \cdot k \cdot (100 \cdot p_{1x} \cdot f_{ck})^{0.333} = 0.551 \text{ N/mm²} \)
Min punching shear resistance \( v_{min} = 0.035 \cdot k \cdot 1.5 \cdot f_{ck} \cdot 0.5 = 0.6261 \text{ N/mm²} \)

Hence, the minimum design punching shear resistance governs.

Design punching shear resistance \( v_{Rdc} = 0.6261 \text{ N/mm²} \)
As \( v_{Ed} \leq v_{Rdc} \) \( 0.2384 \text{ N/mm²} \leq 0.6261 \text{ N/mm²} \), punching shear reinforcement is not necessary for this load case.

(B) **Shear stress on plate perimeter due to tie force**

An estimate of the shear reinforcement required for tying action will be made, assuming the tie force is carried into the wall through the cast-in plate (no bending checks on the wall will be carried out).

The tie force is a direct force \( V_{Ed}'' = F_{tEd} = 500 \text{ kN} \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.2 \) (accidental load case)
Load perimeter \( u_0 = 2 \cdot (\text{cx} + \text{cy}) = 1620 \text{ mm} \)
Shear stress at load perimeter \( v_{Ed} = V_{Ed}'' \cdot 1000 / (u_0 \cdot d) = 1.816 \text{ N/mm²} \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5 \cdot v \cdot f_{cd} = 7.14 \text{ N/mm²} \)
As \( v_{Ed} \leq v_{Rdmax} \) \( 1.816 \text{ N/mm²} \leq 7.14 \text{ N/mm²} \), shear stress at load perimeter is within the maximum punching shear resistance.
Check shear stress at control perimeter $u_1$ at 2d from load face

Basic control perimeter at 2d

$$u_1 = 2(c_x + c_y) + 4 \pi d = 3756 \text{ mm}$$

Design punching shear resistance

$$v_{Rdc} = (0.18 / \gamma_{mc}) \cdot k \cdot (100 \cdot p_1 \cdot f_{ck})^{0.333} = 0.6888 \text{ kN/mm}$$

Min punching shear resistance

$$v_{min} = 0.035 \cdot k^{1.5} \cdot f_{ck}^{0.5} = 0.6261 \text{ kN/mm}$$

As $v_{Ed} > v_{Rdc}$ ($0.783 \text{ kN/mm}^2 > 0.6888 \text{ kN/mm}^2$), punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required

Outer control perimeter

$$U_{out} = V_{Ed} \cdot 1000 / (d \cdot v_{Rdc}) = 4270 \text{ mm}$$

Radius to $U_{out}$

$$r_{out} = (U_{out} - lcf) / (2 \pi) = 421.8 \text{ mm}$$

The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter $U_{out}$. Radius to outer perimeter of shear reinforcement $Outper = r_{out} - 1.5d = 166.8 \text{ mm from load face}$.

Shear links on basic control perimeter $u_1$

The partial safety factor for reinforcement will be taken as $\gamma_{ms} = 1.15$.

Spac.inside 2d control perimeter

$$s_t = 255 \text{ mm}$$

Spac.outside control perimeter $u_1$

$$s_t' = 340 \text{ mm}$$

Char yield strength of reinft.

$$f_{yk} = 500 \text{ N/mm}^2$$

Diameter of shear reinforcement

$$d_{al} = 6 \text{ mm}$$

Spacing of links to be used

$$s_{pc} = 255 \text{ mm}$$

Use minimum H 6 (28.27 mm$^2$) legs of links @ 255 mm c/c around perimeter $u_1$ (i.e. in the tangential direction).

NOTE:

Punching shear links on perimeter 3 (i.e. at 2d from the load face) shown below are optional and could be omitted. This is because the radius $Outper < 1.25d$ (166.8 mm < 212.5 mm).

Summary of punching shear links

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.

RC wall

<table>
<thead>
<tr>
<th>3</th>
<th>2</th>
<th>1</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Connecting beam</td>
<td>1,2,3 above indicate link perimeters</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

basic control perimeter $u_1$

radial direction
Links on perimeter 1:
Use minimum H 6 (28.27 mm$^2$) legs of links @ 210 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 6 (28.27 mm$^2$) legs of links @ 255 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 6 (28.27 mm$^2$) legs of links @ 255 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

DESIGN SUMMARY - nominally pinned joint

Design loads:
- Design shear: 570 kN
- Shear resistance: 848.2 kN (stud group)
- Shear resistance: 141.4 kN (per stud)
- Design tying force: 500 kN
- Tension resistance: 982 kN (bar group)
- Tension resistance: 213.4 kN (per bar)

Grade/strength:
- Concrete: 40 N/mm$^2$
- Fin plate: S 275
- Cast-in plate: S 355
- M20 bolts: 8.8

FP = fin plate
CP = cast-in plate
tb = top bars
bb = bottom bars
ss = shear stud

Fin plate:
- No. of bolts: 7
- Plate depth: 500 mm
- Plate width: 140 mm
- Plate thickness: 10 mm

Cast-in plate:
- Plate depth: 560 mm
- Plate width: 250 mm
- Plate thickness: 25 mm
Weld leg length: Fin to cast-in plate 2 No/8 mm FW need to be provided for connecting the fin plate to cast-in plate to eliminate the possibility of weld failure.

Stud welding: It is assumed that studs will be welded using electric arc stud-welding equipment. See SCI publication P300 for further information.

Weld leg length: Reinforcement 10 mm FW with two welds round the bar as shown in the diagram. PPFW is a partial penetration fillet weld (FW) in countersunk hole.

\[
\text{dr}=25 \text{ mm (bar diameter)} \\
\text{tp}=25 \text{ mm (plate thickness)}
\]

Shear studs: No. and diameter 6 No/25 mm
Height 100 mm
Spacing vertically 125 mm
Spacing horizontally 130 mm
Vertical edge distance 155 mm
Horiz. edge distance 60 mm

Shear studs:
2x3 shear stud array
Total 6 studs
All dimensions in mm

Reinforcement: Top bars 2 No/25 mm diameter
Bottom bars 2 No/25 mm diameter
Top bar spacing (bsp) 155 mm (laterally)
Bar edge distance (n') 47.5 mm (top & bottom bars)
Bar end distance (ex) 70 mm (top & bottom bars)
Bend bars mandrel 263.3 mm diameter
Bar anchorage length 1052 mm
Punching shear links are required, refer to the punching shear section of the calculations for diameter & spacing of links.

Punching shear links on perimeter 3 are optional.
General design information


The shear stud resistance check will be as per EC4 Part 1-1. The shear force is transferred to the fin plate by bolts. The cast-in plate and its welded connection to the fin plate will resist the eccentricity moment due to the offset of the bolt group from the face of the plate. The standard arrangement for fin plates in the Green Book (i.e. SCI publication P358) is 10 mm thick fin plate using S275 material, with two 8 mm FW. This will eliminate the possibility of weld failure. The face of the cast-in plate is normally inset to allow for the sliding form.

Both the cast-in plate and fin plate will need site-applied corrosion protection system if corrosion protection of steelwork is specified. Furthermore parts of the connection plate (e.g. the fin plate) will also require fire protection as for the connecting beam.

Design shear (+ve value) $V_{Ed}=570\ kN$
Design tying force (+ve value) $F_{tEd}=500\ kN$

Fin plate arrangement

<table>
<thead>
<tr>
<th>No. of rows of M20 bolts</th>
<th>nbr=7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fin plate depth</td>
<td>hfp=500 mm</td>
</tr>
<tr>
<td>Fin plate width</td>
<td>bfp=140 mm</td>
</tr>
</tbody>
</table>

Choose initial shear stud arrangement

<table>
<thead>
<tr>
<th>Stud diameter</th>
<th>ds=19 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stud height</td>
<td>hs=100 mm</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>fck=40 N/mm²</td>
</tr>
<tr>
<td>Spacing of studs (vertically)</td>
<td>d1=106 mm</td>
</tr>
<tr>
<td>Spacing of studs (horizontally)</td>
<td>d2=75 mm</td>
</tr>
<tr>
<td>Edge distance (vertically)</td>
<td>e1=121 mm</td>
</tr>
<tr>
<td>Edge distance (horizontally)</td>
<td>e2=50 mm</td>
</tr>
</tbody>
</table>
Initial reinforcement arrangement

Reinforcement welded to cast-in plate for bending and tying resistance will need to be provided.
Bar diameter (top & bottom bars) \( dr=25 \text{ mm} \)
Total number of bars \( nb=4 \)
Tension resistance (bar group) \( PtRd=\text{TABLE 3 for nb=4, } dr=25=982 \text{ kN} \)
Design tying force \( FtEd=500 \text{ kN} \)
As \( PtRd \geq FtEd \) (982 kN ≥ 500 kN), the tension resistance exceeds the design tying force.

Confirm the shear stud arrangement - EC4 Part 1-1 Cl. 6.6.3.1

Governing group shear resistance \( PRd=PRd1=980 \text{ kN} \)
As \( PRd \geq VEd \) (980 kN ≥ 570 kN), the shear resistance exceeds the design shear and hence studs provided are satisfactory.
Horizontal deviation \( \text{ecc}=35 \text{ mm} \)
Angle \( \beta \) (rotational deviation) \( \beta=5.71^\circ \)
Maximum shear in corner stud \( Vmax2=\sqrt{(VEd/ns+V2)^2+V1^2)}=57.7 \text{ kN} \)
Shear resistance per stud \( PRd1=PRd/ns=81.7 \text{ kN} \)
As \( Vmax2 \leq PRd1 \) (57.7 kN ≤ 81.7 kN), corner stud shear OK.

Reinforcement arrangement and bar size

(A) Eccentricity moment on the cast-in plate
Maximum eccentricity \( ep=150 \text{ mm} \)
Design moment due to eccentricity \( Md=VEd*ep/1000=85.5 \text{ kNm} \)

(B) Vertical lever arm for tension - compression couple
Design tension force \( T=Md/l=202 \text{ kN} \)
Design compression force \( C=T2=202 \text{ kN} \)

(C) Confirm the bar size
Top bar spacing (laterally) \( bsp=150 \text{ mm} \)
As \( T \leq FtRd \) (202 kN ≤ 427 kN), bars provided are OK.
As \( FtEd' \leq FtRd1 \) (148 kN ≤ 213 kN), maximum tensile force due to eccentricity of loading is less than the bar resistance.

Bending of the cast-in plate

The following calculations will check the resistance of the cast-in plate in bending for the eccentricity moment tension. The resistance model for end-plate from BS EN 1993-1-8 Clause 6.2.4 will be used.

(A) Check Mode 2 failure: bolt failure with yielding of the flange. In this case the bars are substituted for bolts.
Steel design to Eurocode 3

Cast-in plate to connect structural steel beam to

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Plan on cast-in plate

ecc is the eccentricity to the centre line (CL) of the cast-in plate.

This was defined earlier as 35 mm.

fin = fin plate

---

Equation 1: \[ M_c + R_a((n+(m+ecc))) - aF_t(m+ecc) = 0 \]

Equation 2: \[ M_c + R_b((n+(m-ecc))) - F_t(m-ecc) = 0 \]

---

Bar end distance \( ex = 70 \text{ mm} \)

Cast-in plate thickness \( tp = 25 \text{ mm} \)

Force \( F_t = F_{tRd}/ntb = 213 \text{ kN} \)

Moment \( M_c = M_{plRd} = 12.3 \text{ kNm} \)

From equation 2

\[ R_b = (F_t^*(m-ecc)-M_c*1E3)/(n+(m-ecc)) 
\]

\[ = -79 \text{ kN} \]

---

Tension resistance

\[ F_t^* = F_{tRd}/(a*n+(n+bsp))/(n+(bsp/2+ecc)) \]

\[ = 283 \text{ kN} \]

Design tension force \( T = M_d/l = 202 \text{ kN} \)

As \( T \leq F_t \ (202 \text{ kN} \leq 283 \text{ kN}) \), Mode 2 failure check OK.

Bending moment at fin plate face

\[ M = F_t^* (m-ecc) / 1E3 = 6.1 \text{ kNm} \]

As \( M \leq M_{plRd} \ (6.1 \text{ kNm} \leq 12.3 \text{ kNm}) \), bending moment at the face of the fin plate is less than the cast-in plate moment resistance.

(B) Check Mode 1 failure: consider a vertical centre line through the cast-in plate and fin plate.

---

Total resistance force

\[ F_{RdT1} = F_{RdT1A} + F_{RdT1B} = 1274 \text{ kN} \]

As \( T \leq F_{RdT1} \ (202 \text{ kN} \leq 1274 \text{ kN}) \), Mode 1 failure check OK.
CAST-IN PLATE
DETAILS
Depth  560 mm
Breadth  250 mm
Thickness  25 mm

(C) Check the cast-in plate under the tying load case
Tension resistance FT=Ft*(a*n+(n+bsp))/com1=326 kN
As FT ≥ 2*FtEd/nb ( 326 kN ≥ 250 kN ), tension resistance exceeds
the design tying force.
As M ≤ MplRd ( 7.02 kNm ≤ 15.3 kNm ), bending moment in cast-in
plate is less than the cast-in plate moment resistance. Hence the
25 mm thick cast-in plate is acceptable for the tying load case.

Resistance of the combined cast-in plate and fin plate

The resistance of the combined cast-in plate and fin plate to bending
in the vertical plane will be considered. The fin plate will need to
be checked in bending at the face of the cast-in plate. The nominal
eccentricity moment at the back of the plate calculated previously for
determining the bar sizes is Md=85.5 kNm. The latter will be reduced
by the assumed plate thickness.
As M ≤ MRd ( 71.3 kNm ≤ 115 kNm ), the reduced bending moment is
less than the fin plate moment resistance.

(A) Tying load case
The fin plate and cast-in plate acting as a Tee section spans between
the top and bottom tie bars to resist tie forces. The span of the Tee
section is the distance
\[ b \]
\[ \text{between the top & bottom tie bars.} \]
\[ ls \]
\[ \text{= length of stem} \]
\[ y \]
\[ \text{= depth to NA} \]
\[ tp \]
\[ =25 \text{ mm} \]
\[ tfp=10 \text{ mm} \]

Consider the cast-in plate and fin plate acting as a Tee section.

Assumed Tee section flange width b=100 mm
Length of Tee stem ls=100 mm
Span of Tee section L=0.42 m

(B) Plastic modulus
Plastic modulus of Tee section \[ Z=z1+z2+z3=75625 \text{ mm}^3 \]
Moment resistance \[ MRd=fufp*Z/(gammu*1E6)=28.2 \text{ kN} \]
As M ≤ MRd ( 26.3 kNm ≤ 28.2 kNm ), the bending moment is less
than the Tee section moment resistance.

Welding reinforcement to cast-in plate

Bar strength Ft=245 kN
Partial factor (cast-in plate) gamm=1.1
Weld coefficient betaw=0.9
Design strength of weld \[ fvwd = \frac{f_{up}}{\beta_{w} \gamma_{w} (3)^{0.5}} \]
\[ = 274 \text{ N/mm}^2 \]

Two welds round the bar will need to be provided as shown in diagram.

Throat length required \[ a = \frac{F_{t} \cdot 1E3}{2 \cdot fvwd \cdot \pi \cdot dr} = 5.7 \text{ mm} \]
Weld leg length (reinf'ment) \[ sr = \frac{a}{0.7071} = 8.06 \text{ mm} \]
Weld leg length to be adopted \[ sr = 10 \text{ mm} \]

**PUNCHING SHEAR**

The shear stress expression below is as per Clause 6.4.4(2).

\[ \nu_{Ed} = \frac{V_{Ed}}{u_{i} \cdot d} + \frac{k \cdot M_{Ed}}{W_{i} \cdot d} \]

Consider the bending component only for the eccentricity moment and the direct component only for the tie force as these are separate load cases.

Definitions:
- loaded area refers to the cross-sectional area of the stiff bearing
- load perimeter refers to the stiff bearing plate perimeter
- load face refers to the stiff bearing plate face
- face of the load refers to the face of the stiff bearing plate
- loaded area dimensions refers to stiff bearing area dimensions
(A) Shear stress on plate perimeter due to eccentricity moment

\[ \text{cx} \text{ and } \text{cy} \text{ are the loaded area dimensions.} \]

First link perimeter at \( \leq 0.5d \) from face of loaded area.

Link perimeters at \( \leq 0.75d \) apart.

\[ \leq 1.5d \quad \leq 0.5d \]

Average effective depth \( d = 170 \text{ mm} \)
Dimension of loaded area \( cx = 250 \text{ mm} \)
Dimension of loaded area \( cy = 560 \text{ mm} \)
As \( v_{Ed} \leq v_{Rdmax} (1.227 \text{ N/mm}^2 \leq 5.712 \text{ N/mm}^2) \), shear stress at load perimeter is within the maximum punching shear resistance.

Shear stress on the basic control perimeter \( u_1 \) due to bending

Determine the value of the shear resistance assuming there is no shear reinforcement as per EC2 Clause 6.4.4(1).

Reinforcement ratio in x-dir \( p_{lx} = 0.004 \)
Reinforcement ratio in y-dir \( p_{ly} = 0.0023 \)
Design punching shear resistance \( v_{Rdc} = (0.18/\gamma_{mc}) \ast k \ast (100 \ast p_1 \ast f_{ck})^{0.333} = 0.551 \text{ N/mm}^2 \)

Min punching shear resistance \( v_{min} = 0.035 \ast k \ast 1.5 \ast f_{ck} \ast 0.5 = 0.6261 \text{ N/mm}^2 \)

Hence, the minimum design punching shear resistance governs.

Design punching shear resistance \( v_{Rdc} = 0.6261 \text{ N/mm}^2 \)
As \( v_{Ed} \leq v_{Rdc} (0.2384 \text{ N/mm}^2 \leq 0.6261 \text{ N/mm}^2) \), punching shear reinforcement is not necessary for this load case.

(B) Shear stress on plate perimeter due to tie force

An estimate of the shear reinforcement required for tying action will be made, assuming the tie force is carried into the wall through the cast-in plate (no bending checks on the wall will be carried out).

The tie force is a direct force \( V_{Ed}' = F_{tEd} = 500 \text{ kN} \)
Partial safety factor (concrete) \( \gamma_{mc} = 1.2 \) (accidental load case)
Load perimeter \( u_0 = 2 \ast (cx + cy) = 1620 \text{ mm} \)
Shear stress at load perimeter \( v_{Ed} = V_{Ed}' \ast 1000 / (u_0 \ast d) = 1.816 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5 \ast v \ast f_{cd} = 7.14 \text{ N/mm}^2 \)
As \( v_{Ed} \leq v_{Rdmax} (1.816 \text{ N/mm}^2 \leq 7.14 \text{ N/mm}^2) \), shear stress at load perimeter is within the maximum punching shear resistance.
Check shear stress at control perimeter $u_1$ at 2d from load face

Basic control perimeter at 2d $u_1=2*(cx+cy)+4*\pi*d=3756$ mm

Design punching shear resistance $V_Rdc=(0.18/\gamma_c)*k^*(100*p_1^*f_{ck})^0.333=0.6888$ N/mm²

Min punching shear resistance $v_{min}=0.035*k^1.5*f_{ck}^0.5=0.6261$ N/mm²

As $V_{Ed} > V_Rdc$, punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required

Outer control perimeter $U_{out}=V_{Ed}^'*1000/(d^*V_{Rdc})=4270$ mm

Radius to $U_{out}$ $r_{out}=(U_{out}-lcf)/(2*\pi)=421.8$ mm

The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter $U_{out}$. Radius to outer perimeter of shear reinforcement $Outper=r_{out}-1.5*d=166.8$ mm from load face

Shear links on basic control perimeter $u_1$

The partial safety factor for reinforcement will be taken as $\gamma_m=1.15$.

Spacing inside 2d control perimeter $St=255$ mm

Spacing outside control perimeter $u_1$ $St'=340$ mm

Char yield strength of reinft. $f_{yk}=500$ N/mm²

Diameter of shear reinforcement $dia=6$ mm

Spacing of links to be used $spac=255$ mm

Use minimum H 6 (28.27 mm²) legs of links @ 255 mm c/c around perimeter $u_1$ (i.e. in the tangential direction).

NOTE:
Punching shear links on perimeter 3 (i.e. at 2d from the load face) shown below are optional and could be omitted. This is because the radius $Outper < 1.25*d$ (166.8 mm < 212.5 mm).

Summary of punching shear links

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.
Links on perimeter 1:
Use minimum H 6 (28.27 mm²) legs of links @ 210 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 6 (28.27 mm²) legs of links @ 255 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 6 (28.27 mm²) legs of links @ 255 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

**DESIGN SUMMARY - nominally pinned joint**

<table>
<thead>
<tr>
<th>Design loads:</th>
<th>Design shear 570 kN</th>
<th>0</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>979.9 kN (stud group)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear resistance</td>
<td>81.66 kN (per stud)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design tying force</td>
<td>500 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension resistance</td>
<td>982 kN (bar group)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension resistance</td>
<td>213.4 kN (per bar)</td>
<td></td>
<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Grade/strength:</th>
<th>Concrete 40 N/mm²</th>
<th>0</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fin plate</td>
<td>S 275</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cast-in plate</td>
<td>S 355</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M20 bolts</td>
<td>8.8</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fin plate:
- FP = fin plate
- CP = cast-in plate
- tb = top bars
- bb = bottom bars
- ss = shear stud

Cast-in plate:
- Plate depth 560 mm
- Plate width 250 mm
- Plate thickness 25 mm

RC wall

SCALE 5.48 Office 1007 Proforma 495
Weld leg length: Fin to cast-in plate
2 No/8 mm FW need to be provided for connecting the fin plate to cast-in plate to eliminate the possibility of weld failure.

Stud welding: It is assumed that studs will be welded using electric arc stud-welding equipment. See SCI publication P300 for further information.

Weld leg length: Reinforcement
10 mm FW with two welds round the bar as shown in the diagram. PPFW is a partial penetration fillet weld (FW) in countersunk hole.

\[ \text{dr} = 25 \text{ mm (bar diameter)} \]
\[ \text{tp} = 25 \text{ mm (plate thickness)} \]

Shear studs:
- No. and diameter: 12 No/19 mm
- Height: 100 mm
- Spacing vertically: 106 mm
- Spacing horizontally: 75 mm
- Vertical edge distance: 121 mm
- Horiz. edge distance: 50 mm

Reinforcement:
- Top bars: 2 No/25 mm diameter
- Bottom bars: 2 No/25 mm diameter
- Top bar spacing (bsp): 150 mm (laterally)
- Bar edge distance (n'): 50 mm (top & bottom bars)
- Bar end distance (ex): 70 mm (top & bottom bars)
- Bend bars mandrel: 266.8 mm diameter
- Bar anchorage length: 1052 mm
Cast-in plate with a total of 4 bars as shown
i.e. 2 top bars and 2 bottom bars.
n'=50 mm (edge distance)
bsp=150 mm (top bars)
bp=250 mm
hp=560 mm
ex=70 mm

Punching shear links are required, refer to the punching shear section of the calculations for diameter & spacing of links.
Punching shear links on perimeter 3 are optional.
Location: Ex3 - Cast-in plate design (2x3 Shear stud Array)

General design information


The shear stud resistance check will be as per EC4 Part 1-1. The shear force is transferred to the fin plate by bolts. The cast-in plate and its welded connection to the fin plate will resist the eccentricity moment due to the offset of the bolt group from the face of the plate. The standard arrangement for fin plates in the Green Book (i.e. SCI publication P358) is 10 mm thick fin plate using S275 material, with two 8 mm FW. This will eliminate the possibility of weld failure. The face of the cast-in plate is normally inset to allow for the sliding form.

Both the cast-in plate and fin plate will need site-applied corrosion protection system if corrosion protection of steelwork is specified. Furthermore parts of the connection plate (e.g. the fin plate) will also require fire protection as for the connecting beam.

Design shear (+ve value) \( V_{Ed} = 702 \) kN

Design tying force (+ve value) \( F_{tEd} = 700 \) kN

Fin plate arrangement

No. of rows of M20 bolts \( n_{br} = 7 \)

Fin plate depth \( h_{fp} = 500 \) mm

Fin plate width \( b_{fp} = 140 \) mm

Choose initial shear stud arrangement

Stud diameter \( d_{s} = 25 \) mm

Stud height \( h_{s} = 100 \) mm

Concrete strength \( f_{ck} = 40 \) N/mm²

Spacing of studs (vertically) \( d_{1} = 125 \) mm

Spacing of studs (horizontally) \( d_{2} = 130 \) mm

Edge distance (vertically) \( e_{1} = 155 \) mm

Edge distance (horizontally) \( e_{2} = 60 \) mm
**Initial reinforcement arrangement**

Reinforcement welded to cast-in plate for bending and tying resistance will need to be provided.

- **Bar diameter (top & bottom bars)**: $d_r = 25$ mm
- **Total number of bars**: $n_b = 6$
- **Tension resistance (bar group)**: $P_{tRd} = TABLE\ 3\ for\ n_b = 6,\ d_r = 25$
  
  
  = 1473 kN
- **Design tying force**: $F_{tEd} = 700$ kN

As $P_{tRd} \geq F_{tEd}$ (1473 kN $\geq$ 700 kN), the tension resistance exceeds the design tying force.

**Confirm the shear stud arrangement - EC4 Part 1-1 Cl. 6.6.3.1**

- **Governing group shear resistance**: $P_{Rd} = P_{Rd1} = 848$ kN
- **As $P_{Rd} \geq V_{Ed}$ (848 kN $\geq$ 702 kN)**, the shear resistance exceeds the design shear and hence studs provided are satisfactory.

  - **Horizontal deviation**: $e_{cc} = 35$ mm
  - **Angle $\beta$ (rotational deviation)**: $\beta = 1^\circ$
  - **Maximum shear in corner stud**: $V_{max2} = \sqrt{\left(V_{Ed}/n_s + V_2\right)^2 + V_1^2} = 140$ kN
  - **Shear resistance per stud**: $P_{Rd1} = P_{Rd}/n_s = 141$ kN

As $V_{max2} \leq P_{Rd1}$ (140 kN $\leq$ 141 kN), corner stud shear OK.

**Reinforcement arrangement and bar size**

- **(A) Eccentricity moment on the cast-in plate**
  - **Maximum eccentricity**: $e_p = 150$ mm
  - **Design moment due to eccentricity**: $M_d = V_{Ed}e_p/1000 = 105$ kNm

- **(B) Vertical lever arm for tension - compression couple**
  - **Design tension force**: $T = M_d/l = 255$ kN
  - **Design compression force**: $C = T_2 = 255$ kN

- **(C) Confirm the bar size**
  - **Top bar spacing (laterally)**: $b_{sp} = 150$ mm
  - **As $T \leq F_{tRd}$ (255 kN $\leq$ 427 kN)**, bars provided are OK.
  - **As $F_{tEd}' \leq F_{tRd1}$ (187 kN $\leq$ 213 kN)**, maximum tensile force due to eccentricity of loading is less than the bar resistance.

**Bending of the cast-in plate**

The following calculations will check the resistance of the cast-in plate in bending for the eccentricity moment tension. The resistance model for end-plate from BS EN 1993-1-8 Clause 6.2.4 will be used.

- **(A) Check Mode 2 failure: bolt failure with yielding of the flange. In this case the bars are substituted for bolts.**
Equation 1: \[ M_c + R_a((n+(m+ecc))) - aF_t(m+ecc) = 0 \]
Equation 2: \[ M_c + R_b((n+(m-ecc))) - F_t(m-ecc) = 0 \]

Bar end distance: \( ex = 70 \text{ mm} \)
Cast-in plate thickness: \( tp = 25 \text{ mm} \)
Force: \( F_t = F_{tRd}/ntb = 213 \text{ kN} \)
Moment: \( M_c = M_{plRd} = 12.3 \text{ kNm} \)
From equation 2: \( R_b = (F_t*(m-ecc)-M_c*1E3)/(n+(m-ecc)) = -79 \text{ kN} \)

Tension resistance: \( F_T = F_{tRd}*(a*n+(n+bsp))/(n+(bsp/2+ecc)) = 283 \text{ kN} \)
Design tension force: \( T = M_d/l = 255 \text{ kN} \)
As \( T \leq F_T \) (255 kN \leq 283 kN), Mode 2 failure check OK.
Bending moment at fin plate face: \( M = F_t*(m-ecc)/1E3 = 6.1 \text{ kNm} \)
As \( M \leq M_{plRd} \) (6.1 kNm \leq 12.3 kNm), bending moment at the face of the fin plate is less than the cast-in plate moment resistance.

(B) Check Mode 1 failure: consider a vertical centre line through the cast-in plate and fin plate.

Total resistance force: \( F_{RdT1} = F_{RdT1A} + F_{RdT1B} = 1274 \text{ kN} \)
As \( T \leq F_{RdT1} \) (255 kN \leq 1274 kN), Mode 1 failure check OK.
CAST-IN PLATE DETAILS
Depth 560 mm                 Breadth 250 mm
Thickness 25 mm

(C) Check the cast-in plate under the tying load case
Tension resistance
FT=Ft*(a*n+(n+bsp))/com1=326 kN
As FT ≥ 2*FtEd/nb (326 kN ≥ 233 kN), tension resistance exceeds the design tying force.
As M ≤ MplRd (7.02 kNm ≤ 15.3 kNm), bending moment in cast-in plate is less than the cast-in plate moment resistance. Hence the 25 mm thick cast-in plate is acceptable for the tying load case.

Resistance of the combined cast-in plate and fin plate

The resistance of the combined cast-in plate and fin plate to bending in the vertical plane will be considered. The fin plate will need to be checked in bending at the face of the cast-in plate. The nominal eccentricity moment at the back of the plate calculated previously for determining the bar sizes is Md=105 kNm. The latter will be reduced by the assumed plate thickness.
As M ≤ MRd (87.8 kNm ≤ 115 kNm), the reduced bending moment is less than the fin plate moment resistance.

(A) Tying load case
The fin plate and cast-in plate acting as a Tee section spans between the top and bottom tie bars to resist tie forces. The span of the Tee section is the distance between the top & bottom tie bars.
ls = length of stem
y = depth to NA
tp =25 mm
tfp=10 mm
Consider the cast-in plate and fin plate acting as a Tee section.

Assumed Tee section flange width b=150 mm
Length of Tee stem ls=110 mm
Span of Tee section L=0.42 m

(B) Plastic modulus
Plastic modulus of Tee section Z=z1+z2+z3=117671 mm³
Moment resistance MRd=fufp*Z/(gammu*1E6)=43.9 kNm
As M ≤ MRd (36.8 kNm ≤ 43.9 kNm), the bending moment is less than the Tee section moment resistance.

Welding reinforcement to cast-in plate

Bar strength Ft=245 kN
Partial factor (cast-in plate) gamm=1.1
Weld coefficient betaw=0.9
Design strength of weld \[ f_{vwd} = \frac{f_{up}}{(\beta_{aw} \gamma_{aw} (3)^{0.5})} \]

\[ = 274 \text{ N/mm}^2 \]

Two welds round the bar will need to be provided as shown in diagram.

Throat length required \[ a = \frac{F_t \cdot 1 \times 10^3}{2 \cdot f_{vwd} \cdot \pi \cdot d_r} = 5.7 \text{ mm} \]

Weld leg length (reinf’ment) \[ s_r = \frac{a}{0.7071} = 8.06 \text{ mm} \]

Weld leg length to be adopted \[ s_r = 10 \text{ mm} \]

**PUNCHING SHEAR**

The shear stress expression below is as per Clause 6.4.4(2).

\[ \frac{v_{Ed}}{\mu_i \cdot d} = \frac{V_{Ed}}{\mu_i \cdot d} + \frac{k \cdot M_{Ed}}{\mu_i \cdot d} \]

Consider the bending component only for the eccentricity moment and the direct component only for the tie force as these are separate load cases.

Definitions:
- loaded area refers to the cross-sectional area of the stiff bearing
- load perimeter refers to the stiff bearing plate perimeter
- load face refers to the stiff bearing plate face
- face of the load refers to the face of the stiff bearing plate
- loaded area dimensions refers to stiff bearing area dimensions
(A) Shear stress on plate perimeter due to eccentricity moment

First link perimeter at ≤ 0.5d from face of loaded area.

Link perimeters at ≤ 0.75d apart.

Average effective depth d = 170 mm
Dimension of loaded area cx = 250 mm
dimension of loaded area cy = 560 mm
As vEd ≤ vRdmax (1.511 N/mm² ≤ 5.712 N/mm²), shear stress at load perimeter is within the maximum punching shear resistance.

Shear stress on the basic control perimeter u₁ due to bending

Determine the value of the shear resistance assuming there is no shear reinforcement as per EC2 Clause 6.4.4(1).

Reinforcement ratio in x-dir plx = 0.004
Reinforcement ratio in y-dir ply = 0.0023
Design punching shear resistance VRdc = (0.18/gamc)*k*(100*p1 *fck)^0.333 = 0.551 N/mm²
Min punching shear resistance vmin = 0.035*k*1.5*fck*0.5 = 0.6261 N/mm²
Hence, the minimum design punching shear resistance governs.

Design punching shear resistance VRdc = 0.6261 N/mm²
As vEd ≤ VRdc (0.2936 N/mm² ≤ 0.6261 N/mm²), punching shear reinforcement is not necessary for this load case.

(B) Shear stress on plate perimeter due to tie force

An estimate of the shear reinforcement required for tying action will be made, assuming the tie force is carried into the wall through the cast-in plate (no bending checks on the wall will be carried out).

The tie force is a direct force VEd’ = FtEd = 700 kN
Partial safety factor (concrete) gamc = 1.2 (accidental load case)
Load perimeter uo = 2*(cx + cy) = 1620 mm
Shear stress at load perimeter vEd = VEd’*1000/(uo*d) = 2.542 N/mm²
Maximum punching shear resistance VRdmax = 0.5*v*fcd = 7.14 N/mm²
As vEd ≤ VRdmax (2.542 N/mm² ≤ 7.14 N/mm²), shear stress at load perimeter is within the maximum punching shear resistance.
Check shear stress at control perimeter $u_1$ at 2d from load face

Basic control perimeter at 2d

$$u_1 = 2(cx + cy) + 4\pi d = 3756 \text{ mm}$$

Design punching shear resistance

$$\nu_{Rdc} = (0.18/\gamma_{Mc}) \times k \times (100 \times p_1 \times f_{ck})^{0.333} = 0.6888 \text{ N/mm}^2$$

Min punching shear resistance

$$\nu_{min} = 0.035 \times k^{1.5} \times f_{ck}^{0.5} = 0.6261 \text{ N/mm}^2$$

As $\nu_{Ed} > \nu_{Rdc}$ ($1.096 \text{ N/mm}^2 > 0.6888 \text{ N/mm}^2$), punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required

Outer control perimeter

$$U_{out} = \frac{V_{Ed} \times 1000}{d \times \nu_{Rdc}} = 5978 \text{ mm}$$

Radius to $U_{out}$

$$r_{out} = \frac{U_{out} - lcf}{2\pi} = 693.6 \text{ mm}$$

The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter $U_{out}$. Radius to outer perimeter of shear reinforcement

$$Out_{per} = r_{out} - 1.5 \times d = 438.6 \text{ mm from load face}$$

Shear links on basic control perimeter $u_1$

The partial safety factor for reinforcement will be taken as $\gamma_{Ms} = 1.15$.

Spac. inside 2d control perimeter $St = 255 \text{ mm}$

Spac. outside control perimeter $u_1$ $St' = 340 \text{ mm}$

Char yield strength of reinf. $f_{yk} = 500 \text{ N/mm}^2$

Diameter of shear reinforcement $d_{al} = 6 \text{ mm}$

Spacing of links to be used $spac = 170 \text{ mm}$

Use minimum H 6 (28.27 mm$^2$) legs of links @ 170 mm c/c around perimeter $u_1$ (i.e. in the tangential direction).

Summary of punching shear links

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.
Links on perimeter 1:
Use minimum H 6 (28.27 mm²) legs of links @ 95 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 6 (28.27 mm²) legs of links @ 130 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 6 (28.27 mm²) legs of links @ 170 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

Links on perimeter 4:
Use minimum H 6 (28.27 mm²) legs of links @ 205 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

DESIGN SUMMARY - nominally pinned joint

Design loads:
- Design shear: 702 kN
- Shear resistance: 848.2 kN (stud group)
- Shear resistance: 141.4 kN (per stud)
- Design tying force: 700 kN
- Tension resistance: 1473 kN (bar group)
- Tension resistance: 213.4 kN (per bar)

Grade/strength:
- Concrete: 40 N/mm²
- Fin plate: S 275
- Cast-in plate: S 355
- M20 bolts: 8.8

FP = fin plate
CP = cast-in plate
tb = top bars
bb = bottom bars
ss = shear stud

Fin plate:
- No. of bolts: 7
- Plate depth: 500 mm
- Plate width: 140 mm
- Plate thickness: 10 mm

Cast-in plate:
- Plate depth: 560 mm
- Plate width: 250 mm
- Plate thickness: 25 mm
Weld leg length: Fin to cast-in plate
2 No/8 mm FW need to be provided for connecting the fin plate to cast-in plate to eliminate the possibility of weld failure.

Stud welding: It is assumed that studs will be welded using electric arc stud-welding equipment. See SCI publication P300 for further information.

Weld leg length: Reinforcement
10 mm FW with two welds round the bar as shown in the diagram. PPFW is a partial penetration fillet weld (FW) in countersunk hole.

Shear studs:
No. and diameter 6 No/25 mm
Height 100 mm
Spacing vertically 125 mm
Spacing horizontally 130 mm
Vertical edge distance 155 mm
Horiz. edge distance 60 mm

Reinforcement:
Top bars 2 No/25 mm diameter
Bottom bars 4 No/25 mm diameter
Top bar spacing (bsp) 150 mm (laterally)
Bar edge distance (n') 50 mm (top & bottom bars)
Bar end distance (ex) 70 mm (top & bottom bars)
Bend bars mandrel 266.8 mm diameter
Bar anchorage length 1052 mm
Cast-in plate with a total of 6 bars as shown
i.e. 2 top bars and
4 bottom bars.
n'=50 mm (edge distance)
bsp=150 mm (top bars)
bp=250 mm
hp=560 mm
ex=70 mm

Punching shear links are required, refer to the punching shear section of the calculations for diameter & spacing of links.
Location: Ex4 - Cast-in plate design (3x3 Shear stud Array)

General design information


The shear stud resistance check will be as per EC4 Part 1-1. The shear force is transferred to the fin plate by bolts. The cast-in plate and its welded connection to the fin plate will resist the eccentricity moment due to the offset of the bolt group from the face of the plate. The standard arrangement for fin plates in the Green Book (i.e. SCI publication P358) is 10 mm thick fin plate using S275 material, with two 8 mm FW. This will eliminate the possibility of weld failure. The face of the cast-in plate is normally inset to allow for the sliding form.

Both the cast-in plate and fin plate will need site-applied corrosion protection system if corrosion protection of steelwork is specified. Furthermore parts of the connection plate (e.g. the fin plate) will also require fire protection as for the connecting beam.

Design shear (+ve value) $V_{Ed}=725$ kN
Design tying force (+ve value) $F_{tEd}=700$ kN

Fin plate arrangement

<table>
<thead>
<tr>
<th>No. of rows of M20 bolts</th>
<th>nbr=7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fin plate depth</td>
<td>hfp=500 mm</td>
</tr>
<tr>
<td>Fin plate width</td>
<td>bfp=140 mm</td>
</tr>
</tbody>
</table>

Choose initial shear stud arrangement

<table>
<thead>
<tr>
<th>Stud diameter</th>
<th>ds=25 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stud height</td>
<td>hs=100 mm</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>$f_{ck}=40$ N/mm$^2$</td>
</tr>
<tr>
<td>Spacing of studs (vertically)</td>
<td>$d_1=125$ mm</td>
</tr>
<tr>
<td>Spacing of studs (horizontally)</td>
<td>$d_2=100$ mm</td>
</tr>
<tr>
<td>Edge distance (vertically)</td>
<td>$e_1=155$ mm</td>
</tr>
<tr>
<td>Edge distance (horizontally)</td>
<td>$e_2=60$ mm</td>
</tr>
</tbody>
</table>
**Initial reinforcement arrangement**

Reinforcement welded to cast-in plate for bending and tying resistance will need to be provided.
- Bar diameter (top & bottom bars): $d_r = 25$ mm
- Total number of bars: $n_b = 6$
- Tension resistance (bar group): $P_{trd} = $ TABLE 3 for $n_b = 6, d_r = 25$
  
  $= 1473$ kN
- Design tying force: $F_{ted} = 700$ kN

As $P_{trd} \geq F_{ted}$ ($1473$ kN $\geq 700$ kN), the tension resistance exceeds the design tying force.

**Confirm the shear stud arrangement - EC4 Part 1-1 Cl. 6.6.3.1**

- Governing group shear resistance: $P_{rd} = P_{rd1} = 1272$ kN
- As $P_{rd} \geq V_{Ed}$ ($1272$ kN $\geq 725$ kN), the shear resistance exceeds the design shear and hence studs provided are satisfactory.
- Horizontal deviation: $e_{cc} = 35$ mm
- Angle $\beta$ (rotational deviation): $\beta = 3.81^\circ$
- Maximum shear in corner stud: $V_{max2} = \sqrt{(V_{Ed}/n_s + V_2)^2 + V_1^2} = 100$ kN
- Shear resistance per stud: $P_{rd1} = P_{rd}/n_s = 141$ kN

As $V_{max2} \leq P_{rd1}$ ($100$ kN $\leq 141$ kN), corner stud shear OK.

**Reinforcement arrangement and bar size**

- (A) Eccentricity moment on the cast-in plate
  - Maximum eccentricity: $e_p = 150$ mm
  - Design moment due to eccentricity: $M_d = V_{Ed} * e_p / 1000 = 109$ kNm

- (B) Vertical lever arm for tension - compression couple
  - Design tension force: $T = M_d / l = 264$ kN
  - Design compression force: $C = T_2 = 264$ kN

- (C) Confirm the bar size
  - Top bar spacing (laterally): $b_{sp} = 150$ mm

As $T \leq F_{td}$ ($264$ kN $\leq 427$ kN), bars provided are OK.
As $F_{ted}' \leq F_{td1}$ ($194$ kN $\leq 213$ kN), maximum tensile force due to eccentricity of loading is less than the bar resistance.

**Bending of the cast-in plate**

The following calculations will check the resistance of the cast-in plate in bending for the eccentricity moment tension. The resistance model for end-plate from BS EN 1993-1-8 Clause 6.2.4 will be used.

- (A) Check Mode 2 failure: bolt failure with yielding of the flange. In this case the bars are substituted for bolts.
Steel design to Eurocode 3  
Cast-in plate to connect structural steel beam to  
Made by: IFB  
Date: 02/12/19  
Ref No: SC495 EC

Equation 1: \[ M_c + R_a((n+(m+ecc))) - aF_t(m+ecc) = 0 \]
Equation 2: \[ M_c + R_b((n+(m-ecc))) - F_t(m-ecc) = 0 \]

Bar end distance \( e_x = 70 \text{ mm} \)
Cast-in plate thickness \( t_p = 25 \text{ mm} \)
Force \( F_t = FtRd/ntb = 213 \text{ kN} \)
Moment \( M_c = MplRd = 13.5 \text{ kNm} \)
From equation 2 \[ R_b = (F_t*(m-ecc)-M_c*1E3)/(n+(m-ecc)) = -68.4 \text{ kN} \]

Tension resistance \[ F_T = F_t*(a*n+(n+bsp))/(n+(bsp/2+ecc)) = 289 \text{ kN} \]
Design tension force \( T = M_d/l = 264 \text{ kN} \)
As \( T \leq F_T \) (264 kN \( \leq 289 \text{ kN} \)), Mode 2 failure check OK.
Bending moment at fin plate face \( M = F_t*(m-ecc)/1E3 = 6.1 \text{ kNm} \)
As \( M \leq MplRd \) (6.1 kNm \( \leq 13.5 \text{ kNm} \)), bending moment at the face of the fin plate is less than the cast-in plate moment resistance.

(B) Check Mode 1 failure: consider a vertical centre line through the cast-in plate and fin plate.

Total resistance force \( F_{RdT1} = F_{RdT1A} + F_{RdT1B} = 1375 \text{ kN} \)
As \( T \leq F_{RdT1} \) (264 kN \( \leq 1375 \text{ kN} \)), Mode 1 failure check OK.
CAST-IN PLATE                          Details
Depth  560 mm                      Breadth  320 mm
Thickness 25 mm

(C) Check the cast-in plate under the tying load case
Tension resistance  \( FT=2\times F_t (a\times n+(n+b/np))/\text{com1}=333 \text{ kN} \)
As \( FT \geq 2\times F_t E_d/\text{nb} \) (333 kN \( \geq \) 233 kN), tension resistance exceeds the design tying force.
As \( M \leq M_{pl,Rd} \) (7.02 kNm \( \leq \) 16.7 kNm), bending moment in cast-in plate is less than the cast-in plate moment resistance. Hence the 25 mm thick cast-in plate is acceptable for the tying load case.

Resistance of the combined cast-in plate and fin plate

The resistance of the combined cast-in plate and fin plate to bending in the vertical plane will be considered. The fin plate will need to be checked in bending at the face of the cast-in plate. The nominal eccentricity moment at the back of the plate calculated previously for determining the bar sizes is \( M_d=109 \text{ kNm} \). The latter will be reduced by the assumed plate thickness.
As \( M \leq M_{Rd} \) (90.6 kNm \( \leq \) 115 kNm), the reduced bending moment is less than the fin plate moment resistance.

(A) Tying load case
The fin plate and cast-in plate acting as a Tee section spans between the top and bottom tie bars to resist tie forces. The span of the Tee section is the distance between the top & bottom tie bars.
\[
\begin{align*}
\text{tp} & = \text{length of stem} \\
\text{y} & = \text{depth to NA} \\
\text{tp} & = 25 \text{ mm} \\
\text{tfp} & = 10 \text{ mm}
\end{align*}
\]
Consider the cast-in plate and fin plate acting as a Tee section.

Assumed Tee section flange width \( b=150 \text{ mm} \)
Length of Tee stem \( l_s=110 \text{ mm} \)
Span of Tee section \( L=0.42 \text{ m} \)

(B) Plastic modulus
Plastic modulus of Tee section \( Z=z_1+z_2+z_3=117671 \text{ mm}^3 \)
Moment resistance \( M_{Rd}=f_{ufp}\times Z/(\gamma_m\times 1\times 10^6)=43.9 \text{ kNm} \)
As \( M \leq M_{Rd} \) (36.8 kNm \( \leq \) 43.9 kNm), the bending moment is less than the Tee section moment resistance.

Welding reinforcement to cast-in plate

Bar strength \( F_t=245 \text{ kN} \)
Partial factor (cast-in plate) \( \gamma_m=1.1 \)
Weld coefficient \( \beta_{aw}=0.9 \)
Design strength of weld:

\[ f_{vwd} = \frac{f_{up}}{(\beta_{w} \gamma_{w} (3)^{0.5})} \]

\[ = 274 \text{ N/mm}^2 \]

Two welds round the bar will need to be provided as shown in diagram.

Throat length required:

\[ a = \frac{F_t \times 1E3}{2 \times f_{vwd} \pi \times d} = 5.7 \text{ mm} \]

Weld leg length (reinf'ment):

\[ s_r = a / 0.7071 = 8.06 \text{ mm} \]

Weld leg length to be adopted:

\[ s_r = 10 \text{ mm} \]

**PUNCHING SHEAR**

The shear stress expression below is as per Clause 6.4.4(2).

\[ \frac{V_{Ed}}{u_i \cdot d} + \frac{k \cdot M_{Ed}}{W_i \cdot d} \]

Consider the bending component only for the eccentricity moment and the direct component only for the tie force as these are separate load cases.

Definitions:
- loaded area refers to the cross-sectional area of the stiff bearing
- load perimeter refers to the stiff bearing plate perimeter
- load face refers to the stiff bearing plate face
- face of the load refers to the face of the stiff bearing plate
- loaded area dimensions refers to stiff bearing area dimensions
(A) Shear stress on plate perimeter due to eccentricity moment

First link perimeter at ≤ 0.5d from face of loaded area.

Link perimeters at ≤ 0.75d apart.

 cx and cy are the loaded area dimensions.

Basic control perimeter is at 2d from the face of the loaded area.

Sr = spacing of links in the radial direction.

Average effective depth d=170 mm
Dimension of loaded area cx=250 mm
dimension of loaded area cy=560 mm

As vEd ≤ vRdmax (1.56 N/mm² ≤ 5.712 N/mm²), shear stress at load perimeter is within the maximum punching shear resistance.

Shear stress on the basic control perimeter u₁ due to bending

Determine the value of the shear resistance assuming there is no shear reinforcement as per EC2 Clause 6.4.4(1).

Reinforcement ratio in x-dir pl₁ = 0.004
Reinforcement ratio in y-dir ply = 0.0023
Design punching shear resistance vRdc=(0.18/gamc)*k*(100*p₁ *fck)^0.333=0.551 N/mm²

Min punching shear resistance vmin=0.035*k'*fck'*0.5=0.6261 N/mm²

Hence, the minimum design punching shear resistance governs.

Design punching shear resistance vRdc=0.6261 N/mm²

As vEd ≤ vRdc (0.3032N/mm² ≤ 0.6261 N/mm²), punching shear reinforcement is not necessary for this load case.

(B) Shear stress on plate perimeter due to tie force

An estimate of the shear reinforcement required for tying action will be made, assuming the tie force is carried into the wall through the cast-in plate (no bending checks on the wall will be carried out).

The tie force is a direct force VEd'=FtEd=700 kN
Partial safety factor (concrete) gamc=1.2 (accidental load case)
Load perimeter u₀=2*(cx+cy)=1620 mm
Shear stress at load perimeter vEd=VEd'*(uo*d) = 2.542 N/mm²

Maximum punching shear resistance vRdmax=0.5*v*fcd=7.14 N/mm²

As vEd ≤ vRdmax (2.542 N/mm² ≤ 7.14 N/mm²), shear stress at load perimeter is within the maximum punching shear resistance.
Check shear stress at control perimeter \( u_1 \) at 2d from load face

Basic control perimeter at 2d \( u_1 = 2*(c_x + c_y) + 4\pi d = 3756 \text{ mm} \)
Design punching shear resistance \( v_{Rdc} = (0.18/gamc) \cdot k \cdot (100 \cdot p_1 \cdot f_{ck})^{0.333} = 0.6888 \text{ N/mm}^2 \)
Min punching shear resistance \( v_{min} = 0.035 \cdot k^{1.5} \cdot f_{ck}^{0.5} = 0.6261 \text{ N/mm}^2 \)
As \( v_{Ed} > v_{Rdc} \) (1.096 N/mm² > 0.6888 N/mm²), punching shear reinforcement needs to be provided.

Outer control perimeter where shear links are no longer required

Outer control perimeter \( U_{out} = V_{Ed} \cdot 1000 / (d \cdot v_{Rdc}) = 5978 \text{ mm} \)
Radius to \( U_{out} \) \( r_{out} = (U_{out} - lcf) / (2 \cdot \pi) = 693.6 \text{ mm} \)
The outer perimeter of shear reinforcement should be at a distance not greater than 1.5d from the outer control perimeter \( U_{out} \). Radius to outer perimeter of shear reinforcement \( r_{outer} = r_{out} - 1.5 \cdot d \)
\( = 438.6 \text{ mm from load face} \)

Shear links on basic control perimeter \( u_1 \)
The partial safety factor for reinforcement will be taken as \( \gamma_{ms} = 1.15 \).
Spac. inside 2d control perimeter \( S_t = 255 \text{ mm} \)
Spac. outside control perimeter \( u_1 \) \( S_t' = 340 \text{ mm} \)
Char yield strength of reinforcement \( f_{yk} = 500 \text{ N/mm}^2 \)
Diameter of shear reinforcement \( \phi_{dl} = 6 \text{ mm} \)
Spacing of links to be used \( S_p = 170 \text{ mm} \)
Use minimum H 6 (28.27 mm²) legs of links @ 170 mm c/c around perimeter \( u_1 \) (i.e. in the tangential direction).

Summary of punching shear links

In the following calculation the first perimeter will be taken at 0.5d from the face of the load and subsequent perimeters will be spaced at 0.75d apart.
Links on perimeter 1:
Use minimum H 6 (28.27 mm$^2$) legs of links @ 95 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

Links on perimeter 2:
Use minimum H 6 (28.27 mm$^2$) legs of links @ 130 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

Links on perimeter 3:
Use minimum H 6 (28.27 mm$^2$) legs of links @ 170 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

Links on perimeter 4:
Use minimum H 6 (28.27 mm$^2$) legs of links @ 205 mm c/c in the tangential direction and 125 mm c/c in the radial direction.

**DESIGN SUMMARY - nominally pinned joint**

<table>
<thead>
<tr>
<th>Design loads:</th>
<th>Design shear</th>
<th>725 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>1272 kN (stud group)</td>
<td></td>
</tr>
<tr>
<td>Shear resistance</td>
<td>141.4 kN (per stud)</td>
<td></td>
</tr>
<tr>
<td>Design tying force</td>
<td>700 kN</td>
<td></td>
</tr>
<tr>
<td>Tension resistance</td>
<td>1473 kN (bar group)</td>
<td></td>
</tr>
<tr>
<td>Tension resistance</td>
<td>213.4 kN (per bar)</td>
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</table>

<table>
<thead>
<tr>
<th>Grade/strength:</th>
<th>Concrete</th>
<th>40 N/mm$^2$</th>
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<tbody>
<tr>
<td>Fin plate</td>
<td>S 275</td>
<td></td>
</tr>
<tr>
<td>Cast-in plate</td>
<td>S 355</td>
<td></td>
</tr>
<tr>
<td>M20 bolts</td>
<td>8.8</td>
<td></td>
</tr>
</tbody>
</table>

**Fin plate:**
- No. of bolts: 7
- Plate depth: 500 mm
- Plate width: 140 mm
- Plate thickness: 10 mm

**Cast-in plate:**
- Plate depth: 560 mm
- Plate width: 320 mm
- Plate thickness: 25 mm
Weld leg length: Fin to cast-in plate 2 No/8 mm FW need to be provided for connecting the fin plate to cast-in plate to eliminate the possibility of weld failure.

Stud welding: It is assumed that studs will be welded using electric arc stud-welding equipment. See SCI publication P300 for further information.

Weld leg length: Reinforcement 10 mm FW with two welds round the bar as shown in the diagram. PPFW is a partial penetration fillet weld (FW) in countersunk hole.

\[ \text{dr} = 25 \text{ mm (bar diameter)} \]
\[ \text{tp} = 25 \text{ mm (plate thickness)} \]

Shear studs: 9 No/25 mm

- No. and diameter: 9 No/25 mm
- Height: 100 mm
- Spacing vertically: 125 mm
- Spacing horizontally: 100 mm
- Vertical edge distance: 155 mm
- Horiz. edge distance: 60 mm

\[ \begin{array}{cccc}
\text{60} & \text{100} & \text{100} & \text{60} \\
\end{array} \]

\[ \begin{array}{cccc}
\text{155} & & & \\
\text{125} & & & \\
\text{125} & & & \\
\text{155} & & & \\
\end{array} \]

Shear studs: 3x3 shear stud array

Total 9 studs

All dimensions in mm

Reinforcement:

- Top bars: 2 No/25 mm diameter
- Bottom bars: 4 No/25 mm diameter
- Top bar spacing (bsp): 150 mm (laterally)
- Bar edge distance (n'): 85 mm (top & bottom bars)
- Bar end distance (ex): 70 mm (top & bottom bars)
- Bend bars mandrel: 266.8 mm diameter
- Bar anchorage length: 1052 mm
Cast-in plate with a total of 6 bars as shown
i.e. 2 top bars and 4 bottom bars.
n' = 85 mm (edge distance)
bsp = 150 mm (top bars)
bp = 320 mm
hp = 560 mm
ex = 70 mm

Punching shear links are required, refer to the punching shear section of the calculations for diameter & spacing of links.
Load bearing stiffener

Calculations are in accordance with BS 5950-1:2000, Clause 4.5.

Force applied through flange  \( F_c = 639.7 \text{ kN} \)

**Section properties**

457 x 191 x 74 UB.  
Young's Modulus  \( E = 205 \text{ kN/mm}^2 \)

**Check for bearing on the web - Clause 4.5.2.1**

\[
\text{Bearing capacity of web } P_{bw} = (b_1 + n \cdot k) \cdot t \cdot \sigma_w / 10^3 = 436.34 \text{ kN}
\]

Since \( F_c > P_{bw} \) (639.7 kN > 436.34 kN) a bearing stiffener is required.

**Check for buckling of the web - Clause 4.5.3.1**

\[
\text{Buckling resistance } P_x = 25 \cdot e \cdot t / ((b_1 + n \cdot k) \cdot d)^{0.5} \cdot P_{bw} = 366.24 \text{ kN}
\]

Since \( F_c > P_x \) (639.7 kN > 366.24 kN) a load carrying stiffener will be required to prevent the web from buckling.

**Stiffener design**

Assuming initially that the stiffener will be less than or equal to 16 mm thick, two stiffeners are to be used and allowing for a cut-back of 13 mm at the root of the member.

Design strength  \( \sigma_s = 275 \text{ N/mm}^2 \)

Minimum area of stiffener  \( A_s = F_s / \sigma_s = 739.48 \text{ mm}^2 \)

Allowable stiffener outstand  \( \text{out} = 13 \cdot t \cdot s = 195 \text{ mm} \)

As \( b_s < \text{out} \) (80 mm < 195 mm) the actual stiffener outstand is less than the allowable, hence OK.

The effective section of the stiffener, is shown.

**Check stiffener as a bearing stiffener**

\[
\text{Force on bearing stiffener } F_{B} = F_c - P_{bw} = 203.36 \text{ kN}
\]

\[
\text{Bearing capacity of stiffener } P_B = \sigma_s \cdot A_{s\text{iff}} / 10^3 = 552.75 \text{ kN}
\]

Since \( F_B \geq P_B \), stiffener is OK for bearing.
Connection of stiffeners to web

Effective length of weld \( lw = D - 2*(\text{cut} + T + 0.7*t) = 389.4 \text{ mm} \)

Weld size \( s = 6 \text{ mm} \)

Capacity of weld \( pws = 0.7*s*pw = 924 \text{ N/mm} \)

6 mm weld is suitable.

DESIGN SUMMARY

457 x 191 x 74 UB

Steel Grade S 275

External load 639.7 kN

Local capacity 436.3 kN

Buckling capacity 1328 kN

STIFFENER DETAILS

Plate 2No/80 mm x 15 mm

Welding 4No/6 mm FW's

Both flanges restrained against rotation in plane of stiffener.
Location: Ex2 - End stiffener (Plate Girder)

Load bearing stiffener

Calculations are in accordance with BS 5950-1:2000, Clause 4.5.

Force applied through flange \( F_c = 1560 \text{ kN} \)

Section properties

Plate Girder
Depth of section \( D = 1500 \text{ mm} \)
Width of section \( B = 450 \text{ mm} \)
Thickness of flange \( T = 40 \text{ mm} \)
Thickness of web \( t = 15 \text{ mm} \)
Dimensions (mm): \( D = 1500 \) \( B = 450 \) \( T = 40 \) \( t = 15 \)

Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Check for bearing on the web - Clause 4.5.2.1

Stiff bearing length \( b_1 = 75 \text{ mm} \)
from the end of stiff bearing \( b_e = 0 \text{ mm} \)
Design strength \( p_y w = 275 \text{ N/mm}^2 \)
Bearing capacity of web \( P_{bw} = (b_1 + n \cdot k) \cdot t \cdot p_y w / 10^3 = 639.38 \text{ kN} \)

Since \( F_c > P_{bw} \) (1560 kN > 639.38 kN) a bearing stiffener is required.

Check for buckling of the web - Clause 4.5.3.1

Stiff bearing length \( b_1 = 75 \text{ mm} \)
distance \( a_e = 0 \text{ mm} \)
Buckling resistance \( P_x = p_x \cdot 25 \cdot e \cdot t / ((b_1 + n \cdot k) \cdot d)^0.5 \cdot P_{bw} = 255.53 \text{ kN} \)

Since \( F_c > P_x \) (1560 kN > 255.53 kN) a load carrying stiffener will be required to prevent the web from buckling.

Stiffener design

Assuming initially that the stiffener will be less than or equal to 16 mm thick, two stiffeners are to be used and allowing for a cut-back of 15 mm at the root of the member.

Design strength \( p_y s = 275 \text{ N/mm}^2 \)
Minimum area of stiffener \( A_{min} = F_s \cdot 10^3 / p_y s = 3347.7 \text{ mm}^2 \)
Allowable stiffener outstand \( out = 13 \cdot t_s \cdot e = 208 \text{ mm} \)

As \( b_s < out \) (150 mm < 208 mm) the actual stiffener outstand is less than the allowable, hence OK.
The effective section of the stiffener, is shown.
Check stiffener as a bearing stiffener

Force on bearing stiffener \( FB = F_c - P_{bw} = 920.63 \text{ kN} \)
Bearing capacity of stiffener \( PB = p_y s * A_{stiff} / 10^3 = 1188 \text{ kN} \)
Since \( FB \geq PB \), stiffener is OK for bearing.

Connection of stiffeners to web

Effective length of weld \( lw = D - 2 \times (cut + T + 0.7t) = 1369 \text{ mm} \)
Weld size \( s = 6 \text{ mm} \)
Capacity of weld \( p_{ws} = 0.7s p_w = 924 \text{ N/mm} \)
6 mm weld is suitable.

PLATE GIRDER
DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Steel Grade</th>
<th>1500 mm x 450 mm Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>S 275</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>External load</th>
<th>1560 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local capacity</td>
<td>639.4 kN</td>
</tr>
<tr>
<td>Buckling capacity</td>
<td>2281 kN</td>
</tr>
</tbody>
</table>

STIFFENER DETAILS

<table>
<thead>
<tr>
<th>Plate</th>
<th>315 mm x 16 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welding</td>
<td>2No/6 mm FW's</td>
</tr>
</tbody>
</table>

Both flanges restrained against rotation in plane of stiffener.
Location: Ex3 - Check load bearing stiffener (End stiffener)

Load bearing stiffener

Calculations are in accordance with BS 5950-1:2000, Clause 4.5.

Force applied through flange \( F_c = 800 \text{ kN} \)

Section properties

- **254 x 254 x 73 UC.**
- Young's Modulus \( E = 205 \text{ kN/mm}^2 \)

Stiffener design

Assuming initially that the stiffener will be less than or equal to 16 mm thick, two stiffeners are to be used and allowing for a cut-back of 15 mm at the root of the member.

- **Design strength** \( p_{ys} = 275 \text{ N/mm}^2 \)
- **Design strength** \( p_{yw} = 275 \text{ N/mm}^2 \)
- **Bearing capacity** \( P_{bw} = (b_1 + 2k) t p_{yw}/10^3 = 127.24 \text{ kN} \)
- **Minimum area of stiffener** \( A_{s_{min}} = F_s 10^3/p_{ys} = 2446.4 \text{ mm}^2 \)
- **Allowable stiffener outstand** \( o_{ut} = 13 t_s e = 208 \text{ mm} \)

As \( A_s < o_{ut} \) (100 mm < 208 mm) the actual stiffener outstand is less than the allowable, hence OK. The effective section of the stiffener, is shown.

Check stiffener as a bearing stiffener

Assuming a stiff bearing length \( b_1 = 0 \text{ mm} \)

- **Bearing capacity of web** \( P_{bw} = (b_1 + n k) t p_{yw}/10^3 = 254.47 \text{ kN} \)
- **Force on bearing stiffener** \( F_B = F_c - P_{bw} = 545.53 \text{ kN} \)
- **Bearing capacity of stiffener** \( P_B = p_{ys} A_{s_{\text{stiff}}} 10^3 = 748 \text{ kN} \)

Since \( F_B \geq P_B \), stiffener is OK for bearing.

Connection of stiffeners to web

- **Effective length of weld** \( l_w = D - 2(cut + T + 0.7t) = 183.66 \text{ mm} \)
- **Weld size** \( s = 8 \text{ mm} \)
- **Capacity of weld** \( p_{ws} = 0.7s p_{w} = 1232 \text{ N/mm} \)

8 mm weld is suitable.

**DESIGN SUMMARY**

- **Steel Grade**: S 275
- **External load**: 800 kN
- **254 x 254 x 73 UC**

**STIFFENER DETAILS**

- **Plate**: 2No/100 mm x 16 mm
- **Welding**: 4No/8 mm FW's
- **Flanges not restrained against rotation**.
Load bearing stiffener

Calculations are in accordance with BS EN 1993-1-1:2005. References to EC3 Part 1-5 and EC3 Part 1-8 are also made where appropriate.

Force applied through flange $V_{zEd} = 639.7$ kN

Section properties

457 x 191 x 74 UKB
Stiff bearing length $ss = 52.8$ mm
Stiff bearing position $C = 0$ mm

Check for bearing on the web

Design strength of web $f_{yw} = 275$ N/mm$^2$
Bearing resistance of web $FR_{dbr} = \frac{(ss+n*k)*tw*fyw}{(\gamma_{M0}*10^3)}$

$$= 436.34$$ kN

Since $V_{zEd} > FR_{dbr}$ ($639.7$ kN > 436.34 kN) a bearing stiffener is required.

Stiffener design

Assuming initially that the stiffener will be less than or equal to 16 mm thick, two stiffeners are to be used and allowing for a cut-back of 13 mm at the root of the member.

Design strength of stiffener $f_{ys} = 275$ N/mm$^2$
Assumed stiffener thickness $ts = 15$ mm
Assumed stiffener width $bs = 80$ mm
Allowable stiffener outstand $out = 13*ts*e = 180.26$ mm

As $bs \leq out$ (80 mm ≤ 180.26 mm) the actual stiffener outstand is less than the allowable, hence OK.

The effective section of the stiffener, is shown.

Check stiffener as a bearing stiffener

Force on bearing stiffener $F_{EdB} = V_{zEd} - FR_{dbr} = 203.36$ kN
Bearing resistance of stiffener $FR_{dB} = f_{ys}*Astiff/(\gamma_{M0}*10^3) = 552.75$ kN

Since $F_{EdB} \geq FR_{dB}$, stiffener is OK for bearing.

Connection of stiffeners to web

Design strength of fillet weld $f_{wvd} = f_{u}/(\sqrt{3}*betw*\gamma_{M2})$

$$= 222.79$$ N/mm$^2$

Design load on weld $F_{wEd} = F_{EdB}/(4*lw) = 0.13056$ kN/mm
Fillet weld size (leg length) $s = 6$ mm
Design weld resistance $F_{wRd} = 0.7*s*f_{wvd}/1000 = 0.93571$ kN/mm

Hence, 6 mm FW's are suitable and will be adopted.
<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>457 x 191 x 74 UB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Grade</td>
<td>S 275</td>
</tr>
<tr>
<td>External load</td>
<td>639.7 kN</td>
</tr>
<tr>
<td>Bearing resistance</td>
<td>436.3 kN</td>
</tr>
<tr>
<td>Buckling resistance</td>
<td>1278 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>STIFFENER DETAILS</th>
<th>Plate</th>
<th>2No/80 mm x 15 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welding</td>
<td>4No/6 mm FW's</td>
<td>Both flanges restrained against rotation in plane of stiffener.</td>
</tr>
</tbody>
</table>
Location: Ex2 - End stiffener (Plate Girder)

Load bearing stiffener

Calculations are in accordance with BS EN 1993-1-1:2005. References to EC3 Part 1-5 and EC3 Part 1-8 are also made where appropriate.

Force applied through flange \( V_{zEd} = 1560 \text{ kN} \)

Section properties

<table>
<thead>
<tr>
<th>Plate Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of section ( h = 1500 \text{ mm} )</td>
</tr>
<tr>
<td>Width of section ( b = 450 \text{ mm} )</td>
</tr>
<tr>
<td>Thickness of flange ( t_f = 40 \text{ mm} )</td>
</tr>
<tr>
<td>Thickness of web ( t_w = 15 \text{ mm} )</td>
</tr>
<tr>
<td>Dimensions (mm) : ( h = 1500 ), ( b = 450 ), ( t_f = 40 ), ( t_w = 15 )</td>
</tr>
<tr>
<td>Depth between flanges : ( h_w = h - 2t_f = 1420 \text{ mm} )</td>
</tr>
<tr>
<td>Stiff bearing length ( s_s = 75 \text{ mm} )</td>
</tr>
<tr>
<td>Stiff bearing position ( C = 0 \text{ mm} )</td>
</tr>
</tbody>
</table>

Check for bearing on the web

Distance to nearer end of the member from the end of stiff bearing \( b_e = 0 \text{ mm} \)

Design strength of web \( f_{yw} = 275 \text{ N/mm}^2 \)

Bearing resistance of web \( F_{Rdbr} = (s_s + n k) * t_w * f_{yw} / (\gamma M_0 * 10^3) = 639.38 \text{ kN} \)

Since \( V_{zEd} > F_{Rdbr} \) (1560 kN > 639.38 kN) a bearing stiffener is required.

Stiffener design

Assuming initially that the stiffener will be less than or equal to 16 mm thick, two stiffeners are to be used and allowing for a cut-back of 15 mm at the root of the member.

Design strength of stiffener \( f_{ys} = 275 \text{ N/mm}^2 \)

Assumed stiffener thickness \( t_s = 16 \text{ mm} \)

Assumed stiffener width \( b_s = 150 \text{ mm} \)

Allowable stiffener outstand \( \text{out} = 13t_s \text{e} = 192.28 \text{ mm} \)

As \( b_s \leq \text{out} \) (150 mm ≤ 192.28 mm) the actual stiffener outstand is less than the allowable, hence OK.

The effective section of the stiffener, is shown.
Check stiffener as a bearing stiffener

Force on bearing stiffener  \[ F_{EdB} = V_{zEd} - F_{RdBr} = 920.63 \text{ kN} \]
Bearing resistance of stiffener  \[ F_{RdB} = f_{ys} \times A_{stiff} / (\gamma M0 \times 10^3) = 1188 \text{ kN} \]
Since \( F_{EdB} \geq F_{RdB} \), stiffener is OK for bearing.

Connection of stiffeners to web

Design strength of fillet weld  \[ f_{wvd} = f_u / (\sqrt{3} \times \beta_{tw} \times \gamma M2) = 222.79 \text{ N/mm}^2 \]
Design load on weld  \[ F_{wEd} = F_{EdB} / (2 \times \ell_w) = 0.33624 \text{ kN/mm} \]
Fillet weld size (leg length)  \( s = 6 \text{ mm} \)
Design weld resistance  \[ F_{wRd} = 0.7 \times s \times f_{wvd} / 1000 = 0.93571 \text{ kN/mm} \]
Hence, 6 mm FW’s are suitable and will be adopted.

PLATE GIRDER

DESIGN SUMMARY  1500 mm x 450 mm Section
Steel Grade  S 275
External load  1560 kN
Bearing resistance  639.4 kN
Buckling resistance  2050 kN

STIFFENER DETAILS  Plate  315 mm x 16 mm
Welding  2No/6 mm FW’s
Both flanges restrained against rotation in plane of stiffener.
Location: Ex3 - Check load bearing stiffener (End stiffener)

Load bearing stiffener

Calculations are in accordance with BS EN 1993-1-1:2005. References to EC3 Part 1-5 and EC3 Part 1-8 are also made where appropriate.

Force applied through flange \( V_{zEd} = 800 \) kN

Section properties

254 x 254 x 73 UKC

Stiffener design

Assuming initially that the stiffener will be less than or equal to 16 mm thick, two stiffeners are to be used and allowing for a cut-back of 15 mm at the root of the member.

Design strength of stiffener \( f_{ys} = 275 \) N/mm²
Design strength of web \( f_{yw} = 275 \) N/mm²
Bearing resistance \( F_{Rdbr} = (ss+2k)*tw*fyw/(\gamma_M0*10^3) \)
\( = 127.24 \) kN

Assumed stiffener thickness \( ts = 16 \) mm
Assumed stiffener width \( bs = 100 \) mm
Allowable stiffener outstand \( out = 13*ts*e = 192.28 \) mm
As \( bs \leq out \) (100 mm \( \leq 192.28 \) mm) the actual stiffener outstand is less than the allowable, hence OK.

The effective section of the stiffener, is shown.

Check stiffener as a bearing stiffener

Assuming a stiff bearing length \( ss = 0 \) mm
Permissible stress of web \( f_{yw} = 275 \) N/mm²
Bearing resistance of web \( F_{Rdbr} = (ss+n*k)*tw*fyw/(\gamma_M0*10^3) \)
\( = 254.47 \) kN

Force on bearing stiffener \( F_{EdB} = V_{zEd} - F_{Rdbr} = 545.53 \) kN
Bearing resistance of stiffener \( F_{RdB} = f_{ys}*A_{stiff}/(\gamma_M0*10^3) = 748 \) kN
Since \( F_{EdB} \geq F_{RdB} \), stiffener is OK for bearing.

Connection of stiffeners to web

Design strength of fillet weld \( f_{wvd} = fu/(SQR(3) * betw * \gamma_M2) \)
\( = 222.79 \) N/mm²
Design load on weld \( F_{wEd} = F_{EdB}/(4*lw) = 0.74258 \) kN/mm
Fillet weld size (leg length) \( s = 8 \) mm
Design weld resistance \( F_{wRd} = 0.7*s*f_{wvd}/1000 = 1.2476 \) kN/mm

Hence, 8 mm FW's are suitable and will be adopted.
DESIGN SUMMARY

254 x 254 x 73 UC
Steel Grade S 275
External load 800 kN

STIFFENER DETAILS
Plate 2No/100 mm x 16 mm
Welding 4No/8 mm FW's
Both flanges restrained against rotation in plane of stiffener.
Location: Ex1 - Equal area in parent section flange

Properties of haunched section

Universal beam or column section with a haunch. Parent section and haunch are assumed to be of the same steel grade. Calculations relating to the welding of the haunch to the parent section are not included.

Section properties - Parent section

406 x 178 x 54 UB.
Dimensions (mm): D=402.6 B=177.7 t=7.7 T=10.9 r=10.2
Properties (cm): Iz=18700 Iy=1020 Sz=1060 Sy=178 J=23.1 A=69
Elastic modulus about major axis Z=Iz/(D/20)=928.96 cm³

Section properties - haunch cutting

457 x 191 x 67 UB.
Dimensions (mm): Da=453.4 Ba=189.9 ta=8.5 Ta=12.7 ra=10.2
Properties (cm): Aa=85.5
Depth between root fillets da=Da-2*(ra+Ta)=407.6 mm

Combined properties

Overall depth of section Dh=750 mm
Depth of haunch section Dc=Dh-D=347.4 mm
Neutral axis depth to top flange ynat=SigZ/At=39.641 cm
Neutral axis depth to bot.flange ynab=Dh/10-ynat=35.359 cm
Location: Ex2 - Equal area in haunch

Properties of haunched section

Universal beam or column section with a haunch. Parent section and haunch are assumed to be of the same steel grade. Calculations relating to the welding of the haunch to the parent section are not included.

Section properties - Parent section

406 x 178 x 54 UB.
Dimensions (mm): D=402.6 B=177.7 t=7.7 T=10.9 r=10.2
Properties (cm): Iz=18700 Iy=1020 Sz=1060 Sy=178 J=23.1
A=69
Elastic modulus about major axis \( Z = \frac{I_z}{(D/20)} = 928.96 \text{ cm}^3 \)

Section properties - haunch cutting

533 x 210 x 101 UB.
Dimensions (mm): Da=536.7 Ba=210 ta=10.8 Ta=17.4 ra=12.7
Properties (cm): Aa=129
Depth between root fillets \( d_a = D_a - 2*(r_a + T_a) = 476.5 \text{ mm} \)

Combined properties

Overall depth of section \( D_h = 780 \text{ mm} \)
Depth of haunch section \( D_c = D_h - D = 377.4 \text{ mm} \)
Neutral axis depth to top flange \( y_{nat} = \frac{S_y}{A} = 44.965 \text{ cm} \)
Neutral axis depth to bot.flange \( y_{nab} = \frac{D_h}{10} - y_{nat} = 33.035 \text{ cm} \)
Location: Ex3 - Equal area in parent section web

Properties of haunched section

Universal beam or column section with a haunch. Parent section and haunch are assumed to be of the same steel grade. Calculations relating to the welding of the haunch to the parent section are not included.

Section properties - Parent section

406 x 178 x 54 UB.
Dimensions (mm): D=402.6 B=177.7 t=7.7 T=10.9 r=10.2
Properties (cm): Iz=18700 Iy=1020 Sz=1060 Sy=178 J=23.1 A=69
Elastic modulus about major axis Z=Iz/(D/20)=928.96 cm³

Section properties - haunch cutting

356 x 171 x 45 UB.
Dimensions (mm): Da=351.4 Ba=171.1 ta=7 Ta=9.7 ra=10.2
Properties (cm): Aa=57.3
Depth between root fillets da=Da-2*(ra+Ta)=311.6 mm

Combined properties

Overall depth of section Dh=550 mm
Depth of haunch section Dc=Dh-D=147.4 mm
Neutral axis depth to top flange ynat=SigZ/At=28.973 cm
Neutral axis depth to bot.flange ynab=Dh/10-ynat=26.027 cm
Location: Ex1 - Equal area in parent section flange

**Properties of haunched section**

Universal beam or column section with a haunch.

Parent section and haunch are assumed to be of the same steel grade.

Calculations relating to the welding of the haunch to the parent section are not included.

**Section properties - parent section**

406 x 178 x 54 UKB

Dimensions (mm): h=402.6 b=177.7 tw=7.7 tf=10.9 r=10.2

Properties (cm): Iy=18700 Iz=1020 Wply=1060 Wplz=178

It=23.1 A=69 Wely=928.96

**Section properties - haunch cutting**

457 x 191 x 67 UKB

Dimensions (mm): ha=453.4 ba=189.9 twa=8.5 tfa=12.7 ra=10.2

Properties (cm): Aa=85.5

Depth between root fillets da=ha-2*(ra+tfa)=407.6 mm

**Combined properties**

Overall depth of section ho=750 mm

Depth of haunch section hh=ho-h=347.4 mm

Neutral axis depth to top flange znat=SigZ/At=39.641 cm

Neutral axis depth to bot.flange znab=ho/10-znat=35.359 cm
Location: Ex2 - Equal area in haunch

Properties of haunched section

Universal beam or column section with a haunch.

Parent section and haunch are assumed to be of the same steel grade.

Calculations relating to the welding of the haunch to the parent section are not included.

Section properties - parent section

406 x 178 x 54 UKB
Dimensions (mm): h=402.6 b=177.7 tw=7.7 tf=10.9 r=10.2
Properties (cm): Iy=18700 Iz=1020 Wply=1060 Wplz=178
It=23.1 A=69 Wely=928.96

Section properties - haunch cutting

533 x 210 x 101 UKB
Dimensions (mm): ha=536.7 ba=210 twa=10.8 tfa=17.4 ra=12.7
Properties (cm): Aa=129
Depth between root fillets da=ha-2*(ra+tfa)=476.5 mm

Combined properties

Overall depth of section ho=780 mm
Depth of haunch section hh=ho-h=377.4 mm
Neutral axis depth to top flange znat=SigZ/At=44.965 cm
Neutral axis depth to bot.flange znab=ho/10-znat=33.035 cm
Location: Ex3 - Equal area in parent section web

Properties of haunched section

Universal beam or column section with a haunch.

Parent section and haunch are assumed to be of the same steel grade.

Calculations relating to the welding of the haunch to the parent section are not included.

Section properties - parent section

406 x 178 x 54 UKB
Dimensions (mm): h=402.6 b=177.7 tw=7.7 tf=10.9 r=10.2
Properties (cm): Iy=18700 Iz=1020 Wply=1060 Wplz=178
It=23.1 A=69 Wely=928.96

Section properties - haunch cutting

356 x 171 x 45 UKB
Dimensions (mm): ha=351.4 ba=171.1 twa=7 tfa=9.7 ra=10.2
Properties (cm): Aa=57.3
Depth between root fillets da=ha-2*(ra+tfa)=311.6 mm

Combined properties

Overall depth of section ho=550 mm
Depth of haunch section hh=ho-h=147.4 mm
Neutral axis depth to top flange znat=SigZ/At=28.973 cm
Neutral axis depth to bot.flange znab=ho/10-znat=26.027 cm
Bolted connection

Calculations in accordance with BS5950-1:2000. Fasteners are subject to both shear and torsion. Fasteners are in the plane of the load.

Axial force $F_v = 0$ kN
Eccentric force $F = 212$ kN
Eccentricity of force $ecc = 525$ mm
Bolt diameter $bd = 24$ mm
Bolt pitch (vertically) $p = 70$ mm
Distance between bolt centres $g = 500$ mm

**BOLT SUMMARY**

- Bolt diameter: 24 mm
- Number: 12
- Bolt grade: 4.6
Location: Example preloaded bolts shear and torsion

Bolted connection

Calculations in accordance with BS5950-1:2000. Fasteners are subject to both shear and torsion. Fasteners are in the plane of the load.

- Axial force: $F_v = 0$ kN
- Eccentric force: $F = 280$ kN
- Eccentricity of force: $ecc = 650$ mm
- Bolt diameter: $bd = 20$ mm
- Bolt pitch (vertically): $p = 70$ mm
- Distance between bolt centres: $S_1 = 120$ mm
- Distance between bolt centres: $S_2 = 90$ mm
- Slip factor for preloaded bolts: $\mu = 0.5$
- Coefficient for type of hole: $K_s = 1$

BOLT SUMMARY

- Bolt diameter: 20 mm
- Number: 20
- Bolt grade: HSFG
  - Higher grade to BS4395-2
- Nut grade: 12
Location: Example HSFG not preloaded shear and tension

Bolted connection

Calculations in accordance with BS5950-1:2000. Fasteners are subject to both shear and tension. Fasteners are not in the plane of the load.

Applied shear force \( F_v = 147 \text{ kN} \)

Applied moment \( M = 257 \text{ kNm} \)

Bolt diameter \( b_d = 24 \text{ mm} \)

Bolt pitch (vertically) \( p = 90 \text{ mm} \)

Distance between bolt centres \( g = 140 \text{ mm} \)

below the bottom bolts \( a = 60 \text{ mm} \)

BOLT SUMMARY

```
  o  o
  o  o
  o  o
  o  o
  o  o
```

Bolt diameter 24 mm
Number 10
Bolt grade HSFG
Nut grade 10

General grade to BS4395-1
Location: Example HSFG preloaded shear and tension

**Bolted connection**

Calculations in accordance with BS5950-1:2000. Fasteners are subject to both shear and tension. Fasteners are not in the plane of the load.

- **Applied shear force** \( F_v = 247 \, \text{kN} \)
- **Applied moment** \( M = 657 \, \text{kNm} \)
- **Bolt diameter** \( b_d = 24 \, \text{mm} \)
- **Bolt pitch (vertically)** \( p = 90 \, \text{mm} \)
- **Distance between bolt centres** \( S_1 = 100 \, \text{mm} \)
- **Distance between bolt centres** \( S_2 = 140 \, \text{mm} \)
- **Below the bottom bolts** \( a = 60 \, \text{mm} \)
- **Slip factor for preloaded bolts** \( \mu = 0.5 \)
- **Coefficient for type of hole** \( K_s = 1 \)

**BOLT SUMMARY**

- **Bolt diameter** 24 mm
- **Number** 20
- **Bolt grade** HSFG
  - Higher grade to BS4395-2
- **Nut grade** 12
Bolted connection

Calculations are in accordance with EC3 Part 1-1. Fasteners are subject to both shear and torsion. Fasteners are in the plane of the load.

Axial force \( F_v = 0 \) kN

Eccentric force \( F = 212 \) kN

Eccentricity of force \( \text{ecc} = 525 \) mm

Bolt diameter \( \text{db} = 24 \) mm

Bolt pitch (vertically) \( \text{p1} = 70 \) mm

Distance between bolt centres \( \text{p3} = 140 \) mm

Plate thickness \( \text{tp} = 15 \) mm

BOLT SUMMARY - all dimensions in diagram are in mm

Bolt diameter 24 mm

Number of bolts 12

Bolt grade 8.8
Location: Ex2 - Grade 8.8 bolts shear and torsion

Bolted connection

Calculations are in accordance with EC3 Part 1-1. Fasteners are subject to both shear and torsion. Fasteners are in the plane of the load.

- Applied shear force       $F_v = 150 \text{ kN}$
- Applied torsional moment  $M = 180 \text{ kNm}$
- Bolt diameter             $d_b = 20 \text{ mm}$
- Bolt pitch (vertically)   $p_1 = 70 \text{ mm}$
- Distance between bolt centres $S_1 = 120 \text{ mm}$
- Distance between bolt centres $S_2 = 90 \text{ mm}$
- Plate thickness           $t_p = 15 \text{ mm}$

BOLT SUMMARY - all dimensions in diagram are in mm

- Bolt diameter             20 mm
- Number of bolts            20
- Bolt grade                8.8
Location: Ex3 - Grade 8.8 bolts shear and tension

Bolted connection

Calculations in accordance with BS5950-1:2000. Fasteners are subject to both shear and tension. Fasteners are not in the plane of the load.

- Applied shear force \( F_v = 147 \text{ kN} \)
- Applied moment \( M = 257 \text{ kNm} \)
- Bolt diameter \( d_b = 24 \text{ mm} \)
- Bolt pitch (vertically) \( p_1 = 90 \text{ mm} \)
- Distance between bolt centres \( p_3 = 140 \text{ mm} \)
- Plate thickness \( t_p = 15 \text{ mm} \)

below the bottom bolts \( a = 60 \text{ mm} \)

BOLT SUMMARY - all dimensions in diagram are in mm

- Bolt diameter \( 24 \text{ mm} \)
- Number of bolts \( 10 \)
- Bolt grade \( 8.8 \)
Location: Ex4 - Grade 8.8 bolts shear and tension

Bolted connection

Calculations in accordance with BS5950-1:2000. Fasteners are subject to both shear and tension. Fasteners are not in the plane of the load.

Applied shear force \( F_v = 247 \text{ kN} \)

Applied moment \( M = 657 \text{ kNm} \)

Bolt diameter \( d_b = 24 \text{ mm} \)

Bolt pitch (vertically) \( p_1 = 90 \text{ mm} \)

Distance between bolt centres \( S_1 = 100 \text{ mm} \)

Distance between bolt centres \( S_2 = 140 \text{ mm} \)

Plate thickness \( t_p = 15 \text{ mm} \)

below the bottom bolts \( a = 60 \text{ mm} \)

BOLT SUMMARY - all dimensions in diagram are in mm

<table>
<thead>
<tr>
<th>S2</th>
<th>S1</th>
<th>S2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Bolt diameter: 24 mm

Number of bolts: 20

Bolt grade: 8.8
Location: Unrestrained notched beam to EC3 - top notch

Partial depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3 and BS EN 1993-1: 2005.

Universal section to universal section bolted connection.
Factored end reaction $V_{Ed}=200$ kN

Supporting member details

457 x 191 x 82 UKB
Thickness of connecting ply $ts=9.9$ mm

Supported beam details

356 x 127 x 33 UKB

<table>
<thead>
<tr>
<th>ln</th>
<th>dnt</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>tp</td>
</tr>
<tr>
<td></td>
<td></td>
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<tr>
<td>/</td>
<td>/</td>
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<tr>
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</tr>
<tr>
<td>/</td>
<td></td>
</tr>
</tbody>
</table>

Length of notch to end face $ln=110$ mm
Depth of notch $dnt=50$ mm
Thickness of end plate $tp=10$ mm
Length of end plate $hp=290$ mm
Reduced depth of sectn.at notch $hn=299$ mm

Design moment about y-y axis $My_{Ed}=40$ kNm
Length of supported beam $L=5000$ mm
Coefficient $C_1=1.13$

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of section</td>
<td>349 mm</td>
</tr>
<tr>
<td>Length of notch</td>
<td>110 mm</td>
</tr>
<tr>
<td>Depth of notch</td>
<td>50 mm</td>
</tr>
<tr>
<td>Yield strength of steel</td>
<td>275 N/mm$^2$</td>
</tr>
<tr>
<td>Factored end reaction</td>
<td>200 kN</td>
</tr>
<tr>
<td>Shear resistance</td>
<td>248.64 kN</td>
</tr>
<tr>
<td>Classification</td>
<td>Class 1</td>
</tr>
<tr>
<td>Design moment (y-y axis)</td>
<td>40 kNm</td>
</tr>
<tr>
<td>Design buckling resistance</td>
<td>42.113 kNm</td>
</tr>
<tr>
<td>Unity factor</td>
<td>0.94984</td>
</tr>
</tbody>
</table>

SCALE 5.48 Office 1007 Proforma 499
Location: Restrained notched beam to EC3 - top notch

Partial depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3 and BS EN 1993-1: 2005.

Universal section to universal section bolted connection.

Factored end reaction $V_{Ed}=200$ kN

Supporting member details

457 x 191 x 82 UKB
 Thickness of connecting ply $t_s=9.9$ mm

Supported beam details

356 x 127 x 33 UKB

\[
\begin{array}{c}
\text{ln} \\
\text{tp} \\
/ \\
/ \\
/ \\
/ \\
\text{dnt} \\
/ \\
/ \\
/ \\
\text{hp} \\
\text{hn}
\end{array}
\]

Length of notch to end face $l_n=110$ mm
 Depth of notch $d_{nt}=50$ mm
 Thickness of end plate $t_p=10$ mm
 Length of end plate $h_{p}=290$ mm
 Reduced depth of section at notch $h_{n}=h-d_{nt}=299$ mm

| DESIGN | Depth of section 349 mm |
| SUMMARY | Length of notch 110 mm |
|         | Depth of notch 50 mm |
|         | Yield strength of steel 275 N/mm² |
|         | Factored end reaction 200 kN |
|         | Shear resistance 248.64 kN |
Location: Unrestrained beam to EC3 - notches top and bottom

Partial depth end plate

Analysis of the connection follows the treatment in the BCSA/SCI publication 'Joints in Steel Construction: Simple Joints to EC3 and BS EN 1993-1: 2005.

Universal section to universal section bolted connection.
Factored end reaction \( V_{Ed} = 160 \text{ kN} \)

Supporting member details

457 x 191 x 82 UKB
Thickness of connecting ply \( t_s = 9.9 \text{ mm} \)

Supported beam details

356 x 127 x 39 UKB

\[
\begin{array}{c|c|c}
| \text{ln} & \text{dnt} & \text{tp} \\
/ & / & / \\
/ & / & \text{tp = end plate thickness} \\
/ & / & \text{hn} \\
/ & / & \text{hp = end plate length} \\
/ & / & \text{dnb} \\
\end{array}
\]

Length of notch to end face \( ln = 110 \text{ mm} \)
Depth of notch \( dnt = 50 \text{ mm} \)
Depth of lower notch \( dnb = 50 \text{ mm} \)
Thickness of end plate \( tp = 10 \text{ mm} \)
Reduced depth of sectn.at notch \( hn = h - dnt - dnb = 253.4 \text{ mm} \)
Length of end plate \( hp = 253.4 \text{ mm} \)
Design moment about y-y axis \( My_{Ed} = 40 \text{ kNm} \)
Length of supported beam \( L = 5000 \text{ mm} \)
Coefficient of supported beam \( C_1 = 1.13 \)

DESIGN

Depth of section \( 353.4 \text{ mm} \)

SUMMARY

Depth of notch \( 110 \text{ mm} \)
Depth of notch \( 50 \text{ mm} \)
Yield strength of steel \( 275 \text{ N/mm}^2 \)
Factored end reaction \( 160 \text{ kN} \)
Shear resistance \( 238.98 \text{ kN} \)
Classification Class 1
Design moment (y-y axis) \( 40 \text{ kNm} \)
Design buckling resistance \( 48.918 \text{ kNm} \)
Unity factor \( 0.81769 \)
Location: Cavity wall spanning horizontally adjacent to an opening

Cavity wall spanning horizontally

Wall design to BS5628-1:2005

Wall section between windows will be considered as a cantilever spanning horizontally. The window is assumed to be spanning horizontally transferring wind loading to the end of the cantilever. Between the windows is assumed to be a wall, a column or a wind post secured to floors. The wall sections above and below the windows are beyond the scope of this program.

Length of cantilever \( L = 0.5 \text{ m} \)
Thickness of outer skin \( t_1 = 100 \text{ mm} \)
Thickness of inner skin \( t_2 = 140 \text{ mm} \)
Mortar designation to be used \( \text{mort} = 3 \)
Blockwork compressive strength \( \text{bcs}_1 = 7.3 \text{ N/mm}^2 \)
Outer skin flexural strength \( \text{fkx}_1 = \text{TABLE 3.8 for bcs}_1 = 7.3, \text{mort} = 3 = 0.6 \text{ N/mm}^2 \)

Inner skin is blockwork:
Blockwork compressive strength \( \text{bcs}_2 = 7.3 \text{ N/mm}^2 \)
Inner skin flexural strength \( \text{fkx}_2 = 0.53333 \text{ N/mm}^2 \)
Width of window \( \text{width} = 1.5 \text{ m} \)
Wind loading on wall \( w_2 = 0.4 \text{ kN/m}^2 \)
Outer skin flexural strength \( \text{fkx}_1 = 0.6 \text{ N/mm}^2 \)
Inner skin flexural strength \( \text{fkx}_2 = 0.53333 \text{ N/mm}^2 \)
Partial safety factor of material \( \gamma_m = 3 \)
Wall design moment of resistance \( Mr = mrl + mr2 = 0.91407 \text{ kNm} \)

Cavity wall spanning horizontally

The wall moment of resistance is greater than moment on the wall and therefore the wall is satisfactory for the proposed lateral load of 0.4 kN/m²

Adopt an outer skin of blockwork 100 mm
Adopt an inner skin of blockwork 140 mm
Mortar designation to be used 3
Moment of resistance 0.91407 kNm
Ultimate Moment on wall 0.24 kNm
Factor of safety for wind load 1.2
Location: Cavity wall spanning horizontally adjacent to an opening

Cavity wall spanning horizontally

![Wall elevation diagram]

Wall design to BS5628-1:2005

Wall section between windows will be considered as a cantilever spanning horizontally. The window is assumed to be spanning horizontally transferring wind loading to the end of the cantilever. Between the windows is assumed to be a wall, a column or a wind post secured to floors. The wall sections above and below the windows are beyond the scope of this program.

Length of cantilever
L = 0.75 m

Thickness of outer skin
t1 = 100 mm

Thickness of inner skin
t2 = 140 mm

Mortar designation to be used
mort = 3

Water absorption of bricks
wa = 13

Outer skin flexural strength
fkx1 = TABLE 3.2 for wa = 13, mort = 3 = 0.9 N/mm²

Inner skin is blockwork:
Blockwork compressive strength
bcs2 = 7.3 N/mm²

Inner skin flexural strength
fkx2 = 0.53333 N/mm²

Width of window
width = 1.5 m

Wind loading on wall
w2 = 0.4 kN/m²

Outer skin flexural strength
fkx1 = 0.9 N/mm²

Inner skin flexural strength
fkx2 = 0.53333 N/mm²

Partial safety factor of material
gamma_m = 3

Wall design moment of resistance
Mr = Mr1 + Mr2 = 1.0807 kNm

Cavity wall spanning horizontally

The wall moment of resistance is greater than moment on the wall and therefore the wall is satisfactory for the proposed lateral load of 0.4 kN/m²

Adopt an outer skin of brickwork
100 mm

Water absorption of bricks
13 %

Adopt an inner skin of blockwork
140 mm

Mortar designation to be used
3

Moment of resistance
1.0807 kNm

Ultimate Moment on wall
0.405 kNm

Factor of safety for wind load
1.2
Location: Cavity wall spanning horizontally adjacent to an opening

Cavity wall spanning horizontally

Wall design to EC6 Part 1-1

Wall section between windows will be considered as a cantilever spanning horizontally. The window is assumed to be spanning horizontally transferring wind loading to the cantilever. Between the windows is assumed to be a wall, a column or a wind post secured to floors. The wall sections above and below the windows are beyond the scope of this program.

Length of cantilever  
L=0.5 m

Thickness of outer skin  
t1=100 mm

Thickness of inner skin  
t2=140 mm

Mortar designation to be used  
mort=3

The outer skin is blockwork:
Compressive strength of units  
fb1=7.3 N/mm²

Outer skin flexural strength  
f(xk)2=TABLE 6.8 for fb=7.3, mort=3  
=0.6 N/mm²

Inner skin is blockwork:
Compressive strength of units  
fb2=7.3 N/mm²

Inner skin flexural strength  
f(xk)2'=0.53333 N/mm²

Width of window  
width=1.5 m

Wind loading on wall  
Qkw=0.4 kN/m²

Partial safety factor (material)  
γM=2.4

Wall design moment of resistance  
Mr=mr1+mr2=1.1426 kNm

SUMMARY OF RESULTS

The wall moment of resistance is greater than moment on the wall and therefore the wall is satisfactory for the proposed lateral load of 0.4 kN/m²

Adopt an outer skin of blockwork  
100 mm

Adopt an inner skin of blockwork  
140 mm

Mortar designation to be used  
3

Design moment of resistance  
1.1426 kNm

Design moment on wall  
0.3 kNm

Factor of safety for wind load  
1.5
Location: Cavity wall spanning horizontally adjacent to an opening

Cavity wall spanning horizontally

Wall design to EC6 Part 1-1

Wall section between windows will be considered as a cantilever spanning horizontally. The window is assumed to be spanning horizontally transferring wind loading to the cantilever. Between the windows is assumed to be a wall, a column or a wind post secured to floors. The wall sections above and below the windows are beyond the scope of this program.

Length of cantilever \( L = 0.75 \) m
Thickness of outer skin \( t_1 = 100 \) mm
Thickness of inner skin \( t_2 = 140 \) mm
Mortar designation to be used \( \text{mort} = 3 \)

Outer skin is brickwork:
Water absorption of bricks \( w_a = 13 \% \)
Outer skin flexural strength \( f_{xk2} = \text{TABLE 6.2 for } w_a = 13, \text{mort} = 3 \)
= 0.9 N/mm²

Inner skin is blockwork:
Compressive strength of units \( f_b = 7.3 \) N/mm²
Inner skin flexural strength \( f_{xk2}' = 0.53333 \) N/mm²
Width of window \( w_e = 1.5 \) m
Wind loading on wall \( Q_{kw} = 0.4 \) kN/m²
Partial safety factor (material) \( \gamma_m = 2.4 \)
Wall design moment of resistance \( M_{rd} = m_r1 + m_r2 = 1.3509 \) kNm

SUMMARY OF RESULTS

The wall moment of resistance is greater than moment on the wall and therefore the wall is satisfactory for the proposed lateral load of 0.4 kN/m²

Adopt an outer skin of brickwork 100 mm
Water absorption of bricks 13 %
Adopt an inner skin of blockwork 140 mm
Mortar designation to be used 3
Design moment of resistance 1.3509 kNm
Design moment on wall 0.50625 kNm
Factor of safety for wind load 1.5
Location: Wall on grid line A/1-2

Length of wall panel \( L = 4 \) m
Height of wall panel \( h = 3 \) m
Outer leaf density \( \text{den}(1) = 10 \) kN/m\(^2\)
Thickness of outer leaf \( t(1) = 100 \) mm
Inner leaf density \( \text{den}(2) = 10 \) kN/m\(^2\)
Thickness of inner leaf \( t(2) = 100 \) mm
Mortar designation (Table 1) \( \text{mortar} = 3 \)
Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block) \( \text{typ}(1) = 4 \)
Compressive strength \( \text{po}(1) = 10.4 \) N/mm\(^2\)
Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block) \( \text{typ}(2) = 4 \)
Compressive strength \( \text{po}(2) = 10.4 \) N/mm\(^2\)
Partial safety factor materials \( \gamma_{m} = 3 \)

Characteristic wind load per unit area \( WL = 0.85 \) kN/m\(^2\)
Additional vert DL on panel \( w(1) = 0 \) kN/m
Additional vert DL on panel \( w(2) = 0 \) kN/m

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Panel length</th>
<th>4 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel height</td>
<td>3 m</td>
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</tbody>
</table>

**OUTER LEAF**

<table>
<thead>
<tr>
<th>Thickness</th>
<th>100 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char flex strength (bed)</td>
<td>0.25 N/mm(^2)</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
<td>0.75 N/mm(^2)</td>
</tr>
</tbody>
</table>

**INNER LEAF**

<table>
<thead>
<tr>
<th>Thickness</th>
<th>100 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Char flex strength (bed)</td>
<td>0.25 N/mm(^2)</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
<td>0.75 N/mm(^2)</td>
</tr>
<tr>
<td>Effective wall thickness</td>
<td>133.33 mm</td>
</tr>
<tr>
<td>Char wind pressure</td>
<td>0.85 kN/m(^2)</td>
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<tr>
<td>Char wind capacity (bend)</td>
<td>0.98078 kN/m(^2)</td>
</tr>
<tr>
<td>Design shear on sides</td>
<td>6.12 kN</td>
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<td>Shear strength of sides</td>
<td>84 kN</td>
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<tr>
<td>Design shear on founds</td>
<td>12.24 kN</td>
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<tr>
<td>Shear strength of founds</td>
<td>53.184 kN</td>
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<td>Material safety factor</td>
<td>3</td>
</tr>
<tr>
<td>Load safety factor</td>
<td>1.2</td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

Length of wall panel $L=2\ m$
Height of wall panel $h=1.5\ m$
Outer leaf density $\text{den}(1)=10\ \text{kN/m}^2$
Thickness of outer leaf $t(1)=150\ \text{mm}$
Mortar designation (Table 1) $\text{mortar}=1$
Outer leaf (1=clay 2=calc silicate 3=concrete brick 4=block) $\text{typ}(1)=1$
Water absorption %age of leaf $\text{water1}=7$
Partial safety factor materials $\gamma_{\text{m}}=3$

Characteristic wind load per unit area $WL=0.85\ \text{kN/m}^2$
Additional vert DL on panel $w(1)=10\ \text{kN/m}$

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Panel length</td>
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<tr>
<td>Panel height</td>
<td>1.5 m</td>
</tr>
<tr>
<td>Thickness</td>
<td>150 mm</td>
</tr>
<tr>
<td>Char flex strength (bed)</td>
<td>0.5 N/mm$^2$</td>
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<tr>
<td>Char flex strength (perp)</td>
<td>1.5 N/mm$^2$</td>
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<tr>
<td>Char wind pressure</td>
<td>0.85 kN/m$^2$</td>
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<tr>
<td>Char wind capacity (bend)</td>
<td>7.0176 kN/m$^2$</td>
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<tr>
<td>Design shear on sides</td>
<td>1.53 kN</td>
</tr>
<tr>
<td>Shear strength of sides</td>
<td>63 kN</td>
</tr>
<tr>
<td>Design shear on founds</td>
<td>3.06 kN</td>
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<tr>
<td>Shear strength of founds</td>
<td>47.292 kN</td>
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<tr>
<td>Material safety factor</td>
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</tr>
<tr>
<td>Load safety factor</td>
<td>1.2</td>
</tr>
</tbody>
</table>
**Location:** Wall on grid line A/1-2

- Length of wall panel: $L = 10$ m
- Height of wall panel: $h = 3$ m
- Outer leaf density: $\text{den}(1) = 3$ kN/m$^3$
- Thickness of outer leaf: $t(1) = 100$ mm
- Inner leaf density: $\text{den}(2) = 20$ kN/m$^3$
- Thickness of inner leaf: $t(2) = 450$ mm
- Mortar designation (Table 1): $\text{mortar} = 2$
- Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block): $\text{typ}(1) = 2$
- Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block): $\text{typ}(2) = 4$
- Compressive strength: $p(2) = 60$ N/mm$^2$
- Partial safety factor materials: $\gamma_m = 3$

Characteristic wind load per unit area: $WL = 0.85$ kN/m$^2$

Additional vert DL on panel:
- $w(1) = 0$ kN/m
- $w(2) = 0$ kN/m

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Panel length</th>
<th>10 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel height</td>
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<td>3 m</td>
</tr>
<tr>
<td><strong>OUTER LEAF</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thickness</td>
<td></td>
<td>100 mm</td>
</tr>
<tr>
<td>Char flex strength (bed)</td>
<td></td>
<td>0.3 N/mm$^2$</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
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<td>0.9 N/mm$^2$</td>
</tr>
<tr>
<td><strong>INNER LEAF</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thickness</td>
<td></td>
<td>450 mm</td>
</tr>
<tr>
<td>Char flex strength (bed)</td>
<td></td>
<td>0.25 N/mm$^2$</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
<td></td>
<td>0.9 N/mm$^2$</td>
</tr>
<tr>
<td>Effective wall thickness</td>
<td></td>
<td>450 mm</td>
</tr>
<tr>
<td>Char wind pressure</td>
<td></td>
<td>0.85 kN/m$^2$</td>
</tr>
<tr>
<td>Char wind capacity (bend)</td>
<td></td>
<td>4.2393 kN/m$^2$</td>
</tr>
<tr>
<td>Design shear on sides</td>
<td></td>
<td>17.85 kN</td>
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<tr>
<td>Shear strength of sides</td>
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<tr>
<td>Design shear on founds</td>
<td></td>
<td>17.85 kN</td>
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<td>Shear strength of founds</td>
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<td>Material safety factor</td>
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<tr>
<td>Load safety factor</td>
<td></td>
<td>1.4</td>
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**Location:** Wall on grid line A/1-2

<table>
<thead>
<tr>
<th>Property</th>
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<tbody>
<tr>
<td>Length of wall panel</td>
<td>L=1 m</td>
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<tr>
<td>Height of wall panel</td>
<td>h=0.9 m</td>
</tr>
<tr>
<td>Outer leaf density</td>
<td>den(1)=10 kN/m²</td>
</tr>
<tr>
<td>Thickness of outer leaf</td>
<td>t(1)=150 mm</td>
</tr>
<tr>
<td>Inner leaf density</td>
<td>den(2)=10 kN/m³</td>
</tr>
<tr>
<td>Thickness of inner leaf</td>
<td>t(2)=350 mm</td>
</tr>
<tr>
<td>Mortar designation (Table 1)</td>
<td>mortar=3</td>
</tr>
<tr>
<td>Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block)</td>
<td>typ(1)=3</td>
</tr>
<tr>
<td>Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block)</td>
<td>typ(2)=4</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>po(2)=10.4 N/mm²</td>
</tr>
<tr>
<td>Partial safety factor materials</td>
<td>gammam=3</td>
</tr>
<tr>
<td>Characteristic wind load per unit area</td>
<td>WL=0.85 kN/m²</td>
</tr>
<tr>
<td>Additional vert DL on panel</td>
<td>w(1)=0 kN/m</td>
</tr>
<tr>
<td>Additional vert DL on panel</td>
<td>w(2)=0 kN/m</td>
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</tbody>
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**DESIGN SUMMARY**

<p>| | |</p>
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<thead>
<tr>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>Panel length</td>
<td>1 m</td>
</tr>
<tr>
<td>Panel height</td>
<td>0.9 m</td>
</tr>
<tr>
<td>OUTER LEAF</td>
<td></td>
</tr>
<tr>
<td>Thickness</td>
<td>150 mm</td>
</tr>
<tr>
<td>Char flex strength (bed)</td>
<td>0.3 N/mm²</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
<td>0.9 N/mm²</td>
</tr>
<tr>
<td>INNER LEAF</td>
<td></td>
</tr>
<tr>
<td>Thickness</td>
<td>350 mm</td>
</tr>
<tr>
<td>Char flex strength (bed)</td>
<td>0.25 N/mm²</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
<td>0.75 N/mm²</td>
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<tr>
<td>Effective wall thickness</td>
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<tr>
<td>Char wind pressure</td>
<td>0.85 kN/m²</td>
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<tr>
<td>Char wind capacity (bend)</td>
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<tr>
<td>Shear strength of sides</td>
<td>52.2 kN</td>
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<tr>
<td>Design shear on founds</td>
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<td>Shear strength of founds</td>
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<td>Material safety factor</td>
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<tr>
<td>Load safety factor</td>
<td>1.2</td>
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</table>
Location: Wall on grid line A/1-2

Length of wall panel          L=3 m
Height of wall panel          h=3 m
Outer leaf density           den(1)=10 kN/m²
Thickness of outer leaf      t(1)=150 mm
Mortar designation (Table 1) mortar=4
Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block) typ(1)=4
compressive strength         po(1)=10.4 N/mm²
Partial safety factor materials gammam=3
Characteristic wind load per unit area    WL=0.6 kN/m²
Additional vert DL on panel      w(1)=0 kN/m

DESIGN SUMMARY
Panel length                   3 m
Panel height                   3 m
Thickness                      150 mm
Char flex strength (bed)       0.2 N/mm²
Char flex strength (perp)      0.6 N/mm²
Char wind pressure             0.6 kN/m²
Char wind capacity (bend)      0.61428 kN/m²
Design shear on sides         7.56 kN
Shear strength of sides       63 kN
Design shear on founds        3.78 kN
Shear strength of founds      29.916 kN
Material safety factor        3
Load safety factor            1.4
### Location: Wall on grid line A/1-2

Length of wall panel \( L = 4 \text{ m} \)
Height of wall panel \( h = 3 \text{ m} \)
Outer leaf density \( \text{den}(1) = 10 \text{ kN/m}^2 \)
Thickness of outer leaf \( t(1) = 100 \text{ mm} \)
Inner leaf density \( \text{den}(2) = 10 \text{ kN/m}^2 \)
Thickness of inner leaf \( t(2) = 100 \text{ mm} \)
Mortar designation (Table NA.2) \( \text{mortar} = 3 \)
Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block) \( \text{typ}(1) = 4 \)
Compressive strength of units \( p_o(1) = 10.4 \text{ N/mm}^2 \)
Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block) \( \text{typ}(2) = 4 \)
Compressive strength of units \( p_o(2) = 10.4 \text{ N/mm}^2 \)
Partial safety factor (material) \( \gamma_M = 2.4 \)
Characteristic wind load per unit area \( Q_{kw} = 0.85 \text{ kN/m}^2 \)
Additional vert DL on panel \( w(1) = 0 \text{ kN/m} \)
Height of outer leaf units \( h_{u(1)} = 215 \text{ mm} \)

#### DESIGN SUMMARY

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<th>Panel length</th>
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<tbody>
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<td>Panel height</td>
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<tr>
<td>OUTER LEAF</td>
<td></td>
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<tr>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char flex strength (bed)</td>
<td>0.25 N/mm²</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
<td>0.75 N/mm²</td>
</tr>
<tr>
<td>INNER LEAF</td>
<td></td>
</tr>
<tr>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char flex strength (bed)</td>
<td>0.25 N/mm²</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
<td>0.75 N/mm²</td>
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<tr>
<td>Effective wall thickness</td>
<td>125.93 mm</td>
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<td>Char wind pressure</td>
<td>0.85 kN/m²</td>
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<tr>
<td>Char wind capacity (bend)</td>
<td>0.97602 kN/m²</td>
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<tr>
<td>Design shear on sides</td>
<td>7.65 kN</td>
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<tr>
<td>Shear strength of sides</td>
<td>36 kN</td>
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<tr>
<td>Design shear on founds</td>
<td>15.3 kN</td>
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<td>51.456 kN</td>
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Location: Wall on grid line A/1-2

<table>
<thead>
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<tbody>
<tr>
<td>Length of wall panel</td>
<td>L=2 m</td>
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<tr>
<td>Height of wall panel</td>
<td>h=1.5 m</td>
</tr>
<tr>
<td>Outer leaf density</td>
<td>den(1)=10 kN/m²</td>
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<tr>
<td>Thickness of outer leaf</td>
<td>t(1)=150 mm</td>
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<tr>
<td>Mortar designation (Table NA.2)</td>
<td>mortar=1</td>
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<td>Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block)</td>
<td>typ(1)=1</td>
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<tr>
<td>Water absorption %age of leaf</td>
<td>water1=7</td>
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<tr>
<td>Compressive strength of units</td>
<td>po(1)=10.4 N/mm²</td>
</tr>
<tr>
<td>Partial safety factor (material)</td>
<td>gamM=2.4</td>
</tr>
<tr>
<td>Characteristic wind load per unit area</td>
<td>Qkw=0.85 kN/m²</td>
</tr>
<tr>
<td>Additional vert DL on panel</td>
<td>w(1)=10 kN/m</td>
</tr>
<tr>
<td>Height of the units</td>
<td>hu(1)=215 mm</td>
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</table>

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
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<tbody>
<tr>
<td>Panel length</td>
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<tr>
<td>Panel height</td>
<td>1.5 m</td>
</tr>
<tr>
<td>Thickness</td>
<td>150 mm</td>
</tr>
<tr>
<td>Char flex strength (bed)</td>
<td>0.5 N/mm²</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
<td>1.5 N/mm²</td>
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<tr>
<td>Char wind pressure</td>
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<tr>
<td>Char wind capacity (bend)</td>
<td>6.951 kN/m²</td>
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<tr>
<td>Design shear on sides</td>
<td>1.9125 kN</td>
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<tr>
<td>Shear strength of sides</td>
<td>27 kN</td>
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<tr>
<td>Design shear on founds</td>
<td>3.825 kN</td>
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<tr>
<td>Shear strength of founds</td>
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<td>Material safety factor</td>
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<td>Load safety factor</td>
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</table>
Location: Wall on grid line A/1-2

Length of wall panel          L=10 m
Height of wall panel          h=3 m
Outer leaf density            den(1)=3 kN/m³
Thickness of outer leaf       t(1)=100 mm
Inner leaf density            den(2)=20 kN/m³
Thickness of inner leaf       t(2)=450 mm
Mortar designation (Table NA.2) mortar=2
Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block) typ(1)=2
Compressive strength of units po(1)=20 N/mm²
Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block) typ(2)=4
Compressive strength of units po(2)=60 N/mm²
Partial safety factor (material) gamM=2.7
Characteristic wind load per unit area Qkw=0.85 kN/m²
Additional vert DL on panel   w(1)=0 kN/m
Height of outer leaf units    hu(1)=215 mm
Additional vert DL on panel   w(2)=0 kN/m
Height of inner leaf units    hu(2)=215 mm

DESIGN SUMMARY

Panel length          10 m
Panel height          3 m
OUTER LEAF
Thickness             100 mm
Char flex strength (bed) 0.3 N/mm²
Char flex strength (perp) 0.9 N/mm²
INNER LEAF
Thickness             450 mm
Char flex strength (bed) 0.25 N/mm²
Char flex strength (perp) 0.9 N/mm²
Effective wall thickness 451.36 mm
Char wind pressure     0.85 kN/m²
Char wind capacity (bend) 4.3956 kN/m²
Design shear on sides  19.125 kN
Shear strength of sides 99 kN
Design shear on founds  19.125 kN
Shear strength of founds 370.18 kN
Material safety factor  2.7
Load safety factor      1.5
Location: Wall on grid line A/1-2

Length of wall panel \( L = 1 \text{ m} \)
Height of wall panel \( h = 0.9 \text{ m} \)
Outer leaf density \( \text{den}(1) = 10 \text{ kN/m}^3 \)
Thickness of outer leaf \( t(1) = 150 \text{ mm} \)
Inner leaf density \( \text{den}(2) = 10 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2) = 350 \text{ mm} \)
Mortar designation (Table NA.2) \( \text{mortar} = 3 \)
Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block) \( \text{typ}(1) = 3 \)
Compressive strength of units \( p(1) = 10.4 \text{ N/mm}^2 \)
Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block) \( \text{typ}(2) = 4 \)
Compressive strength of units \( p(2) = 10.4 \text{ N/mm}^2 \)
Partial safety factor (material) \( \gamma_M = 2.4 \)
Characteristic wind load per unit area \( Q_{kw} = 0.85 \text{ kN/m}^2 \)
Additional vert DL on panel \( w(1) = 0 \text{ kN/m} \)
Height of outer leaf units \( h_u(1) = 215 \text{ mm} \)
Additional vert DL on panel \( w(2) = 0 \text{ kN/m} \)
Height of inner leaf units \( h_u(2) = 215 \text{ mm} \)

**DESIGN SUMMARY**

| Panel length | 1 m |
| Panel height | 0.9 m |
| **OUTER LEAF** | |
| Thickness | 150 mm |
| Char flex strength (bed) | 0.3 N/mm² |
| Char flex strength (perp) | 0.9 N/mm² |
| **INNER LEAF** | |
| Thickness | 350 mm |
| Char flex strength (bed) | 0.25 N/mm² |
| Char flex strength (perp) | 0.75 N/mm² |
| Effective wall thickness | 358.74 mm |
| Char wind pressure | 0.85 kN/m² |
| Char wind capacity (bend) | 174.41 kN/m² |
| Design shear on sides | 0.57375 kN |
| Shear strength of sides | 27 kN |
| Design shear on founds | 0.57375 kN |
| Shear strength of founds | 30.648 kN |
| Material safety factor | 2.4 |
| Load safety factor | 1.5 |
Location: Wall on grid line A/1-2

Length of wall panel \( L = 3 \) m  
Height of wall panel \( h = 3 \) m  
Outer leaf density \( \text{den}(1) = 10 \) kN/m\(^2\)  
Thickness of outer leaf \( t(1) = 150 \) mm  
Mortar designation (Table NA.2) \( \text{mortar} = 4 \)  
Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block) \( \text{typ}(1) = 4 \)  
Compressive strength of units \( p(1) = 10.4 \) N/mm\(^2\)  
Partial safety factor (material) \( \gamma_M = 2.7 \)  
Characteristic wind load per unit area \( Q_{kw} = 0.6 \) kN/m\(^2\)  
Additional vert DL on panel \( w(1) = 0 \) kN/m  
Height of the units \( h_u(1) = 215 \) mm

**DESIGN SUMMARY**  
Panel length 3 m  
Panel height 3 m  
Thickness 150 mm  
Char flex strength (bed) 0.2 N/mm\(^2\)  
Char flex strength (perp) 0.6 N/mm\(^2\)  
Char wind pressure 0.6 kN/m\(^2\)  
Char wind capacity (bend) 0.63099 kN/m\(^2\)  
Design shear on sides 8.1 kN  
Shear strength of sides 18 kN  
Design shear on founds 4.05 kN  
Shear strength of founds 19.944 kN  
Material safety factor 2.7  
Load safety factor 1.5
Location: Wall on grid line A/1-2

UDL on panel (including opening)  \( w = 0.85 \text{ kN/m}^2 \)
Overall panel length horizontally  \( L = 4.2 \text{ m} \)
Overall panel height  \( H = 2.8 \text{ m} \)
Vertical dimension of opening  \( c = 1.3 \text{ m} \)
Horizontal dimension of opening  \( d = 1.1 \text{ m} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sub-panel A:</td>
<td>length 1.1 m</td>
</tr>
<tr>
<td></td>
<td>height 1.5 m</td>
</tr>
<tr>
<td></td>
<td>udl 0.85 kN/m²</td>
</tr>
<tr>
<td>Sub-panel B:</td>
<td>length 3.1 m</td>
</tr>
<tr>
<td></td>
<td>height 2.8 m</td>
</tr>
<tr>
<td></td>
<td>udl 1.6848 kN/m²</td>
</tr>
</tbody>
</table>

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Location: Wall on grid line A/1-2

UDL on panel (including opening) \( w = 0.85 \, \text{kN/m}^2 \)

Overall panel length horizontally \( L = 10.2 \, \text{m} \)

Overall panel height \( H = 7.8 \, \text{m} \)

Vertical dimension of opening \( c = 0.2 \, \text{m} \)

Horizontal dimension of opening \( d = 0.2 \, \text{m} \)

Distance to underside of opening \( e = 3.5 \, \text{m} \)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Sub-panel A: length 10.2 m</th>
<th>height 4.1 m</th>
<th>udl 0.89167 kN/m²</th>
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<tr>
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<td>Sub-panel B: length 10 m</td>
<td>height 0.2 m</td>
<td>udl 0.85 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Sub-panel C: length 10.2 m</td>
<td>height 3.5 m</td>
<td>udl 0.89857 kN/m²</td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

<table>
<thead>
<tr>
<th></th>
<th>Sub-panel A: length 1.2 m</th>
<th>Sub-panel B: length 0.9 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>height 0.6 m</td>
<td>height 0.2 m</td>
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<tr>
<td>udl</td>
<td>6.8708 kN/m²</td>
<td>4.85 kN/m²</td>
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</table>

UDL on panel (including opening) \( w = 4.85 \) kN/m²
Overall panel length horizontally \( L = 1.2 \) m
Overall panel height \( H = 0.8 \) m
Vertical dimension of opening \( c = 0.2 \) m
Horizontal dimension of opening \( d = 0.3 \) m
**Location:** Wall on grid line A/1-2

UDL on panel (including opening) \( w = 0.85 \text{ kN/m}^2 \)
Overall panel length horizontally \( L = 4.2 \text{ m} \)
Overall panel height \( H = 1.8 \text{ m} \)
Vertical dimension of opening \( c = 0.8 \text{ m} \)
Horizontal dimension of opening \( d = 0.8 \text{ m} \)
Distance to side of opening \( e = 1.7 \text{ m} \)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Sub-panel A: length 1.7 m  ( \text{udl} \ 1.7944 \text{ kN/m}^2 )</th>
<th>Sub-panel B: length 0.8 m  ( \text{udl} \ 0.85 \text{ kN/m}^2 )</th>
<th>Sub-panel C: length 1.7 m  ( \text{udl} \ 1.7944 \text{ kN/m}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>height 1.8 m</td>
<td>height 1 m</td>
<td>height 1.8 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**Location:** Wall on grid line A/1-2

UDL on panel (including opening) \( w = 0.85 \text{ kN/m}^2 \)
Overall panel length horizontally \( L = 4.2 \text{ m} \)
Overall panel height \( H = 3.8 \text{ m} \)
Vertical dimension of opening \( c = 0.8 \text{ m} \)
Horizontal dimension of opening \( d = 4.2 \text{ m} \)

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<th>DESIGN</th>
<th>Sub-panel A: length</th>
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<td>SUMMARY</td>
<td>height</td>
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</tr>
<tr>
<td></td>
<td>udl</td>
<td>1.2548 kN/m²</td>
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</tbody>
</table>

- Location: Wall on grid line A/1-2
- UDL on panel (including opening) \( w = 0.85 \text{ kN/m}^2 \)
- Overall panel length horizontally \( L = 4.2 \text{ m} \)
- Overall panel height \( H = 3.8 \text{ m} \)
- Vertical dimension of opening \( c = 0.8 \text{ m} \)
- Horizontal dimension of opening \( d = 4.2 \text{ m} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Sub-panel A: length</th>
<th>4.2 m</th>
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<tbody>
<tr>
<td>SUMMARY</td>
<td>height</td>
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</tr>
<tr>
<td></td>
<td>udl</td>
<td>1.2548 kN/m²</td>
</tr>
</tbody>
</table>
### Location: Wall on grid line A/1-2

- **UDL on panel (including opening)**: $w=0.85 \text{ kN/m}^2$
- **Overall panel length horizontally**: $L=4.2 \text{ m}$
- **Overall panel height**: $H=2.8 \text{ m}$
- **Vertical dimension of opening**: $c=1.3 \text{ m}$
- **Horizontal dimension of opening**: $d=1.1 \text{ m}$

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sub-panel A:</strong></td>
<td><strong>Sub-panel B:</strong></td>
</tr>
<tr>
<td>length</td>
<td>length</td>
</tr>
<tr>
<td>1.1 m</td>
<td>3.1 m</td>
</tr>
<tr>
<td>height</td>
<td>height</td>
</tr>
<tr>
<td>1.5 m</td>
<td>2.8 m</td>
</tr>
<tr>
<td>udl</td>
<td>udl</td>
</tr>
<tr>
<td>0.85 kN/m$^2$</td>
<td>1.6848 kN/m$^2$</td>
</tr>
</tbody>
</table>
**Location:** Wall on grid line A/1-2

- UDL on panel (including opening) $w = 0.85 \text{ kN/m}^2$
- Overall panel length horizontally $L = 10.2 \text{ m}$
- Overall panel height $H = 7.8 \text{ m}$
- Vertical dimension of opening $c = 0.2 \text{ m}$
- Horizontal dimension of opening $d = 0.2 \text{ m}$
- Distance to underside of opening $e = 3.5 \text{ m}$

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Sub-panel A: length 10.2 m</th>
<th>height 4.1 m</th>
<th>udl 0.89167 kN/m²</th>
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<tbody>
<tr>
<td>Sub-panel B: length 10 m</td>
<td>height 0.2 m</td>
<td>udl 0.85 kN/m²</td>
</tr>
<tr>
<td>Sub-panel C: length 10.2 m</td>
<td>height 3.5 m</td>
<td>udl 0.89857 kN/m²</td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

UDL on panel (including opening) \( w = 4.85 \text{ kN/m}^2 \)
Overall panel length horizontally \( L = 1.2 \text{ m} \)
Overall panel height \( H = 0.8 \text{ m} \)
Vertical dimension of opening \( c = 0.2 \text{ m} \)
Horizontal dimension of opening \( d = 0.3 \text{ m} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Sub-panel A: length 1.2 m</th>
<th>SUMMARIZE</th>
<th>Sub-panel B: length 0.9 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>height 0.6 m</td>
<td>udl 6.8708 kN/m²</td>
<td>height 0.2 m</td>
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</table>
Location: Wall on grid line A/1-2

UDL on panel (including opening) \( w = 0.85 \text{ kN/m}^2 \)
Overall panel length horizontally \( L = 4.2 \text{ m} \)
Overall panel height \( H = 1.8 \text{ m} \)
Vertical dimension of opening \( c = 0.8 \text{ m} \)
Horizontal dimension of opening \( d = 0.8 \text{ m} \)
Distance to side of opening \( e = 1.7 \text{ m} \)

| DESIGN SUMMARY | Sub-panel A: length 1.7 m | height 1.8 m | udl 1.7944 kN/m² |
|               | Sub-panel B: length 0.8 m | height 1 m   | udl 0.85 kN/m²   |
|               | Sub-panel C: length 1.7 m | height 1.8 m | udl 1.7944 kN/m² |
Location: Wall on grid line A/1-2

UDL on panel (including opening) \( w = 0.85 \, \text{kN/m}^2 \)
Overall panel length horizontally \( L = 4.2 \, \text{m} \)
Overall panel height \( H = 3.8 \, \text{m} \)
Vertical dimension of opening \( c = 0.8 \, \text{m} \)
Horizontal dimension of opening \( d = 4.2 \, \text{m} \)

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<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARÝY</th>
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</thead>
<tbody>
<tr>
<td>Sub-panel A: length 4.2 m</td>
<td>height 3 m</td>
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<tr>
<td></td>
<td>udl 1.2548 kN/m²</td>
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</tbody>
</table>
Location: Wall on grid line A/1-2

Clear height of wall \( h = 2.2 \)
Length of panel \( L = 4 \text{ m} \)
Thickness of wall \( t = 140 \text{ mm} \)
Support restraint factor \( rf = 1 \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 1 \)
Compressive Strength of Units \( po = 20 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{water} = 7 \)
Material partial safety factor \( \text{gammam} = 2.5 \)
Density \( \text{dens} = 10 \text{ kN/m}^3 \)
Eccentricity of roof load \( \text{ecc} = 23.333 \text{ mm} \)
Dead load excluding self weight \( DL = 1.2 \text{ kN/m} \)
Imposed load \( LL = 1.6 \text{ kN/m} \)
Wind load \( WL = 0.25 \text{ kN/m}^2 \)
Moment reduction factor \( \text{mrf} = 1 \)

**DESIGN SUMMARY**

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<th>Value</th>
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<td>Char. flexural strength</td>
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<tr>
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<tr>
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<tr>
<td>Design bending moment</td>
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<tr>
<td>Bending moment resistance</td>
<td>0.58021 kNm/m</td>
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<td>Partial safety factors:</td>
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<td>Material</td>
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<tr>
<td>Dead load (max)</td>
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</tr>
<tr>
<td>Dead load (min)</td>
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<tr>
<td>Imposed load</td>
<td>1.6</td>
</tr>
<tr>
<td>Wind load</td>
<td>1.4</td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

- Clear height of wall: h=2.2
- Length of panel: L=4 m
- Thickness of wall: t=150 mm
- Support restraint factor: rf=0.9
- Mortar designation: mortar=3
- Masonry type: typ(1)=4
- Compressive Strength of Units: po=22.5 N/mm²
- Material partial safety factor: gammam=2.5
- Density: dens=10 kN/m³
- Eccentricity of roof load: ecc=20 mm
- Dead load excluding self weight: DL=1.2 kN/m
- Imposed load: LL=1.6 kN/m
- Wind load: WL=0.25 kN/m²
- Moment reduction factor: mrf=1.2

**DESIGN SUMMARY**

<table>
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<tr>
<th>Parameter</th>
<th>Value</th>
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<tbody>
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<tr>
<td>Wall thickness</td>
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<td>Char. flexural strength</td>
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<tr>
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<td>Bending moment resistance</td>
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<td>Dead load (min)</td>
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<tr>
<td>Imposed load</td>
<td>1.6</td>
</tr>
<tr>
<td>Wind load</td>
<td>1.4</td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

Clear height of wall \( h = 3.5 \) 
Length of panel \( L = 8 \text{ m} \) 
Thickness of wall \( t = 150 \text{ mm} \) 
Support restraint factor \( rf = 1 \) 
Mortar designation to Table 1 \( \text{mortar} = 2 \) 
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ} (1) = 2 \) 
Compressive Strength of Units \( po = 60 \text{ N/mm}^2 \) 
Material partial safety factor \( \gamma_{ma} = 2.5 \) 
Density \( \text{dens} = 10 \text{ kN/m}^3 \) 
Eccentricity of roof load \( \text{ecc} = 20 \text{ mm} \) 
Dead load excluding self weight \( DL = 0.2 \text{ kN/m} \) 
Imposed load \( LL = 0.6 \text{ kN/m} \) 
Wind load \( WL = 0.15 \text{ kN/m}^2 \) 
Moment reduction factor \( mrf = 1 \)

**DESIGN SUMMARY**

| Wall height | 3.5 m |
| Wall length | 8 m   |
| Wall thickness | 150 mm |
| Char. compressive strength | 10.5 N/mm² |
| Char. flexural strength | 0.3 N/mm² |
| Design vertical load | 4.915 kN/m |
| Vertical load resistance | 287.7 kN/m |
| Design bending moment | 0.32156 kNm/m |
| Bending moment resistance | 0.51356 kNm/m |
| Partial safety factors: |
| Material | 2.5 |
| Dead load (max) | 1.4 |
| Dead load (min) | 0.9 |
| Imposed load | 1.6 |
| Wind load | 1.4 |
Location: Wall on grid line A/1-2

Clear height of wall h=0.8
Length of panel L=1 m
Thickness of wall t=215 mm
Support restraint factor rf=1
Mortar designation to Table 1 mortar=3
Masonry type (1=clay 2=cal silicate 3=concrete brick 4=block) typ(1)=3
Compressive Strength of Units po=5 N/mm²
Material partial safety factor gammam=2.5
Density dens=10 kN/m³
Eccentricity of roof load ecc=20 mm
Dead load excluding self weight DL=1.2 kN/m
Imposed load LL=1.6 kN/m
Wind load WL=3 kN/m²
Moment reduction factor mrf=1

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
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<tr>
<td>Wall length</td>
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<tr>
<td>Wall thickness</td>
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<tr>
<td>Char. compressive strength</td>
<td>2.4 N/mm²</td>
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<tr>
<td>Char. flexural strength</td>
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<tr>
<td>Design vertical load</td>
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<tr>
<td>Vertical load resistance</td>
<td>185.09 kN/m</td>
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<td>Design bending moment</td>
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<tr>
<td>Bending moment resistance</td>
<td>0.99094 kNm/m</td>
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<td>Material</td>
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<td>Dead load (min)</td>
<td>0.9</td>
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<tr>
<td>Imposed load</td>
<td>1.6</td>
</tr>
<tr>
<td>Wind load</td>
<td>1.4</td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

Clear height of wall   \( h = 2.2 \)
Length of panel   \( L = 4 \text{ m} \)
Thickness of wall   \( t = 225 \text{ mm} \)
Support restraint factor   \( r_f = 1 \)
Mortar designation to Table 1   \( \text{mortar}=4 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block)   \( \text{typ}(1)=4 \)
Compressive Strength of Units   \( p_o = 22.5 \text{ N/mm}^2 \)
Material partial safety factor   \( \gamma_m = 2.5 \)
Density   \( \text{dens}=10 \text{ kN/m}^3 \)
Eccentricity of roof load   \( \text{ecc}=20 \text{ mm} \)
Dead load excluding self weight   \( \text{DL}=1.2 \text{ kN/m} \)
Imposed load   \( \text{LL}=1.6 \text{ kN/m} \)
Wind load   \( \text{WL}=0.25 \text{ kN/m}^2 \)
Moment reduction factor   \( m_r f=1 \)

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Wall height</td>
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<td>Wall length</td>
<td>4 m</td>
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<tr>
<td>Wall thickness</td>
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<td>Char. compressive strength</td>
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<td>Char. flexural strength</td>
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<td>Design vertical load</td>
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<tr>
<td>Vertical load resistance</td>
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<td>Bending moment resistance</td>
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<tr>
<td>Imposed load</td>
<td>1.6</td>
</tr>
<tr>
<td>Wind load</td>
<td>1.4</td>
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</tbody>
</table>
Location: Wall on grid line A/1-2

Clear height of wall \( h = 2.2 \text{ m} \)
Length of panel \( L = 4 \text{ m} \)
Thickness of wall \( t = 140 \text{ mm} \)
Support restraint factor \( rf = 1 \)
Mortar designation to Table NA.2 mortar=3
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=1
Compressive strength of units \( po = 20 \text{ N/mm}^2 \)
Water absorption %age of leaf water=7%
Constant K from NA Table NA.4 \( K(1) = 0.5 \)
Height of masonry unit \( hu = 65 \text{ mm} \)
Least horiz. dimension of units \( lhd = 100 \text{ mm} \)
Partial safety factor (material) \( \gamma_M = 2.3 \)
Density of masonry units \( dens = 10 \text{ kN/m}^3 \)
Ecc. of floor load on inner leaf \( ecc = 23.333 \text{ mm} \)
Permanent load from above \( Gk(1) = 0 \text{ kN/m} \)
Variable load from above \( Qk(1) = 0 \text{ kN/m} \)
Permanent load from floor \( Gk(2) = 1.2 \text{ kN/m} \)
Variable load from floor \( Qk(2) = 1.6 \text{ kN/m} \)
Horizontal wind load (positive) \( Wk = 0.25 \text{ kN/m}^2 \)
Perm actions (upper levels) \( \gamma_G(1) = 1.35 \)
Perm actions (floor) \( \gamma_G(2) = 1.35 \)
Variable actions (vertical load) \( \gamma_Q = 1.5 \)
Variable actions (horiz.wind) \( \gamma_Qw = 0.75 \)
Design moment at mid-height \( Mmd = 0.085385 \text{ kNm/m} \)

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
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<td>Wall length</td>
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<td>Wall thickness</td>
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<tr>
<td>Char. compressive strength</td>
<td>5.5068 N/mm²</td>
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<tr>
<td>Char. flexural strength</td>
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<tr>
<td>Design vertical load</td>
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<td>Design vert load resistance</td>
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<td>Partial safety factors:</td>
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<td>Permanent load (min)</td>
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<td>Variable load</td>
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<tr>
<td>Wind load</td>
<td>0.75</td>
</tr>
<tr>
<td>Wall to be constructed using Group 1 units</td>
<td></td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

Clear height of wall \( h = 2.2 \text{ m} \)
Length of panel \( L = 4 \text{ m} \)
Thickness of wall \( t = 150 \text{ mm} \)
Support restraint factor \( rf = 0.9 \)
Mortar designation to Table NA.2 \( \text{mortar}=3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1)=4 \)
Compressive strength of units \( po = 22.5 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1)=0.5 \)
Height of masonry unit \( hu = 215 \text{ mm} \)
Least horiz. dimension of units \( lhd = 150 \text{ mm} \)
Partial safety factor (material) \( \text{gamM}=2.3 \)
Density of masonry units \( \text{dens}=10 \text{ kN/m}^3 \)
Ecc. of floor load on inner leaf \( \text{ecc}=25 \text{ mm} \)
Permanent load from above \( Gk(1)=0 \text{ kN/m} \)
Variable load from above \( Qk(1)=0 \text{ kN/m} \)
Permanent load from floor \( Gk(2)=1.2 \text{ kN/m} \)
Variable load from floor \( Qk(2)=1.6 \text{ kN/m} \)
Horizontal wind load (positive) \( Wk=0.25 \text{ kN/m}^2 \)
Perm actions (upper levels) \( \text{gamG1}=1.35 \)
Perm actions (floor) \( \text{gamG2}=1.35 \)
Variable actions (vertical load) \( \text{gamQ}=1.5 \)
Variable actions (horiz.wind) \( \text{gamQw}=0.75 \)

Design moment at mid-height \( Mmd=0.093713 \text{ kNm/m} \)

**DESIGN SUMMARY**

<table>
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<th>Parameter</th>
<th>Value</th>
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<td>Wall length</td>
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<tr>
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<tr>
<td>Char. compressive strength</td>
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<td>Char. flexural strength</td>
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<td>Permanent load (max)</td>
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<tr>
<td>Variable load</td>
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<tr>
<td>Wind load</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Wall to be constructed using Group 1 units
Location: Wall on grid line A/1-2

Clear height of wall  \( h = 3.5 \) m
Length of panel  \( L = 8 \) m
Thickness of wall  \( t = 150 \) mm
Support restraint factor  \( rf = 1 \)
Mortar designation to Table NA.2  \( \text{mortar} = 2 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block)  \( \text{typ}(1) = 2 \)
Compressive strength of units  \( po = 60 \) N/mm²
Constant K from NA Table NA.4  \( K(1) = 0.5 \)
Height of masonry unit  \( hu = 65 \) mm
Least horiz. dimension of units  \( lhd = 100 \) mm
Partial safety factor (material)  \( \gamma_{M} = 2.3 \)
Density of masonry units  \( \text{dens} = 10 \) kN/m³
Ecc. of floor load on inner leaf  \( \text{ecc} = 25 \) mm
Permanent load from above  \( Gk(1) = 0 \) kN/m
Variable load from above  \( Qk(1) = 0 \) kN/m
Permanent load from floor  \( Gk(2) = 0.2 \) kN/m
Variable load from floor  \( Qk(2) = 0.6 \) kN/m
Horizontal wind load (positive)  \( Wk = 0.15 \) kN/m²
Perm actions (upper levels)  \( \gamma_{G1} = 1.35 \)
Perm actions (floor)  \( \gamma_{G2} = 1.35 \)
Variable actions (vertical load)  \( \gamma_{Q} = 1.5 \)
Variable actions (horiz.wind)  \( \gamma_{Qw} = 0.75 \)

Design moment at mid-height  \( M_{md} = 0.070707 \) kNm/m

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Wall height</th>
<th>3.5 m</th>
</tr>
</thead>
<tbody>
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<tr>
<td>Wall thickness</td>
<td>150 mm</td>
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<tr>
<td>Char. compressive strength</td>
<td>13.419 N/mm²</td>
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<tr>
<td>Char. flexural strength</td>
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<tr>
<td>Design vertical load</td>
<td>4.7138 kN/m</td>
</tr>
<tr>
<td>Design vert load resistance</td>
<td>7.0922 kN/m</td>
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</tbody>
</table>

Partial safety factors:

- Material  \( \gamma_{M} = 2.3 \)
- Permanent load (max)  \( \gamma_{Q} = 1.35 \)
- Permanent load (min)  \( \gamma_{Q} = 1.0 \)
- Variable load  \( \gamma_{Q} = 1.5 \)
- Wind load  \( \gamma_{Q} = 0.75 \)

Wall to be constructed using Group 1 units
Location: Wall on grid line A/1-2

Clear height of wall \( h = 0.8 \) m
Length of panel \( L = 1 \) m
Thickness of wall \( t = 215 \) mm
Support restraint factor \( rf = 1 \)
Mortar designation to Table NA.2 \( \text{mortar} = 3 \)
Masonry type \((1=\text{clay} \ 2=\text{cal silicate} \ 3=\text{conc brick} \ 4=\text{block})\) \( \text{typ(1)} = 3 \)
Compressive strength of units \( po = 5 \) N/mm²
Constant K from NA Table NA.4 \( K(1) = 0.75 \)
Height of masonry unit \( hu = 65 \) mm
Least horiz. dimension of units \( lhd = 100 \) mm
Partial safety factor (material) \( \gamma_m = 2.3 \)
Density of masonry units \( \text{dens} = 10 \) kN/m²
Ecc. of floor load on inner leaf \( \text{ecc} = 35.833 \) mm
Permanent load from above \( Gk(1) = 0 \) kN/m
Variable load from above \( Qk(1) = 0 \) kN/m
Permanent load from floor \( Gk(2) = 1.2 \) kN/m
Variable load from floor \( Qk(2) = 1.6 \) kN/m
Horizontal wind load (positive) \( Wk = 3 \) kN/m²
Perm actions (upper levels) \( \gamma_G1 = 1.35 \)
Perm actions (floor) \( \gamma_G2 = 1.35 \)
Variable actions (vertical load) \( \gamma_Q = 1.5 \)
Variable actions (horiz.wind) \( \gamma_Qw = 0.75 \)

Design moment at mid-height \( M_{md} = 0.11139 \) kNm/m

**DESIGN SUMMARY**

**Wall height** 0.8 m
**Wall length** 1 m
**Wall thickness** 215 mm
**Char. compressive strength** 3.1301 N/mm²
**Char. flexural strength** 0.3 N/mm²
**Design vertical load** 5.181 kN/m
**Design vert load resistance** 133.5 kN/m
**Partial safety factors:**
**Material** 2.3
**Permanent load (max)** 1.35
**Permanent load (min)** 1.0
**Variable load** 1.5
**Wind load** 0.75
**Wall to be constructed using Group 1 units**
Location: Wall on grid line A/1-2

Clear height of wall \( h = 2.2 \text{ m} \)
Length of panel \( L = 4 \text{ m} \)
Thickness of wall \( t = 225 \text{ mm} \)
Support restraint factor \( rf = 1 \)
Mortar designation to Table NA.2 \( \text{mortar}=4 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1)=4 \)
Compressive strength of units \( po = 22.5 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1)=0.75 \)
Height of masonry unit \( hu = 215 \text{ mm} \)
Least horiz. dimension of units \( lhd = 225 \text{ mm} \)
Partial safety factor (material) \( \gamma_M = 2.3 \)
Density of masonry units \( \text{dens} = 10 \text{ kN/m}^3 \)
Ecc. of floor load on inner leaf \( \text{ecc} = 37.5 \text{ mm} \)
Permanent load from above \( Gk(1)=0 \text{ kN/m} \)
Variable load from above \( Qk(1)=0 \text{ kN/m} \)
Permanent load from floor \( Gk(2)=1.2 \text{ kN/m} \)
Variable load from floor \( Qk(2)=1.6 \text{ kN/m} \)
Horizontal wind load (positive) \( Wk=0.25 \text{ kN/m}^2 \)
Perm actions (upper levels) \( \gamma_{G1}=1.35 \)
Perm actions (floor) \( \gamma_{G2}=1.35 \)
Variable actions (vertical load) \( \gamma_Q=1.5 \)
Variable actions (horiz.wind) \( \gamma_{Qw}=0.75 \)

Design moment at mid-height \( M_{md}=0.16563 \text{ kNm/m} \)

**DESIGN SUMMARY**

| Wall height | 2.2 m |
| Wall length | 4 m   |
| Wall thickness | 225 mm |
| Char. compressive strength | 8.9893 N/mm² |
| Char. flexural strength | 0.2 N/mm² |
| Design vertical load | 73613 kN/m |
| Design vert load resistance | 483.86 kN/m |

Partial safety factors:

| Material | 2.3 |
| Permanent load (max) | 1.35 |
| Permanent load (min) | 1.0 |
| Variable load | 1.5 |
| Wind load | 0.75 |

Wall to be constructed using Group 1 units
Location: Wall on grid line A/1-2

Masonry single leaf wall by
effective eccentricities

This calculation are in accordance with BS5628-1:2005 using effective eccentricities as per Clause 28.

If no restraint is offered by the ground floor slab then take h to the top of foundation. Else take h to be the distance between floors as shown.

Support Restraint factor \( rf = 1 \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 4 \)
Compressive strength of units \( \text{po} = 10.4 \text{ N/mm}^2 \)
Height \( \text{hb} = 215 \text{ mm} \)
Least horizontal dimension \( \text{lhd} = 150 \text{ mm} \)
Partial Safety Factor Material \( \gamma_m = 3.1 \)
Dead load \( \text{DL}(1) = 7 \text{ kN/m} \)
Imposed load \( \text{LL}(1) = 10 \text{ kN/m} \)
Dead load \( \text{DL}(2) = 10.2 \text{ kN/m} \)
Imposed load \( \text{LL}(2) = 12.6 \text{ kN/m} \)
Eccentricity of roof/floor load \( \text{ecc}(2) = 20 \text{ mm} \)
Dead load \( \text{DL}(3) = 8.6 \text{ kN/m} \)
Imposed load \( \text{LL}(3) = 9.5 \text{ kN/m} \)
Self weight \( \text{DL}(5) = 10 \text{ kN/m} \)
Eccentricity of roof/floor load \( \text{ecc}(3) = 20 \text{ mm} \)
Wind load \( \text{WL} = 0.45 \text{ kN/m}^2 \)

**DESIGN**
- Wall height \( 2.2 \text{ m} \)

**SUMMARY**
- Wall length \( 4 \text{ m} \)
- Wall thickness \( 150 \text{ mm} \)
- Char comp strength \( 6.5405 \text{ N/mm}^2 \)
- Slenderness ratio \( 14.667 \)
<table>
<thead>
<tr>
<th>Case</th>
<th>Load (kN/m)</th>
<th>Design Moment (kNm)</th>
<th>Eff Ecc'y (m)</th>
<th>Ecc'y Ratio</th>
<th>Beta</th>
<th>Vert load R'stance (kN/m)</th>
<th>Factor of safety</th>
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</table>
Location: Wall on grid line A/1-2

Masonry single leaf wall by effective eccentricities

This calculation are in accordance with BS5628-1:2005 using effective eccentricities as per Clause 28.

If no restraint is offered by the ground floor slab then take h to the top of foundation. Else take h to be the distance between floors as shown.

Support Restraint factor \( rf = 1 \)
Mortar designation to Table 1 \( \text{mortar} = 1 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ(1)} = 4 \)
Compressive strength of units \( \text{po} = 10.4 \ \text{N/mm}^2 \)
Height \( \text{hb} = 215 \ \text{mm} \)
Least horizontal dimension \( \text{lhd} = 200 \ \text{mm} \)
Partial Safety Factor Material \( \text{gammam} = 3.1 \)
Dead load \( \text{DL(1)} = 7 \ \text{kN/m} \)
Imposed load \( \text{LL(1)} = 10 \ \text{kN/m} \)
Dead load \( \text{DL(2)} = 10.2 \ \text{kN/m} \)
Imposed load \( \text{LL(2)} = 12.6 \ \text{kN/m} \)
Eccentricity of roof/floor load \( \text{ecc(2)} = 20 \ \text{mm} \)
Dead load \( \text{DL(3)} = 8.6 \ \text{kN/m} \)
Imposed load \( \text{LL(3)} = 9.5 \ \text{kN/m} \)
Self weight \( \text{DL(5)} = 10 \ \text{kN/m} \)
Eccentricity of roof/floor load \( \text{ecc(3)} = 100 \ \text{mm} \)
Wind load \( \text{WL} = 0.45 \ \text{kN/m}^2 \)

**DESIGN**
- Wall height \( 2.2 \ \text{m} \)

**SUMMARY**
- Wall length \( 4 \ \text{m} \)
- Wall thickness \( 200 \ \text{mm} \)
- Char comp strength \( 5.8929 \ \text{N/mm}^2 \)
- Slenderness ratio \( 11 \)
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<tr>
<th>Case</th>
<th>Vert Load (kN/m)</th>
<th>Design Moment (kNm)</th>
<th>Eff Ecc'y</th>
<th>Ecc'y Ratio (&gt;0.05)</th>
<th>Beta load R'stance (kN/m)</th>
<th>Factor of safety</th>
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</table>
**Location:** Wall on grid line A/1-2

**Masonry single leaf wall by**

**effective eccentricities**

This calculation are in accordance with BS5628-1:2005 using effective eccentricities as per Clause 28.

If no restraint is offered by the ground floor slab then take h to the top of foundation. Else take h to be the distance between floors as shown.

Support Restraint factor \( rf = 1 \)
Mortar designation to Table 1 \( \text{mortar}=2 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1)=4 \)
Compressive strength of units \( p_0 = 60 \text{ N/mm}^2 \)
Height \( h_b = 215 \text{ mm} \)
Least horizontal dimension \( l_{hd} = 150 \text{ mm} \)
Partial Safety Factor Material \( \gamma_{ma} = 3.1 \)
Dead load \( \text{DL}(1) = 7 \text{ kN/m} \)
Imposed load \( \text{LL}(1) = 10 \text{ kN/m} \)
Dead load \( \text{DL}(2) = 10.2 \text{ kN/m} \)
Imposed load \( \text{LL}(2) = 12.6 \text{ kN/m} \)
Eccentricity of roof/floor load \( \text{ecc}(2) = 20 \text{ mm} \)
Dead load \( \text{DL}(3) = 8.6 \text{ kN/m} \)
Imposed load \( \text{LL}(3) = 9.5 \text{ kN/m} \)
Self weight \( \text{DL}(5) = 10 \text{ kN/m} \)
Eccentricity of roof/floor load \( \text{ecc}(3) = 20 \text{ mm} \)
Wind load \( \text{WL} = 0.15 \text{ kN/m}^2 \)

<table>
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<th>DESIGN</th>
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<td>Wall thickness</td>
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<td>----------------</td>
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<td>32.2</td>
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</table>
Location: Wall on grid line A/1-2

Masonry single leaf wall by effective eccentricities

This calculation are in accordance with BS5628-1:2005 using effective eccentricities as per Clause 28.

If no restraint is offered by the ground floor slab then take h to the top of foundation. Else take h to be the distance between floors as shown.

Support Restraint factor \( rf = 1 \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Masonry type (1=clay 2=cal silicate 3=concrete brick 4=block) \( \text{typ}(1) = 4 \)
Compressive strength of units \( po = 5.2 \text{ N/mm}^2 \)
Height \( hb = 215 \text{ mm} \)
Least horizontal dimension \( lhd = 450 \text{ mm} \)
Partial Safety Factor Material \( \gamma_m = 3.1 \)
Dead load \( DL(1) = 7 \text{ kN/m} \)
Imposed load \( LL(1) = 10 \text{ kN/m} \)
Dead load \( DL(2) = 10.2 \text{ kN/m} \)
Imposed load \( LL(2) = 12.6 \text{ kN/m} \)
Eccentricity of roof/floor load \( \text{ecc}(2) = 20 \text{ mm} \)
Dead load \( DL(3) = 8.6 \text{ kN/m} \)
Imposed load \( LL(3) = 9.5 \text{ kN/m} \)
Self weight \( DL(5) = 10 \text{ kN/m} \)
Eccentricity of roof/floor load \( \text{ecc}(3) = 20 \text{ mm} \)
Wind load \( WL = 4.5 \text{ kN/m}^2 \)

**DESIGN**
- Wall height \( 0.8 \text{ m} \)

**SUMMARY**
- Wall length \( 1 \text{ m} \)
- Wall thickness \( 450 \text{ mm} \)
- Char comp strength \( 2.5 \text{ N/mm}^2 \)
- Slenderness ratio \( 1.7778 \)
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<th>Case</th>
<th>Load (kN/m)</th>
<th>Design Moment (kNm)</th>
<th>Eff Ecc'y (m)</th>
<th>Ecc'y Ratio (&gt;0.05)</th>
<th>Beta</th>
<th>Vert load R'stance (kN/m)</th>
<th>Factor of safety</th>
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</thead>
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</table>
Location: Wall on grid line A/1-2

Masonry single leaf wall by effective eccentricities

This calculation is in accordance with BS5628-1:2005 using effective eccentricities as per Clause 28.

If no restraint is offered by the ground floor slab then take h to the top of foundation. Else take h to be the distance between floors as shown.

Support Restraint factor rf=1
Mortar designation to Table 1 mortar=4
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Compressive strength of units po=10.4 N/mm²
Height hb=215 mm
Least horizontal dimension lhd=100 mm
Partial Safety Factor Material gammam=3.1
Dead load DL(1)=7 kN/m
Imposed load LL(1)=10 kN/m
Dead load DL(2)=10.2 kN/m
Imposed load LL(2)=12.6 kN/m
Eccentricity of roof/floor load ecc(2)=20 mm
Dead load DL(3)=8.6 kN/m
Imposed load LL(3)=9.5 kN/m
Self weight DL(5)=10 kN/m
Eccentricity of roof/floor load ecc(3)=20 mm
Wind load WL=0.45 kN/m²

DESIGN
Wall height 2.2 m

SUMMARY
Wall length 4 m
Wall thickness 100 mm
Char comp strength 7 N/mm²
Slenderness ratio 22
<table>
<thead>
<tr>
<th>Case</th>
<th>Vert Load (kN/m)</th>
<th>Design Moment (kNm)</th>
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<th>Ecc'y Ratio (&gt;0.05)</th>
<th>Beta Vert R'stance (kN/m)</th>
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Location: Wall on grid line A/1-2

Masonry single leaf wall by effective eccentricities

This calculation is in accordance with EC6 and is based on effective eccentricities.

If no restraint is offered by the ground floor slab then take $h$ to the top of foundation, otherwise take $h$ to be the distance between floors as shown.

Support Restraint factor $rf = 1$
Mortar designation to Table NA.2 $mortar = 3$
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) $typ(1) = 4$
Compressive strength of units $po = 10.4 \text{ N/mm}^2$
Constant K from NA Table NA.4 $K(1) = 0.75$
Height of masonry unit $hu = 215 \text{ mm}$
Least horiz. dimension of units $lhd = 150 \text{ mm}$
Partial safety factor (material) $\gamma_M = 3$
Permanent Load(1) $G_k(1) = 7 \text{ kN/m}$
Variable Load(1) $Q_k(1) = 10 \text{ kN/m}$
Permanent Load(2) $G_k(2) = 10.2 \text{ kN/m}$
Variable Load(2) $Q_k(2) = 12.6 \text{ kN/m}$
Eccentricity of roof/floor load $ecc(2) = 20 \text{ mm}$
Permanent Load(3) $G_k(3) = 8.6 \text{ kN/m}$
Variable Load(3) $Q_k(3) = 9.5 \text{ kN/m}$
Wall weight at mid-height $G_k(5) = 10 \text{ kN/m}$
Eccentricity of roof/floor load $ecc(3) = 20 \text{ mm}$
Factor $\psi_0$ for floor variable load $\psi_{0f} = 0.7$

Design moment at mid-height $M_{md} = 0.08172 \text{ kNm/m}$

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height</td>
<td>2.2 m</td>
</tr>
<tr>
<td>Wall length</td>
<td>4 m</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>150 mm</td>
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<tr>
<td>Char comp strength</td>
<td>6.9606 N/mm²</td>
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<tr>
<td>Slenderness ratio</td>
<td>14.667</td>
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<tr>
<td>Vert Load at mid height (kN/m)</td>
<td>Design Moment top of wall (kNm)</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>---------------------------------</td>
</tr>
<tr>
<td>LC 1 96.5</td>
<td>0.136</td>
</tr>
<tr>
<td>LC 2 78.4</td>
<td>0.499</td>
</tr>
<tr>
<td>LC 3 73</td>
<td>0.334</td>
</tr>
<tr>
<td>LC 4 55.7</td>
<td>0.499</td>
</tr>
<tr>
<td>LC 5 50.3</td>
<td>0.334</td>
</tr>
<tr>
<td>LC 6 82</td>
<td>0.108</td>
</tr>
<tr>
<td>LC 7 48.3</td>
<td>0.043</td>
</tr>
<tr>
<td>LC 8 32.2</td>
<td>0.029</td>
</tr>
<tr>
<td>LC 9 86.5</td>
<td>0.108</td>
</tr>
<tr>
<td>LC 10 87.7</td>
<td>0.222</td>
</tr>
<tr>
<td>LC 11 86.3</td>
<td>0.023</td>
</tr>
</tbody>
</table>

The vertical load resistance taking into consideration the resultant eccentricity due to vertical and wind loads is greater than the design load for each load combination. Hence wall is satisfactory.
Location: Wall on grid line A/1-2

Masonry single leaf wall by effective eccentricities

This calculation is in accordance with EC6 and is based on effective eccentricities.

If no restraint is offered by the ground floor slab then take h to the top of foundation, otherwise take h to be the distance between floors as shown.

Support Restraint factor \( rf = 1 \)
Mortar designation to Table NA.2 \( \text{mortar} = 1 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ(1)} = 4 \)
Compressive strength of units \( \text{po} = 10.4 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( \text{K(1)} = 0.75 \)
Height of masonry unit \( \text{hu} = 215 \text{ mm} \)
Least horiz. dimension of units \( \text{lhd} = 200 \text{ mm} \)
Partial safety factor (material) \( \text{gamM} = 3 \)
Permanent Load(1) \( \text{Gk(1)} = 7 \text{ kN/m} \)
Variable Load(1) \( \text{Qk(1)} = 10 \text{ kN/m} \)
Permanent Load(2) \( \text{Gk(2)} = 10.2 \text{ kN/m} \)
Variable Load(2) \( \text{Qk(2)} = 12.6 \text{ kN/m} \)
Eccentricity of roof/floor load \( \text{ecc(2)} = 20 \text{ mm} \)
Permanent Load(3) \( \text{Gk(3)} = 8.6 \text{ kN/m} \)
Variable Load(3) \( \text{Qk(3)} = 9.5 \text{ kN/m} \)
Wall weight at mid-height \( \text{Gk(5)} = 10 \text{ kN/m} \)
Eccentricity of roof/floor load \( \text{ecc(3)} = 100 \text{ mm} \)
Factor \( \Psi_0 \) for floor variable load \( \text{psi0f} = 0.7 \)

Design moment at mid-height \( \text{Mmd} = 1.1596 \text{ kNm/m} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height</td>
<td>2.2 m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY</th>
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<tbody>
<tr>
<td>Wall length</td>
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<td>Wall thickness</td>
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<td>Char comp strength</td>
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<td>Slenderness ratio</td>
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<tr>
<td>Vert Load at mid height (kN/m)</td>
<td>Design Moment top of wall (kNm)</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>--------------------------------</td>
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<tr>
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<td>LC 3 73 2.4 45.2 24.6 0.548</td>
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</table>

The vertical load resistance taking into consideration the resultant eccentricity due to vertical and wind loads is greater than the design load for each load combination. Hence wall is satisfactory.
Location: Wall on grid line A/1-2

Masonry single leaf wall by
effective eccentricities

This calculation is in accordance
with EC6 and is based on effective eccentricities.

If no restraint is offered by the
ground floor slab then take h to
to the top of foundation, otherwise
take h to be the distance between
floors as shown.

Support Restraint factor rf=1
Mortar designation to Table NA.2 mortar=2
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Compressive strength of units po=60 N/mm²
Constant K from NA Table NA.4 K(1)=0.75
Height of masonry unit hu=215 mm
Least horiz. dimension of units lhd=150 mm
Partial safety factor (material) gamM=3
Permanent Load(1) Gk(1)=7 kN/m
Variable Load(1) Qk(1)=10 kN/m
Permanent Load(2) Gk(2)=10.2 kN/m
Variable Load(2) Qk(2)=12.6 kN/m
Eccentricity of roof/floor load ecc(2)=20 mm
Permanent Load(3) Gk(3)=8.6 kN/m
Variable Load(3) Qk(3)=9.5 kN/m
Wall weight at mid-height Gk(5)=10 kN/m
Eccentricity of roof/floor load ecc(3)=20 mm
Factor ψ₀ for floor variable load psi0f=0.7

Design moment at mid-height Mmd=0.08172 kNm/m

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Wall height</th>
<th>4 m</th>
</tr>
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<tbody>
<tr>
<td>SUMMARY</td>
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<td>Char comp strength</td>
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<td></td>
<td>Slenderness ratio</td>
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### Table: Masonry Design

<table>
<thead>
<tr>
<th>Vert Load at mid height (kN/m)</th>
<th>Design Moment at mid (kNm)</th>
<th>Ecc'y at mid (mm)</th>
<th>Ecc'y of wall (mm)</th>
<th>Capacity reduction factor (kN/m)</th>
<th>Vert load R'stance (kN/m)</th>
<th>Factor of safety</th>
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</thead>
<tbody>
<tr>
<td>LC 1 96.5</td>
<td>0.136 10.5</td>
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<td>LC 3 73</td>
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<tr>
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<tr>
<td>LC 9 86.5</td>
<td>0.108 10.4</td>
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<td>LC 10 87.7</td>
<td>0.222 11.9</td>
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</tbody>
</table>

The vertical load resistance taking into consideration the resultant eccentricity due to vertical and wind loads is greater than the design load for each load combination. Hence wall is satisfactory.
Location: Wall on grid line A/1-2

Masonry single leaf wall by effective eccentricities

This calculation is in accordance with EC6 and is based on effective eccentricities.

If no restraint is offered by the ground floor slab then take h to to the top of foundation, otherwise take h to be the distance between floors as shown.

Support Restraint factor \( rf = 1 \)
Mortar designation to Table NA.2 mortar = 3
Masonry type (1 = clay 2 = cal silicate 3 = conc brick 4 = block) typ(1) = 4
Compressive strength of units \( po = 5.2 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1) = 0.75 \)
Height of masonry unit \( hu = 215 \text{ mm} \)
Least horiz. dimension of units \( lhd = 450 \text{ mm} \)
Partial safety factor (material) \( \gamma_M = 3 \)
Permanent Load(1) \( Gk(1) = 7 \text{ kN/m} \)
Variable Load(1) \( Qk(1) = 10 \text{ kN/m} \)
Permanent Load(2) \( Gk(2) = 10.2 \text{ kN/m} \)
Variable Load(2) \( Qk(2) = 12.6 \text{ kN/m} \)
Eccentricity of roof/floor load \( ecc(2) = 20 \text{ mm} \)
Permanent Load(3) \( Gk(3) = 8.6 \text{ kN/m} \)
Variable Load(3) \( Qk(3) = 9.5 \text{ kN/m} \)
Wall weight at mid-height \( Gk(5) = 10 \text{ kN/m} \)
Eccentricity of roof/floor load \( ecc(3) = 20 \text{ mm} \)
Factor \( \psi_0 \) for floor variable load \( psi0f = 0.7 \)

Design moment at mid-height \( Mmd = 0.08172 \text{ kNm/m} \)

- **DESIGN**
  - Wall height: 0.8 m
- **SUMMARY**
  - Wall length: 1 m
  - Wall thickness: 450 mm
  - Char comp strength: 3.8902 N/mm²
  - Slenderness ratio: 1.7778
<table>
<thead>
<tr>
<th>Vert Load at mid height (kN/m)</th>
<th>Design Moment top of wall (kNm)</th>
<th>Ecc'y top of wall (mm)</th>
<th>Ecc'y at mid height of wall (mm)</th>
<th>Capacity reduction factor (Nrd)</th>
<th>Vert load R'stance (kN/m)</th>
<th>Factor of safety</th>
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<tbody>
<tr>
<td>LC 1</td>
<td>96.5</td>
<td>0.136</td>
<td>22.5</td>
<td>22.5</td>
<td>0.9</td>
<td>525</td>
</tr>
<tr>
<td>LC 2</td>
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<td>0.499</td>
<td>22.5</td>
<td>22.5</td>
<td>0.9</td>
<td>525</td>
</tr>
<tr>
<td>LC 3</td>
<td>73</td>
<td>0.334</td>
<td>22.5</td>
<td>22.5</td>
<td>0.9</td>
<td>525</td>
</tr>
<tr>
<td>LC 4</td>
<td>55.7</td>
<td>0.499</td>
<td>22.5</td>
<td>22.5</td>
<td>0.9</td>
<td>525</td>
</tr>
<tr>
<td>LC 5</td>
<td>50.3</td>
<td>0.334</td>
<td>22.5</td>
<td>22.5</td>
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<td>525</td>
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<tr>
<td>LC 6</td>
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<td>0.108</td>
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<td>22.5</td>
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<td>525</td>
</tr>
<tr>
<td>LC 7</td>
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<td>LC 8</td>
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<td>22.5</td>
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<tr>
<td>LC 9</td>
<td>86.5</td>
<td>0.108</td>
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<td>22.5</td>
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<td>525</td>
</tr>
<tr>
<td>LC 10</td>
<td>87.7</td>
<td>0.222</td>
<td>22.5</td>
<td>22.5</td>
<td>0.9</td>
<td>525</td>
</tr>
<tr>
<td>LC 11</td>
<td>86.3</td>
<td>0.023</td>
<td>22.5</td>
<td>22.5</td>
<td>0.9</td>
<td>525</td>
</tr>
</tbody>
</table>

The vertical load resistance taking into consideration the resultant eccentricity due to vertical and wind loads is greater than the design load for each load combination. Hence wall is satisfactory.
Location: Wall on grid line A/1-2

Masonry single leaf wall by effective eccentricities

This calculation is in accordance with EC6 and is based on effective eccentricities.

If no restraint is offered by the ground floor slab then take h to the top of foundation, otherwise take h to be the distance between floors as shown.

Support Restraint factor  \( rf = 1 \)
Mortar designation to Table NA.2  mortar = 4
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1) = 4
Compressive strength of units  \( po = 10.4 \text{ N/mm}^2 \)
Constant K from NA Table NA.4  \( K(1) = 0.75 \)
Height of masonry unit  \( hu = 215 \text{ mm} \)
Least horiz. dimension of units  \( lhd = 100 \text{ mm} \)
Partial safety factor (material)  \( \gamma_M = 3 \)
Permanent Load(1)  \( Gk(1) = 7 \text{ kN/m} \)
Variable Load(1)  \( Qk(1) = 10 \text{ kN/m} \)
Permanent Load(2)  \( Gk(2) = 10.2 \text{ kN/m} \)
Variable Load(2)  \( Qk(2) = 12.6 \text{ kN/m} \)
Eccentricity of roof/floor load  \( \text{ecc}(2) = 20 \text{ mm} \)
Permanent Load(3)  \( Gk(3) = 8.6 \text{ kN/m} \)
Variable Load(3)  \( Qk(3) = 9.5 \text{ kN/m} \)
Wall weight at mid-height  \( Gk(5) = 10 \text{ kN/m} \)
Eccentricity of roof/floor load  \( \text{ecc}(3) = 20 \text{ mm} \)
Factor \( \psi_0 \) for floor variable load  \( \psi_0 = 0.7 \)

Design moment at mid-height  \( M_{md} = 0.08172 \text{ kNm/m} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Wall height</th>
<th>2.2 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Wall length</td>
<td>4 m</td>
</tr>
<tr>
<td></td>
<td>Wall thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td></td>
<td>Char comp strength</td>
<td>5.9595 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Slenderness ratio</td>
<td>22</td>
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</tbody>
</table>

SCALE 5.48                Office 1007                Proforma 511
The vertical load resistance taking into consideration the resultant eccentricity due to vertical and wind loads is greater than the design load for each load combination. Hence wall is satisfactory.
Location: Wall on grid line A/1-2

Lateral resistance of vertically loaded single leaf wall by arching

This calculation is in accordance with BS5628-1:2005 being the Code of practice for use of masonry - Part 1: Structural use of unreinforced masonry.

The form of this calculation is in accordance with Clause 32.8 of the code of practice.

Clear height of wall h=2 m
Thickness of wall t=100 mm
Length of wall panel l=3.5 m
Support restraint factor rf=0.75
Mortar designation to Table 1 mortar=3
Masonry type (1=clay 2=cal silicate 3=concrete brick 4=block) typ(1)=1
Compressive Strength of Units po=20 N/mm²
Partial Safety Factor Material gammam=2.5
Dead load DL(1)=20 kN/m
Imposed load LL(1)=10 kN/m
Dead load DL(2)=8 kN/m
Imposed load LL(2)=5 kN/m
Eccentricity of floor loads ecc(2)=16.667 mm
Number of returns nr=1

DESIGN SUMMARY
Wall height 2 m
Wall length 3.5 m
Wall thickness 100 mm
Char comp strength 5 N/mm²
Horizontal capacity by arching (Clause 32.8)
Case 1 - normal loading 3.36 kN/m²
Case 2 - abnormal loading 3.822 kN/m²
Location: Wall on grid line A/1-2

Load(1) \[\Rightarrow \]\ Load(2)

Lateral resistance of vertically loaded single leaf wall by arching

This calculation is in accordance with BS5628-1:2005 being the Code of practice for use of masonry - Part 1: Structural use of unreinforced masonry.

The form of this calculation is in accordance with Clause 32.8 of the code of practice.

Clear height of wall \[h=2 \text{ m}\]
Thickness of wall \[t=100 \text{ mm}\]
Length of wall panel \[l=3.5 \text{ m}\]
Support restraint factor \[rf=0.75\]
Mortar designation to Table 1 \[mortar=2\]
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \[typ(1)=4\]
Height \[hb=215 \text{ mm}\]
Least horiz dimension \[lhd=100 \text{ mm}\]
Compressive Strength of Units \[po=10.4 \text{ N/mm}^2\]
Partial Safety Factor Material \[\gamma_{m}=3.1\]
Dead load \[DL(1)=20 \text{ kN/m}\]
Imposed load \[LL(1)=10 \text{ kN/m}\]
Dead load \[DL(2)=8 \text{ kN/m}\]
Imposed load \[LL(2)=5 \text{ kN/m}\]
Eccentricity of floor loads \[ecc(2)=16.667 \text{ mm}\]
Number of returns \[nr=0\]

DESIGN SUMMARY

Wall height \[2 \text{ m}\]
Wall length \[3.5 \text{ m}\]
Wall thickness \[100 \text{ mm}\]
Char comp strength \[8.4 \text{ N/mm}^2\]
Horizontal capacity by arching (Clause 32.8)
Case 1 - normal loading \[2.8 \text{ kN/m}^2\]
Case 2 - abnormal loading \[3.185 \text{ kN/m}^2\]
Location: Wall on grid line A/1-2

Lateral resistance of vertically loaded single leaf wall by arching

This calculation is in accordance with BS5628-1:2005 being the Code of practice for use of masonry - Part 1: Structural use of unreinforced masonry.

The form of this calculation is in accordance with Clause 32.8 of the code of practice.

<table>
<thead>
<tr>
<th>Clear height of wall</th>
<th>h=3 m</th>
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<td>t=150 mm</td>
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<tr>
<td>Length of wall panel</td>
<td>l=8 m</td>
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<tr>
<td>Support restraint factor</td>
<td>rf=0.75</td>
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<tr>
<td>Mortar designation to Table 1</td>
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<tr>
<td>Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4</td>
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</tr>
<tr>
<td>Height</td>
<td>hb=215 mm</td>
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<tr>
<td>Least horiz dimension</td>
<td>lhd=100 mm</td>
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<tr>
<td>Compressive Strength of Units</td>
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<td>Partial Safety Factor Material</td>
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<td>Dead load</td>
<td>DL(1)=20 kN/m</td>
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<tr>
<td>Imposed load</td>
<td>LL(1)=10 kN/m</td>
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<tr>
<td>Dead load</td>
<td>DL(2)=8 kN/m</td>
</tr>
<tr>
<td>Imposed load</td>
<td>LL(2)=5 kN/m</td>
</tr>
<tr>
<td>Eccentricity of floor loads</td>
<td>ecc(2)=25 mm</td>
</tr>
<tr>
<td>Number of returns</td>
<td>nr=2</td>
</tr>
</tbody>
</table>

**DESIGN SUMMARY**

| Wall height | 3 m |
| Wall length | 8 m |
| Wall thickness | 150 mm |
| Char comp strength | 18.7 N/mm² |

Horizontal capacity by arching (Clause 32.8)

| Case 1 - normal loading | 2.4267 kN/m² |
| Case 2 - abnormal loading | 2.7603 kN/m² |
Location: Wall on grid line A/1-2

Lateral resistance of vertically loaded single leaf wall by arching

This calculation is in accordance with BS5628-1:2005 being the Code of practice for use of masonry - Part 1: Structural use of unreinforced masonry.

The form of this calculation is in accordance with Clause 32.8 of the code of practice.

Clear height of wall \( h = 0.8 \) m
Thickness of wall \( t = 450 \) mm
Length of wall panel \( l = 1 \) m
Support restraint factor \( rf = 0.75 \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ(1)} = 4 \)
Height \( hb = 215 \) mm
Least horiz dimension \( lhd = 100 \) mm
Compressive Strength of Units \( po = 5.2 \) N/mm\(^2\)
Partial Safety Factor Material \( \gamma_{mm} = 3.1 \)
Dead load \( DL(1) = 50 \) kN/m
Imposed load \( LL(1) = 20 \) kN/m
Dead load \( DL(2) = 8 \) kN/m
Imposed load \( LL(2) = 5 \) kN/m
Eccentricity of floor loads \( ecc(2) = 75 \) mm
Number of returns \( nr = 1 \)

DESIGN SUMMARY

Wall height \( 0.8 \) m
Wall length \( 1 \) m
Wall thickness \( 450 \) mm
Char comp strength \( 5 \) N/mm\(^2\)
Horizontal capacity by arching (Clause 32.8)
Case 1 - normal loading \( 228.37 \) kN/m\(^2\)
Case 2 - abnormal loading \( 251.41 \) kN/m\(^2\)
Case 3 - protected member \( 477.68 \) kN/m\(^2\)
Location: Wall on grid line A/1-2

**Lateral resistance of vertically loaded single leaf wall by arching**

This calculation is in accordance with BS5628-1:2005 being the Code of practice for use of masonry - Part 1: Structural use of unreinforced masonry.

The form of this calculation is in accordance with Clause 32.8 of the code of practice.

---

Clear height of wall \( h = 2 \text{ m} \)
Thickness of wall \( t = 100 \text{ mm} \)
Length of wall panel \( l = 3.5 \text{ m} \)
Support restraint factor \( rf = 0.75 \)
Mortar designation to Table 1 \( \text{mortar} = 4 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ(1)} = 4 \)
Height \( h_b = 215 \text{ mm} \)
Least horiz dimension \( h_{ld} = 100 \text{ mm} \)
Compressive Strength of Units \( p_0 = 10.4 \text{ N/mm}^2 \)
Partial Safety Factor Material \( \gamma_m = 3.1 \)
Dead load \( DL(1) = 20 \text{ kN/m} \)
Imposed load \( LL(1) = 10 \text{ kN/m} \)
Dead load \( DL(2) = 8 \text{ kN/m} \)
Imposed load \( LL(2) = 5 \text{ kN/m} \)
Eccentricity of floor loads \( \text{ecc}(2) = 16.667 \text{ mm} \)
Number of returns \( nr = 1 \)

**DESIGN SUMMARY**

- Wall height \( 2 \text{ m} \)
- Wall length \( 3.5 \text{ m} \)
- Wall thickness \( 100 \text{ mm} \)
- Char comp strength \( 7 \text{ N/mm}^2 \)
- Horizontal capacity by arching (Clause 32.8)
  - Case 1 - normal loading \( 3.36 \text{ kN/m}^2 \)
  - Case 2 - abnormal loading \( 3.822 \text{ kN/m}^2 \)
Location: Wall on grid line A/1-2

Lateral resistance of vertically loaded single leaf wall by arching

This Calculation is in accordance with EC6 - Design of masonry structures Part 1-1: General rules for reinforced and unreinforced masonry structures.

The form of this calculation is in accordance with section 6 of the code of practice.

Wk = design lateral strength

Clear height of wall  
Thickness of wall  
Length of wall panel  
Support restraint factor  
Mortar designation to Table NA.2  
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block)  
Compressive strength of units  
Constant K from NA Table NA.4  
Height of masonry unit  
Least horiz. dimension of units  
Partial safety factor (material)  
Permanent Load(1)  
Variable Load(1)  
Permanent Load(2)  
Variable Load(2)  
Ecc. of floor load on inner leaf  
Perm actions (upper levels)  
Perm actions (floor)  
Variable actions (vertical load)  
Weight of wall at mid-height  
Horizontal loads (e.g. wind load)  
Design moment at mid-height

h=2 m
t=100 mm
L=3.5 m
rf=0.75
mortar=3
typ(1)=1
po=20 N/mm²
K(1)=0.5
hu=65 mm
hd=100 mm
gamM=2.6
Gk(1)=20 kN/m
Qk(1)=10 kN/m
Gk(2)=8 kN/m
Qk(2)=5 kN/m
ecc(2)=16.667 mm
gamG1=1.35
gamG2=1.35
gamQ=1.5
Gkws=2.2 kN/m
ehe=0 mm
ehm=0 mm
Mmd=0.19202 kNm/m
Leading variable load factor $\psi_1$ psi1=0.5
Other variable load factor $\psi_2$ psi2=0.3

DESIGN SUMMARY

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height</td>
<td>2 m</td>
</tr>
<tr>
<td>Wall length</td>
<td>3.5 m</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char compressive strength</td>
<td>5.5068 N/mm²</td>
</tr>
</tbody>
</table>

Design lateral strength by arching (Cl. 6.3.2)
Case 1 - normal loading 5.295 kN/m²
Case 2 - abnormal loading 5.295 kN/m²
Location: Wall on grid line A/1-2

Lateral resistance of vertically loaded single leaf wall by arching

This Calculation is in accordance with EC6 - Design of masonry structures Part 1-1: General rules for reinforced and unreinforced masonry structures.

The form of this calculation is in accordance with section 6 of the code of practice.

Wk = design lateral strength

Clear height of wall h = 2 m
Thickness of wall t = 100 mm
Length of wall panel L = 3.5 m
Support restraint factor rf = 0.75
Mortar designation to Table NA.2 mortar = 2
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1) = 4
Compressive strength of units po = 10.4 N/mm²
Constant K from NA Table NA.4 K(1) = 0.75
Normalised mean comp strength fbn(1) = 10.4 N/mm²
Partial safety factor (material) gamM = 3
Permanent Load(1) Gk(1) = 20 kN/m
Variable Load(1) Qk(1) = 10 kN/m
Permanent Load(2) Gk(2) = 8 kN/m
Variable Load(2) Qk(2) = 5 kN/m
Ecc. of floor load on inner leaf ecc(2) = 16.667 mm
Perm actions (upper levels) gamG1 = 1.35
Perm actions (floor) gamG2 = 1.35
Variable actions (vertical load) gamQ = 1.5

Weight of wall at mid-height Gkws = 1 kN/m
horizontal loads (e.g. wind load) ehe = 0 mm
loads (e.g. wind load) ehm = 0 mm
Design moment at mid-height Mmd = 0.1871 kNm/m
Leading variable load factor \( \psi_1 \) psi1=0.5
Other variable load factor \( \psi_2 \) psi2=0.3

**DESIGN SUMMARY**
- Wall height: 2 m
- Wall length: 3.5 m
- Wall thickness: 100 mm
- Char compressive strength: 6.6135 N/mm\(^2\)

Design lateral strength by arching (Cl. 6.3.2)
- Case 1 - normal loading: 5.5112 kN/m\(^2\)
- Case 2 - abnormal loading: 5.5112 kN/m\(^2\)
Location: Wall on grid line A/1-2

Load(1)   Load(2)

Lateral resistance of vertically loaded single leaf wall by arching

This Calculation is in accordance with EC6 - Design of masonry structures Part 1-1: General rules for reinforced and unreinforced masonry structures.

The form of this calculation is in accordance with section 6 of the code of practice.

Wk = design lateral strength

Clear height of wall  h=3 m
Thickness of wall  t=150 mm
Length of wall panel  L=8 m
Support restraint factor  rf=0.75
Mortar designation to Table NA.2  mortar=2
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Compressive strength of units  po=60 N/mm²
Constant K from NA Table NA.4  K(1)=0.75
Normalised mean comp strength  fbn(1)=60 N/mm²
Partial safety factor (material)  gamM=3
Permanent Load(1)  Gk(1)=20 kN/m
Variable Load(1)  Qk(1)=10 kN/m
Permanent Load(2)  Gk(2)=8 kN/m
Variable Load(2)  Qk(2)=5 kN/m
Ecc. of floor load on inner leaf  ecc(2)=25 mm
Perm actions (upper levels)  gamG1=1.35
Perm actions (floor)  gamG2=1.35
Variable actions (vertical load)  gamQ=1.5

Weight of wall at mid-height  Gkws=2.25 kN/m
horizontal loads (e.g. wind load)  ehe=0 mm
loads (e.g. wind load)  ehm=0 mm
Design moment at mid-height  Mmd=0.28833 kNm/m
Leading variable load factor \( \psi_1 \) \( \psi_1=0.5 \)
Other variable load factor \( \psi_2 \) \( \psi_2=0.3 \)

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Design parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height</td>
<td>3 m</td>
</tr>
<tr>
<td>Wall length</td>
<td>8 m</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>150 mm</td>
</tr>
<tr>
<td>Char compressive strength</td>
<td>22.553 N/mm(^2)</td>
</tr>
</tbody>
</table>

**Design lateral strength by arching (Cl. 6.3.2)**

- Case 1 - normal loading: 18.795 kN/m\(^2\)
- Case 2 - abnormal loading: 18.795 kN/m\(^2\)
- Case 3 - protected member: 37.589 kN/m\(^2\)
Location: Wall on grid line A/1-2

Lateral resistance of vertically loaded single leaf wall by arching

This Calculation is in accordance with EC6 - Design of masonry structures Part 1-1: General rules for reinforced and unreinforced masonry structures.

The form of this calculation is in accordance with section 6 of the code of practice.

Wk = design lateral strength

Clear height of wall \( h = 0.8 \) m
Thickness of wall \( t = 450 \) mm
Length of wall panel \( L = 1 \) m
Support restraint factor \( rf = 0.75 \)
Mortar designation to Table NA.2 \( \text{mortar} = 3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ(1)} = 4 \)
Compressive strength of units \( p_0 = 5.2 \) N/mm\(^2\)
Constant K from NA Table NA.4 \( K(1) = 0.75 \)
Normalised mean comp strength \( f_{bn}(1) = 5.2 \) N/mm\(^2\)
Partial safety factor (material) \( \gamma_M = 3 \)
Permanent Load(1) \( G_k(1) = 50 \) kN/m
Variable Load(1) \( Q_k(1) = 20 \) kN/m
Permanent Load(2) \( G_k(2) = 8 \) kN/m
Variable Load(2) \( Q_k(2) = 5 \) kN/m
Ecc. of floor load on inner leaf \( e_{cc}(2) = 75 \) mm
Perm actions (upper levels) \( \gamma_G = 1.35 \)
Perm actions (floor) \( \gamma_G = 1.35 \)
Variable actions (vertical load) \( \gamma_Q = 1.5 \)

Weight of wall at mid-height \( G_{kws} = 1.8 \) kN/m
horizontal loads (e.g. wind load) \( e_{he} = 0 \) mm
loads (e.g. wind load) \( e_{hm} = 0 \) mm
Design moment at mid-height \( M_{md} = 0.84078 \) kNm/m
Leading variable load factor $\psi_1$ \(\psi_1=0.5\)
Other variable load factor $\psi_2$ \(\psi_2=0.3\)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height</td>
<td>0.8 m</td>
</tr>
<tr>
<td>Wall length</td>
<td>1 m</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>450 mm</td>
</tr>
<tr>
<td>Char compressive strength</td>
<td>3.6048 N/mm$^2$</td>
</tr>
</tbody>
</table>

Design lateral strength by arching (Cl. 6.3.2)
- Case 1 - normal loading \(380.19\) kN/m$^2$
- Case 2 - abnormal loading \(380.19\) kN/m$^2$
- Case 3 - protected member \(760.39\) kN/m$^2$
Location: Wall on grid line A/1-2

Lateral resistance of vertically loaded single leaf wall by arching

This Calculation is in accordance with EC6 - Design of masonry structures Part 1-1: General rules for reinforced and unreinforced masonry structures.

The form of this calculation is in accordance with section 6 of the code of practice.

Wk = design lateral strength

Clear height of wall h = 2 m
Thickness of wall t = 100 mm
Length of wall panel L = 3.5 m
Support restraint factor rf = 0.75
Mortar designation to Table NA.2 mortar = 4
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1) = 4
Compressive strength of units po = 10.4 N/mm²
Constant K from NA Table NA.4 K(1) = 0.75
Normalised mean comp strength fbn(1) = 10.4 N/mm²
Partial safety factor (material) gamM = 3
Permanent Load(1) Gk(1) = 20 kN/m
Variable Load(1) Qk(1) = 10 kN/m
Permanent Load(2) Gk(2) = 8 kN/m
Variable Load(2) Qk(2) = 5 kN/m
Ecc. of floor load on inner leaf ecc(2) = 16.667 mm
Perm actions (upper levels) gamG1 = 1.35
Perm actions (floor) gamG2 = 1.35
Variable actions (vertical load) gamQ = 1.5
Weight of wall at mid-height Gkws = 1 kN/m
horizontal loads (e.g. wind load) ehe = 0 mm
loads (e.g. wind load) ehm = 0 mm
Design moment at mid-height Mmd = 0.1871 kNm/m
LEADING VARIABLE LOAD FACTOR $\psi_1$ psi1=0.5
Other variable load factor $\psi_2$ psi2=0.3

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height</td>
<td>2 m</td>
</tr>
<tr>
<td>Wall length</td>
<td>3.5 m</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char compressive strength</td>
<td>4.7566 N/mm$^2$</td>
</tr>
</tbody>
</table>

Design lateral strength by arching (Cl. 6.3.2)

Case 1 - normal loading 3.9638 kN/m$^2$
Case 2 - abnormal loading 3.9638 kN/m$^2$
Location: Column on grid line A/1

Masonry column constructed
with bricks or blocks


Height of col perp to major axis  \( h_{xx} = 2.7 \text{ m} \)
Height of col perp to minor axis  \( h_{yy} = 2.7 \text{ m} \)
Width of column                     \( b = 450 \text{ mm} \)
Thickness of column                 \( t = 250 \text{ mm} \)
Effective ht for major axis bend    \( h_{ef{xx}} = 5.4 \text{ m} \)
Effective ht for minor axis bend    \( h_{ef{yy}} = 2.7 \text{ m} \)
Mortar designation to Table 1       mortar=3
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=1
Compressive Strength of Units       \( p_o = 20 \text{ N/mm}^2 \)
Partial safety factor material      \( \gamma_m = 2.5 \)
Dead load at top                    \( DL = 25 \text{ kN} \)
Live load at top                    \( LL = 60 \text{ kN} \)
Self weight                         \( SW = 2.89 \text{ kN} \)
Eccentricity for major axis bend    \( ex' = 20 \text{ mm} \)
Eccentricity for minor axis bend    \( ey' = 20 \text{ mm} \)

**DESIGN**
Major axis column height  \( 2.7 \text{ m} \)

**SUMMARY**
Minor axis column height  \( 2.7 \text{ m} \)
Column width              \( 450 \text{ mm} \)
Column thickness          \( 250 \text{ mm} \)
Char comp strength        \( 4.3438 \text{ N/mm}^2 \)
Slenderness ratio         \( 12 \)
Design vertical load      \( 135.05 \text{ kN} \)
Vertical load resistance  \( 174.75 \text{ kN} \)
**Location:** Column on grid line A/1-2

**Masonry column constructed**

**with bricks or blocks**

Design column in accordance with BS5628-1:2005 the Code of Practice for use of masonry. Part 1 - Structural use of unreinforced masonry.

Height of col perp to major axis \( h_{xx} = 2.7 \text{ m} \)
Height of col perp to minor axis \( h_{yy} = 2.7 \text{ m} \)
Width of column \( b = 350 \text{ mm} \)
Thickness of column \( t = 200 \text{ mm} \)
Effective ht for major axis bend \( h_{effxx} = 2.7 \text{ m} \)
Effective ht for minor axis bend \( h_{effyy} = 2.7 \text{ m} \)
Mortar designation to Table 1 \( \text{mortar} = 1 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 4 \)
Height \( h_b = 215 \text{ mm} \)
Least horiz dimension \( l_{hd} = 100 \text{ mm} \)
Compressive Strength of Units \( p_0 = 10.4 \text{ N/mm}^2 \)
Partial safety factor material \( \gamma_m = 2.5 \)
Dead load at top \( D_L = 25 \text{ kN} \)
Live load at top \( L_L = 60 \text{ kN} \)
Self weight \( S_W = 1.8 \text{ kN} \)
Eccentricity for major axis bend \( e_{xx} = 20 \text{ mm} \)
Eccentricity for minor axis bend \( e_{yy} = 20 \text{ mm} \)

**DESIGN**

<table>
<thead>
<tr>
<th>Major axis column height</th>
<th>2.7 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minor axis column height</td>
<td>2.7 m</td>
</tr>
<tr>
<td>Column width</td>
<td>350 mm</td>
</tr>
<tr>
<td>Column thickness</td>
<td>200 mm</td>
</tr>
<tr>
<td>Char comp strength</td>
<td>7.084 N/mm²</td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td>13.5</td>
</tr>
<tr>
<td>Design vertical load</td>
<td>133.52 kN</td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td>157.09 kN</td>
</tr>
</tbody>
</table>

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SCALE 5.48  Office 1007  Proforma 515
Location: Column on grid line A/1-2

Masonry column constructed
with bricks or blocks


Height of col perp to major axis  \( h_{xx} = 2.7 \) m
Height of col perp to minor axis  \( h_{yy} = 2.7 \) m
Width of column  \( b = 450 \) mm
Thickness of column  \( t = 150 \) mm
Effective ht for major axis bend  \( h_{efxx} = 5.4 \) m
Effective ht for minor axis bend  \( h_{efyy} = 2.7 \) m
Mortar designation to Table 1  mortar=2
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Height  \( h_{b} = 215 \) mm
Least horiz dimension  \( l_{hd} = 100 \) mm
Compressive Strength of Units  \( p_{o} = 10.4 \) N/mm\(^2\)
Partial safety factor material  \( \gamma_{mm} = 3.1 \)
Dead load at top  \( D_{L} = 25 \) kN
Live load at top  \( L_{L} = 25 \) kN
Self weight  \( S_{W} = 1.73 \) kN
Eccentricity for major axis bend  \( e_{x}' = 20 \) mm
Eccentricity for minor axis bend  \( e_{y}' = 20 \) mm

DESIGN
Major axis column height  2.7 m
SUMMARY
Minor axis column height  2.7 m
Column width  450 mm
Column thickness  150 mm
Char comp strength  6.7305 N/mm\(^2\)
Slenderness ratio  18
Design vertical load  77.422 kN
Vertical load resistance  96.235 kN
Location: Column on grid line A/1-2

Masonry column constructed
with bricks or blocks

Design column in accordance
with BS5628-1:2005 the Code of
practice for use of masonry.
Part 1 - Structural use of
unreinforced masonry.

Height of col perp to major axis  $h_{xx}=12$ m
Height of col perp to minor axis  $h_{yy}=12$ m
Width of column  $b=600$ mm
Thickness of column  $t=600$ mm
Effective ht for major axis bend  $h_{efxx}=12$ m
Effective ht for minor axis bend  $h_{efyy}=12$ m
Mortar designation to Table 1  mortar=3
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Height  $h_b=215$ mm
Least horiz dimension  $l_{hd}=100$ mm
Compressive Strength of Units  $p_0=60$ N/mm$^2$
Partial safety factor material  $\gamma_m=3.1$
Dead load at top  $D_L=600$ kN
Live load at top  $L_L=200$ kN
Self weight  $S_W=19.24$ kN
Eccentricity for major axis bend  $e_x'=20$ mm
Eccentricity for minor axis bend  $e_y'=20$ mm

| DESIGN    | Major axis column height 12 m |
| SUMMARY   | Minor axis column height 12 m |
|           | Column width 600 mm           |
|           | Column thickness 600 mm       |
|           | Char comp strength 16.8 N/mm$^2$|
|           | Slenderness ratio 20          |
|           | Design vertical load 1186.9 kN|
|           | Vertical load resistance 1365.7 kN |
Location: Column on grid line A/1-2

Masonry column constructed with bricks or blocks


Height of col perp to major axis  hxx=1.8 m
Height of col perp to minor axis  hyy=1.8 m
Width of column                  b=300 mm
Thickness of column              t=100 mm
Effective ht for major axis bend  hefxx=3.6 m
Effective ht for minor axis bend  hefyy=1.8 m
Mortar designation to Table 1    mortar=4
Masonry type  (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Height                          hb=215 mm
Least horiz dimension            lhd=100 mm
Compressive Strength of Units   po=17.5 N/mm²
Partial safety factor material   gammam=3.1
Dead load at top                DL=5 kN
Live load at top                LL=10 kN
Self weight                     SW=0.5 kN
Eccentricity for major axis bend ex'=20 mm
Eccentricity for minor axis bend ey'=20 mm

DESIGN
Major axis column height         1.8 m
SUMMARY
Minor axis column height         1.8 m
Column width                     300 mm
Column thickness                 100 mm
Char comp strength               6.7795 N/mm²
Slenderness ratio                18
Design vertical load             23.7 kN
Vertical load resistance         34.676 kN
Location: Column on grid line A/1

Masonry column constructed with bricks or blocks

Design column in accordance with EC6 - Design of masonry structures Part 1-1: General rules for reinforced and unreinforced masonry.

Column dimensions

- Height of col perp to major axis: $h_{yy} = 2.7$ m
- Height of col perp to minor axis: $h_{zz} = 2.7$ m
- Width of column: $b = 450$ mm
- Thickness of column: $t = 250$ mm
- Effective ht for major axis bend: $h_{ef_{yy}} = 5.4$ m
- Effective ht for minor axis bend: $h_{ef_{zz}} = 2.7$ m

Masonry details

- Mortar designation to Table 1: $\text{mortar}=3$
- Masonry type (1=clay 2=cal silicate 3=conc brick 4=block): $\text{typ}(1)=1$
- Compressive strength of units: $p_0 = 20$ N/mm$^2$
- Constant K from NA Table NA.4: $K(1)=0.5$
- Height of masonry unit: $h_u = 65$ mm
- Least horiz. dimension of units: $l_{hd} = 100$ mm
- Partial safety factor material: $\gamma_{M}=2.3$

Load factors

- Permanent actions: $\gamma_G=1.35$
- Variable actions (vertical load): $\gamma_Q=1.5$

Characteristic loads

- Permanent load at top: $G_k = 25$ kN
- Variable load at top: $Q_k = 60$ kN
- Column self weight: $G_{ksw} = 2.89$ kN
- Eccentricity for major axis bend: $e_{y}' = 20$ mm
- Eccentricity for minor axis bend: $e_{z}' = 25$ mm
**Design load resistance**

Eccentricity at the top or bottom of the column resulting from horizontal loads (e.g. wind load) \( e_{he} = 0 \text{ mm} \)

Design moment at mid-height \( M_{md} = 1.9148 \text{ kNm} \)

Eccentricity at mid-height of the column resulting from horizontal loads (e.g. wind load) \( e_{hm} = 0 \text{ mm} \)

<table>
<thead>
<tr>
<th><strong>DESIGN</strong></th>
<th>Major axis column height 2.7 m</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SUMMARY</strong></td>
<td>Minor axis column height 2.7 m</td>
</tr>
<tr>
<td>Column width</td>
<td>450 mm</td>
</tr>
<tr>
<td>Column thickness (depth)</td>
<td>250 mm</td>
</tr>
<tr>
<td>Char comp strength</td>
<td>5.5068 N/mm²</td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td>12</td>
</tr>
<tr>
<td>Design vertical load</td>
<td>127.65 kN</td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td>197.51 kN</td>
</tr>
</tbody>
</table>
Location: Column on grid line A/1-2

Masonry column constructed
with bricks or blocks

Design column in accordance
with EC6 - Design of masonry
structures Part 1-1: General
rules for reinforced and
unreinforced masonry.

Column dimensions

- Height of col perp to major axis: hyy = 2.7 m
- Height of col perp to minor axis: hzz = 2.7 m
- Width of column: b = 350 mm
- Thickness of column: t = 200 mm
- Effective ht for major axis bend: hefyy = 2.7 m
- Effective ht for minor axis bend: hefzz = 2.7 m

Masonry details

- Mortar designation to Table 1: mortar = 1
- Masonry type (1=clay 2=cal silicate 3=conc brick 4=block): typ(1) = 4
- Compressive strength of units: po = 10.4 N/mm²
- Constant K from NA Table NA.4: K(1) = 0.75
- Height of masonry unit: hu = 215 mm
- Least horiz. dimension of units: lhd = 100 mm
- Partial safety factor material: gamM = 2.3

Load factors

- Permanent actions: gamG = 1.35
- Variable actions (vertical load): gamQ = 1.5

Characteristic loads

- Permanent load at top: Gk = 25 kN
- Variable load at top: Qk = 60 kN
- Column self weight: Gksw = 1.8 kN
- Eccentricity for major axis bend: ey' = 20 mm
- Eccentricity for minor axis bend: ez' = 25 mm
Design load resistance

Eccentricity at the top or bottom of the column resulting from horizontal loads (e.g. wind load) ehe=0 mm
Design moment at mid-height Mmd=2.2892 kNm
Eccentricity at mid-height of the column resulting from horizontal loads (e.g. wind load) ehm=0 mm

| DESIGN | Minor axis column height | 2.7 m |
| SUMMARY | Minor axis column height | 2.7 m |
| Column width | 350 mm |
| Column thickness (depth) | 200 mm |
| Char comp strength | 9.2832 N/mm² |
| Slenderness ratio | 13.5 |
| Design vertical load | 126.18 kN |
| Vertical load resistance | 176.79 kN |
Location: Column on grid line A/1-2

Masonry column constructed with bricks or blocks

Design column in accordance with EC6 - Design of masonry structures Part 1-1: General rules for reinforced and unreinforced masonry.

Column dimensions

- Height of col perp to major axis: $h_{yy} = 2.7$ m
- Height of col perp to minor axis: $h_{zz} = 2.7$ m
- Width of column: $b = 450$ mm
- Thickness of column: $t = 150$ mm
- Effective ht for major axis bend: $h_{efyy} = 5.4$ m
- Effective ht for minor axis bend: $h_{efzz} = 2.7$ m

Masonry details

- Mortar designation to Table 1: mortar = 2
- Masonry type (1=clay 2=cal silicate 3=conc brick 4=block): typ(1) = 4
- Compressive strength of units: $p_0 = 10.4$ N/mm$^2$
- Constant K from NA Table NA.4: $K(1) = 0.75$
- Height of masonry unit: $h_u = 215$ mm
- Least horiz. dimension of units: $l_{hd} = 100$ mm
- Partial safety factor material: $\gamma_m = 2.7$

Load factors

- Permanent actions: $\gamma_g = 1.35$
- Variable actions (vertical load): $\gamma_q = 1.5$

Characteristic loads

- Permanent load at top: $G_k = 25$ kN
- Variable load at top: $Q_k = 25$ kN
- Column self weight: $G_{ksw} = 1.73$ kN
- Eccentricity for major axis bend: $e_y' = 20$ mm
- Eccentricity for minor axis bend: $e_z' = 25$ mm
Design load resistance

Eccentricity at the top or bottom of the column resulting from horizontal loads (e.g. wind load) \( e_{he} = 0 \) mm

Design moment at mid-height \( M_{md} = 1.1038 \) kNm

Eccentricity at mid-height of the column resulting from horizontal loads (e.g. wind load) \( e_{hm} = 0 \) mm

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Minor axis column height</th>
<th>2.7 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Major axis column height</td>
<td>2.7 m</td>
</tr>
<tr>
<td>Column width</td>
<td></td>
<td>450 mm</td>
</tr>
<tr>
<td>Column thickness (depth)</td>
<td></td>
<td>150 mm</td>
</tr>
<tr>
<td>Char comp strength</td>
<td></td>
<td>7.4781 N/mm²</td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td></td>
<td>18</td>
</tr>
<tr>
<td>Design vertical load</td>
<td></td>
<td>73.586 kN</td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td></td>
<td>89.954 kN</td>
</tr>
</tbody>
</table>
Location: Column on grid line A/1-2

Masonry column constructed with bricks or blocks

Design column in accordance with EC6 - Design of masonry structures Part 1-1: General rules for reinforced and unreinforced masonry.

Column dimensions

- Height of col perp to major axis: $h_{yy}=12$ m
- Height of col perp to minor axis: $h_{zz}=12$ m
- Width of column: $b=600$ mm
- Thickness of column: $t=600$ mm
- Effective ht for major axis bend: $h_{efyy}=12$ m
- Effective ht for minor axis bend: $h_{efzz}=12$ m

Masonry details

- Mortar designation to Table 1: mortar=3
- Masonry type (1=clay 2=cal silicate 3=conc brick 4=block): typ(1)=4
- Compressive strength of units: $p_0=60$ N/mm$^2$
- Constant $K$ from NA Table NA.4: $K(1)=0.75$
- Height of masonry unit: $h_u=215$ mm
- Least horiz. dimension of units: $l_{hd}=100$ mm
- Partial safety factor material: $\gamma_M=2.7$

Load factors

- Permanent actions: $\gamma_G=1.35$
- Variable actions (vertical load): $\gamma_Q=1.5$

Characteristic loads

- Permanent load at top: $G_k=600$ kN
- Variable load at top: $Q_k=200$ kN
- Column self weight: $G_{ksw}=19.24$ kN
- Eccentricity for major axis bend: $e_y'=20$ mm
- Eccentricity for minor axis bend: $e_z'=25$ mm
Design load resistance

Eccentricity at the top or bottom of the column resulting from horizontal loads (e.g. wind load) ehe=0 mm
Design moment at mid-height Mmd=17.04 kNm
Eccentricity at mid-height of the column resulting from horizontal loads (e.g. wind load) ehm=0 mm

**DESIGN**
- Major axis column height: 12 m

**SUMMARY**
- Minor axis column height: 12 m
- Column width: 600 mm
- Column thickness (depth): 600 mm
- Char comp strength: 25.021 N/mm²
- Slenderness ratio: 20
- Design vertical load: 1136 kN
- Vertical load resistance: 1948.8 kN
Location: Column on grid line A/1-2

Masonry column constructed with bricks or blocks

Design column in accordance with EC6 - Design of masonry structures Part 1-1: General rules for reinforced and unreinforced masonry.

Column dimensions

Height of col perp to major axis  \( h_{yy} = 1.8 \) m
Height of col perp to minor axis  \( h_{zz} = 1.8 \) m
Width of column  \( b = 300 \) mm
Thickness of column  \( t = 100 \) mm
Effective ht for major axis bend  \( h_{efy} = 3.6 \) m
Effective ht for minor axis bend  \( h_{efz} = 1.8 \) m

Masonry details

Mortar designation to Table 1  \( \text{mortar}=4 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block)  \( \text{typ}(1)=4 \)
Compressive strength of units  \( p_0 = 17.5 \) N/mm\(^2\)
Constant K from NA Table NA.4  \( K(1)=0.75 \)
Height of masonry unit  \( h_u = 215 \) mm
Least horiz. dimension of units  \( l_{hd} = 100 \) mm
Partial safety factor material  \( \gamma_M = 2.7 \)

Load factors

Permanent actions  \( \gamma_G = 1.35 \)
Variable actions (vertical load)  \( \gamma_Q = 1.5 \)

Characteristic loads

Permanent load at top  \( G_k = 5 \) kN
Variable load at top  \( Q_k = 10 \) kN
Column self weight  \( G_{ksw} = 0.5 \) kN
Eccentricity for major axis bend  \( e_{y'} = 20 \) mm
Eccentricity for minor axis bend  \( e_{z'} = 25 \) mm
Design load resistance

Eccentricity at the top or bottom of the column resulting from horizontal loads (e.g. wind load) $e_{he}=0$ mm
Design moment at mid-height $M_{md}=0.38294$ kNm
Eccentricity at mid-height of the column resulting from horizontal loads (e.g. wind load) $e_{hm}=0$ mm

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Minor axis column height</th>
<th>1.8 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Minor axis column height</td>
<td>1.8 m</td>
</tr>
<tr>
<td>Column width</td>
<td></td>
<td>300 mm</td>
</tr>
<tr>
<td>Column thickness (depth)</td>
<td></td>
<td>100 mm</td>
</tr>
<tr>
<td>Char comp strength</td>
<td></td>
<td>6.777 N/mm²</td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td></td>
<td>18</td>
</tr>
<tr>
<td>Design vertical load</td>
<td></td>
<td>22.425 kN</td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td></td>
<td>25.035 kN</td>
</tr>
</tbody>
</table>
Location: Outer leaf reinforced

Length of wall panel
Height of wall panel
Outer leaf density
Thickness of outer leaf
Inner leaf density
Thickness of inner leaf
Mortar designation (Table 1)
Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block)
Water absorption %age of leaf
Compressive strength of units
Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block)
Height
Least horiz dimension
Compressive strength of units
Partial safety factor materials
Char wind load per unit area
Additional vert. DL on panel
Additional vert. DL on panel
Effective depth of reinforcement
Diameter (main reinforcing bars)
Spacing of reinforcement
Reinforcement tensile strength

DESIGN SUMMARY
Panel length
Panel height
Thickness outer leaf
Thickness inner leaf
Effective wall thickness
Char wind pressure
Char wind capacity
Design shear on sides
Shear strength of sides
Design shear on founds
Shear strength of founds
Material safety factor
Load safety factor
**Location:** Inner leaf reinforced

Panel reference E in BS5628 Table 8

Length of wall panel \( L = 7.5 \text{ m} \)
Height of wall panel \( h = 6 \text{ m} \)
Outer leaf density \( \text{den}(1) = 20 \text{ kN/m}^3 \)
Thickness of outer leaf \( t(1) = 102.5 \text{ mm} \)
Inner leaf density \( \text{den}(2) = 10 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2) = 140 \text{ mm} \)
Mortar designation (Table 1) \( \text{mortar} = 3 \)
Outer leaf \( (1=\text{clay} 2=\text{calc silicate} 3=\text{conc brick} 4=\text{block}) \) \( \text{typ}(1) = 1 \)
Water absorption %age of leaf \( \text{water1} = 12 \)
Compressive strength of units \( \text{po}(1) = 20 \text{ N/mm}^2 \)
Inner leaf \( (1=\text{clay} 2=\text{calc silicate} 3=\text{conc brick} 4=\text{block}) \) \( \text{typ}(2) = 4 \)
Height \( \text{hb} = 215 \text{ mm} \)
Least horiz dimension \( \text{Ihd2} = 140 \text{ mm} \)
Compressive strength of units \( \text{po}(2) = 7.3 \text{ N/mm}^2 \)
Partial safety factor materials \( \gamma_{\text{mm}} = 3.5 \)
Char wind load per unit area \( \text{WL} = 0.4 \text{ kN/m}^2 \)
Additional vert. DL on panel \( w(1) = 0 \text{ kN/m} \)
Additional vert. DL on panel \( w(2) = 0 \text{ kN/m} \)
Effective depth of reinforcement \( d2 = 116 \text{ mm} \)
Diameter of main reinforcing bars \( \text{bard2} = 5 \text{ mm} \)
Spacing of reinforcement \( \text{spacr2} = 450 \text{ mm} \)
Reinforcement tensile strength \( \text{fy} = 460 \text{ N/mm}^2 \)

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel length</td>
<td>7.5 m</td>
</tr>
<tr>
<td>Panel height</td>
<td>6 m</td>
</tr>
<tr>
<td>Thickness outer leaf</td>
<td>102.5 mm</td>
</tr>
<tr>
<td>Thickness inner leaf</td>
<td>140 mm</td>
</tr>
<tr>
<td>Effective wall thickness</td>
<td>161.67 mm</td>
</tr>
<tr>
<td>Char wind pressure</td>
<td>0.4 kN/m²</td>
</tr>
<tr>
<td>Char wind capacity</td>
<td>0.40829 kN/m²</td>
</tr>
<tr>
<td>Design shear on sides</td>
<td>10.8 kN</td>
</tr>
<tr>
<td>Shear strength of sides</td>
<td>240.6 kN</td>
</tr>
<tr>
<td>Design shear on founds</td>
<td>10.8 kN</td>
</tr>
<tr>
<td>Shear strength of founds</td>
<td>142.66 kN</td>
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<tr>
<td>Material safety factor</td>
<td>3.5</td>
</tr>
<tr>
<td>Load safety factor</td>
<td>1.2</td>
</tr>
</tbody>
</table>
**Location:** Both leaves reinforced

**Panel reference A**

Panel reference in BS5628 Table 8

---

**Length of wall panel**  
\( L = 8 \text{ m} \)

**Height of wall panel**  
\( h = 4 \text{ m} \)

**Outer leaf density**  
\( \text{den}(1) = 20 \text{ kN/m}^3 \)

**Thickness of outer leaf**  
\( t(1) = 102.5 \text{ mm} \)

**Inner leaf density**  
\( \text{den}(2) = 8 \text{ kN/m}^3 \)

**Thickness of inner leaf**  
\( t(2) = 150 \text{ mm} \)

**Mortar designation (Table 1)**  
\( \text{mortar} = 3 \)

**Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block) typ(1)=1**

**Water absorption %age of leaf**  
\( \text{water1} = 15 \)

**Compressive strength of units**  
\( \text{po}(1) = 20 \text{ N/mm}^2 \)

**Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block) typ(2)=4**

**Height**  
\( h_b = 215 \text{ mm} \)

**Least horiz dimension**  
\( l_{hd2} = 150 \text{ mm} \)

**Compressive strength of units**  
\( \text{po}(2) = 5.2 \text{ N/mm}^2 \)

**Partial safety factor materials**  
\( \text{gammam} = 2.5 \)

**Char wind load per unit area**  
\( \text{WL} = 0.35 \text{ kN/m}^2 \)

**Additional vert.DL on panel**  
\( w(1) = 0 \text{ kN/m} \)

**Additional vert.DL on panel**  
\( w(2) = 0 \text{ kN/m} \)

**Effective depth of reinforcement**  
\( d_1 = 81 \text{ mm} \)

**Diameter (main reinforcing bars)**  
\( \text{bard1} = 4 \text{ mm} \)

**Spacing of reinforcement**  
\( \text{spacr1} = 225 \text{ mm} \)

**Reinforcement tensile strength**  
\( f_y = 485 \text{ N/mm}^2 \)

**Effective depth of reinforcement**  
\( d_2 = 125 \text{ mm} \)

**Diameter of main reinforcing bars**  
\( \text{bard2} = 4 \text{ mm} \)

**Spacing of reinforcement**  
\( \text{spacr2} = 225 \text{ mm} \)

**Reinforcement tensile strength**  
\( f_y = 485 \text{ N/mm}^2 \)
<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Panel length</th>
<th>8 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel height</td>
<td>4 m</td>
<td></td>
</tr>
<tr>
<td>Thickness outer leaf</td>
<td>102.5 mm</td>
<td></td>
</tr>
<tr>
<td>Thickness inner leaf</td>
<td>150 mm</td>
<td></td>
</tr>
<tr>
<td>Effective wall thickness</td>
<td>168.33 mm</td>
<td></td>
</tr>
<tr>
<td>Char wind pressure</td>
<td>0.35 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Char wind capacity</td>
<td>0.47265 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Design shear on sides</td>
<td>6.72 kN</td>
<td></td>
</tr>
<tr>
<td>Shear strength of sides</td>
<td>118 kN</td>
<td></td>
</tr>
<tr>
<td>Design shear on founders</td>
<td>13.44 kN</td>
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</tr>
<tr>
<td>Shear strength of founders</td>
<td>143.66 kN</td>
<td></td>
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<tr>
<td>Material safety factor</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>Load safety factor</td>
<td>1.2</td>
<td></td>
</tr>
</tbody>
</table>
Location: Unreinforced wall panel

Length of wall panel \( L = 4 \text{ m} \)
Height of wall panel \( h = 3 \text{ m} \)
Outer leaf density \( \text{den}(1) = 10 \text{ kN/m}^3 \)
Thickness of outer leaf \( t(1) = 100 \text{ mm} \)
Inner leaf density \( \text{den}(2) = 10 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2) = 100 \text{ mm} \)
Mortar designation (Table 1) \( \text{mortar} = 3 \)
Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block) \( \text{typ}(1) = 4 \)
Height \( \text{hb} = 215 \text{ mm} \)
Least horiz dimension \( \text{lhd}1 = 100 \text{ mm} \)
Compressive strength of units \( \text{po}(1) = 10.4 \text{ N/mm}^2 \)
Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block) \( \text{typ}(2) = 4 \)
Height \( \text{hb} = 215 \text{ mm} \)
Least horiz dimension \( \text{lhd}2 = 100 \text{ mm} \)
Compressive strength of units \( \text{po}(2) = 10.4 \text{ N/mm}^2 \)
Partial safety factor materials \( \text{gammam} = 3.1 \)
Char wind load per unit area \( \text{WL} = 0.85 \text{ kN/m}^2 \)
Additional vert DL on panel \( \text{w}(1) = 0 \text{ kN/m} \)
Additional vert DL on panel \( \text{w}(2) = 0 \text{ kN/m} \)

**DESIGN SUMMARY**

- Panel length \( 4 \text{ m} \)
- Panel height \( 3 \text{ m} \)
- Thickness outer leaf \( 100 \text{ mm} \)
- Thickness inner leaf \( 100 \text{ mm} \)
- Effective wall thickness \( 133.33 \text{ mm} \)
- Char wind pressure \( 0.85 \text{ kN/m}^2 \)
- Char wind capacity \( 0.94991 \text{ kN/m}^2 \)
- Design shear on sides \( 6.12 \text{ kN} \)
- Shear strength of sides \( 84 \text{ kN} \)
- Design shear on founds \( 12.24 \text{ kN} \)
- Shear strength of founds \( 53.184 \text{ kN} \)
- Material safety factor \( 3.1 \)
- Load safety factor \( 1.2 \)
**Location: Single skin wall reinforced**

![Wall Diagram]

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of wall panel</td>
<td>L=4.5 m</td>
</tr>
<tr>
<td>Height of wall panel</td>
<td>h=4.5 m</td>
</tr>
<tr>
<td>Wall density</td>
<td>den(1)=20 kN/m²</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>t(1)=102.5 mm</td>
</tr>
<tr>
<td>Mortar designation (Table 1)</td>
<td>mortar=3</td>
</tr>
<tr>
<td>Wall type (1=clay 2=calc silicate 3=conc brick 4=block)</td>
<td>typ(1)=1</td>
</tr>
<tr>
<td>Water absorption %age of leaf</td>
<td>water1=15</td>
</tr>
<tr>
<td>Compressive strength of units</td>
<td>po(1)=20 N/mm²</td>
</tr>
<tr>
<td>Partial safety factor materials</td>
<td>gammam=3.5</td>
</tr>
<tr>
<td>Char wind load per unit area</td>
<td>WL=0.65 kN/m²</td>
</tr>
<tr>
<td>Additional vert.DL on panel</td>
<td>w(1)=5 kN/m</td>
</tr>
<tr>
<td>Effective depth of reinforcement</td>
<td>d1=85 mm</td>
</tr>
<tr>
<td>Diameter (main reinforcing bars)</td>
<td>bard1=4 mm</td>
</tr>
<tr>
<td>Spacing of reinforcement</td>
<td>spacr1=150 mm</td>
</tr>
<tr>
<td>Reinforcement tensile strength</td>
<td>fy=485 N/mm²</td>
</tr>
</tbody>
</table>

**DESIGN SUMMARY**

- Panel length: 4.5 m
- Panel height: 4.5 m
- Single skin wall thickness: 102.5 mm
- Char wind pressure: 0.65 kN/m²
- Char wind capacity: 0.84451 kN/m²
- Design shear on sides: 7.8975 kN
- Shear strength of sides: 92.25 kN
- Design shear on founds: 7.8975 kN
- Shear strength of founds: 41.502 kN
- Material safety factor: 3.5
- Load safety factor: 1.2
Location: Single skin wall reinforced

Panel reference E
in BS5628 Table 8

Length of wall panel \( L = 1.8 \text{ m} \)
Height of wall panel \( h = 4.5 \text{ m} \)
Wall density \( \text{den}(1) = 20 \text{ kN/m}^3 \)
Wall thickness \( t(1) = 215 \text{ mm} \)
Mortar designation (Table 1) \( \text{mortar}=3 \)
Wall type \( 1=\text{clay} 2=\text{calc silicate} 3=\text{conc brick} 4=\text{block} \) \( \text{typ}(1)=4 \)
Height \( \text{hb}=215 \text{ mm} \)
Least horiz.dimension \( \text{lhd1}=215 \text{ mm} \)
Compressive strength of units \( \text{po}(1)=7.3 \text{ N/mm}^2 \)
Partial safety factor materials \( \text{gammam}=3.5 \)
Char wind load per unit area \( \text{WL}=4.5 \text{ kN/m}^2 \)
Additional vert.DL on panel \( w(1)=0 \text{ kN/m} \)
Effective depth of reinforcement \( \text{d1}=150 \text{ mm} \)
Diameter (main reinforcing bars) \( \text{bard1}=4.5 \text{ mm} \)
Spacing of reinforcement \( \text{spacr1}=225 \text{ mm} \)
Reinforcement tensile strength \( \text{fy}=460 \text{ N/mm}^2 \)

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel length</td>
<td>1.8 m</td>
</tr>
<tr>
<td>Panel height</td>
<td>4.5 m</td>
</tr>
<tr>
<td>Single skin wall thickness</td>
<td>215 mm</td>
</tr>
<tr>
<td>Char wind pressure</td>
<td>4.5 kN/m²</td>
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<tr>
<td>Char wind capacity</td>
<td>6.8771 kN/m²</td>
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<tr>
<td>Design shear on sides</td>
<td>25.515 kN</td>
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<tr>
<td>Shear strength of sides</td>
<td>135.45 kN</td>
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<tr>
<td>Design shear on founds</td>
<td>25.515 kN</td>
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<tr>
<td>Shear strength of founds</td>
<td>30.743 kN</td>
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<td>Material safety factor</td>
<td>3.5</td>
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<tr>
<td>Load safety factor</td>
<td>1.4</td>
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</tbody>
</table>
Location: Outer leaf reinforced

Panel reference E in EC6 Annex E

Length of wall panel \( L = 4.5 \, \text{m} \)
Height of wall panel \( h = 4.5 \, \text{m} \)
Outer leaf density \( \text{den}(1) = 20 \, \text{kN/m}^3 \)
Thickness of outer leaf \( t(1) = 102.5 \, \text{mm} \)
Inner leaf density \( \text{den}(2) = 8 \, \text{kN/m}^3 \)
Thickness of inner leaf \( t(2) = 100 \, \text{mm} \)
Mortar designation to Table NA.2 \( \text{mortar} = 3 \)
Outer leaf (1=clay 2=calc silicate 3=concrete brick 4=block) \( \text{typ}(1) = 1 \)
Water absorption %age of leaf \( \text{water1} = 15 \)
Compressive strength of units \( \text{po}(1) = 20 \, \text{N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1) = 0.5 \)
Height of masonry unit \( h(1) = 65 \, \text{mm} \)
Least horiz. dimension of units \( lhd(1) = 100 \, \text{mm} \)
Inner leaf (1=clay 2=calc silicate 3=concrete brick 4=block) \( \text{typ}(2) = 4 \)
Compressive strength of units \( \text{po}(2) = 2.9 \, \text{N/mm}^2 \)
Constant K from NA Table NA.4 \( K(2) = 0.75 \)
Height of masonry unit \( h(2) = 215 \, \text{mm} \)
Least horiz. dimension of units \( lhd(2) = 100 \, \text{mm} \)
Partial safety factor (material) \( \text{gamM} = 2.4 \)
Character wind load per unit area \( Qkw = 0.6 \, \text{kN/m}^2 \)
Additional vert.DL on panel \( Gk(1) = 0 \, \text{kN/m} \)
Height of outer leaf units \( h(1) = 65 \, \text{mm} \)
Additional vert.DL on panel \( Gk(2) = 0 \, \text{kN/m} \)
Height of inner leaf units \( h(2) = 215 \, \text{mm} \)
Effective depth of reinforcement \( d1 = 75 \, \text{mm} \)
Diameter main reinforcing bars \( b1 = 4 \, \text{mm} \)
Spacing of reinforcement \( s1 = 150 \, \text{mm} \)
Reinforcement tensile strength \( f_{y} = 485 \, \text{N/mm}^2 \)
<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
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<tbody>
<tr>
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<tr>
<td>Panel height</td>
<td>4.5 m</td>
</tr>
<tr>
<td>Thickness outer leaf</td>
<td>102.5 mm</td>
</tr>
<tr>
<td>Thickness inner leaf</td>
<td>100 mm</td>
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<tr>
<td>Effective wall thickness</td>
<td>127.52 mm</td>
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<tr>
<td>Char wind pressure</td>
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<tr>
<td>Char wind capacity</td>
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<td>Design shear on sides</td>
<td>9.1125 kN</td>
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<td>Shear strength of sides</td>
<td>72.9 kN</td>
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<tr>
<td>Design shear on founds</td>
<td>9.1125 kN</td>
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<td>Shear strength of founds</td>
<td>81.211 kN</td>
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<td>Material safety factor</td>
<td>2.4</td>
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<td>Load safety factor</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Outer leaf - use Group 1 units (i.e. solid)
Inner leaf - use Group 1 units (i.e. solid)
Location: Inner leaf reinforced

Panel reference E
in EC6 Annex E

Length of wall panel  L=7.5 m
Height of wall panel  h=6 m
Outer leaf density  den(1)=20 kN/m³
Thickness of outer leaf  t(1)=102.5 mm
Inner leaf density  den(2)=10 kN/m³
Thickness of inner leaf  t(2)=140 mm
Mortar designation to Table NA.2  mortar=3
Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block)  typ(1)=1
Water absorption %age of leaf  water1=12
Compressive strength of units  po(1)=20 N/mm²
Constant K from NA Table NA.4  K(1)=0.5
Height of masonry unit  hu(1)=65 mm
Least horiz. dimension of units  lhd(1)=100 mm
Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block)  typ(2)=4
Compressive strength of units  po(2)=7.3 N/mm²
Constant K from NA Table NA.4  K(2)=0.55
Height of masonry unit  hu(2)=215 mm
Least horiz. dimension of units  lhd(2)=140 mm
Partial safety factor (material)  gamM=2
Charact wind load per unit area  Qkw=0.4 kN/m²
Additional vert.DL on panel  Gk(1)=0 kN/m
Height of outer leaf units  hu(1)=65 mm
Additional vert.DL on panel  Gk(2)=0 kN/m
Height of inner leaf units  hu(2)=215 mm

DESIGN SUMMARY  Panel length  7.5 m
Panel height  6 m
Thickness outer leaf  102.5 mm
Thickness inner leaf  140 mm
Effective wall thickness  156.26 mm
Char wind pressure  0.4 kN/m²
Char wind capacity  0.46072 kN/m²
Design shear on sides  13.5 kN
Shear strength of sides  116.4 kN
Design shear on founds  13.5 kN
Shear strength of founds  167.86 kN
Material safety factor  2
Load safety factor  1.5
Outer leaf - use Group 1 units (i.e. solid)
Inner leaf - use Group 1 units (i.e. solid)
Location: Both leaves reinforced

Panel reference A
in EC6 Annex E

Length of wall panel \(L=8\) m
Height of wall panel \(h=4\) m
Outer leaf density \(\text{den}(1)=20\) kN/m\(^3\)
Thickness of outer leaf \(t(1)=102.5\) mm
Inner leaf density \(\text{den}(2)=8\) kN/m\(^3\)
Thickness of inner leaf \(t(2)=150\) mm
Mortar designation to Table NA.2 \(\text{mortar}=3\)
Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block) \(\text{typ}(1)=1\)
Water absorption %age of leaf \(\text{water}(1)=15\)
Compressive strength of units \(p(1)=20\) N/mm\(^2\)
Constant K from NA Table NA.4 \(K(1)=0.5\)
Normalised mean comp strength \(f_{bn}(1)=17\) N/mm\(^2\)
Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block) \(\text{typ}(2)=4\)
Compressive strength of units \(p(2)=5.2\) N/mm\(^2\)
Constant K from NA Table NA.4 \(K(2)=0.75\)
Normalised mean comp strength \(f_{bn}(2)=5.2\) N/mm\(^2\)
Partial safety factor (material) \(\gamma_M=2\)
Charact wind load per unit area \(Q_{kw}=0.35\) kN/m\(^2\)
Additional vert.DL on panel \(G_k(1)=0\) kN/m
Height of outer leaf units \(h_u(1)=65\) mm
Additional vert.DL on panel \(G_k(2)=0\) kN/m
Height of inner leaf units \(h_u(2)=215\) mm
Effective depth of reinforcement \(d_1=81\) mm
Diameter main reinforcing bars \(b_{ard1}=4\) mm
Spacing of reinforcement \(s_{pcr1}=225\) mm
Reinforcement tensile strength \(f_y=485\) N/mm\(^2\)
Effective depth of reinforcement \(d_2=125\) mm
Diameter main reinforcing bars \(b_{ard2}=4\) mm
Spacing of reinforcement \(s_{pcr2}=225\) mm
Reinforcement tensile strength \(f_y=485\) N/mm\(^2\)
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<td>Panel height</td>
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<td>Thickness outer leaf</td>
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<tr>
<td>Thickness inner leaf</td>
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<td>Char wind capacity</td>
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<td>Shear strength of sides</td>
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<td>Design shear on founds</td>
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<td>Shear strength of founds</td>
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<td>Load safety factor</td>
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<tr>
<td>Outer leaf - use Group 1 units (i.e. solid)</td>
<td></td>
</tr>
<tr>
<td>Inner leaf - use Group 1 units (i.e. solid)</td>
<td></td>
</tr>
</tbody>
</table>
### Location: Unreinforced wall panel

![Diagram of wall panel]

- **Panel reference C**
- **in EC6 Annex E**
- **Length of wall panel** $L=4$ m
- **Height of wall panel** $h=3$ m
- **Outer leaf density** $\text{den}(1)=10$ kN/m³
- **Thickness of outer leaf** $t(1)=100$ mm
- **Inner leaf density** $\text{den}(2)=10$ kN/m³
- **Thickness of inner leaf** $t(2)=100$ mm
- **Mortar designation to Table NA.2** $\text{mortar}=3$
- **Outer leaf (1=clay 2=calc silicate 3=conc brick 4=block)** $\text{typ}(1)=4$
- **Compressive strength of units** $p(1)=10.4$ N/mm²
- **Constant K from NA Table NA.4** $K(1)=0.75$
- **Normalised mean comp strength** $f(1)=10.4$ N/mm²
- **Inner leaf (1=clay 2=calc silicate 3=conc brick 4=block)** $\text{typ}(2)=4$
- **Compressive strength of units** $p(2)=10.4$ N/mm²
- **Constant K from NA Table NA.4** $K(2)=0.75$
- **Normalised mean comp strength** $f(2)=10.4$ N/mm²
- **Partial safety factor (material)** $\gamma_M=2.4$
- **Charact wind load per unit area** $Q_{kw}=0.85$ kN/m²
- **Additional vert. DL on panel** $G_k(1)=0$ kN/m
- **Height of outer leaf units** $h_u(1)=215$ mm
- **Additional vert. DL on panel** $G_k(2)=0$ kN/m
- **Height of inner leaf units** $h_u(2)=215$ mm

### DESIGN SUMMARY

- **Panel length** 4 m
- **Panel height** 3 m
- **Thickness outer leaf** 100 mm
- **Thickness inner leaf** 100 mm
- **Effective wall thickness** 125.93 mm
- **Char wind pressure** 0.85 kN/m²
- **Char wind capacity** 0.97602 kN/m²
- **Design shear on sides** 7.65 kN
- **Shear strength of sides** 36 kN
- **Design shear on founds** 15.3 kN
- **Shear strength of founds** 51.456 kN
- **Material safety factor** 2.4
- **Load safety factor** 1.5

- **Outer leaf - use Group 1 units (i.e. solid)**
- **Inner leaf - use Group 1 units (i.e. solid)**
Location: Single skin wall reinforced

Panel reference E in EC6 Annex E

Length of wall panel \( L = 4.5 \) m
Height of wall panel \( h = 4.5 \) m
Wall density \( \text{den}(1) = 20 \) kN/m\(^2\)
Wall thickness \( t(1) = 102.5 \) mm
Mortar designation to Table NA.2 mortar=3
Wall type \( \text{typ}(1) = 1 \)
Water absorption %age of leaf \( \text{water1} = 15 \)
Compressive strength of units \( \text{po}(1) = 20 \) N/mm\(^2\)
Constant K from NA Table NA.4 \( K(1) = 0.5 \)
Height of masonry unit \( \text{hu}(1) = 65 \) mm
Least horiz. dimension of units \( \text{lhd}(1) = 100 \) mm
Partial safety factor (material) \( \text{gamM} = 2.4 \)
Charact wind load per unit area \( \text{Qkw} = 0.65 \) kN/m\(^2\)
Additional vert.DL on panel \( \text{Gk}(1) = 5 \) kN/m
Height of the units \( \text{hu}(1) = 65 \) mm
Effective depth of reinforcement \( d1 = 85 \) mm
Diameter main reinforcing bars \( \text{bard1} = 4 \) mm
Spacing of reinforcement \( \text{spacr1} = 150 \) mm
Reinforcement tensile strength \( \text{fy} = 485 \) N/mm\(^2\)

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tbody>
<tr>
<td>Panel length</td>
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<tr>
<td>Panel height</td>
<td>4.5 m</td>
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<tr>
<td>Single skin wall thickness</td>
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<tr>
<td>Char wind pressure</td>
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<tr>
<td>Char wind capacity</td>
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<td>Design shear on sides</td>
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<td>Shear strength of sides</td>
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<tr>
<td>Design shear on founds</td>
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<td>Shear strength of founds</td>
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<tr>
<td>Outer leaf - use Group 1 units (i.e. solid)</td>
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</tbody>
</table>
Location: Single skin wall reinforced

Length of wall panel  L=1.8 m
Height of wall panel  h=4.5 m
Wall density  den(1)=20 kN/m³
Wall thickness  t(1)=215 mm
Mortar designation to Table NA.2  mortar=3
Wall type  (1=clay 2=calc silicate 3=conc brick 4=block)  typ(1)=4
Compressive strength of units  po(1)=7.3 N/mm²
Constant K from NA Table NA.4  K(1)=0.75
Normalised mean comp strength  fbn(1)=7.3 N/mm²
Partial safety factor (material)  gamM=2.7
Charact wind load per unit area  Qkw=4.5 kN/m²
Additional vert.DL on panel  Gk(1)=0 kN/m
Height of the units  hu(1)=215 mm
Effective depth of reinforcement  d1=150 mm
Diameter main reinforcing bars  bard1=4.5 mm
Spacing of reinforcement  spacr1=225 mm
Reinforcement tensile strength  fy=460 N/mm²

DESIGN SUMMARY

Panel length  1.8 m
Panel height  4.5 m
Single skin wall thickness  215 mm
Char wind pressure  4.5 kN/m²
Char wind capacity  6.5895 kN/m²
Design shear on sides  27.337 kN
Shear strength of sides  58.05 kN
Design shear on founds  27.337 kN
Shear strength of founds  28.236 kN
Material safety factor  2.7
Load safety factor  1.5
Outer leaf - use Group 1 units (i.e. solid)
Location: Wall on grid line A/1-2

Clear height of wall \( h = 1.7 \) m
Depth of soil \( d = 0 \) m
Thickness of wall \( t = 150 \) mm
Overall thickness of wall \( tp = 450 \) mm
Offset of pier \( T1 = 0 \) mm
Width of pier \( b = 200 \) mm
Spacing of piers (width of wall) \( B = 2 \) m
Partial safety factor materials \( \gamma_{ma} = 2.5 \)
Mortar designation to Table 1 \( \gamma_{ma} = 2 \)
Masonry density \( \rho_{(1)} = 15 \) kN/m\(^3\) 
Typical density \( \gamma_{(1)} = 1 \)
Water absorption %age of leaf \( \text{water} = 7 \)
Soil density \( \rho_{sd} = 18 \) kN/m\(^3\)
Soil angle of internal friction \( \theta' = 30^\circ \)
Surcharge pressure \( \text{surch} = 0 \) kN/m\(^2\)
Wind pressure \( \text{WL} = 0.4 \) kN/m\(^2\)

**DESIGN**
- Wall height \( 1.7 \) m
- Wall thickness \( 150 \) mm
- Effective thickness \( 210 \) mm
- Section modulus \( 0.0105 \) m\(^3\)
- Characteristic flex strength \( 0.4 \) N/mm\(^2\)
- Design bending moment \( 1.3872 \) kNm
- Bending moment resistance \( 1.921 \) kNm
- Design shear \( 1.632 \) kN
- Ultimate shear resistance \( 52.383 \) kN
Location: Wall on grid line A/1-2

Clear height of wall \( h = 0.8 \) m
Depth of soil \( d = 0 \) m
Thickness of wall \( t = 400 \) mm
Overall thickness of wall \( t_p = 450 \) mm
Offset of pier \( T_1 = 0 \) mm
Width of pier \( b = 200 \) mm
Spacing of piers (width of wall) \( B = 2 \) m
Partial safety factor materials \( \gamma_{mat} = 3.1 \)
Mortar designation to Table 1 \( \text{mortar} = 1 \)
Masonry density
\[ \text{den}(1) = 15 \text{ kN/m}^3 \]
\[ \text{typ}(1) = 4 \]
Compressive strength \( p_0 = 10.4 \text{ N/mm}^2 \)
Soil density \( s_{dens} = 18 \text{ kN/m}^3 \)
Soil angle of internal friction \( \theta' = 30^\circ \)
Surcharge pressure \( s_{urch} = 0 \text{ kN/m}^2 \)
Wind pressure \( W_L = 0.4 \text{ kN/m}^2 \)

DESIGN
Wall height \( 0.8 \) m
SUMMARY
Wall thickness \( 400 \) mm
Effective thickness \( 410 \) mm
Section modulus \( 0.057978 \text{ m}^3 \)
Characteristic flex strength \( 0.25 \text{ N/mm}^2 \)
Design bending moment \( 0.39936 \text{ kNm} \)
Bending moment resistance \( 5.3018 \text{ kNm} \)
Design shear \( 0.9984 \text{ kN} \)
Ultimate shear resistance \( 149.72 \text{ kN} \)
Location: Wall on grid line A/1-2

Clear height of wall \( h = 2.4 \) m
Depth of soil \( d = 0.3 \) m
Thickness of wall \( t = 150 \) mm
Overall thickness of wall \( tp = 450 \) mm
Offset of pier \( T1 = 0 \) mm
Width of pier \( b = 200 \) mm
Spacing of piers (width of wall) \( B = 2 \) m
Partial safety factor materials \( \gamma_{m} = 3.1 \)
Mortar designation to Table 1 \( \text{mortar}=2 \)
Masonry density \( \text{den}(1)=15 \text{ kN/m}^3 \)
\( \text{typ}(1)=4 \)
Compressive strength \( \text{po}=60 \text{ N/mm}^2 \)
Soil density \( \text{sdens}=18 \text{ kN/m}^3 \)
Soil angle of internal friction \( \theta' = 30^\circ \)
Surcharge pressure \( \text{surch}=0 \text{ kN/m}^2 \)
Wind pressure \( \text{WL}=0.05 \text{ kN/m}^2 \)

**DESIGN**
- Wall height \( 2.4 \) m
- Wall thickness \( 150 \) mm
- Effective thickness \( 210 \) mm
- Section modulus \( 0.0098654 \text{ m}^3 \)
- Characteristic flex strength \( 0.25 \text{ N/mm}^2 \)
- Design bending moment \( 0.4374 \text{ kNm} \)
- Bending moment resistance \( 1.1152 \text{ kNm} \)
- Design shear \( 1.224 \text{ kN} \)
- Ultimate shear resistance \( 39.9 \text{ kN} \)
Location: Wall on grid line A/1-2

Clear height of wall \( h = 1.8 \) m
Depth of soil \( d = 0 \) m
Thickness of wall \( t = 450 \) mm
Overall thickness of wall \( tp = 450 \) mm
Partial safety factor materials \( \gamma_m = 3.1 \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Masonry density \( \text{den}(1) = 15 \) kN/m\(^3\) \( \text{typ}(1) = 4 \)
Compressive strength \( p_o = 10.4 \) N/mm\(^2\)
Soil density \( \text{sdens} = 18 \) kN/m\(^3\)
Soil angle of internal friction \( \theta' = 30^\circ \)
Surcharge pressure \( \text{surch} = 0 \) kN/m\(^2\)
Wind pressure \( \text{WL} = 0.4 \) kN/m\(^2\)

**DESIGN**
- Wall height \( 1.8 \) m

**SUMMARY**
- Wall thickness \( 450 \) mm
- Section modulus \( 0.03375 \) m\(^3\)
- Characteristic flex strength \( 0.25 \) N/mm\(^2\)
- Design bending moment \( 0.7776 \) kNm
- Bending moment resistance \( 3.5419 \) kNm
- Design shear \( 0.864 \) kN
- Ultimate shear resistance \( 29.624 \) kN
**Location:** Wall on grid line A/1-2

Clear height of wall $h = 1.1$ m
Depth of soil $d = 0$ m
Thickness of wall $t = 100$ mm
Overall thickness of wall $tp = 450$ mm
Offset of pier $T1 = 0$ mm
Width of pier $b = 200$ mm
Spacing of piers (width of wall) $B = 2$ m
Partial safety factor materials $\gamma_m = 3.1$
Mortar designation to Table 1 $\text{mortar} = 4$
Masonry density $\text{den}(1) = 15$ kN/m$^3$
$\text{typ}(1) = 4$
Compressive strength $p_0 = 10.4$ N/mm$^2$
Soil density $s_{dens} = 18$ kN/m$^3$
Soil angle of internal friction $\theta' = 30^\circ$
Surcharge pressure $\text{surch} = 0$ kN/m$^2$
Wind pressure $WL = 0.4$ kN/m$^2$

**DESIGN**
- Wall height $1.1$ m

**SUMMARY**
- Wall thickness $100$ mm
- Effective thickness $140$ mm
- Section modulus $0.0098269$ m$^3$
- Characteristic flex strength $0.2$ N/mm$^2$
- Design bending moment $0.5808$ kNm
- Bending moment resistance $0.77992$ kNm
- Design shear $1.056$ kN
- Ultimate shear resistance $13.348$ kN
Location: Wall on grid line A/1-2

Clear height of wall \( h = 1.7 \text{ m} \)
Depth of soil \( d = 0 \text{ m} \)
Thickness of wall \( t = 150 \text{ mm} \)
Overall thickness of wall \( tp = 450 \text{ mm} \)
Offset of pier \( T_1 = 0 \text{ mm} \)
Width of pier \( b = 200 \text{ mm} \)
Spacing of piers (width of wall) \( B = 2 \text{ m} \)
Partial safety factor (material) \( \gamma_M = 2 \)
Mortar designation to Table NA.2 \( \text{mortar} = 2 \)
Masonry density \( \text{den}(1) = 15 \text{ kN/m}^3 \)
\( \text{typ}(1) = 1 \)
Water absorption %age of leaf \( \text{water} = 7 \)
Compressive strength of units \( \text{po} = 10.4 \text{ N/mm}^2 \)
Soil density \( \text{sdens} = 18 \text{ kN/m}^3 \)
Soil angle of internal friction \( \theta' = 30^\circ \)
Surcharge pressure \( \text{surch} = 0 \text{ kN/m}^2 \)
Wind pressure \( Wk = 0.4 \text{ kN/m}^2 \)

**DESIGN**
- Wall height \( 1.7 \text{ m} \)
**SUMMARY**
- Wall thickness \( 150 \text{ mm} \)
- Effective thickness \( 210 \text{ mm} \)
- Section modulus \( 0.0091431 \text{ m}^3 \)
- Characteristic flex strength \( 0.4 \text{ N/mm}^2 \)
- Design bending moment \( 1.734 \text{ kNm} \)
- Bending moment resistance \( 2.0384 \text{ kNm} \)
- Design shear \( 2.04 \text{ kN} \)
- Design shear resistance \( 16.065 \text{ kN} \)
Location: Wall on grid line A/1-2

Clear height of wall \( h = 0.8 \) m  
Depth of soil \( d = 0 \) m  
Thickness of wall \( t = 400 \) mm  
Overall thickness of wall \( tp = 450 \) mm  
Offset of pier \( T1 = 0 \) mm  
Width of pier \( b = 200 \) mm  
Spacing of piers (width of wall) \( B = 2 \) m  
Partial safety factor (material) \( \gamma_{M} = 2.4 \)  
Mortar designation to Table NA.2 \( \text{mortar} = 1 \)  
Masonry density \( \text{den}(1) = 15 \) kN/m\(^2\)  
\( \text{typ}(1) = 4 \)  
Compressive strength of units \( p_{0} = 10.4 \) N/mm\(^2\)  
Soil density \( s_{\text{dens}} = 18 \) kN/m\(^3\)  
Soil angle of internal friction \( \theta' = 30^\circ \)  
Surcharge pressure \( \text{surch} = 0 \) kN/m\(^2\)  
Wind pressure \( W_{k} = 0.4 \) kN/m\(^2\)  

**DESIGN**  
Wall height \( 0.8 \) m  
Wall thickness \( 400 \) mm  
Effective thickness \( 410 \) mm  
Section modulus \( 0.010177 \) m\(^3\)  
Characteristic flex strength \( 0.25 \) N/mm\(^2\)  
Design bending moment \( 0.06912 \) kNm  
Bending moment resistance \( 1.17 \) kNm  
Design shear \( 0.1728 \) kN  
Design shear resistance \( 12.586 \) kN
Location: Wall on grid line A/1-2

Elevation

Pressure diagram

SECTION A-A

Clear height of wall \( h = 2.4 \text{ m} \)
Depth of soil \( d = 0.3 \text{ m} \)
Thickness of wall \( t = 150 \text{ mm} \)
Overall thickness of wall \( tp = 450 \text{ mm} \)
Offset of pier \( T1 = 0 \text{ mm} \)
Width of pier \( b = 200 \text{ mm} \)
Spacing of piers (width of wall) \( B = 2 \text{ m} \)
Partial safety factor (material) \( \gamma_M = 2.4 \)
Mortar designation to Table NA.2 \( \text{mortar}=2 \)
Masonry density \( \text{den}(1)=15 \text{ kN/m}^3 \)
\( \text{typ}(1)=4 \)
Compressive strength of units \( p_0=60 \text{ N/mm}^2 \)
Soil density \( \text{sdens}=18 \text{ kN/m}^3 \)
Soil angle of internal friction \( \theta'=30^\circ \)
Surcharge pressure \( \text{surch}=0 \text{ kN/m}^2 \)
Wind pressure \( W_k=0.05 \text{ kN/m}^2 \)

DESIGN

Wall height \( 2.4 \text{ m} \)
Wall thickness \( 150 \text{ mm} \)
Effective thickness \( 210 \text{ mm} \)
Section modulus \( 0.0095645 \text{ m}^3 \)
Characteristic flex strength \( 0.25 \text{ N/mm}^2 \)
Design bending moment \( 0.54675 \text{ kNm} \)
Bending moment resistance \( 1.3062 \text{ kNm} \)
Design shear \( 1.53 \text{ kN} \)
Design shear resistance \( 15.253 \text{ kN} \)
Location: Wall on grid line A/1-2

Clear height of wall  $h=1.8$ m
Depth of soil  $d=0$ m
Thickness of wall  $t=450$ mm
Overall thickness of wall  $tp=450$ mm
Partial safety factor (material)  $\gamma_M=2.4$
Mortar designation to Table NA.2  $\text{mortar}=3$
Masonry density  
\[
\text{den}(1)=15 \text{ kN/m}^3
\]
$\text{typ}(1)=4$
Compressive strength of units  $p_0=10.4$ N/mm$^2$
Soil density  $\text{sdens}=18$ kN/m$^3$
Soil angle of internal friction  $\theta'=30^\circ$
Surcharge pressure  $\text{surch}=0$ kN/m$^2$
Wind pressure  $Wk=0.4$ kN/m$^2$

**DESIGN**
Wall height  1.8 m

**SUMMARY**
Wall thickness  450 mm
Section modulus  0.03375 m$^3$
Characteristic flex strength  0.25 N/mm$^2$
Design bending moment  0.972 kNm
Bending moment resistance  4.3357 kNm
Design shear  1.08 kN
Design shear resistance  28.75 kN
Location: Wall on grid line A/1-2

Clear height of wall $h=1.1$ m
Depth of soil $d=0$ m
Thickness of wall $t=100$ mm
Overall thickness of wall $tp=450$ mm
Offset of pier $T1=0$ mm
Width of pier $b=200$ mm
Spacing of piers (width of wall) $B=2$ m
Partial safety factor (material) $\gamma_M=2.4$
Mortar designation to Table NA.2 $\text{mortar}=4$
Masonry density $\text{den}(1)=15$ kN/m$^3$
$\text{typ}(1)=4$
Compressive strength of units $p_0=10.4$ N/mm$^2$
Soil density $s\text{dens}=18$ kN/m$^3$
Soil angle of internal friction $\theta'=30^\circ$
Surcharge pressure $s\text{urch}=0$ kN/m$^2$
Wind pressure $Wk=0.4$ kN/m$^2$

DESIGN
Wall height 1.1 m
Wall thickness 100 mm
Effective thickness 140 mm
Section modulus 0.0087107 m$^3$
Characteristic flex strength 0.2 N/mm$^2$
Design bending moment 0.726 kNm
Bending moment resistance 0.85525 kNm
Design shear 1.32 kN
Design shear resistance 5.6784 kN
Location: Ex1 - Beam on grid line A/1-2

![Diagram of supported rectangular brick beam]

Sample output for SCALE Proforma 521. (ans=1)  
Masonry design to BS5628, MEKE and Eurocode 6  
Brick beam  
Made by: IFB  
Date: 02/12/19  
Ref No: SC521 BS

Ultimate bending moment \( \mu = 40 \text{ kNm} \)
Ultimate shear force \( \upsilon = 25 \text{ kN} \)
Effective depth of beam \( d = 600 \text{ mm} \)
Width of beam \( b = 250 \text{ mm} \)
Span of brick beam \( L = 4000 \text{ mm} \)
Spacing of lateral restraints \( L_r = 4000 \text{ mm} \)
Compressive strength of units \( S_{\text{units}} = 20 \text{ N/mm}^2 \)
Characteristic comp strength \( f_k = 5.6 \text{ N/mm}^2 \)
Partial safety factor \( \gamma_m = 2.3 \)
Characteristic tensile strength of reinforcement \( f_y = 500 \text{ N/mm}^2 \)
Diameter of reinforcing bars \( \text{diam} = 10 \text{ mm} \)
Charac anchorage bond strength \( f_b = 2 \text{ N/mm}^2 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Ultimate bending moment</th>
<th>40 kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Ultimate shear force</td>
<td>25 kN</td>
</tr>
<tr>
<td></td>
<td>Beam effective depth</td>
<td>600 mm</td>
</tr>
<tr>
<td></td>
<td>Beam width</td>
<td>250 mm</td>
</tr>
<tr>
<td></td>
<td>Beam span</td>
<td>4000 mm</td>
</tr>
<tr>
<td></td>
<td>Tension reinforcement area</td>
<td>235.62 mm²</td>
</tr>
<tr>
<td></td>
<td>Tensile strength of reinft.</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Resistance moment of section</td>
<td>87.652 kNm</td>
</tr>
<tr>
<td></td>
<td>Mortar designation</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Compressive strength of units</td>
<td>20 N/mm²</td>
</tr>
</tbody>
</table>

Links are not required but according to Clause 8.2.5 nominal links need to be considered by the designer because of the sudden nature of shear failure.

Partial safety factors:
- Masonry \( \gamma_m = 2.3 \)
- Reinforcement strength \( \gamma_f = 1.15 \)
- Shear strength \( \gamma_s = 2 \)
- Bond strength \( \gamma_b = 1.5 \)
Location: Ex2 - Beam on grid line A/1-2

Singly reinforced simply

supported rectangular brick

beam to BS5628-2:2005

Ultimate bending moment \( M_u = 40 \text{ kNm} \)
Ultimate shear force \( V_u = 35 \text{ kN} \)
Effective depth of beam \( d = 600 \text{ mm} \)
Width of beam \( b = 250 \text{ mm} \)
Span of brick beam \( L = 4000 \text{ mm} \)
Spacing of lateral restraints \( L_r = 4000 \text{ mm} \)
Compressive strength of units \( S_{units} = 20 \text{ N/mm}^2 \)
Characteristic comp strength \( f_k = 5.6 \text{ N/mm}^2 \)
Partial safety factor \( \gamma_m = 2.3 \)
Characteristics tensile strength of reinforcement \( f_y = 500 \text{ N/mm}^2 \)
Diameter of reinforcing bars \( \text{diam} = 10 \text{ mm} \)
Spacing (<0.75d & <500mm) \( s_v = 200 \text{ mm} \)
Tensile strength of links \( f_y' = 250 \text{ N/mm}^2 \)
Diameter of bars for shear \( D_s = 8 \text{ mm} \)
Charac anchorage bond strength \( f_b = 2 \text{ N/mm}^2 \)

DESIGN

Ultimate bending moment \( 40 \text{ kNm} \)
Ultimate shear force \( 35 \text{ kN} \)
Beam effective depth \( 600 \text{ mm} \)
Beam width \( 250 \text{ mm} \)
Beam span \( 4000 \text{ mm} \)
Tension reinforcement area \( 235.62 \text{ mm}^2 \)
Tensile strength of reinft. \( 500 \text{ N/mm}^2 \)
Resistance moment of section \( 87.652 \text{ kNm} \)
Mortar designation \( 2 \)
Compressive strength of units \( 20 \text{ N/mm}^2 \)
Diameter of bars for shear \( 8 \text{ mm}^2 \)
Spacing of shear links \( 200 \text{ mm} \)
Tensile strength of links \( 250 \text{ N/mm}^2 \)

Partial safety factors:
Masonry \( 2.3 \)
Reinforcement strength \( 1.15 \)
Shear strength \( 2 \)
Bond strength \( 1.5 \)
Location: Ex3 - R2 Solution. Reinforced Brickwork Beam by J Roberts

Singly reinforced simply

supported rectangular brick

beam to BS5628-2:2005

Ultimate bending moment $M_u = 86.4$ kNm
Ultimate shear force $V_u = 82.1$ kN
Effective depth of beam $d = 410$ mm
Width of beam $b = 440$ mm
Span of brick beam $L = 4210$ mm
Spacing of lateral restraints $L_r = 4000$ mm
Mortar designation (Table 1) mortar=2
Compressive strength of units $S_{units} = 40$ N/mm$^2$
Characteristic comp strength $f_k = 10.1$ N/mm$^2$
Partial safety factor $\gamma_m = 2$
Characteristic tensile strength of reinforcement $f_y = 500$ N/mm$^2$
Diameter of reinforcing bars $diam = 20$ mm
Spacing (<0.75d & <500mm) $s_v = 300$ mm
Tensile strength of links $f_y' = 200$ N/mm$^2$
Diameter of bars for shear $D_s = 8$ mm
Charac anchorage bond strength $f_b = 2.5$ N/mm$^2$

DESIGN

Ultimate bending moment $86.4$ kNm
Ultimate shear force $82.1$ kN
Beam effective depth $410$ mm
Beam width $440$ mm
Beam span $4210$ mm
Tension reinforcement area $628.32$ mm$^2$
Tensile strength of reinf. $500$ N/mm$^2$
Resistance moment of section $149.41$ kNm
Mortar designation $2$
Compressive strength of units $40$ N/mm$^2$

SUMMARY

Diameter of bars for shear $8$ mm$^2$
Spacing of shear links $300$ mm
Tensile strength of links $200$ N/mm$^2$

Partial safety factors:
Masonry $2$
Reinforcement strength $1.15$
Shear strength $2$
Bond strength $1.5$
**Location:** Ex1 - Beam on grid line A/1-2

**Singly reinforced simply supported rectangular brick beam to EC6 Part 1-1**

Design bending moment \( Md' = 40 \text{kNm} \)
Design shear force \( V_d = 25 \text{kN} \)
Depth of beam \( h = 670 \text{mm} \)
Effective depth of beam \( d = 600 \text{mm} \)
Width of beam \( b = 250 \text{mm} \)
Bearing length (left support) \( t_1 = 328 \text{mm} \)
Bearing length (right support) \( t_2 = 328 \text{mm} \)
Clear distance between supports \( L_{cl} = 3672 \text{mm} \)
Spacing of lateral restraints \( L_r = 4000 \text{mm} \)
Mortar designation (Table NA.2) \( \text{mortar}=2 \)
Compressive strength of units \( p_0 = 20 \text{N/mm}^2 \)
Constant \( K \) from NA Table NA.4 \( K(1) = 0.5 \)
Height of masonry unit \( h_u = 65 \text{mm} \)
Least horiz. dimension of units \( l_{hd} = 100 \text{mm} \)
Partial safety factor \( \gamma_M = 2.3 \)
Characteristic tensile strength \( f_y = 500 \text{N/mm}^2 \)
Diameter of reinforcing bars \( \text{diam}=10 \text{mm} \)
Char anchorage bond strength \( f_{bok}=3.4 \text{N/mm}^2 \)

**DESIGN**

Design bending moment \( 40 \text{kNm} \)
Design shear force \( 25 \text{kN} \)
Beam effective depth \( 600 \text{mm} \)
Beam width \( 250 \text{mm} \)
Beam span \( 4000 \text{mm} \)
Tension reinforcement area \( 235.62 \text{mm}^2 \)
Tensile strength of reinft \( 500 \text{N/mm}^2 \)
Resistance moment of section \( 65.706 \text{kNm} \)
Mortar designation \( 2 \)
Compressive strength of units \( 20 \text{N/mm}^2 \)
Wall to be constructed using Group 1 units

Links are not required, however nominal links need to be considered by the designer because of the sudden nature of shear failure.

Partial safety factors:

- Masonry \( 2.3 \)
- Reinforcement strength \( 1.15 \)
- Shear strength of masonry \( 2 \)
- Bond strength \( 1.5 \)
**Location:** Ex2 - Beam on grid line A/1-2

---

**Singly reinforced simply supported rectangular brick beam to EC6 Part 1-1**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design bending moment</td>
<td>$M_d' = 40$ kNm</td>
</tr>
<tr>
<td>Design shear force</td>
<td>$V_d = 25$ kN</td>
</tr>
<tr>
<td>Depth of beam</td>
<td>$h = 670$ mm</td>
</tr>
<tr>
<td>Effective depth of beam</td>
<td>$d = 600$ mm</td>
</tr>
<tr>
<td>Width of beam</td>
<td>$b = 250$ mm</td>
</tr>
<tr>
<td>Bearing length (left support)</td>
<td>$t_1 = 328$ mm</td>
</tr>
<tr>
<td>Bearing length (right support)</td>
<td>$t_2 = 328$ mm</td>
</tr>
<tr>
<td>Clear distance between supports</td>
<td>$L_{cl} = 3672$ mm</td>
</tr>
<tr>
<td>Spacing of lateral restraints</td>
<td>$L_r = 4000$ mm</td>
</tr>
<tr>
<td>Mortar designation (Table NA.2)</td>
<td>Mortar = 2</td>
</tr>
<tr>
<td>Compressive strength of units</td>
<td>$p_o = 20$ N/mm²</td>
</tr>
<tr>
<td>Constant K from NA Table NA.4</td>
<td>$K(1) = 0.5$</td>
</tr>
<tr>
<td>Height of masonry unit</td>
<td>$h_u = 65$ mm</td>
</tr>
<tr>
<td>Least horiz. dimension of units</td>
<td>$l_{hd} = 100$ mm</td>
</tr>
<tr>
<td>Partial safety factor</td>
<td>$\gamma_M = 2.3$</td>
</tr>
<tr>
<td>Characteristic tensile strength</td>
<td>$f_y = 500$ N/mm²</td>
</tr>
<tr>
<td>Diameter of reinforcing bars</td>
<td>$d_{iam} = 10$ mm</td>
</tr>
<tr>
<td>Char anchorage bond strength</td>
<td>$f_{bo,k} = 3.4$ N/mm²</td>
</tr>
</tbody>
</table>

**DESIGN**

- Design bending moment: 40 kNm
- Design shear force: 25 kN
- Beam effective depth: 600 mm
- Beam width: 250 mm
- Beam span: 4000 mm
- Tension reinforcement area: 235.62 mm²
- Tensile strength of reinft: 500 N/mm²
- Resistance moment of section: 65.706 kNm
- Mortar designation: 2

**SUMMARY**

- Compressive strength of units: 20 N/mm²
- Wall to be constructed using Group 1 units

Links are not required, however nominal links need to be considered by the designer because of the sudden nature of shear failure.

**Partial safety factors:**

- Masonry: 2.3
- Reinforcement strength: 1.15
- Shear strength of masonry: 2
- Bond strength: 1.5
Location: Ex3 - R2 Solution. Reinforced Brickwork Beam by J Roberts

Singly reinforced simply
supported rectangular brick
beam to EC6 Part 1-1

Design bending moment \( M_d' = 86.4 \text{ kNm} \)
Design shear force \( V_d = 82.1 \text{ kN} \)
Depth of beam \( h = 480 \text{ mm} \)
Effective depth of beam \( d = 410 \text{ mm} \)
Width of beam \( b = 440 \text{ mm} \)
Bearing length (left support) \( t_1 = 328 \text{ mm} \)
Bearing length (right support) \( t_2 = 328 \text{ mm} \)
Clear distance between supports \( L_c = 3882 \text{ mm} \)
Spacing of lateral restraints \( L_r = 4000 \text{ mm} \)
Mortar designation (Table NA.2) \( \text{mortar} = 2 \)
Compressive strength of units \( p_o = 40 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1) = 0.5 \)
Height of masonry unit \( h_u = 65 \text{ mm} \)
Least horiz. dimension of units \( l_{hd} = 100 \text{ mm} \)
Partial safety factor \( \gamma_M = 2 \)
Characteristic tensile strength \( f_y = 500 \text{ N/mm}^2 \)
Spacing (<0.75d & <300mm) \( s_v = 300 \text{ mm} \)
Char tensile strength of links \( f_{y'} = 200 \text{ N/mm}^2 \)
Diameter of shear link \( D_s = 8 \text{ mm} \)
Number of link legs \( n_{leg} = 2 \)
Char anchorage bond strength \( f_{bok} = 3.4 \text{ N/mm}^2 \)

Concrete anchorage to reinforcement \( \text{cover} = 40 \text{ mm} \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design bending moment</td>
<td>86.4 kNm</td>
</tr>
<tr>
<td>Design shear force</td>
<td>82.1 kN</td>
</tr>
<tr>
<td>Beam effective depth</td>
<td>410 mm</td>
</tr>
<tr>
<td>Beam width</td>
<td>440 mm</td>
</tr>
<tr>
<td>Beam span</td>
<td>4210 mm</td>
</tr>
<tr>
<td>Tension reinforcement area</td>
<td>628.32 mm²</td>
</tr>
<tr>
<td>Tensile strength of reinft</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Resistance moment of section</td>
<td>100.88 kNm</td>
</tr>
<tr>
<td>Mortar designation</td>
<td>2</td>
</tr>
<tr>
<td>Compressive strength of units</td>
<td>40 N/mm²</td>
</tr>
</tbody>
</table>

Wall to be constructed using Group 1 units

Diameter of bars for shear | 8 mm² |
Spacing of shear links | 300 mm |
Tensile strength of links | 200 N/mm² |

Partial safety factors:
Masonry | 2
Reinforcement strength | 1.15
<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Shear strength of masonry</strong></td>
<td>2</td>
</tr>
<tr>
<td><strong>Bond strength</strong></td>
<td>1.5</td>
</tr>
</tbody>
</table>
Location: Ex4 - Beam on grid line A/1-2

Singly reinforced simply
supported rectangular brick
beam to EC6 Part 1-1

Design bending moment \( Md' = 40 \text{ kNm} \)
Design shear force \( V_d = 25 \text{ kN} \)
Depth of beam \( h = 670 \text{ mm} \)
Effective depth of beam \( d = 600 \text{ mm} \)
Width of beam \( b = 250 \text{ mm} \)
Bearing length (left support) \( t_1 = 328 \text{ mm} \)
Bearing length (right support) \( t_2 = 328 \text{ mm} \)
Clear distance between supports \( L_{cl} = 3672 \text{ mm} \)
Spacing of lateral restraints \( L_r = 4000 \text{ mm} \)
Mortar designation (Table NA.2) \( \text{mortar} = 2 \)
Compressive strength of units \( p_0 = 20 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1) = 0.5 \)
Normalized mean comp strength \( f_{bn} = 8.5 \text{ N/mm}^2 \)
Partial safety factor \( \gamma_M = 2.3 \)
Characteristic tensile strength \( f_y = 500 \text{ N/mm}^2 \)
Diameter of reinforcing bars \( \text{diam} = 10 \text{ mm} \)
Char anchorage bond strength \( f_{bok} = 3.4 \text{ N/mm}^2 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design bending moment</td>
<td>40 kNm</td>
</tr>
<tr>
<td>Design shear force</td>
<td>25 kN</td>
</tr>
<tr>
<td>Beam effective depth</td>
<td>600 mm</td>
</tr>
<tr>
<td>Beam width</td>
<td>250 mm</td>
</tr>
<tr>
<td>Beam span</td>
<td>4000 mm</td>
</tr>
<tr>
<td>Tension reinforcement area</td>
<td>235.62 mm²</td>
</tr>
<tr>
<td>Tensile strength of reinft</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Resistance moment of section</td>
<td>40.447 kNm</td>
</tr>
<tr>
<td>Mortar designation</td>
<td>2</td>
</tr>
<tr>
<td>Compressive strength of units</td>
<td>20 N/mm²</td>
</tr>
<tr>
<td>Wall to be constructed using Group 1 units</td>
<td></td>
</tr>
</tbody>
</table>

Links are not required, however nominal links need to be considered by the designer because of the sudden nature of shear failure.

Partial safety factors:
Masonry \( 2.3 \)
Reinforcement strength \( 1.15 \)
Shear strength of masonry | 2 |
Bond strength | 1.5 |
Location: Bearing under beam B1

Plan view on wall

Bearing under beam B1

Proposed width of bearing  \( B = 150 \text{ mm} \)
Proposed length of bearing  \( L = 100 \text{ mm} \)
Mortar designation to Table 1  mortar=3
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Height  \( h_b = 215 \text{ mm} \)
Least horiz dimension  \( l_h = 100 \text{ mm} \)
Compressive strength of units  \( p_o = 10.4 \text{ N/mm}^2 \)
Material partial safety factor  \( \gamma_m = 2.5 \)
Characteristic dead load  \( D_L = 20 \text{ kN} \)
Characteristic live load  \( L_L = 10 \text{ kN} \)

DESIGN

Bearing width  \( 150 \text{ mm} \)

SUMMARY

Length of bearing  \( 100 \text{ mm} \)
Clear edge distance  \( 0 \text{ mm} \)
Char comp strength  \( 8.2 \text{ N/mm}^2 \)
Local bearing safety factor 1.25
Design bearing resistance  \( 61.5 \text{ kN} \)
Design load on bearing  \( 44 \text{ kN} \)
Location: Bearing under beam B2

Plan view on wall

Bearing type 1
see Figure 5(a) in BS5628

ed=0

Proposed width of bearing B=100 mm
Proposed length of bearing L=250 mm
Mortar designation to Table 1 mortar=3
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=2
Compressive strength of units po=10 N/mm²
Material partial safety factor gammam=2.5
Characteristic dead load DL=20 kN
Characteristic live load LL=10 kN
Length of bearing to be used L=250 mm

WARNING:
Length of bearing should be within range 258.82 mm to 300 mm
Location: Bearing under beam B3

Plan view on wall

Bearing type 2
see Figure 5(b) in BS5628

Proposed width of bearing $B=100 \text{ mm}$
Proposed length of bearing $L=200 \text{ mm}$
Mortar designation to Table 1 $\text{mortar}=3$
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) $\text{typ}(1)=3$
Compressive strength of units $\text{po}=10.4 \text{ N/mm}^2$
Material partial safety factor $\gamma_m=2.5$
Characteristic dead load $\text{DL}=20 \text{ kN}$
Characteristic live load $\text{LL}=10 \text{ kN}$

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>Bearing width</th>
<th>100 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Length of bearing</td>
<td>200 mm</td>
</tr>
<tr>
<td></td>
<td>Clear edge distance</td>
<td>0 mm</td>
</tr>
<tr>
<td></td>
<td>Char comp strength</td>
<td>4.1 N/mm$^2$</td>
</tr>
<tr>
<td></td>
<td>Local bearing safety factor</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Design bearing resistance</td>
<td>49.2 kN</td>
</tr>
<tr>
<td></td>
<td>Design load on bearing</td>
<td>44 kN</td>
</tr>
</tbody>
</table>
Location: Bearing under beam B4

Plan view on wall

Proposed width of bearing $B = 150$ mm
Proposed length of bearing $L = 100$ mm
Clear edge distance to beam $ed = 100$ mm
Mortar designation to Table 1 $mortar = 3$
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) $typ(1)=4$
Height $hb = 215$ mm
Least horiz dimension $lhd = 100$ mm
Compressive strength of units $po = 10.4$ N/mm$^2$
Material partial safety factor $gammam = 2.5$
Characteristic dead load $DL = 20$ kN
Characteristic live load $LL = 10$ kN

**DESIGN**
- Bearing width $150$ mm

**SUMMARY**
- Length of bearing $100$ mm
- Clear edge distance $100$ mm
- Char comp strength $8.2$ N/mm$^2$
- Local bearing safety factor $1.5$
- Design bearing resistance $73.8$ kN
- Design load on bearing $44$ kN
Location: Bearing under beam B5

Plan view on wall

Bearing type 3
see Figure 5(c) in BS5628.

$t=wall$ $thicknes$ $L=length$ $of$ $bearing$ $ed=0$ $L=t$

Proposed width of bearing $B=133$ $mm$
Proposed length of bearing $L=225$ $mm$
Mortar designation to Table 1 $mortar=3$
Masonry type ($1=clay$ $2=cal$ $silicate$ $3=conc$ $brick$ $4=block$) typ(1)=$1
Compressive strength of units $po=20$ $N/mm^2$
Material partial safety factor $gammam=3.1$
Characteristic dead load $DL=35.3$ $kN$
Characteristic live load $LL=11.6$ $kN$
Length of spreader beam $sl=450$ $mm$
Offset load distance $x1=200$ $mm$

**DESIGN**
- Length of spreader beam $450$ $mm$

**SUMMARY**
- Width of spreader beam $225$ $mm$
- Eccentricity of load $25$ $mm$
- Char comp strength $3.2258$ $N/mm^2$
- Local bearing safety factor $2$
- Design stress capacity $3.2258$ $N/mm^2$
- Max stress under spreader $0.89521$ $N/mm^2$
Location: Bearing under beam B6

Plan view on wall

B

Plan view on wall

Beam

Bearing type 1
see Figure 5(a) in BS5628

Proposed width of bearing: B = 150 mm
Proposed length of bearing: L = 50 mm
Clear edge distance to beam: ed = 50 mm
Mortar designation to Table 1: mortar = 3
Masonry type (1 = clay, 2 = cal silicate, 3 = conc brick, 4 = block): typ(1) = 4
Height: hb = 215 mm
Least horiz dimension: lhd = 100 mm
Compressive strength of units: po = 10.4 N/mm²
Material partial safety factor: γm = 2.5
Characteristic dead load: DL = 20 kN
Characteristic live load: LL = 10 kN

Length of spreader beam: sl = 450 mm
Offset load distance: xl = 225 mm

DESIGN
Length of spreader beam: 450 mm
Summary
Width of spreader beam: 50 mm
Eccentricity of load: 0 mm
Char comp strength: 4.1 N/mm²
Local bearing safety factor: 1.25
Design stress capacity: 4.1 N/mm²
Max stress under spreader: 1.9556 N/mm²
**Location:** Bearing under beam B7

- Proposed width of bearing: \( B = 200 \text{ mm} \)
- Proposed length of bearing: \( L = 75 \text{ mm} \)
- Clear edge distance to beam: \( ed = 100 \text{ mm} \)

- Mortar designation to Table 1: mortar = 3
- Masonry type (1=clay 2=cal silicate 3=concrete brick 4=block): typ(1) = 4
- Height: \( h_b = 215 \text{ mm} \)
- Least horiz dimension: \( l_{hd} = 100 \text{ mm} \)
- Compressive strength of units: \( p_o = 10.4 \text{ N/mm}^2 \)
- Material partial safety factor: \( \gamma_{m} = 2.5 \)
- Characteristic dead load: \( D_L = 30 \text{ kN} \)
- Characteristic live load: \( L_L = 20 \text{ kN} \)
- Length of bearing to be used: \( L = 75 \text{ mm} \)

**WARNING:**

Length of bearing should be within range 90.244 mm to 100 mm
Location: Bearing under beam B8

Plan view on wall

Bearing type 2
see Figure 5(b) in BS5628

Proposed width of bearing \( B = 150 \text{ mm} \)
Proposed length of bearing \( L = 50 \text{ mm} \)
Clear edge distance to beam \( ed = 50 \text{ mm} \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ(1)} = 4 \)
Height \( h_b = 215 \text{ mm} \)
Least horiz dimension \( l_{hd} = 100 \text{ mm} \)
Compressive strength of units \( p_o = 10.4 \text{ N/mm}^2 \)
Material partial safety factor \( \gamma_m = 2.5 \)
Characteristic dead load \( D_L = 20 \text{ kN} \)
Characteristic live load \( L_L = 10 \text{ kN} \)

Length of spreader beam \( s_l = 450 \text{ mm} \)
Offset load distance \( x_1 = 75 \text{ mm} \)

WARNING:
Max stress under spreader > Max permissible
i.e. \( f_{km} \ (7.8222 \text{ N/mm}^2) > f_{k1} \ (6.56 \text{ N/mm}^2) \)
Location: Bearing under beam B1

Plan on loaded area

Wall thickness \( t = 100 \text{ mm} \)
Load bearing length // to wall \( a_2 = 150 \text{ mm} \)
Load bearing width perp. to wall \( a_3 = 100 \text{ mm} \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 4 \)
Compressive strength of units \( p_0 = 10.4 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1) = 0.75 \)
Height of masonry unit \( h_u = 215 \text{ mm} \)
Least horiz. dimension of units \( l_{hd} = 100 \text{ mm} \)
Partial safety factor (material) \( \gamma_{M} = 2.3 \)
Characteristic permanent load \( G_k = 20 \text{ kN} \)
Characteristic variable load \( Q_k = 10 \text{ kN} \)

DESIGN

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of bearing</td>
<td>150 mm</td>
</tr>
</tbody>
</table>

SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing width</td>
<td>100 mm</td>
</tr>
<tr>
<td>Distance from end of wall</td>
<td>0 mm</td>
</tr>
<tr>
<td>Char compressive strength</td>
<td>7.337 N/mm²</td>
</tr>
<tr>
<td>Design load resistance</td>
<td>59.812 kN</td>
</tr>
<tr>
<td>Design load on bearing</td>
<td>42 kN</td>
</tr>
</tbody>
</table>

The wall will also need to be checked for vertical loading at the top, middle and bottom. At the middle of the wall account will need to be taken of any overlap of the spread of the concentrated loads plus the applied design vertical loads. Adopt Group 1 units.
Location: Bearing under beam B2

Wall thickness \( t = 100 \text{ mm} \)

Load bearing length // to wall \( a_2 = 250 \text{ mm} \)

Load bearing width perp. to wall \( a_3 = 100 \text{ mm} \)

Mortar designation to Table 1 \( \text{mortar} = 3 \)

Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ(1)} = 2 \)

Compressive strength of units \( P_o = 10 \text{ N/mm}^2 \)

Constant K from NA Table NA.4 \( K(1) = 0.5 \)

Height of masonry unit \( h_u = 65 \text{ mm} \)

Least horiz. dimension of units \( l_{hd} = 100 \text{ mm} \)

Partial safety factor (material) \( \gamma_{M} = 2.3 \)

Characteristic permanent load \( G_k = 20 \text{ kN} \)

Characteristic variable load \( Q_k = 10 \text{ kN} \)

**DESIGN**

Length of bearing \( 250 \text{ mm} \)

**SUMMARY**

Bearing width \( 100 \text{ mm} \)

Distance from end of wall \( 0 \text{ mm} \)

Char compressive strength \( 3.3899 \text{ N/mm}^2 \)

Design load resistance \( 44.841 \text{ kN} \)

Design load on bearing \( 42 \text{ kN} \)

The wall will also need to be checked for vertical loading at the top, middle and bottom. At the middle of the wall account will need to be taken of any overlap of the spread of the concentrated loads plus the applied design vertical loads. Adopt Group 1 units.
Location: Bearing under beam B3

Plan on loaded area

Wall thickness \( t=100 \text{ mm} \)
Load bearing length \( // \) to wall \( a_2=200 \text{ mm} \)
Load bearing width perp. to wall \( a_3=100 \text{ mm} \)
Mortar designation to Table 1 \( \text{mortar}=3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1)=3 \)
Compressive strength of units \( p_0=10.4 \text{ N/mm}^2 \)
Constant \( K \) from NA Table NA.4 \( K(1)=0.75 \)
Height of masonry unit \( h_u=65 \text{ mm} \)
Least horiz. dimension of units \( l_{hd}=100 \text{ mm} \)
Partial safety factor (material) \( \gamma_{M}=2.3 \)
Characteristic permanent load \( G_k=20 \text{ kN} \)
Characteristic variable load \( Q_k=10 \text{ kN} \)

**DESIGN**

- Length of bearing \( 200 \text{ mm} \)

**SUMMARY**

- Bearing width \( 100 \text{ mm} \)
- Distance from end of wall \( 0 \text{ mm} \)
- Char compressive strength \( 5.2263 \text{ N/mm}^2 \)
- Design load resistance \( 56.808 \text{ kN} \)
- Design load on bearing \( 42 \text{ kN} \)

The wall will also need to be checked for vertical loading at the top, middle and bottom. At the middle of the wall account will need to be taken of any overlap of the spread of the concentrated loads plus the applied design vertical loads. Adopt Group 1 units.
Location: Bearing under beam B4

Wall thickness $t = 100 \text{ mm}$
Load bearing length // to wall $a_2 = 150 \text{ mm}$
Load bearing width perp. to wall $a_3 = 100 \text{ mm}$
Mortar designation to Table 1 $\text{mortar}=3$
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Compressive strength of units $p_0 = 10.4 \text{ N/mm}^2$
Constant $K$ from NA Table NA.4 $K(1)=0.75$
Height of masonry unit $h_u = 215 \text{ mm}$
Least horiz. dimension of units $l_{hd}=100 \text{ mm}$
Partial safety factor (material) $\gamma_{M}=2.3$
Characteristic permanent load $G_k=20 \text{ kN}$
Characteristic variable load $Q_k=10 \text{ kN}$

**DESIGN**
- Length of bearing 150 mm

**SUMMARY**
- Bearing width 100 mm
- Distance from end of wall 100 mm
- Char compressive strength 7.337 N/mm²
- Design load resistance 60.769 kN
- Design load on bearing 42 kN

The wall will also need to be checked for vertical loading at the top, middle and bottom. At the middle of the wall account will need to be taken of any overlap of the spread of the concentrated loads plus the applied design vertical loads. Adopt Group 1 units.
Location: Bearing under beam B5

Wall thickness \( t = 225 \text{ mm} \)
Load bearing length // to wall \( a_2 = 133 \text{ mm} \)
Load bearing width perp. to wall \( a_3 = 225 \text{ mm} \)
Mortar designation to Table 1 \( \text{mortar}=3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1)=1 \)
Compressive strength of units \( p_0 = 20 \text{ N/mm}^2 \)
Constant \( K \) from NA Table NA.4 \( K(1)=0.5 \)
Height of masonry unit \( h_u = 65 \text{ mm} \)
Least horiz. dimension of units \( l_{hd} = 100 \text{ mm} \)
Partial safety factor (material) \( \gamma_M = 2.7 \)
Characteristic permanent load \( G_k = 35.3 \text{ kN} \)
Characteristic variable load \( Q_k = 11.6 \text{ kN} \)

**DESIGN**
- Length of bearing \( 133 \text{ mm} \)
**SUMMARY**
- Bearing width \( 225 \text{ mm} \)
- Distance from end of wall \( 0 \text{ mm} \)
- Char compressive strength \( 5.5068 \text{ N/mm}^2 \)
- Design load resistance \( 76.293 \text{ kN} \)
- Design load on bearing \( 65.055 \text{ kN} \)

The wall will also need to be checked for vertical loading at the top, middle and bottom. At the middle of the wall account will need to be taken of any overlap of the spread of the concentrated loads plus the applied design vertical loads. Adopt Group 1 units.
Location: Bearing under beam B6

Wall thickness \( t = 100 \text{ mm} \)
Load bearing length // to wall \( a_2 = 150 \text{ mm} \)
Load bearing width perp. to wall \( a_3 = 50 \text{ mm} \)
Mortar designation to Table 1 \( \text{mortar}=3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1)=4 \)
Compressive strength of units \( p_0 = 10.4 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1)=0.75 \)
Height of masonry unit \( h_u = 215 \text{ mm} \)
Least horiz. dimension of units \( l_{hd}=100 \text{ mm} \)
Partial safety factor (material) \( \gamma_{M}=2.3 \)
Characteristic permanent load \( G_k=20 \text{ kN} \)
Characteristic variable load \( Q_k=10 \text{ kN} \)

\[
\begin{align*}
&\text{Nedc} \\
&\quad x_1 \\
&\quad \downarrow \\
&\quad \text{padstone} \\
&\quad \downarrow \\
&\quad \text{wall} \\
&\quad s_1
\end{align*}
\]
\[
\begin{align*}
&x_1 = \text{offset load distance} \\
&s_d = \text{padstone height} \\
&s_l = \text{padstone length}
\end{align*}
\]
Padstone length \( s_l=450 \text{ mm} \)
Padstone width \( s_b=50 \text{ mm} \)
Offset load distance \( x_1=225 \text{ mm} \)
Padstone height to be adopted \( s_d=259.81 \text{ mm} \)

**DESIGN**
- Length of concrete padstone \( 450 \text{ mm} \)

**SUMMARY**
- Width of concrete padstone \( 50 \text{ mm} \)
- Concrete padstone height \( 259.81 \text{ mm} \)
- Eccentricity of load \( 0 \text{ mm} \)
- Design strength under bearing \( 4.0194 \text{ N/mm}^2 \)
- Design stress under spreader \( 1.8667 \text{ N/mm}^2 \)

The wall will also need to be checked for vertical loading at the top, middle and bottom. At the middle of the wall account will need to be taken of any overlap of the spread of the concentrated loads plus the applied design vertical loads. Adopt Group 1 units.
Location: Bearing under beam B7

Plan on loaded area

Wall thickness \( t = 100 \text{ mm} \)
Load bearing length // to wall \( a_2 = 200 \text{ mm} \)
Load bearing width perp. to wall \( a_3 = 75 \text{ mm} \)
Mortar designation to Table 1 \( \text{mortar}=3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1)=4 \)
Compressive strength of units \( p_0=10.4 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1)=0.75 \)
Height of masonry unit \( h_u=215 \text{ mm} \)
Least horiz. dimension of units \( h_d=100 \text{ mm} \)
Partial safety factor (material) \( \gamma_M=2.3 \)
Characteristic permanent load \( G_k=30 \text{ kN} \)
Characteristic variable load \( Q_k=20 \text{ kN} \)

---

LENGTH OF CONCRETE PADSTONE

**SUMMARY**

- Length of concrete padstone: 450 mm
- Width of concrete padstone: 100 mm
- Concrete padstone height: 216.51 mm
- Eccentricity of load: 0 mm
- Design strength under bearing: 4.0513 N/mm²
- Design stress under spreader: 1.5667 N/mm²

The wall will also need to be checked for vertical loading at the top, middle and bottom. At the middle of the wall account will need to be taken of any overlap of the spread of the concentrated loads plus the applied design vertical loads. Adopt Group 1 units.
Location: Bearing under beam B8

Plan on loaded area

Wall thickness $t=100$ mm
Load bearing length // to wall $a_2=150$ mm
Load bearing width perp. to wall $a_3=50$ mm
Mortar designation to Table 1 $\text{mortar}=3$
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) $\text{typ}(1)=4$
Compressive strength of units $p_0=10.4$ N/mm$^2$
Constant K from NA Table NA.4 $K(1)=0.75$
Height of masonry unit $h_u=215$ mm
Least horiz. dimension of units $l_{hd}=100$ mm
Partial safety factor (material) $\gamma_M=2.3$
Characteristic permanent load $G_k=20$ kN
Characteristic variable load $Q_k=10$ kN

$\text{Nedc}$

$x_1 = \text{offset load distance}$
$sd = \text{padstone height}$
$sl = \text{padstone length}$

Padstone length $sl=450$ mm
Padstone width $sb=100$ mm
Offset load distance $x_1=225$ mm
Padstone height to be adopted $sd=259.81$ mm

**DESIGN**

Length of concrete padstone $450$ mm

**SUMMARY**

Width of concrete padstone $100$ mm
Concrete padstone height $259.81$ mm
Eccentricity of load $0$ mm
Design strength under bearing $4.0194$ N/mm$^2$
Design stress under spreader $0.93333$ N/mm$^2$

The wall will also need to be checked for vertical loading at the top, middle and bottom. At the middle of the wall account will need to be taken of any overlap of the spread of the concentrated loads plus the applied design vertical loads. Adopt Group 1 units.
BS EN 1996-1-1:2005, Annex C, gives a simplified method for calculating the out-of-plane eccentricity of loading on masonry walls. The structure is simplified to a sub-frame for the evaluation of moments at the top and bottom of the wall.

Partial safety factors

<table>
<thead>
<tr>
<th>Permanent actions</th>
<th>gamG=1.35</th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable actions</td>
<td>gamQ=1.5</td>
</tr>
</tbody>
</table>

Masonry and floor details

<table>
<thead>
<tr>
<th>Mortar designation (Table NA.2)</th>
<th>mortar=3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry type (1=clay 2=cal silicate 3=conc brick 4=block)</td>
<td>typ(1)=4</td>
</tr>
<tr>
<td>Compressive strength of units</td>
<td>po=7.3 N/mm²</td>
</tr>
<tr>
<td>Constant K from NA Table NA.4</td>
<td>K(1)=0.75</td>
</tr>
<tr>
<td>Height of masonry unit</td>
<td>hu=215 mm</td>
</tr>
<tr>
<td>Least horiz. dimension of units</td>
<td>lhd=140 mm</td>
</tr>
</tbody>
</table>

Moment at the top of wall - (at node point A)

\[
\begin{align*}
\text{M1} = & \text{moment at top of wall} \\
A-B = & \text{represents the wall under consideration} \\
1 \& 4 \text{ are member numbers}
\end{align*}
\]

Masonry Wall 1

<table>
<thead>
<tr>
<th>Wall thickness</th>
<th>t1'=140 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall clear height</td>
<td>h1'=2800 mm</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>E1'=5492.4 N/mm²</td>
</tr>
</tbody>
</table>

Roof slab 4

<table>
<thead>
<tr>
<th>Slab thickness</th>
<th>t4'=200 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of member 4</td>
<td>14'=6500 mm</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>E4'=36000 N/mm²</td>
</tr>
<tr>
<td>Char permanent load on member 4</td>
<td>Gk4'=4.3 kN/m²</td>
</tr>
<tr>
<td>Char variable load on member 4</td>
<td>Qk4'=1.5 kN/m²</td>
</tr>
</tbody>
</table>
Moment at the bottom of the wall - (at node point B)

1, 2 & 4 are member numbers
M2 = moment at bot of wall
A-B = represents the wall under consideration

Masonry wall 1
- Wall thickness: t1=140 mm
- Wall clear height: h1=2800 mm
- Modulus of elasticity of wall: E1=5492.4 N/mm²

Masonry wall 2
- Wall thickness: t2=140 mm
- Wall height: h2=2800 mm
- Modulus of elasticity of wall: E2=5492.4 N/mm²
- 2nd moment of area of wall: I2=228.67E6 mm⁴
- Compute: c2=448.55 kNm

Floor slab member 4
- Slab thickness: t4=200 mm
- Length of member 4: l4=6500 mm
- Modulus of elasticity: E4=36000 N/mm²
- Char permanent load on member 4: Gk4=4.3 kN/m²
- Char variable load on member 4: Qk4=3.5 kN/m²

Eccentricity of roof load
The eccentricity of roof load on masonry wall can be evaluated by dividing the design moment at the top of wall by the design vertical load at the top of the wall.
- Design vert. load @ top of wall: Nid=26.3 kN/m
- Eccentricity of roof load: ecc=ABS(M1)*1000/Nid=67.422 mm
### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall clear height</td>
<td>2800 mm</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td>Eccentricity of floor load</td>
<td>67.422 mm</td>
</tr>
<tr>
<td>Design moment (top of wall)</td>
<td>-1.7732 kNm/m</td>
</tr>
<tr>
<td>Design moment @ mid-height</td>
<td>0.19497 kNm/m</td>
</tr>
<tr>
<td>Design moment (bot of wall)</td>
<td>-2.1631 kNm/m</td>
</tr>
</tbody>
</table>
Location: Wall panel 2

BS EN 1996-1-1:2005, Annex C, gives a simplified method for calculating the out-of-plane eccentricity of loading on masonry walls. The structure is simplified to a sub-frame for the evaluation of moments at the top and bottom of the wall.

Partial safety factors

- Permanent actions: \( \gamma_G = 1.35 \)
- Variable actions: \( \gamma_Q = 1.5 \)

Moment at the top of wall - (at node point A)

\[
M_1 = \frac{1}{2} l l_1 (h_1')^2
\]

Masonry Wall 1

- Wall thickness: \( t_1' = 140 \text{ mm} \)
- Wall clear height: \( h_1' = 2800 \text{ mm} \)
- Modulus of elasticity: \( E_1' = 4050 \text{ N/mm}^2 \)

Masonry Wall 2

- Wall thickness: \( t_2' = 140 \text{ mm} \)
- Wall height: \( h_2' = 2800 \text{ mm} \)
- Modulus of elasticity of wall: \( E_2' = 4050 \text{ N/mm}^2 \)
- 2nd moment of area of wall: \( I_2' = \frac{1000 \times t_2'^3}{12} = 228.67 \text{E6 mm}^4 \)
- Compute: \( c_2 = \frac{E_2' I_2'}{h_2'^2} \times 1\text{E-6} = 330.75 \text{ kNm} \)

Floor slab 4

- Slab thickness: \( t_4' = 200 \text{ mm} \)
- Length of member 4: \( l_4' = 6500 \text{ mm} \)
- Modulus of elasticity: \( E_4' = 36000 \text{ N/mm}^2 \)
- Char permanent load on member 4: \( G_{k4}' = 4.3 \text{ kN/m}^2 \)
- Char variable load on member 4: \( Q_{k4}' = 3.5 \text{ kN/m}^2 \)
Moment at the bottom of the wall - (at node point B)

\[
A \quad - - -
\]
\[
/ (1)
\]
\[
/ (2)
\]
\[
/ (4)
\]
\[
M2 \quad - - - \quad / \text{floor}
\]
\[
B
\]

1, 2 & 4 are member numbers
M2 = moment at bot of wall
A-B = represents the wall under consideration

Masonry wall 1

Wall thickness \( t_1 = 140 \text{ mm} \)
Wall clear height \( h_1 = 3300 \text{ mm} \)
Modulus of elasticity of wall \( E_1 = 4050 \text{ N/mm}^2 \)

Masonry wall 2

Wall thickness \( t_2 = 140 \text{ mm} \)
Wall height \( h_2 = 2800 \text{ mm} \)
Modulus of elasticity of wall \( E_2 = 4050 \text{ N/mm}^2 \)
2nd moment of area of wall \( I_2 = 228.67 \text{E6 mm}^4 \)
compute \( c_2 = 330.75 \text{ kNm} \)

Floor slab member 4

Slab thickness \( t_4 = 200 \text{ mm} \)
Length of member 4 \( l_4 = 6500 \text{ mm} \)
Modulus of elasticity \( E_4 = 36000 \text{ N/mm}^2 \)
Char permanent load on member 4 \( G_{k4} = 4.3 \text{ kN/m}^2 \)
Char variable load on member 4 \( Q_{k4} = 3.5 \text{ kN/m}^2 \)

DESIGN SUMMARY

Wall clear height \( 2800 \text{ mm} \)
Wall thickness \( 140 \text{ mm} \)
Design moment (top of wall) \( -1.6939 \text{ kNm/m} \)
Design moment @ mid-height \( 0.011319 \text{ kNm/m} \)
Design moment (bot of wall) \( -1.7166 \text{ kNm/m} \)
Location: Wall panel 1

BS EN 1996-1-1:2005, Annex C, gives a simplified method for calculating the out-of-plane eccentricity of loading on masonry walls. The structure is simplified to a sub-frame for the evaluation of moments at the top and bottom of the wall.

Partial safety factors

Permanent actions \( \gamma_{G} = 1.35 \)
Variable actions \( \gamma_{Q} = 1.5 \)

Moment at the top of wall - (at node point A)

\[
\begin{aligned}
A \quad (2) & \quad (4) / \quad \text{floor} \\
M_1 / & \quad (1) \\
/ 1,2 & \text{ & } 4 \text{ are member numbers} \\
/ M_1 = \text{moment at top of wall} \\
/ A-B = \text{represents the wall} \\
/ \text{under consideration} \\
\end{aligned}
\]

Masonry Wall 1

- Wall thickness: \( t_1' = 140 \text{ mm} \)
- Wall clear height: \( h_1' = 3300 \text{ mm} \)
- Modulus of elasticity: \( E_1' = 4050 \text{ N/mm}^2 \)

Masonry Wall 2

- Wall thickness: \( t_2' = 140 \text{ mm} \)
- Wall height: \( h_2' = 2800 \text{ mm} \)
- Modulus of elasticity of wall: \( E_2' = 4050 \text{ N/mm}^2 \)
- 2nd moment of area of wall: \( I_2' = 1000 \times t_2'^3 / 12 = 228.67 \text{E6 mm}^4 \)
- Compute: \( c_2 = E_2' I_2' / h_2'^*1 \text{E}-6 = 330.75 \text{ kNm} \)

Floor slab 4

- Slab thickness: \( t_4' = 200 \text{ mm} \)
- Length of member 4: \( 14' = 6500 \text{ mm} \)
- Modulus of elasticity: \( E_4' = 36000 \text{ N/mm}^2 \)
- Char permanent load on member 4: \( G_{k4'} = 4.3 \text{ kN/m}^2 \)
- Char variable load on member 4: \( Q_{k4'} = 3.5 \text{ kN/m}^2 \)
Moment at the bottom of the wall - (at node point B)

A --- floor
/     \\
/     \\
/     \\
(1) = are member numbers
M2 = moment at bot of wall
A-B = represents the wall under consideration

DESIGN SUMMARY
Wall clear height 3300 mm
Wall thickness 140 mm
Design moment (top of wall) -1.4565 kNm/m
Design moment @ mid-height -0.36412 kNm/m
Design moment (bot of wall) -0.72824 kNm/m
Location: Wall panel 6

BS EN 1996-1-1:2005, Annex C, gives a simplified method for calculating the out-of-plane eccentricity of loading on masonry walls. The structure is simplified to a sub-frame for the evaluation of moments at the top and bottom of the wall.

Partial safety factors

<table>
<thead>
<tr>
<th>Permanent actions</th>
<th>gamG=1.35</th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable actions</td>
<td>gamQ=1.5</td>
</tr>
</tbody>
</table>

Moment at the top of wall - (at node point A)

\[
\begin{align*}
\text{M1} & \quad 1, 3 & \quad 4 \quad \text{are member numbers} \\
\text{M1} & \quad \text{M1} = \text{moment at top of wall} \\
\text{A-B} & \quad \text{A-B} = \text{represents the wall consideration} \\
\end{align*}
\]

Masonry Wall 1

<table>
<thead>
<tr>
<th>Wall thickness</th>
<th>t1' = 140 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall clear height</td>
<td>h1' = 2800 mm</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>E1' = 4050 N/mm²</td>
</tr>
</tbody>
</table>

Roof slab 3

<table>
<thead>
<tr>
<th>Slab thickness</th>
<th>t3' = 200 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of member 3</td>
<td>l3' = 5000 mm</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>E3' = 36000 N/mm²</td>
</tr>
</tbody>
</table>

Roof slab 4

<table>
<thead>
<tr>
<th>Slab thickness</th>
<th>t4' = 200 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of member 4</td>
<td>l4' = 6500 mm</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>E4' = 36000 N/mm²</td>
</tr>
<tr>
<td>Char permanent load on member 3</td>
<td>Gk3' = 4.3 kN/m²</td>
</tr>
<tr>
<td>Char variable load on member 3</td>
<td>Qk3' = 1.5 kN/m²</td>
</tr>
<tr>
<td>Char permanent load on member 4</td>
<td>Gk4' = 4.3 kN/m²</td>
</tr>
<tr>
<td>Char variable load on member 4</td>
<td>Qk4' = 1.5 kN/m²</td>
</tr>
</tbody>
</table>
Moment at the bottom of the wall - (at node point B)

1,2,3 & 4 are member numbers

M2 = moment at bot of wall

A-B = represents the wall under consideration

Masonry wall 1

Wall thickness \( t1 = 140 \text{ mm} \)
Wall clear height \( h1 = 2800 \text{ mm} \)
Modulus of elasticity of wall \( E1 = 4050 \text{ N/mm}^2 \)

Masonry wall 2

Wall thickness \( t2 = 140 \text{ mm} \)
Wall height \( h2 = 2800 \text{ mm} \)
Modulus of elasticity of wall \( E2 = 4050 \text{ N/mm}^2 \)
2nd moment of area of wall \( I2 = 228.67E6 \text{ mm}^4 \)
compute \( c2 = 330.75 \text{ kNm} \)

Floor slab member 3

Slab thickness \( t3 = 200 \text{ mm} \)
Length of member 3 \( l3 = 5000 \text{ mm} \)
Modulus of elasticity \( E3 = 36000 \text{ N/mm}^2 \)

Floor slab member 4

Slab thickness \( t4 = 200 \text{ mm} \)
Length of member 4 \( l4 = 6500 \text{ mm} \)
Modulus of elasticity \( E4 = 36000 \text{ N/mm}^2 \)
Char permanent load on member 3 \( Gk3 = 4.3 \text{ kN/m}^2 \)
Char variable load on member 3 \( Qk3 = 3.5 \text{ kN/m}^2 \)
Char permanent load on member 4 \( Gk4 = 4.3 \text{ kN/m}^2 \)
Char variable load on member 4 \( Qk4 = 3.5 \text{ kN/m}^2 \)
### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall clear height</td>
<td>2800 mm</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td>Design moment (top of wall)</td>
<td>-0.25365 kNm/m</td>
</tr>
<tr>
<td>Design moment @ mid-height</td>
<td>0.039929 kNm/m</td>
</tr>
<tr>
<td>Design moment (bot of wall)</td>
<td>-0.33351 kNm/m</td>
</tr>
</tbody>
</table>
Location: Wall panel 5

BS EN 1996-1-1:2005, Annex C, gives a simplified method for calculating the out-of-plane eccentricity of loading on masonry walls. The structure is simplified to a sub-frame for the evaluation of moments at the top and bottom of the wall.

Partial safety factors

<table>
<thead>
<tr>
<th>Permanent actions</th>
<th>gamG=1.35</th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable actions</td>
<td>gamQ=1.5</td>
</tr>
</tbody>
</table>

Moment at the top of wall - (at node point A)

Masonry Wall 1

<table>
<thead>
<tr>
<th>Wall thickness</th>
<th>t1'=140 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall clear height</td>
<td>h1'=2800 mm</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>E1'=4050 N/mm²</td>
</tr>
</tbody>
</table>

Masonry wall 2

<table>
<thead>
<tr>
<th>Wall thickness</th>
<th>t2'=140 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height</td>
<td>h2'=2800 mm</td>
</tr>
<tr>
<td>Modulus of elasticity of wall</td>
<td>E2'=4050 N/mm²</td>
</tr>
<tr>
<td>2nd moment of area of wall</td>
<td>I2'=1000*t2'^3/12=228.67E6 mm⁴</td>
</tr>
<tr>
<td>Compute</td>
<td>c2=E2'*I2'/h2'^1E-6=330.75 kNm</td>
</tr>
</tbody>
</table>

Floor slab 3

<table>
<thead>
<tr>
<th>Slab thickness</th>
<th>t3'=200 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of member 3</td>
<td>l3'=5000 mm</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>E3'=36000 N/mm²</td>
</tr>
</tbody>
</table>

Floor slab 4

| Slab thickness | t4'=200 mm |
Length of member 4 $l_4' = 6500 \text{ mm}$

Modulus of elasticity $E_4' = 36000 \text{ N/mm}^2$

Char permanent load on member 3 $G_{k3}' = 4.3 \text{ kN/m}^2$

Char variable load on member 3 $Q_{k3}' = 1.5 \text{ kN/m}^2$

Char permanent load on member 4 $G_{k4}' = 4.3 \text{ kN/m}^2$

Char variable load on member 4 $Q_{k4}' = 1.5 \text{ kN/m}^2$

Moment at the bottom of the wall - (at node point B)

1, 2, 3 & 4 are member numbers

$M_2$ = moment at bot of wall

A-B = represents the wall under consideration

Masonry wall 1

Wall thickness $t_1 = 140 \text{ mm}$

Wall clear height $h_1 = 3300 \text{ mm}$

Modulus of elasticity of wall $E_1 = 4050 \text{ N/mm}^2$

Masonry wall 2

Wall thickness $t_2 = 140 \text{ mm}$

Wall height $h_2 = 2800 \text{ mm}$

Modulus of elasticity of wall $E_2 = 4050 \text{ N/mm}^2$

2nd moment of area of wall $I_2 = 228.67E6 \text{ mm}^4$

compute

$c_2 = 330.75 \text{ kNm}$

Floor slab member 3

Slab thickness $t_3 = 200 \text{ mm}$

Length of member 3 $l_3 = 5000 \text{ mm}$

Modulus of elasticity $E_3 = 36000 \text{ N/mm}^2$

Floor slab member 4

Slab thickness $t_4 = 200 \text{ mm}$

Length of member 4 $l_4 = 6500 \text{ mm}$

Modulus of elasticity $E_4 = 36000 \text{ N/mm}^2$

Char permanent load on member 3 $G_{k3} = 4.3 \text{ kN/m}^2$

Char variable load on member 3 $Q_{k3} = 3.5 \text{ kN/m}^2$

Char permanent load on member 4 $G_{k4} = 4.3 \text{ kN/m}^2$

Char variable load on member 4 $Q_{k4} = 3.5 \text{ kN/m}^2$
## DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall clear height</td>
<td>2800 mm</td>
<td></td>
</tr>
<tr>
<td>Wall thickness</td>
<td>140 mm</td>
<td></td>
</tr>
<tr>
<td>Design moment (top of wall)</td>
<td>-0.24301 kNm/m</td>
<td></td>
</tr>
<tr>
<td>Design moment @ mid-height</td>
<td>0.04632 kNm/m</td>
<td></td>
</tr>
<tr>
<td>Design moment (bot of wall)</td>
<td>-0.33565 kNm/m</td>
<td></td>
</tr>
</tbody>
</table>
Location: Wall panel 4

BS EN 1996-1-1:2005, Annex C, gives a simplified method for calculating the out-of-plane eccentricity of loading on masonry walls. The structure is simplified to a sub-frame for the evaluation of moments at the top and bottom of the wall.

Partial safety factors

Permanent actions \[ \text{gam}_G=1.35 \]
Variable actions \[ \text{gam}_Q=1.5 \]

Moment at the top of wall - (at node point A)

\[
\begin{align*}
\text{M1} = & \frac{1}{1,2,3 \& 4} \text{ are member numbers} \\
A-B = & \text{represents the wall under consideration} \\
\end{align*}
\]

Masonry Wall 1

Wall thickness \[ t_1'=140 \text{ mm} \]
Wall clear height \[ h_1'=3300 \text{ mm} \]
Modulus of elasticity \[ E_1'=4050 \text{ N/mm}^2 \]

Masonry wall 2

Wall thickness \[ t_2'=140 \text{ mm} \]
Wall height \[ h_2'=2800 \text{ mm} \]
Modulus of elasticity of wall \[ E_2'=4050 \text{ N/mm}^2 \]
2nd moment of area of wall \[ I_2'=1000*t_2'^3/12=228.67E6 \text{ mm}^4 \]
compute \[ c_2=E_2'*I_2'/h_2'^1E-6=330.75 \text{ kNm} \]

Floor slab 3

Slab thickness \[ t_3'=200 \text{ mm} \]
Length of member 3 \[ l_3'=5000 \text{ mm} \]
Modulus of elasticity \[ E_3'=36000 \text{ N/mm}^2 \]

Floor slab 4

Slab thickness \[ t_4'=200 \text{ mm} \]
Length of member 4  
14' = 6500 mm
Modulus of elasticity  
$E_4' = 36000 \text{ N/mm}^2$
Char permanent load on member 3  
$G_{k3}' = 4.3 \text{ kN/m}^2$
Char variable load on member 3  
$Q_{k3}' = 3.5 \text{ kN/m}^2$
Char permanent load on member 4  
$G_{k4}' = 4.3 \text{ kN/m}^2$
Char variable load on member 4  
$Q_{k4}' = 3.5 \text{ kN/m}^2$

**Moment at the bottom of the wall - (at node point B)**

```
A    - - floor
      /    / (1) = member number
      /    M2 = moment at bot of wall
      /    (1) A-B = represents the wall
      /    under consideration
      /    B foundation
\   \                           
```

**DESIGN SUMMARY**

- **Wall clear height**: 3300 mm
- **Wall thickness**: 140 mm
- **Design moment (top of wall)**: -0.28479 kNm/m
- **Design moment @ mid-height**: -0.071198 kNm/m
- **Design moment (bot of wall)**: -0.1424 kNm/m
Location: Wall panel 1 Single storey structure

BS EN 1996-1-1:2005, Annex C, gives a simplified method for calculating the out-of-plane eccentricity of loading on masonry walls. The structure is simplified to a sub-frame for the evaluation of moments at the top and bottom of the wall.

Moment at the top of wall - (at node point A)

Masonry Wall 1

Wall thickness \( t_1' = 140 \text{ mm} \)
Wall clear height \( h_1' = 3300 \text{ mm} \)
Modulus of elasticity \( E_1' = 4050 \text{ N/mm}^2 \)

Floor slab 4

Slab thickness \( t_4' = 200 \text{ mm} \)
Length of member 4 \( l_4' = 6500 \text{ mm} \)
Modulus of elasticity \( E_4' = 36000 \text{ N/mm}^2 \)
Char permanent load on member 4 \( G_k 4' = 4.3 \text{ kN/m}^2 \)
Char variable load on member 4 \( Q_k 4' = 1.5 \text{ kN/m}^2 \)

Moment at the bottom of the wall - (at node point B)
DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall clear height</td>
<td>3300 mm</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td>Design moment (top of wall)</td>
<td>-1.1639 kNm/m</td>
</tr>
<tr>
<td>Design moment @ mid-height</td>
<td>-0.58195 kNm/m</td>
</tr>
<tr>
<td>Design moment (bot of wall)</td>
<td>0 kNm/m</td>
</tr>
</tbody>
</table>
Location: Wall panel 2 Single storey structure

BS EN 1996-1-1:2005, Annex C, gives a simplified method for calculating the out-of-plane eccentricity of loading on masonry walls. The structure is simplified to a sub-frame for the evaluation of moments at the top and bottom of the wall.

Masonry and floor details

Mortar designation (Table NA.2) mortar=3
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Compressive strength of units po=7.3 N/mm²
Constant K from NA Table NA.4 K(1)=0.75
Height of masonry unit hu=215 mm
Least horiz. dimension of units lhd=140 mm

Moment at the top of wall - (at node point A)

\[ M1 = \text{moment at top of wall} \]
A-B = represents the wall under consideration

Masonry Wall 1

Wall thickness t1'=140 mm
Wall clear height h1'=3300 mm
Modulus of elasticity E1'=5492.4 N/mm²

Floor slab 3

Slab thickness t3'=200 mm
Length of member 3 l3'=5000 mm
Modulus of elasticity E3'=36000 N/mm²

Floor slab 4

Slab thickness t4'=200 mm
Length of member 4 l4'=6500 mm
Modulus of elasticity E4'=36000 N/mm²
Char permanent load on member 3 Gk3'=4.3 kN/m²
Char variable load on member 3 Qk3'=1.5 kN/m²
Char permanent load on member 4 Gk4'=4.3 kN/m²
Char variable load on member 4 Qk4'=1.5 kN/m²
Moment at the bottom of the wall - (at node point B)

\[ 1, 3 \text{ & } 4 \text{ are member numbers} \\
M2 = \text{moment at bottom of wall} \\
A-B = \text{represents the wall under consideration} \\
\]

**DESIGN SUMMARY**
- Wall clear height: 3300 mm
- Wall thickness: 140 mm
- Design moment (top of wall): -0.28996 kNm/m
- Design moment @ mid-height: -0.07249 kNm/m
- Design moment (bot of wall): -0.14498 kNm/m
Location: Internal Partition A1

Wall restrained on three sides but not at the top.

Masonry details

Length of wall panel \( L = 6 \text{ m} \)
Height of wall panel \( H = 2 \text{ m} \)
Effective wall thickness \( t = 200 \text{ mm} \)

DESIGN SUMMARY

Wall length 6 m
Wall height 2 m
Wall effective thickness 200 mm
Location: Internal Partition A2

Wall restrained on three sides but not at the top.

Masonry details

Length of wall panel \( L = 6 \text{ m} \)
Height of wall panel \( H = 4 \text{ m} \)
Effective wall thickness \( t = 200 \text{ mm} \)

DESIGN SUMMARY

Wall length \( 6 \text{ m} \)
Wall height \( 4 \text{ m} \)
Wall effective thickness \( 200 \text{ mm} \)
Location: Internal Partition A3

Wall restrained on three sides but not at the top.

Masonry details

Length of wall panel \( L = 6 \) m
Height of wall panel \( H = 6 \) m
Effective wall thickness \( t = 140 \) mm

DESIGN SUMMARY

Wall length \( 6 \) m
Wall height \( 6 \) m
Wall effective thickness \( 140 \) mm
Location: Internal Partition B1

Wall restrained on four sides.

Masonry details

- Length of wall panel: \( L = 5 \text{ m} \)
- Height of wall panel: \( H = 5 \text{ m} \)
- Effective wall thickness: \( t = 140 \text{ mm} \)

DESIGN SUMMARY

- Wall length: 5 m
- Wall height: 5 m
- Wall effective thickness: 140 mm
Location: Internal Partition B2

Wall restrained on four sides.

Masonry details

Length of wall panel \( L = 8 \text{ m} \)
Height of wall panel \( H = 4 \text{ m} \)
Effective wall thickness \( t = 200 \text{ mm} \)

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall length</td>
<td>8 m</td>
</tr>
<tr>
<td>Wall height</td>
<td>4 m</td>
</tr>
<tr>
<td>Wall effective thickness</td>
<td>200 mm</td>
</tr>
</tbody>
</table>
Location: Internal Partition B3

Wall restrained on four sides.

Masonry details

- Length of wall panel: L=9 m
- Height of wall panel: H=8 m
- Effective wall thickness: t=200 mm

**DESIGN SUMMARY**

- Wall length: 9 m
- Wall height: 8 m
- Wall effective thickness: 200 mm
Location: Internal Partition C1

Wall restrained at the top but not at the ends.

Masonry details

- Length of wall panel: L=6 m
- Height of wall panel: H=4.2 m
- Effective wall thickness: t=140 mm

**DESIGN SUMMARY**

- Wall length: 6 m
- Wall height: 4.2 m
- Wall effective thickness: 140 mm
Location: Internal Partition A1

Wall restrained on three sides but not at the top.

Length of wall panel \( L = 6 \text{ m} \)
Height of wall panel \( H = 2 \text{ m} \)
Effective wall thickness \( t = 200 \text{ mm} \)

**DESIGN SUMMARY**
- Wall length: 6 m
- Wall height: 2 m
- Wall effective thickness: 200 mm
Location: Internal Partition A2

Wall restrained on three sides but not at the top.

Length of wall panel $L=6\ m$
Height of wall panel $H=4\ m$
Effective wall thickness $t=200\ mm$

**DESIGN SUMMARY**

- **Wall length**: 6 m
- **Wall height**: 4 m
- **Wall effective thickness**: 200 mm
Location: Internal Partition A3

Wall restrained on three sides but not at the top.

Length of wall panel L=6 m
Height of wall panel H=6 m
Effective wall thickness t=140 mm

Design Summary

Wall length 6 m
Wall height 6 m
Wall effective thickness 140 mm
Location: Internal Partition B1

Wall restrained on four sides.

Length of wall panel L = 5 m
Height of wall panel H = 5 m
Effective wall thickness t = 140 mm

DESIGN SUMMARY
Wall length 5 m
Wall height 5 m
Wall effective thickness 140 mm
**Location: Internal Partition B2**

Wall restrained on four sides.

Length of wall panel $L = 8 \text{ m}$
Height of wall panel $H = 4 \text{ m}$
Effective wall thickness $t = 200 \text{ mm}$

**DESIGN SUMMARY**

- Wall length: 8 m
- Wall height: 4 m
- Wall effective thickness: 200 mm
Location: Internal Partition B3

Wall restrained on four sides.

Length of wall panel: $L = 9\, m$
Height of wall panel: $H = 8\, m$
Effective wall thickness: $t = 200\, mm$

**DESIGN SUMMARY**

- Wall length: 9 m
- Wall height: 8 m
- Wall effective thickness: 200 mm
Location: Internal Partition C1

Wall restrained at the top but not at the ends.

Length of wall panel \( L = 6 \text{ m} \)
Height of wall panel \( H = 4.2 \text{ m} \)
Effective wall thickness \( t = 140 \text{ mm} \)

DESIGN SUMMARY

- Wall length : 6 m
- Wall height : 4.2 m
- Wall effective thickness : 140 mm
**Location: Wall on grid line A/1-2**

**External cavity wall**

This calculation is based on BS5628-1:2005, the Code of practice for use of masonry, Part 1: Structural use of unreinforced masonry.

WL = wind load

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of outer leaf</td>
<td>t(1)=100 mm</td>
</tr>
<tr>
<td>Density of outer leaf</td>
<td>den(1)=12.4 kN/m³</td>
</tr>
<tr>
<td>Thickness of inner leaf</td>
<td>t(2)=140 mm</td>
</tr>
<tr>
<td>Density of inner leaf</td>
<td>den(2)=12.4 kN/m³</td>
</tr>
<tr>
<td>Clear height of wall</td>
<td>h=2.425 m</td>
</tr>
<tr>
<td>Support Restraint factor</td>
<td>rf=0.75</td>
</tr>
<tr>
<td>Length of wall panel</td>
<td>L=4 m</td>
</tr>
<tr>
<td>Mortar designation to Table 1</td>
<td>mortar=3</td>
</tr>
<tr>
<td>Partial Safety Factor Material</td>
<td>γₚₘₐₙ=3.1</td>
</tr>
<tr>
<td>Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=1</td>
<td></td>
</tr>
<tr>
<td>Compressive Strength of Units po(1)=21 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Water absorption %age of leaf</td>
<td>water=9</td>
</tr>
<tr>
<td>Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(2)=4</td>
<td></td>
</tr>
<tr>
<td>Compressive Strength of Units po(2)=3.6 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Height</td>
<td>h_b(2)=215 mm</td>
</tr>
<tr>
<td>Least horizontal dimension</td>
<td>lhd(2)=140 mm</td>
</tr>
<tr>
<td>Dead load from upper levels DL(1)</td>
<td>4 kN/m</td>
</tr>
<tr>
<td>Dead load from upper levels DL(2)</td>
<td>7 kN/m</td>
</tr>
<tr>
<td>Dead load from supported floor DL(3)</td>
<td>12 kN/m</td>
</tr>
<tr>
<td>Live load from supported floor LL(3)</td>
<td>5.5 kN/m</td>
</tr>
<tr>
<td>Ecc of floor load on inner leaf ecc</td>
<td>23.333 mm</td>
</tr>
<tr>
<td>Horizontal wind load WL</td>
<td>0.62 kN/m²</td>
</tr>
</tbody>
</table>

**Vertical load capacity**

For an external cavity wall it is only necessary to check the inner leaf which carries the majority of load.
At top of Wall:

**Case 1: Max load from above 1.4DL + 1.6LL**

Design vertical load  
\[ g = 1.4 \times (DL(2) + DL(3)) + 1.6 \times (LL(2) + LL(3)) = 43.4 \text{ kN/m} \]

Design vert. load resist of wall  
\[ gd = \beta \times \left( \frac{t(2)}{1000} \times \frac{f_k(2) \times 1000}{\gamma_m} \right) = 115.34 \text{ kN/m} \]

Since \( g \leq gd \) (43.4 kN/m \leq 115.34 kN/m) design vertical load within vertical load resistance of wall, therefore OK.

**Case 2: Min load from above 0.9DL + 1.6LL**

Design vertical load  
\[ g = 0.9 \times DL(2) + 1.4 \times DL(3) + 1.6 \times (LL(2) + LL(3)) = 39.9 \text{ kN/m} \]

Design vert. load resist of wall  
\[ gd = \beta \times \left( \frac{t(2)}{1000} \times \frac{f_k(2) \times 1000}{\gamma_m} \right) = 113.47 \text{ kN/m} \]

Since \( g \leq gd \) (39.9 kN/m \leq 113.47 kN/m) design vertical load within vertical load resistance of wall, therefore OK.

**Lateral load capacity**

Select Case(b) of Clause 18 of the Code: 0.9DL+1.4WL

Design moment (simple supports)  
\[ M = 1.4 \times WL \times h^2 / 8 = 0.63805 \text{ kNm/m} \]

Moment reduction factor  
\[ mrf = 1 \]

Elastic design moment  
\[ M = M / mrf = 0.63805 \text{ kNm/m} \]

Since \( M \leq Mr \) (0.63805 kNm/m \leq 0.97615 kNm/m) elastic design moment within design moment of resistance, therefore OK.

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Wall height</th>
<th>2.425 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char. flex strength</td>
<td>0.4 N/mm²</td>
</tr>
<tr>
<td>Char. comp strength</td>
<td>5.13 N/mm²</td>
</tr>
<tr>
<td>Thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td>Char flex strength</td>
<td>0.22333 N/mm²</td>
</tr>
<tr>
<td>Char. comp strength</td>
<td>2.9031 N/mm²</td>
</tr>
<tr>
<td>Effective wall thickness</td>
<td>160 mm</td>
</tr>
<tr>
<td>Effective wall height</td>
<td>1.8188 m</td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td>11.367</td>
</tr>
<tr>
<td>Max vertical design load</td>
<td>43.4 kN</td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td>115.34 kN</td>
</tr>
<tr>
<td>Vertical load (max ecc)</td>
<td>39.9 kN</td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td>113.47 kN</td>
</tr>
<tr>
<td>Wind pressure</td>
<td>0.62 kN/m²</td>
</tr>
<tr>
<td>Elastic design moment</td>
<td>0.63805 kNm</td>
</tr>
<tr>
<td>Moment of resistance</td>
<td>0.97615 kNm</td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

This calculation is based on BS5628-1:2005, the Code of practice for use of masonry, Part 1: Structural use of unreinforced masonry.

External cavity wall

Thickness of outer leaf \( t(1) = 100 \text{ mm} \)
Density of outer leaf \( \text{den}(1) = 12.4 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2) = 140 \text{ mm} \)
Density of inner leaf \( \text{den}(2) = 12.4 \text{ kN/m}^3 \)
Clear height of wall \( h = 2.425 \text{ m} \)
Support Restraint factor \( \text{rf} = 0.75 \)
Length of wall panel \( L = 4 \text{ m} \)
Mortar designation to Table 1 \( \text{mortar}=3 \)
Partial Safety Factor Material \( \gamma_m = 3.1 \)
Masonry type \( 1=\text{clay} \ 2=\text{cal silicate} \ 3=\text{conc brick} \ 4=\text{block} \) \( \text{typ}(1)=2 \)
Compressive Strength of Units \( \text{po}(1)=60 \text{ N/mm}^2 \)
Masonry type \( 1=\text{clay} \ 2=\text{cal silicate} \ 3=\text{conc brick} \ 4=\text{block} \) \( \text{typ}(2)=3 \)
Compressive Strength of Units \( \text{po}(2)=5.2 \text{ N/mm}^2 \)
Dead load from upper levels \( \text{DL}(1)=4 \text{ kN/m} \)
Dead load from upper levels \( \text{DL}(2)=5 \text{ kN/m} \)
Dead load from upper levels \( \text{DL}(3)=10 \text{ kN/m} \)
Dead load from supported floor \( \text{LL}(2)=5.5 \text{ kN/m} \)
Live load from supported floor \( \text{LL}(3)=5 \text{ kN/m} \)
Ecc of floor load on inner leaf \( \text{ecc}=23.333 \text{ mm} \)
Horizontal wind load \( \text{WL}=0.63 \text{ kN/m}^2 \)

Vertical load capacity

For an external cavity wall it is only necessary to check the inner leaf which carries the majority of load.

At top of Wall:

**Case 1: Max load from above 1.4DL + 1.6LL**

Design vertical load \( g=1.4*(\text{DL}(2)+\text{DL}(3))+1.6*(\text{LL}(2)+\text{LL}(3)) = 37.8 \text{ kN/m} \)
Design vert. load resist of wall \( \text{gd}=\beta \frac{t(2)}{1000} * (f_k(2) * 1000) / \gamma_m = 99.054 \text{ kN/m} \)
Since \( g \leq \text{gd} \) (37.8 kN/m \( \leq 99.054 \text{ kN/m} \)) design vertical load within vertical load resistance of wall, therefore OK.
Case 2: Min load from above  \(0.9DL + 1.6LL\)

Design vertical load
\[ g = 0.9*DL(2) + 1.4*DL(3) + 1.6*(LL(2) + LL(3)) = 35.3 \text{kN/m} \]

Design vert. load resist of wall
\[ gd = \beta*(t(2)/1000)*(fk(2)*1000)/\gamma_m = 97.652 \text{kN/m} \]

Since \(g \leq gd\) (35.3 kN/m \(\leq 97.652 \text{kN/m}\)) design vertical load within vertical load resistance of wall, therefore OK.

Lateral load capacity

Select Case(b) of Clause 18 of the Code: \(0.9DL + 1.4WL\)

Design moment (simple supports)
\[ M = 1.4*WL*h^2/8 = 0.64834 \text{kNm/m} \]

Moment reduction factor
\[ mrf = 1.5 \]

Elastic design moment
\[ M = M/mrf = 0.43223 \text{kNm/m} \]

Since \(M \leq Mr\) (0.43223 kNm/m \(\leq 0.85242 \text{kNm/m}\)) elastic design moment within design moment of resistance, therefore OK.

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>2.425 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTER LEAF</td>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char. flex strength</td>
<td>0.3 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Char. comp strength</td>
<td>9.24 N/mm²</td>
<td></td>
</tr>
<tr>
<td>INNER LEAF</td>
<td>Thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td>Char flex strength</td>
<td>0.3 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Char. comp strength</td>
<td>2.5 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Effective wall thickness</td>
<td>160 mm</td>
<td></td>
</tr>
<tr>
<td>Effective wall height</td>
<td>1.8188 m</td>
<td></td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td>11.367</td>
<td></td>
</tr>
<tr>
<td>Max vertical design load</td>
<td>37.8 kN</td>
<td></td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td>99.054 kN</td>
<td></td>
</tr>
<tr>
<td>Vertical load (max ecc)</td>
<td>35.3 kN</td>
<td></td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td>97.652 kN</td>
<td></td>
</tr>
<tr>
<td>Wind pressure</td>
<td>0.63 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Elastic design moment</td>
<td>0.43223 kNm</td>
<td></td>
</tr>
<tr>
<td>Moment of resistance</td>
<td>0.85242 kNm</td>
<td></td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

Load(1) Load(2) Load(3)

External cavity wall

This calculation is based on BS5628-1:2005, the Code of practice for use of masonry, Part 1: Structural use of unreinforced masonry.

WL = wind load

Thickness of outer leaf \( t(1) = 100 \text{ mm} \)
Density of outer leaf \( \text{den}(1) = 12.4 \text{ kN/m}^2 \)
Thickness of inner leaf \( t(2) = 140 \text{ mm} \)
Density of inner leaf \( \text{den}(2) = 12.4 \text{ kN/m}^2 \)
Clear height of wall \( h = 4.25 \text{ m} \)
Support Restraint factor \( rf = 0.75 \)
Length of wall panel \( L = 4 \text{ m} \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Partial Safety Factor Material \( \text{gammam} = 3.1 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 1 \)
Compressive Strength of Units \( \text{po}(1) = 20 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{water} = 9 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(2) = 4 \)
Compressive Strength of Units \( \text{po}(2) = 22.5 \text{ N/mm}^2 \)
Height \( h_b(2) = 215 \text{ mm} \)
Least horizontal dimension \( lhd(2) = 140 \text{ mm} \)
Dead load from upper levels \( DL(1) = 4 \text{ kN/m} \)
Dead load from upper levels \( DL(2) = 7 \text{ kN/m} \)
Live load from upper levels \( LL(2) = 5.5 \text{ kN/m} \)
Dead load from supported floor \( DL(3) = 12 \text{ kN/m} \)
Live load from supported floor \( LL(3) = 5 \text{ kN/m} \)
Ecc of floor load on inner leaf \( ecc = 23.333 \text{ mm} \)
Horizontal wind load \( WL = 0.25 \text{ kN/m}^2 \)

Vertical load capacity

For an external cavity wall it is only necessary to check the inner leaf which carries the majority of load.
At top of Wall:

**Case 1: Max load from above 1.4DL + 1.6LL**

Design vertical load 
\[ g = 1.4 \times (DL(2)+DL(3)) + 1.6 \times (LL(2) + LL(3)) = 43.4 \text{ kN/m} \]

Design vert. load resist of wall 
\[ gd = \beta \times (t(2)/1000) \times (f_k(2) \times 1000) / \gamma_m = 292.99 \text{ kN/m} \]

Since \( g \geq gd \) (43.4 kN/m \( \geq \) 292.99 kN/m) design vertical load within vertical load resistance of wall, therefore OK.

**Case 2: Min load from above 0.9DL + 1.6LL**

Design vertical load 
\[ g = 0.9 \times DL(2)+1.4 \times DL(3)+1.6 \times (LL(2) + LL(3)) = 39.9 \text{ kN/m} \]

Design vert. load resist of wall 
\[ gd = \beta \times (t(2)/1000) \times (f_k(2) \times 1000) / \gamma_m = 288.28 \text{ kN/m} \]

Since \( g \geq gd \) (39.9 kN/m \( \geq \) 288.28 kN/m) design vertical load within vertical load resistance of wall, therefore OK.

**Lateral load capacity**

Select Case(b) of Clause 18 of the Code: 0.9DL+1.4WL

Design moment (simple supports) 
\[ M = 1.4 \times WL \times h^2 / 8 = 0.79023 \text{ kNm/m} \]

Moment reduction factor 
\[ mrf = 1 \]

Elastic design moment 
\[ M = M / mrf = 0.79023 \text{ kNm/m} \]

Since \( M \geq Mr \) (0.79023 kNm/m \( \geq \) 1.1715 kNm/m) elastic design moment within design moment of resistance, therefore OK.

**DESIGN SUMMARY**

| Wall height | 4.25 m |
| Thickness   | 100 mm |
| Char. flex strength | 0.4 N/mm² |
| Char. comp strength | 5 N/mm² |
| INNER LEAF | 140 mm |
| Char flex strength | 0.25 N/mm² |
| Char. comp strength | 10.01 N/mm² |
| Effective wall thickness | 160 mm |
| Effective wall height | 3.1875 m |
| Slenderness ratio | 19.922 |
| Max vertical design load | 43.4 kN |
| Vertical load resistance | 292.99 kN |
| Vertical load (max ecc) | 39.9 kN |
| Vertical load resistance | 288.28 kN |
| Wind pressure | 0.25 kN/m² |
| Elastic design moment | 0.79023 kNm |
| Moment of resistance | 1.1715 kNm |
Location: Wall on grid line A/1-2

External cavity wall

This calculation is based on BS5628-1:2005, the Code of practice for use of masonry, Part 1: Structural use of unreinforced masonry.

WL = wind load

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of outer leaf</td>
<td>t(1)=102 mm</td>
</tr>
<tr>
<td>Density of outer leaf</td>
<td>den(1)=22 kN/m³</td>
</tr>
<tr>
<td>Thickness of inner leaf</td>
<td>t(2)=102 mm</td>
</tr>
<tr>
<td>Density of inner leaf</td>
<td>den(2)=22 kN/m³</td>
</tr>
<tr>
<td>Clear height of wall</td>
<td>h=2.425 m</td>
</tr>
<tr>
<td>Support Restraint factor</td>
<td>rf=0.75</td>
</tr>
<tr>
<td>Length of wall panel</td>
<td>L=4 m</td>
</tr>
<tr>
<td>Mortar designation to Table 1</td>
<td>mortar=3</td>
</tr>
<tr>
<td>Partial Safety Factor Material</td>
<td>gammam=3.1</td>
</tr>
<tr>
<td>Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)</td>
<td>1</td>
</tr>
<tr>
<td>Compressive Strength of Units</td>
<td>po(1)=20 N/mm²</td>
</tr>
<tr>
<td>Water absorption %age of leaf</td>
<td>water=9</td>
</tr>
<tr>
<td>Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(2)</td>
<td>1</td>
</tr>
<tr>
<td>Compressive Strength of Units</td>
<td>po(2)=30 N/mm²</td>
</tr>
<tr>
<td>Water absorption %age of leaf</td>
<td>water=9</td>
</tr>
<tr>
<td>Dead load from upper levels</td>
<td>DL(1)=4 kN/m</td>
</tr>
<tr>
<td>Dead load from upper levels</td>
<td>DL(2)=7 kN/m</td>
</tr>
<tr>
<td>Live load from upper levels</td>
<td>LL(2)=5.5 kN/m</td>
</tr>
<tr>
<td>Dead load from supported floor</td>
<td>DL(3)=12 kN/m</td>
</tr>
<tr>
<td>Live load from supported floor</td>
<td>LL(3)=5 kN/m</td>
</tr>
<tr>
<td>Ecc of floor load on inner leaf</td>
<td>ecc=17 mm</td>
</tr>
<tr>
<td>Horizontal wind load</td>
<td>WL=0.63 kN/m²</td>
</tr>
</tbody>
</table>

Vertical load capacity

For an external cavity wall it is only necessary to check the inner leaf which carries the majority of load.
At top of Wall:

**Case 1: Max load from above  1.4DL + 1.6LL**

Design vertical load  
\[ g = 1.4 \times (DL(2)+DL(3)) + 1.6 \times (LL(2)+LL(3)) = 43.4 \text{ kN/m} \]

Design vert. load resist of wall  
\[ gd = \beta \times (t(2)/1000) \times (fk(2) \times 1000) / \gamma_m = 175.83 \text{ kN/m} \]

Since \( g \leq gd \) (43.4 kN/m \leq 175.83 kN/m) design vertical load within vertical load resistance of wall, therefore OK.

**Case 2: Min load from above  0.9DL + 1.6LL**

Design vertical load  
\[ g = 0.9 \times DL(2)+1.4 \times DL(3)+1.6 \times (LL(2)+LL(3)) = 39.9 \text{ kN/m} \]

Design vert. load resist of wall  
\[ gd = \beta \times (t(2)/1000) \times (fk(2) \times 1000) / \gamma_m = 173.29 \text{ kN/m} \]

Since \( g \leq gd \) (39.9 kN/m \leq 173.29 kN/m) design vertical load within vertical load resistance of wall, therefore OK.

**Lateral load capacity**

Select Case(b) of Clause 18 of the Code: 0.9DL+1.4WL

Design moment (simple supports)  
\[ M = 1.4 \times WL \times h^2/8 = 0.64834 \text{ kNm/m} \]

Moment reduction factor  
\[ mrf = 1 \]

Elastic design moment  
\[ M = M/mrf = 0.64834 \text{ kNm/m} \]

Since \( M \leq Mr \) (0.64834 kNm/m \leq 0.88264 kNm/m) elastic design moment within design moment of resistance, therefore OK.

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Wall height</th>
<th>2.425 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>102 mm</td>
</tr>
<tr>
<td>Char. flex strength</td>
<td>0.4 N/mm²</td>
</tr>
<tr>
<td>Char. comp strength</td>
<td>5 N/mm²</td>
</tr>
<tr>
<td>Thickness</td>
<td>102 mm</td>
</tr>
<tr>
<td>Char. flex strength</td>
<td>0.4 N/mm²</td>
</tr>
<tr>
<td>Char. comp strength</td>
<td>6.3 N/mm²</td>
</tr>
<tr>
<td>Effective wall thickness</td>
<td>136 mm</td>
</tr>
<tr>
<td>Effective wall height</td>
<td>1.8188 m</td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td>13.373</td>
</tr>
<tr>
<td>Max vertical design load</td>
<td>43.4 kN</td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td>175.83 kN</td>
</tr>
<tr>
<td>Vertical load (max ecc)</td>
<td>39.9 kN</td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td>173.29 kN</td>
</tr>
<tr>
<td>Wind pressure</td>
<td>0.63 kN/m²</td>
</tr>
<tr>
<td>Elastic design moment</td>
<td>0.64834 kNm</td>
</tr>
<tr>
<td>Moment of resistance</td>
<td>0.88264 kNm</td>
</tr>
</tbody>
</table>
Load(1) Load(2) 

\[ \text{Load(3)} \]

External cavity wall

This calculation is based on BS5628-1:2005, the Code of practice for use of masonry, Part 1: Structural use of unreinforced masonry.

\[ \text{WL} = \text{wind load} \]

Thickness of outer leaf \( t(1)=102.5 \text{ mm} \)
Density of outer leaf \( \text{den}(1)=20 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2)=140 \text{ mm} \)
Density of inner leaf \( \text{den}(2)=20 \text{ kN/m}^3 \)
Clear height of wall \( h=3 \text{ m} \)
Support Restraint factor \( \text{rf}=0.75 \)
Length of wall panel \( L=4 \text{ m} \)
Mortar designation to Table 1 \( \text{mortar}=3 \)
Partial Safety Factor Material \( \text{gammam}=3.1 \)
Masonry type \( (1=\text{clay} \ 2=\text{cal silicate} \ 3=\text{conc brick} \ 4=\text{block}) \) \( \text{typ}(1)=1 \)
Compressive Strength of Units \( \text{po}(1)=20 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{water}=12 \)
Masonry type \( (1=\text{clay} \ 2=\text{cal silicate} \ 3=\text{conc brick} \ 4=\text{block}) \) \( \text{typ}(2)=4 \)
Compressive Strength of Units \( \text{po}(2)=7.3 \text{ N/mm}^2 \)
Height \( \text{hb}(2)=215 \text{ mm} \)
Least horizontal dimension \( \text{lhd}(2)=140 \text{ mm} \)
Dead load from upper levels \( \text{DL}(1)=10 \text{ kN/m} \)
Dead load from upper levels \( \text{DL}(2)=88.889 \text{ kN/m} \)
Live load from upper levels \( \text{LL}(2)=0 \text{ kN/m} \)
Dead load from supported floor \( \text{DL}(3)=7.4074 \text{ kN/m} \)
Live load from supported floor \( \text{LL}(3)=0 \text{ kN/m} \)
Ecc of floor load on inner leaf \( \text{ecc}=23.333 \text{ mm} \)
Horizontal wind load \( \text{WL}=0.66667 \text{ kN/m}^2 \)

Vertical load capacity

For an external cavity wall it is only necessary to check the inner leaf which carries the majority of load.
At top of Wall:

**Case 1: Max load from above  1.4DL + 1.6LL**

Design vertical load
\[ g = 1.4 \times (DL(2) + DL(3)) + 1.6 \times (LL(2) + LL(3)) = 134.81 \text{ kN/m} \]

Design vert. load resist of wall
\[ gd = \beta \times \left( \frac{t(2)}{1000} \times (fk(2) \times 1000) \right) / \gamma_m = 214.98 \text{ kN/m} \]

Since \( g \leq gd \) (134.81 kN/m ≤ 214.98 kN/m) design vertical load within vertical load resistance of wall, therefore OK.

**Case 2: Min load from above  0.9DL + 1.6LL**

Design vertical load
\[ g = 0.9 \times DL(2) + 1.4 \times DL(3) + 1.6 \times (LL(2) + LL(3)) = 90.37 \text{ kN/m} \]

Design vert. load resist of wall
\[ gd = \beta \times \left( \frac{t(2)}{1000} \times (fk(2) \times 1000) \right) / \gamma_m = 214.98 \text{ kN/m} \]

Since \( g \leq gd \) (90.37 kN/m ≤ 214.98 kN/m) design vertical load within vertical load resistance of wall, therefore OK.

**Lateral load capacity**

Select Case(b) of Clause 18 of the Code: 0.9DL+1.4WL

Design moment (simple supports)
\[ M = 1.4 \times WL \times h^2 / 8 = 1.05 \text{ kNm/m} \]

Moment reduction factor
\[ mrf = 1.5 \]

Elastic design moment
\[ M = M / mrf = 0.7 \text{ kNm/m} \]

Since \( M \leq Mr \) (0.7 kNm/m ≤ 2.7727 kNm/m) elastic design moment within design moment of resistance, therefore OK.

<table>
<thead>
<tr>
<th><strong>DESIGN SUMMARY</strong></th>
<th><strong>Wall height</strong></th>
<th>3 m</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>OUTER LEAF</strong></td>
<td>Thickness</td>
<td>102.5 mm</td>
</tr>
<tr>
<td></td>
<td>Char. flex strength</td>
<td>0.4 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Char. comp strength</td>
<td>5 N/mm²</td>
</tr>
<tr>
<td><strong>INNER LEAF</strong></td>
<td>Thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td></td>
<td>Char flex strength</td>
<td>0.22333 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Char. comp strength</td>
<td>5.3388 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Effective wall thickness</td>
<td>161.67 mm</td>
</tr>
<tr>
<td></td>
<td>Effective wall height</td>
<td>2.25 m</td>
</tr>
<tr>
<td></td>
<td>Slenderness ratio</td>
<td>13.918</td>
</tr>
<tr>
<td></td>
<td>Max vertical design load</td>
<td>134.81 kN</td>
</tr>
<tr>
<td></td>
<td>Vertical load resistance</td>
<td>214.98 kN</td>
</tr>
<tr>
<td></td>
<td>Vertical load (max ecc)</td>
<td>90.37 kN</td>
</tr>
<tr>
<td></td>
<td>Vertical load resistance</td>
<td>214.98 kN</td>
</tr>
<tr>
<td></td>
<td>Wind pressure</td>
<td>0.66667 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Elastic design moment</td>
<td>0.7 kN</td>
</tr>
<tr>
<td></td>
<td>Moment of resistance</td>
<td>2.7727 kNm</td>
</tr>
</tbody>
</table>
External cavity wall

This calculation is in accordance with EC6, the structural Eurocode for design of masonry structures Part 1-1:2005 General rules for reinforced & unreinforced masonry structures.

Wk = wind load or wind suction

Thickness of outer leaf $t(1)=100$ mm
Density of outer leaf $\text{den}(1)=12.4$ kN/m³
Thickness of inner leaf $t(2)=140$ mm
Density of inner leaf $\text{den}(2)=12.4$ kN/m³
between lateral restraints $h=2.425$ m
Support restraint factor $rf=0.75$
Length of wall panel $L=4$ m
Mortar designation (Table NA.2) $\text{mortar}=3$
Material partial safety factor $\text{gamM}=3$
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=1
Compressive strength of units $p_0(1)=21$ N/mm²
Water absorption %age of leaf $\text{water}=9$
Constant K from NA Table NA.4 $K(1)=0.5$
Height of masonry unit $h_u(1)=65$ mm
Least horiz. dimension of units $l_{hd}(1)=100$ mm
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(2)=4
Compressive strength of units $p_0(2)=3.6$ N/mm²
Constant K from NA Table NA.4 $K(2)=0.55$
Height of masonry unit $h_u(2)=215$ mm
Least horiz. dimension of units $l_{hd}(2)=140$ mm
Permanen load from upper levels $Gk(1)=4$ kN/m
Permanen load from upper levels $Gk(2)=7$ kN/m
Variable load from upper levels $Qk(2)=5.5$ kN/m
Permanen load from floor $Gk(3)=12$ kN/m
Variable load from floor $Qk(3)=5$ kN/m
Ecc of floor loads on inner leaf $\text{ecc}=23,333$ mm
Char. horizontal wind load $Wk=0.62$ kN/m²

Perm actions (upper levels) $\text{gamG2}=1.35$
Perm actions (floor) $\text{gamG3}=1.35$
Variable actions (vertical load) $\text{gamQ}=1.5$
Variable actions (horiz.wind) $\text{gamQw}=0.75$
Design moment at mid-height \( M_{md} = 0.61938 \text{ kNm/m} \)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>2.425 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTER LEAF</td>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td></td>
<td>Char. flexural strength</td>
<td>0.4 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Char. compressive strength</td>
<td>5.6982 N/mm²</td>
</tr>
<tr>
<td>INNER LEAF</td>
<td>Thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td></td>
<td>Char flexural strength</td>
<td>0.22333 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Char. compressive strength</td>
<td>2.4556 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Effective wall thickness</td>
<td>155.2 mm</td>
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<tr>
<td></td>
<td>Effective wall height</td>
<td>1.8188 m</td>
</tr>
<tr>
<td></td>
<td>Slenderness ratio</td>
<td>11.719</td>
</tr>
<tr>
<td></td>
<td>Design vertical load</td>
<td>44.242 kN</td>
</tr>
<tr>
<td></td>
<td>Design vert load resistance</td>
<td>60.473 kN</td>
</tr>
<tr>
<td></td>
<td>Char. horizontal wind load</td>
<td>0.62 kN/m²</td>
</tr>
<tr>
<td>Outer leaf - use Group 1 units (i.e. solid)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inner leaf - use Group 1 units (i.e. solid)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

**External cavity wall**

This calculation is in accordance with EC6, the structural Eurocode for design of masonry structures Part 1-1:2005 General rules for reinforced & unreinforced masonry structures.

\[ W_k = \text{wind load or wind suction} \]

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of outer leaf</td>
<td>t(1) = 100 mm</td>
<td></td>
</tr>
<tr>
<td>Density of outer leaf</td>
<td>( \text{den}(1) = 12.4 \text{ kN/m}^3 )</td>
<td></td>
</tr>
<tr>
<td>Thickness of inner leaf</td>
<td>t(2) = 140 mm</td>
<td></td>
</tr>
<tr>
<td>Density of inner leaf</td>
<td>( \text{den}(2) = 12.4 \text{ kN/m}^3 )</td>
<td></td>
</tr>
<tr>
<td>between lateral restraints</td>
<td>h = 2.425 m</td>
<td></td>
</tr>
<tr>
<td>Support restraint factor</td>
<td>rf = 0.75</td>
<td></td>
</tr>
<tr>
<td>Length of wall panel</td>
<td>L = 4 m</td>
<td></td>
</tr>
<tr>
<td>Mortar designation (Table NA.2)</td>
<td>mortar = 3</td>
<td></td>
</tr>
<tr>
<td>Material partial safety factor</td>
<td>( \text{gam}_M ) = 3</td>
<td></td>
</tr>
<tr>
<td>Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1) = 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength of units</td>
<td>( \text{po}(1) = 60 \text{ N/mm}^2 )</td>
<td></td>
</tr>
<tr>
<td>Constant K from NA Table NA.4</td>
<td>K(1) = 0.5</td>
<td></td>
</tr>
<tr>
<td>Height of masonry unit</td>
<td>( \text{hu}(1) = 65 \text{ mm} )</td>
<td></td>
</tr>
<tr>
<td>Least horiz. dimension of units</td>
<td>( \text{lhd}(1) = 100 \text{ mm} )</td>
<td></td>
</tr>
<tr>
<td>Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(2) = 3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength of units</td>
<td>( \text{po}(2) = 5.2 \text{ N/mm}^2 )</td>
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</tr>
<tr>
<td>Constant K from NA Table NA.4</td>
<td>K(2) = 0.75</td>
<td></td>
</tr>
<tr>
<td>Height of masonry unit</td>
<td>( \text{hu}(2) = 65 \text{ mm} )</td>
<td></td>
</tr>
<tr>
<td>Least horiz. dimension of units</td>
<td>( \text{lhd}(2) = 100 \text{ mm} )</td>
<td></td>
</tr>
<tr>
<td>Permanent load from upper levels</td>
<td>( G_k(1) = 4 \text{ kN/m} )</td>
<td></td>
</tr>
<tr>
<td>Permanent load from upper levels</td>
<td>( G_k(2) = 5 \text{ kN/m} )</td>
<td></td>
</tr>
<tr>
<td>Variable load from upper levels</td>
<td>( Q_k(1) = 5.5 \text{ kN/m} )</td>
<td></td>
</tr>
<tr>
<td>Permanent load from floor</td>
<td>( G_k(3) = 10 \text{ kN/m} )</td>
<td></td>
</tr>
<tr>
<td>Variable load from floor</td>
<td>( Q_k(3) = 5 \text{ kN/m} )</td>
<td></td>
</tr>
<tr>
<td>Ecc of floor loads on inner leaf</td>
<td>ecc = 23.333 mm</td>
<td></td>
</tr>
<tr>
<td>Char. horizontal wind load</td>
<td>( W_k = 0.63 \text{ kN/m}^2 )</td>
<td></td>
</tr>
<tr>
<td>Perm actions (upper levels)</td>
<td>( \text{gamG}_2 = 1.35 )</td>
<td></td>
</tr>
<tr>
<td>Perm actions (floor)</td>
<td>( \text{gamG}_3 = 1.35 )</td>
<td></td>
</tr>
<tr>
<td>Variable actions (vertical load)</td>
<td>( \text{gamQ} = 1.5 )</td>
<td></td>
</tr>
<tr>
<td>Variable actions (horiz.wind)</td>
<td>( \text{gamQ}_w = 0.75 )</td>
<td></td>
</tr>
</tbody>
</table>
### Design moment at mid-height

Mmd = 0 kNm/m

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>2.425 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTER LEAF</td>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char. flexural strength</td>
<td>0.3 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>Char. compressive strength</td>
<td>11.882 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>INNER LEAF</td>
<td>Thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td>Char flexural strength</td>
<td>0.3 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>Char. compressive strength</td>
<td>3.2172 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>Effective wall thickness</td>
<td>155.2 mm</td>
<td></td>
</tr>
<tr>
<td>Effective wall height</td>
<td>1.8188 m</td>
<td></td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td>11.719</td>
<td></td>
</tr>
<tr>
<td>Design vertical load</td>
<td>38.842 kN</td>
<td></td>
</tr>
<tr>
<td>Design vert load resistance</td>
<td>98.479 kN</td>
<td></td>
</tr>
<tr>
<td>Char. horizontal wind load</td>
<td>0.63 kN/m$^2$</td>
<td></td>
</tr>
<tr>
<td>Outer leaf - use Group 1 units (i.e. solid)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inner leaf - use Group 1 units (i.e. solid)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

External cavity wall

This calculation is in accordance with EC6, the structural Eurocode for design of masonry structures Part 1-1:2005 General rules for reinforced & unreinforced masonry structures.

Wk = wind load or wind suction

<table>
<thead>
<tr>
<th>Thickness of outer leaf</th>
<th>t(1) = 100 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density of outer leaf</td>
<td>den(1) = 12.4 kN/m³</td>
</tr>
<tr>
<td>Thickness of inner leaf</td>
<td>t(2) = 140 mm</td>
</tr>
<tr>
<td>Density of inner leaf</td>
<td>den(2) = 12.4 kN/m³</td>
</tr>
<tr>
<td>between lateral restraints</td>
<td>h = 4.25 m</td>
</tr>
<tr>
<td>Support restraint factor</td>
<td>rf = 0.75</td>
</tr>
<tr>
<td>Length of wall panel</td>
<td>L = 4 m</td>
</tr>
<tr>
<td>Mortar designation (Table NA.2)</td>
<td>mortar = 3</td>
</tr>
<tr>
<td>Material partial safety factor</td>
<td>γM = 3</td>
</tr>
<tr>
<td>Masonry type (1=clay 2=cal silicate 3=conc brick 4=block)</td>
<td>typ(1) = 1</td>
</tr>
<tr>
<td>Compressive strength of units</td>
<td>po(1) = 20 N/mm²</td>
</tr>
<tr>
<td>Water absorption %age of leaf</td>
<td>water = 9</td>
</tr>
<tr>
<td>Constant K from NA Table NA.4</td>
<td>K(1) = 0.5</td>
</tr>
<tr>
<td>Height of masonry unit</td>
<td>hu(1) = 65 mm</td>
</tr>
<tr>
<td>Least horiz. dimension of units</td>
<td>lhd(1) = 100 mm</td>
</tr>
<tr>
<td>Masonry type (1=clay 2=cal silicate 3=conc brick 4=block)</td>
<td>typ(2) = 4</td>
</tr>
<tr>
<td>Compressive strength of units</td>
<td>po(2) = 22.5 N/mm²</td>
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<tr>
<td>Constant K from NA Table NA.4</td>
<td>K(2) = 0.75</td>
</tr>
<tr>
<td>Height of masonry unit</td>
<td>hu(2) = 215 mm</td>
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<tr>
<td>Least horiz. dimension of units</td>
<td>lhd(2) = 140 mm</td>
</tr>
<tr>
<td>Permanent load from upper levels</td>
<td>Gk(1) = 4 kN/m</td>
</tr>
<tr>
<td>Permanent load from upper levels</td>
<td>Gk(2) = 7 kN/m</td>
</tr>
<tr>
<td>Variable load from upper levels</td>
<td>Qk(2) = 5.5 kN/m</td>
</tr>
<tr>
<td>Permanent load from floor</td>
<td>Gk(3) = 12 kN/m</td>
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<tr>
<td>Variable load from floor</td>
<td>Qk(3) = 5 kN/m</td>
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<tr>
<td>Ecc of floor loads on inner leaf</td>
<td>ecc = 23.333 mm</td>
</tr>
<tr>
<td>Char. horizontal wind load</td>
<td>Wk = 0.25 kN/m²</td>
</tr>
<tr>
<td>Perm actions (upper levels)</td>
<td>γG2 = 1.35</td>
</tr>
<tr>
<td>Perm actions (floor)</td>
<td>γG3 = 1.35</td>
</tr>
<tr>
<td>Variable actions (vertical load)</td>
<td>γQ = 1.5</td>
</tr>
<tr>
<td>Variable actions (horiz.wind)</td>
<td>γQw = 0.75</td>
</tr>
</tbody>
</table>
Design moment at mid-height  
\[ M_{md} = 0.64931 \text{ kNm/m} \]

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>4.25 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTER LEAF</td>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char. flexural strength</td>
<td>0.4 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Char. compressive strength</td>
<td>5.5068 N/mm²</td>
<td></td>
</tr>
<tr>
<td>INNER LEAF</td>
<td>Thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td>Char flexural strength</td>
<td>0.25 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Char. compressive strength</td>
<td>12.077 N/mm²</td>
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</tr>
<tr>
<td>Effective wall thickness</td>
<td>155.2 mm</td>
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<tr>
<td>Effective wall height</td>
<td>3.1875 m</td>
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<tr>
<td>Slenderness ratio</td>
<td>20.538</td>
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</tr>
<tr>
<td>Design vertical load</td>
<td>46.38 kN</td>
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<tr>
<td>Design vert load resistance</td>
<td>149.52 kN</td>
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</tr>
<tr>
<td>Char. horizontal wind load</td>
<td>0.25 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Outer leaf - use Group 1 units (i.e. solid)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inner leaf - use Group 1 units (i.e. solid)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

External cavity wall

This calculation is in accordance with EC6, the structural Eurocode for design of masonry structures Part 1-1:2005 General rules for reinforced & unreinforced masonry structures.

\[ W_k = \text{wind load or wind suction} \]

Thickness of outer leaf \( t(1) = 102 \text{ mm} \)
Density of outer leaf \( \text{den}(1) = 22 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2) = 102 \text{ mm} \)
Density of inner leaf \( \text{den}(2) = 22 \text{ kN/m}^3 \)
between lateral restraints \( h = 2.425 \text{ m} \)
Support restraint factor \( rf = 0.75 \)
Length of wall panel \( L = 4 \text{ m} \)
Mortar designation (Table NA.2) \( \text{mortar} = 3 \)
Material partial safety factor \( \text{gamM} = 3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 1 \)
Compressive strength of units \( \text{po}(1) = 20 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{water} = 9 \%
Constant K from NA Table NA.4 \( K(1) = 0.5 \)
Height of masonry unit \( hu(1) = 65 \text{ mm} \)
Least horiz. dimension of units \( lhd(1) = 100 \text{ mm} \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(2) = 1 \)
Compressive strength of units \( \text{po}(2) = 30 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{water} = 9 \%
Constant K from NA Table NA.4 \( K(2) = 0.5 \)
Height of masonry unit \( hu(2) = 65 \text{ mm} \)
Least horiz. dimension of units \( lhd(2) = 100 \text{ mm} \)
Permanent load from upper levels \( G_k(1) = 4 \text{ kN/m} \)
Permanent load from upper levels \( G_k(2) = 7 \text{ kN/m} \)
Variable load from upper levels \( Q_k(2) = 5.5 \text{ kN/m} \)
Permanent load from floor \( G_k(3) = 12 \text{ kN/m} \)
Variable load from floor \( Q_k(3) = 5 \text{ kN/m} \)
Ecc of floor loads on inner leaf \( \text{ecc} = 17 \text{ mm} \)
Char. horizontal wind load \( W_k = 0.63 \text{ kN/m}^2 \)
Perm actions (upper levels) \( \text{gamG2} = 1.35 \)
Perm actions (floor) \( \text{gamG3} = 1.35 \)
Variable actions (vertical load) \( \text{gamQ} = 1.5 \)
Variable actions (horiz.wind) \( \text{gamQw} = 0.75 \)
Design moment at mid-height \( M_{md} = 0.45974 \text{ kNm/m} \)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>2.425 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTER LEAF</td>
<td>Thickness</td>
<td>102 mm</td>
</tr>
<tr>
<td></td>
<td>Char. flexural strength</td>
<td>0.4 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Char. compressive strength</td>
<td>5.5068 N/mm²</td>
</tr>
<tr>
<td>INNER LEAF</td>
<td>Thickness</td>
<td>102 mm</td>
</tr>
<tr>
<td></td>
<td>Char flexural strength</td>
<td>0.4 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Char. compressive strength</td>
<td>7.3142 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Effective wall thickness</td>
<td>128.45 mm</td>
</tr>
<tr>
<td></td>
<td>Effective wall height</td>
<td>1.8188 m</td>
</tr>
<tr>
<td></td>
<td>Slenderness ratio</td>
<td>14.159</td>
</tr>
</tbody>
</table>

Design vertical load \( 45.073 \text{ kN} \)
Design vert load resistance \( 102.17 \text{ kN} \)
Char. horizontal wind load \( 0.63 \text{ kN/m}² \)
Outer leaf - use Group 1 units (i.e. solid)
Inner leaf - use Group 1 units (i.e. solid)
Location: Example 5 (reference V6 by JR)

External cavity wall

This calculation is in accordance with EC6, the structural Eurocode for design of masonry structures Part 1-1:2005 General rules for reinforced & unreinforced masonry structures.

\[ W_k = \text{wind load or wind suction} \]

Thickness of outer leaf \( t(1) = 102.5 \, \text{mm} \)
Density of outer leaf \( \text{den}(1) = 20 \, \text{kN/m}^3 \)
Thickness of inner leaf \( t(2) = 140 \, \text{mm} \)
Density of inner leaf \( \text{den}(2) = 20 \, \text{kN/m}^3 \)
between lateral restraints \( h = 3 \, \text{m} \)
Support restraint factor \( rf = 0.75 \)
Length of wall panel \( L = 4 \, \text{m} \)
Mortar designation (Table NA.2) \( \text{mortar} = 3 \)
Material partial safety factor \( \text{gamM} = 2.3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 1 \)
Compressive strength of units \( p_o(1) = 20 \, \text{N/mm}^2 \)
Water absorption %age of leaf \( \text{water} = 12 \)
Constant K from NA Table NA.4 \( K(1) = 0.5 \)
Height of masonry unit \( h_u(1) = 65 \, \text{mm} \)
Least horiz. dimension of units \( lhd(1) = 100 \, \text{mm} \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(2) = 4 \)
Compressive strength of units \( p_o(2) = 7.3 \, \text{N/mm}^2 \)
Constant K from NA Table NA.4 \( K(2) = 0.75 \)
Height of masonry unit \( h_u(2) = 215 \, \text{mm} \)
Least horiz. dimension of units \( lhd(2) = 140 \, \text{mm} \)
Permanent load from upper levels \( G_k(1) = 10 \, \text{kN/m} \)
Permanent load from upper levels \( G_k(2) = 88.889 \, \text{kN/m} \)
Variable load from upper levels \( Q_k(2) = 0 \, \text{kN/m} \)
Permanent load from floor \( G_k(3) = 7.4074 \, \text{kN/m} \)
Variable load from floor \( Q_k(3) = 0 \, \text{kN/m} \)
Ecc of floor loads on inner leaf \( \text{ecc} = 23.333 \, \text{mm} \)
Char. horizontal wind load \( W_k = 0.66667 \, \text{kN/m}^2 \)

Perm actions (upper levels) \( \text{gamG2} = 1.35 \)
Perm actions (floor) \( \text{gamG3} = 1.35 \)
Variable actions (vertical load) \( \text{gamQ} = 1.5 \)
Variable actions (horiz.wind) \( \text{gamQw} = 0.75 \)
Design moment at mid-height  $M_{md}=0 \text{kNm/m}$

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>3 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTER LEAF</td>
<td>Thickness</td>
<td>102.5 mm</td>
</tr>
<tr>
<td></td>
<td>Char. flexural strength</td>
<td>0.4 N/mm$^2$</td>
</tr>
<tr>
<td></td>
<td>Char. compressive strength</td>
<td>5.5068 N/mm$^2$</td>
</tr>
<tr>
<td>INNER LEAF</td>
<td>Thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td></td>
<td>Char flexural strength</td>
<td>0.22333 N/mm$^2$</td>
</tr>
<tr>
<td></td>
<td>Char. compressive strength</td>
<td>5.4924 N/mm$^2$</td>
</tr>
<tr>
<td></td>
<td>Effective wall thickness</td>
<td>156.26 mm</td>
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<td></td>
<td>Effective wall height</td>
<td>2.25 m</td>
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<tr>
<td></td>
<td>Slenderness ratio</td>
<td>14.399</td>
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<tr>
<td></td>
<td>Design vertical load</td>
<td>135.67 kN</td>
</tr>
<tr>
<td></td>
<td>Design vert load resistance</td>
<td>253.39 kN</td>
</tr>
<tr>
<td></td>
<td>Char. horizontal wind load</td>
<td>0.66667 kN/m$^2$</td>
</tr>
<tr>
<td></td>
<td>Outer leaf - use Group 1 units (i.e. solid)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Inner leaf - use Group 1 units (i.e. solid)</td>
<td></td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

**External cavity wall**

This calculation is in accordance with BS5628-1:2005 the Code of practice for use of masonry. Part 1: Structural use of unreinforced masonry.

The form of this calculation is in accordance with Clause 32.8.

Thickness of outer leaf \( t(1)=100 \text{ mm} \)
Density of outer leaf \( \text{den}(1)=12.4 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2)=130 \text{ mm} \)
Density of inner leaf \( \text{den}(2)=12.4 \text{ kN/m}^3 \)
Length of wall panel \( l=3.5 \text{ m} \)
Clear height of wall \( h=2.425 \text{ m} \)
Support Restraint factor \( rf=0.75 \)
Mortar designation to Table 1 \( \text{mortar}=3 \)
Partial Safety Factor Material \( \text{gammam}=3.1 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1)=1 \)
Compressive Strength of Units \( p_{o}(1)=20 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{wat}(1)=9 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(2)=4 \)
Compressive Strength of Units \( p_{o}(2)=3.6 \text{ N/mm}^2 \)
Height \( h_{b}(2)=215 \text{ mm} \)
Least horizontal dimension \( lhd(2)=130 \text{ mm} \)
Dead load from upper levels \( DL(1)=4 \text{ kN/m} \)
Dead load from upper levels \( DL(2)=7 \text{ kN/m} \)
Live load from upper levels \( LL(2)=5.5 \text{ kN/m} \)
Dead load from supported floor \( DL(3)=12 \text{ kN/m} \)
Live load from supported floor \( LL(3)=5 \text{ kN/m} \)
Ecc. of floor loads on inner leaf \( ecc=17 \text{ mm} \)
Number of returns: nr=1

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>2.425 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTER LEAF</td>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td></td>
<td>Char flex strength</td>
<td>0.4 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Char comp strength</td>
<td>5 N/mm²</td>
</tr>
<tr>
<td>INNER LEAF</td>
<td>Thickness</td>
<td>130 mm</td>
</tr>
<tr>
<td></td>
<td>Char flex strength</td>
<td>0.23 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Char comp strength</td>
<td>3.0549 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Effective wall thickness</td>
<td>153.33 mm</td>
</tr>
<tr>
<td></td>
<td>Effective wall height</td>
<td>1.8188 m</td>
</tr>
<tr>
<td></td>
<td>Slenderness ratio</td>
<td>11.861</td>
</tr>
<tr>
<td></td>
<td>Max vertical design load</td>
<td>43.4 kN</td>
</tr>
<tr>
<td></td>
<td>Vertical load resistance</td>
<td>115.57 kN</td>
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<tr>
<td></td>
<td>Vertical load (max ecc)</td>
<td>31.1 kN</td>
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<td>Vertical load resistance</td>
<td>110.39 kN</td>
</tr>
<tr>
<td></td>
<td>Lateral capacity (normal)</td>
<td>2.2222 kN/m²</td>
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<tr>
<td></td>
<td>Lateral capacity (accident)</td>
<td>2.5409 kN/m²</td>
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<tr>
<td></td>
<td>Lateral capacity (protect)</td>
<td>4.8278 kN/m²</td>
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</table>
Location: Wall on grid line A/1-2

This calculation is in accordance with BS5628-1:2005 the Code of practice for use of masonry. Part 1: Structural use of unreinforced masonry.

The form of this calculation is in accordance with Clause 32.8.

Thickness of outer leaf \( t(1) = 75 \text{ mm} \)
Density of outer leaf \( \text{den}(1) = 12.4 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2) = 450 \text{ mm} \)
Density of inner leaf \( \text{den}(2) = 12.4 \text{ kN/m}^3 \)
Length of wall panel \( l = 4 \text{ m} \)
Clear height of wall \( h = 2.425 \text{ m} \)
Support Restraint factor \( rf = 0.9 \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Partial Safety Factor Material \( \gamma_{ma} = 3.1 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 2 \)
Compressive Strength of Units \( \text{po}(1) = 60 \text{ N/mm}^2 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(2) = 3 \)
Compressive Strength of Units \( \text{po}(2) = 5.2 \text{ N/mm}^2 \)
Dead load from upper levels \( \text{DL}(1) = 4 \text{ kN/m} \)
Dead load from upper levels \( \text{DL}(2) = 5 \text{ kN/m} \)
Live load from upper levels \( \text{LL}(1) = 5.5 \text{ kN/m} \)
Dead load from supported floor \( \text{DL}(3) = 10 \text{ kN/m} \)
Live load from supported floor \( \text{LL}(3) = 5 \text{ kN/m} \)
Ecc. of floor loads on inner leaf \( \text{ecc} = 17 \text{ mm} \)

Number of returns \( nr = 0 \)
| DESIGN SUMMARY | Wall height         | 2.425 m |
|               | OUTER LEAF          |         |
|               | Thickness           | 75 mm   |
|               | Char flex strength  | 0.3 N/mm² |
|               | Char comp strength  | 9.24 N/mm² |
|               | INNER LEAF          |         |
|               | Thickness           | 450 mm  |
|               | Char flex strength  | 0.3 N/mm² |
|               | Char comp strength  | 2.5 N/mm² |
|               | Effective wall thickness | 450 mm |
|               | Effective wall height | 2.1825 m  |
|               | Slenderness ratio   | 4.85    |
|               | Max vertical design load | 37.8 kN  |
|               | Vertical load resistance | 362.9 kN  |
|               | Vertical load (max ecc) | 26.5 kN  |
|               | Vertical load resistance | 362.9 kN  |
|               | Lateral capacity (normal) | 0 kN/m²  |
|               | Lateral capacity (accident) | 0 kN/m²  |
|               | Lateral capacity (protect) | 0 kN/m²  |
Location: Wall on grid line A/1-2

**External cavity wall**

This calculation is in accordance with BS5628-1:2005 the Code of practice for use of masonry. Part 1: Structural use of unreinforced masonry.

The form of this calculation is in accordance with Clause 32.8.

Thickness of outer leaf \( t(1) = 100 \text{ mm} \)
Density of outer leaf \( \text{den}(1) = 12.4 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2) = 130 \text{ mm} \)
Density of inner leaf \( \text{den}(2) = 12.4 \text{ kN/m}^3 \)
Length of wall panel \( l = 4 \text{ m} \)
Clear height of wall \( h = 2.5 \text{ m} \)
Support Restraint factor \( rf = 0.75 \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Partial Safety Factor Material \( \gamma_m = 3.1 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 1 \)
Compressive Strength of Units \( \text{po}(1) = 20 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{wat}(1) = 9 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(2) = 4 \)
Compressive Strength of Units \( \text{po}(2) = 3.6 \text{ N/mm}^2 \)
Height \( h_b(2) = 215 \text{ mm} \)
Least horizontal dimension \( l_{hd}(2) = 130 \text{ mm} \)
Dead load from upper levels \( DL(1) = 4 \text{ kN/m} \)
Live load from upper levels \( DL(2) = 7 \text{ kN/m} \)
Dead load from supported floor \( DL(3) = 12 \text{ kN/m} \)
Live load from supported floor \( LL(3) = 5 \text{ kN/m} \)
Ecc. of floor loads on inner leaf \( ecc = 17 \text{ mm} \)
Number of returns \( nr = 2 \)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>2.5 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTER LEAF</td>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char flex strength</td>
<td>0.4 N/mm(^2)</td>
<td></td>
</tr>
<tr>
<td>Char comp strength</td>
<td>5 N/mm(^2)</td>
<td></td>
</tr>
<tr>
<td>INNER LEAF</td>
<td>Thickness</td>
<td>130 mm</td>
</tr>
<tr>
<td>Char flex strength</td>
<td>0.23 N/mm(^2)</td>
<td></td>
</tr>
<tr>
<td>Char comp strength</td>
<td>3.0549 N/mm(^2)</td>
<td></td>
</tr>
<tr>
<td>Effective wall thickness</td>
<td>153.33 mm</td>
<td></td>
</tr>
<tr>
<td>Effective wall height</td>
<td>1.875 m</td>
<td></td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td>12.228</td>
<td></td>
</tr>
<tr>
<td>Max vertical design load</td>
<td>43.4 kN</td>
<td></td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td>114.76 kN</td>
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</tr>
<tr>
<td>Vertical load (max ecc)</td>
<td>31.1 kN</td>
<td></td>
</tr>
<tr>
<td>Vertical load resistance</td>
<td>109.75 kN</td>
<td></td>
</tr>
<tr>
<td>Lateral capacity (normal)</td>
<td>3.3197 kN/m(^2)</td>
<td></td>
</tr>
<tr>
<td>Lateral capacity (accident)</td>
<td>3.7958 kN/m(^2)</td>
<td></td>
</tr>
<tr>
<td>Lateral capacity (protect)</td>
<td>7.212 kN/m(^2)</td>
<td></td>
</tr>
</tbody>
</table>
External cavity wall

This calculation is in accordance with EC6 Part 1-1: General rules for reinforced and unreinforced masonry structures.

The form of this calculation is in accordance with section 6 of the code of practice.

Thickness of outer leaf \( t(1) = 100 \text{ mm} \)
Density of outer leaf \( \text{den}(1) = 12.4 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2) = 130 \text{ mm} \)
Density of inner leaf \( \text{den}(2) = 12.4 \text{ kN/m}^3 \)
Length of wall panel \( l = 3.5 \text{ m} \)
Clear height of wall \( h = 2.425 \text{ m} \)
Support restraint factor \( \text{rf} = 0.75 \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Partial safety factor (material) \( \text{gam}M = 2.7 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 1 \)
Compressive strength of units \( \text{po}(1) = 20 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{wat}(1) = 9 \)
Constant K from NA Table NA.4 \( K(1) = 0.5 \)
Height of masonry unit \( \text{hu}(1) = 65 \text{ mm} \)
Least horiz. dimension of units \( \text{lhd}(1) = 100 \text{ mm} \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(2) = 4 \)
Compressive strength of units \( \text{po}(2) = 3.6 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(2) = 0.75 \)
Height of masonry unit \( \text{hu}(2) = 215 \text{ mm} \)
Least horiz. dimension of units \( \text{lhd}(2) = 130 \text{ mm} \)
Permanent load from upper levels \( \text{Gk}(1) = 4 \text{ kN/m} \)
Variable load from upper levels \( \text{Qk}(2) = 5.5 \text{ kN/m} \)
Permanent load from floor \( \text{Gk}(3) = 12 \text{ kN/m} \)
Variable load from floor \( \text{Qk}(3) = 5 \text{ kN/m} \)
Ecc. of floor load on inner leaf \( \text{ecc} = 21.667 \text{ mm} \)
Perm actions (upper levels) \( \text{gamG2} = 1.35 \)
Perm actions (floor) \( \text{gamG3} = 1.35 \)
Variable actions (vertical load) \( \text{gamQ} = 1.5 \)
horizontal loads (e.g. wind load) $\text{ehe}=0 \text{ mm}$
loads (e.g. wind load) $\text{ehm}=0 \text{ mm}$

Design moment at mid-height $\text{Mmd}=0.57252 \text{ kNm/m}$
Leading variable load factor $\psi_1=0.5$
Other variable load factor $\psi_2=0.3$

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>2.425 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTER LEAF</td>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char flex strength</td>
<td>0.4 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>Char comp strength</td>
<td>5.5068 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>INNER LEAF</td>
<td>Thickness</td>
<td>130 mm</td>
</tr>
<tr>
<td>Char flex strength</td>
<td>0.23 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>Char comp strength</td>
<td>3.3845 N/mm$^2$</td>
<td></td>
</tr>
</tbody>
</table>

Effective wall thickness 147.24 mm
Effective wall height 1.8188 m
Slenderness ratio 12.352

Design vertical load 41.4 kN
Design vert.load resistance 102.18 kN

Lateral capacity (normal) 3.6024 kN/m$^2$
Lateral capacity (accident) 3.6024 kN/m$^2$
Lateral capacity (key elem) 7.2048 kN/m$^2$

Outer leaf - use Group 1 units (i.e. solid)
Inner leaf - use Group 1 units (i.e. solid)
**Location:** Wall on grid line A/1-2

**External cavity wall**

This calculation is in accordance with EC6 Part 1-1: General rules for reinforced and unreinforced masonry structures.

The form of this calculation is in accordance with section 6 of the code of practice.

- Thickness of outer leaf: \( t(1) = 75 \text{ mm} \)
- Density of outer leaf: \( \text{den}(1) = 12.4 \text{ kN/m}^3 \)
- Thickness of inner leaf: \( t(2) = 450 \text{ mm} \)
- Density of inner leaf: \( \text{den}(2) = 12.4 \text{ kN/m}^3 \)
- Length of wall panel: \( l = 4 \text{ m} \)
- Clear height of wall: \( h = 2.425 \text{ m} \)
- Support restraint factor: \( rf = 0.9 \)
- Mortar designation to Table 1: \( \text{mortar} = 3 \)
- Partial safety factor (material): \( \text{gamM} = 2.7 \)
- Masonry type (1=clay 2=cal silicate 3=conc brick 4=block): \( \text{typ}(1) = 2 \)
- Compressive strength of units: \( \text{po}(1) = 60 \text{ N/mm}^2 \)
- Constant K from NA Table NA.4: \( K(1) = 0.5 \)
- Height of masonry unit: \( \text{hu}(1) = 65 \text{ mm} \)
- Least horiz. dimension of units: \( \text{ld}(1) = 100 \text{ mm} \)
- Masonry type (1=clay 2=cal silicate 3=conc brick 4=block): \( \text{typ}(2) = 3 \)
- Compressive strength of units: \( \text{po}(2) = 5.2 \text{ N/mm}^2 \)
- Constant K from NA Table NA.4: \( K(2) = 0.75 \)
- Height of masonry unit: \( \text{hu}(2) = 65 \text{ mm} \)
- Least horiz. dimension of units: \( \text{ld}(2) = 100 \text{ mm} \)
- Permanent load from upper levels: \( Gk(1) = 4 \text{ kN/m} \)
- Variable load from upper levels: \( Qk(2) = 5.5 \text{ kN/m} \)
- Permanent load from floor: \( Gk(3) = 10 \text{ kN/m} \)
- Variable load from floor: \( Qk(3) = 5 \text{ kN/m} \)
- Ecc. of floor load on inner leaf: \( \text{ecc} = 75 \text{ mm} \)
- Perm actions (upper levels): \( \text{gamG2} = 1.35 \)
- Perm actions (floor): \( \text{gamG3} = 1.35 \)
- Variable actions (vertical load): \( \text{gamQ} = 1.5 \)
horizontal loads (e.g. wind load) \( e_{he}=0 \text{ mm} \)
loads (e.g. wind load) \( e_{hm}=0 \text{ mm} \)

Design moment at mid-height \( M_{md}=2.031 \text{ kNm/m} \)
Leading variable load factor \( \psi_1=0.5 \)
Other variable load factor \( \psi_2=0.3 \)

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>2.425 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTER LEAF</td>
<td>Thickness</td>
<td>75 mm</td>
</tr>
<tr>
<td></td>
<td>Char flex strength</td>
<td>0.3 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Char comp strength</td>
<td>11.882 N/mm²</td>
</tr>
<tr>
<td>INNER LEAF</td>
<td>Thickness</td>
<td>450 mm</td>
</tr>
<tr>
<td></td>
<td>Char flex strength</td>
<td>0.3 N/mm²</td>
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<tr>
<td></td>
<td>Char comp strength</td>
<td>3.2172 N/mm²</td>
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<td>Effective wall thickness</td>
<td>450.42 mm</td>
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<td>Effective wall height</td>
<td>2.1825 m</td>
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<td></td>
<td>Slenderness ratio</td>
<td>4.8455</td>
</tr>
<tr>
<td></td>
<td>Design vertical load</td>
<td>36 kN</td>
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<tr>
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<td>Design vert.load resistance</td>
<td>412.7 kN</td>
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<tr>
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<td></td>
<td>Lateral capacity (accident)</td>
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<tr>
<td></td>
<td>Lateral capacity (key elem)</td>
<td>0 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Outer leaf - use Group 1 units (i.e. solid)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Inner leaf - use Group 1 units (i.e. solid)</td>
<td></td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

External cavity wall

This calculation is in accordance with EC6 Part 1-1: General rules for reinforced and unreinforced masonry structures.

The form of this calculation is in accordance with section 6 of the code of practice.

Thickness of outer leaf \( t(1) = 100 \text{ mm} \)
Density of outer leaf \( \text{den}(1) = 12.4 \text{ kN/m}^3 \)
Thickness of inner leaf \( t(2) = 130 \text{ mm} \)
Density of inner leaf \( \text{den}(2) = 12.4 \text{ kN/m}^3 \)
Length of wall panel \( l = 4 \text{ m} \)
Clear height of wall \( h = 2.5 \text{ m} \)
Support restraint factor \( rf = 0.75 \)
Masonry designation to Table 1 \( \text{mortar}=3 \)
Partial safety factor (material) \( \text{gam}_M = 2.7 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1)=1 \)
Compressive strength of units \( po(1)=20 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{wat}(1)=9 \)
Constant K from NA Table NA.4 \( K(1)=0.5 \)
Height of masonry unit \( hu(1)=65 \text{ mm} \)
Least horiz. dimension of units \( lhd(1)=100 \text{ mm} \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(2)=4 \)
Compressive strength of units \( po(2)=3.6 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(2)=0.75 \)
Height of masonry unit \( hu(2)=215 \text{ mm} \)
Least horiz. dimension of units \( lhd(2)=130 \text{ mm} \)
Permanent load from upper levels \( Gk(1)=4 \text{ kN/m} \)
Permanent load from upper levels \( Gk(2)=7 \text{ kN/m} \)
Variable load from upper levels \( Qk(2)=5.5 \text{ kN/m} \)
Permanent load from floor \( Gk(3)=12 \text{ kN/m} \)
Variable load from floor \( Qk(3)=5 \text{ kN/m} \)
Ecc. of floor load on inner leaf \( \text{ecc}=21.667 \text{ mm} \)
Perm actions (upper levels) \( \text{gam}_G2=1.35 \)
Perm actions (floor) \( \text{gam}_G3=1.35 \)
Variable actions (vertical load) \( \text{gam}_Q=1.5 \)
horizontal loads (e.g. wind load) \( e_{he}=0 \text{ mm} \)
loads (e.g. wind load) \( e_{hm}=0 \text{ mm} \)

Design moment at mid-height \( M_{md}=0.57357 \text{ kNm/m} \)
Leading variable load factor \( \psi_1 \)=0.5
Other variable load factor \( \psi_2 \)=0.3

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
<th>Wall height</th>
<th>2.5 m</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>OUTER LEAF</strong></td>
<td>Thickness</td>
<td>100 mm</td>
</tr>
<tr>
<td>Char flex strength</td>
<td>0.4 N/mm(^2)</td>
<td></td>
</tr>
<tr>
<td>Char comp strength</td>
<td>5.5068 N/mm(^2)</td>
<td></td>
</tr>
<tr>
<td><strong>INNER LEAF</strong></td>
<td>Thickness</td>
<td>130 mm</td>
</tr>
<tr>
<td>Char flex strength</td>
<td>0.23 N/mm(^2)</td>
<td></td>
</tr>
<tr>
<td>Char comp strength</td>
<td>3.3845 N/mm(^2)</td>
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</tr>
<tr>
<td>Effective wall thickness</td>
<td>147.24 mm</td>
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</tr>
<tr>
<td>Effective wall height</td>
<td>1.875 m</td>
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<tr>
<td>Slenderness ratio</td>
<td>12.734</td>
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<tr>
<td>Design vertical load</td>
<td>41.4 kN</td>
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<td>Design vert.load resistance</td>
<td>100.61 kN</td>
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<td>Lateral capacity (normal)</td>
<td>3.3895 kN/m(^2)</td>
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<td>Lateral capacity (accident)</td>
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<td>Lateral capacity (key elem)</td>
<td>6.779 kN/m(^2)</td>
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<td>Outer leaf - use Group 1 units (i.e. solid)</td>
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</tr>
<tr>
<td>Inner leaf - use Group 1 units (i.e. solid)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Location: Wall on grid line A/1-2

Masonry design to BS5628, MEXE and Eurocode 6

Made by: IFB
Date: 02/12/19

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Ref No: SC535 BS

**Masonry shear wall**

These calculations are in accordance with BS5628-1:2005 the "Code of practice for the use of masonry".

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**Elevation on shear wall**

- Length of shear wall: $L = 0.7$ m
- Density of masonry: $dens = 12$ kN/m$^3$
- Thickness of wall: $t = 100$ mm
- Between lateral restraints: $h = 2.6$ m
- Support Restraint factor: $rf = 0.75$
- Mortar designation to Table 1: $mortar = 3$
- Masonry type (1=clay 2=cal silicate 3=conc brick 4=block): $typ(1) = 4$
- Compressive strength of units: $po = 10.4$ N/mm$^2$
- Height: $hb = 215$ mm
- Least horiz dimension: $lhd = 100$ mm
- Partial safety factor material: $gammam = 2.5$
- Characteristic dead load: $DL = 40$ kN
- Characteristic imposed load: $LL = 0$ kN
- Characteristic wind load: $WL = 2.05$ kN

**DESIGN SUMMARY**

- Wall height: $2.6$ m
- Wall length: $0.7$ m
- Wall thickness: $100$ mm
- Char comp strength: $4.7362$ N/mm$^2$
- Char flex strength: $0.25$ N/mm$^2$
- Design shear load: $2.87$ kN
- Design shear strength: $13.312$ kN
- Compression throughout
  - Maximum comp (DL+SL+wind): $1.1147$ N/mm$^2$
  - Allowable compn: $1.8945$ N/mm$^2$
  - Maximum compn (DL+wind): $1.3005$ N/mm$^2$
  - Allowable compn (DL+wind): $1.8945$ N/mm$^2$
Location: Wall on grid line A/1-2

Masonry shear wall

These calculations are in accordance with BS5628-1:2005 the "Code of practice for the use of masonry".

Elevation on shear wall

Length of shear wall L=0.7 m
Density of masonry dens=12 kN/m³
Thickness of wall t=140 mm
between lateral restraints h=2.6 m
Support Restraint factor rf=0.75
Mortar designation to Table 1 mortar=2
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Compressive strength of units po=10.4 N/mm²
Height hb=215 mm
Least horiz dimension lhd=140 mm
Partial safety factor material γm=2.5
Characteristic dead load DL=40 kN
Characteristic imposed load LL=0 kN
Characteristic wind load WL=0.85 kN

DESIGN SUMMARY

Wall height 2.6 m
Wall length 0.7 m
Wall thickness 140 mm
Char comp strength 5.2907 N/mm²
Char flex strength 0.25 N/mm²
Design shear load 1.19 kN
Design shear strength 23.02 kN
Compression throughout

Maximum comp (DL+SL+wind) 0.75919 N/mm²
Allowable compn 2.1163 N/mm²
Maximum compn (DL+wind) 0.88572 N/mm²
Allowable compn (DL+wind) 2.1163 N/mm²
Location: Wall on grid line A/1-2

Masonry shear wall

These calculations are in accordance with BS5628-1:2005 the "Code of practice for the use of masonry".

Elevation on shear wall

Length of shear wall \( L = 4 \text{ m} \)
Density of masonry \( \text{dens} = 12 \text{ kN/m}^3 \)
Thickness of wall \( t = 100 \text{ mm} \)
between lateral restraints \( h = 2 \text{ m} \)
Support Restraint factor \( rf = 0.75 \)
Mortar designation to Table 1 \( \text{mortar} = 2 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 1 \)
Compressive strength of units \( po = 50 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{water} = 9 \)
Partial safety factor material \( \gamma_m = 2.5 \)
Characteristic dead load \( DL = 40 \text{ kN} \)
Characteristic imposed load \( LL = 0 \text{ kN} \)
Characteristic wind load \( WL = 0.85 \text{ kN} \)

DESIGN SUMMARY

Wall height \( 2 \text{ m} \)
Wall length \( 4 \text{ m} \)
Wall thickness \( 100 \text{ mm} \)
Char comp strength \( 8.17 \text{ N/mm}^2 \)
Char flex strength \( 0.4 \text{ N/mm}^2 \)
Design shear load \( 1.19 \text{ kN} \)
Design shear strength \( 66.714 \text{ kN} \)
Compression throughout

Maximum comp (DL+SL+wind) \( 0.15645 \text{ N/mm}^2 \)
Allowable compn \( 3.268 \text{ N/mm}^2 \)
Maximum compn (DL+wind) \( 0.18253 \text{ N/mm}^2 \)
Allowable compn (DL+wind) \( 3.268 \text{ N/mm}^2 \)
Location: Wall on grid line A/1-2

Masonry shear wall

These calculations are in accordance with EC6 Part 1-1 General rules for reinforced and unreinforced masonry structures.

Wk = Characteristic wind load

Elevation on shear wall

Length of shear wall \( L = 0.7 \text{ m} \)
Density of masonry \( \text{dens} = 12 \text{ kN/m}^3 \)
Thickness of wall \( t = 100 \text{ mm} \)
between lateral restraints \( h = 2.6 \text{ m} \)
Support Restraint factor \( rf = 0.75 \)
Mortar designation to Table NA.2 \( \text{mortar} = 3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 4 \)
Compressive strength of units \( po = 10.4 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1) = 0.75 \)
Height of masonry unit \( hu = 215 \text{ mm} \)
Least horiz. dimension of units \( lhd = 100 \text{ mm} \)
Partial safety factor (material) \( \text{gamM} = 2.3 \)
Characteristic permanent load \( Gk = 40 \text{ kN} \)
Characteristic variable load \( Qk = 0 \text{ kN} \)
Characteristic wind load \( Wk = 2.05 \text{ kN} \)
Factor \( \psi_0 \) for variable load \( \psi_0 = 0.7 \)

DESIGN SUMMARY

Wall height \( 2.6 \text{ m} \)
Wall length \( 0.7 \text{ m} \)
Wall thickness \( 100 \text{ mm} \)
Char compressive strength \( 6.6767 \text{ N/mm}^2 \)
Char flexural strength \( 0.25 \text{ N/mm}^2 \)
Design shear force \( 3.075 \text{ kN} \)
Design shear resistance \( 10.274 \text{ kN} \)
Compression throughout
Design comp stress \( (Gk+Qk+Wk) \) \( 1.3356 \text{ N/mm}^2 \)
Design compressive strength \( 2.9029 \text{ N/mm}^2 \)
Design comp stress \( (Gk+Wk) \) \( 1.303 \text{ N/mm}^2 \)
Design compressive strength \( 2.9029 \text{ N/mm}^2 \)
Location: Wall on grid line A/1-2

Masonry shear wall

These calculations are in accordance with EC6 Part 1-1 General rules for reinforced and unreinforced masonry structures.

Wk = Characteristic wind load

Elevation on shear wall

Length of shear wall \( L = 0.7 \text{ m} \)
Density of masonry \( \text{dens} = 12 \text{ kN/m}^3 \)
Thickness of wall \( t = 140 \text{ mm} \)
between lateral restraints \( h = 2.6 \text{ m} \)
Support Restraint factor \( rf = 0.75 \)
Mortar designation to Table NA.2 \( \text{mortar} = 2 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 4 \)
Compressive strength of units \( po = 10.4 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1) = 0.75 \)
Height of masonry unit \( h_u = 215 \text{ mm} \)
Least horiz. dimension of units \( l_{hd} = 140 \text{ mm} \)
Partial safety factor (material) \( \gamma_m = 2.3 \)
Characteristic permanent load \( G_k = 40 \text{ kN} \)
Characteristic variable load \( Q_k = 0 \text{ kN} \)
Characteristic wind load \( W_k = 0.85 \text{ kN} \)
Factor \( \psi_0 \) for variable load \( \psi_0 = 0.7 \)

DESIGN SUMMARY

Wall height \( 2.6 \text{ m} \)
Wall length \( 0.7 \text{ m} \)
Wall thickness \( 140 \text{ mm} \)
Char compressive strength \( 7.8991 \text{ N/mm}^2 \)
Char flexural strength \( 0.25 \text{ N/mm}^2 \)
Design shear force \( 1.275 \text{ kN} \)
Design shear resistance \( 12.08 \text{ kN} \)
Compression throughout
Design comp stress \((G_k+Q_k+W_k)\) \( 0.86308 \text{ N/mm}^2 \)
Design compressive strength \( 3.4344 \text{ N/mm}^2 \)
Design comp stress \((G_k+W_k)\) \( 0.88308 \text{ N/mm}^2 \)
Design compressive strength \( 3.4344 \text{ N/mm}^2 \)
Location: Wall on grid line A/1-2

Masonry shear wall

These calculations are in accordance with EC6 Part 1-1 General rules for reinforced and unreinforced masonry structures.

Wk = Characteristic wind load

Elevation on shear wall

Length of shear wall \( L = 4 \text{ m} \)
Density of masonry \( \text{dens} = 12 \text{ kN/m}^3 \)
Thickness of wall \( t = 100 \text{ mm} \)
between lateral restraints \( h = 2 \text{ m} \)
Support Restraint factor \( rf = 0.75 \)
Mortar designation to Table NA.2 \( \text{mortar} = 2 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 1 \)
Compressive strength of units \( po = 50 \text{ N/mm}^2 \)
Water absorption %age of leaf \( \text{water} = 9 \)
Constant K from NA Table NA.4 \( K(1) = 0.5 \)
Height of masonry unit \( h_u = 65 \text{ mm} \)
Least horiz. dimension of units \( l_{hd} = 100 \text{ mm} \)
Partial safety factor (material) \( \gamma_M = 2.3 \)
Characteristic permanent load \( G_k = 40 \text{ kN} \)
Characteristic variable load \( Q_k = 0 \text{ kN} \)
Characteristic wind load \( W_k = 0.85 \text{ kN} \)
Factor \( \psi_0 \) for variable load \( \psi_0 = 0.7 \)

DESIGN SUMMARY

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<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height</td>
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<tr>
<td>Wall length</td>
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<td>Wall thickness</td>
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<td>Char compressive strength</td>
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<tr>
<td>Char flexural strength</td>
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<td>Design shear force</td>
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</tr>
<tr>
<td>Design shear resistance</td>
<td>39.142 kN</td>
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<tr>
<td>Compression throughout</td>
<td></td>
</tr>
<tr>
<td>Design comp stress ( G_k+Q_k+W_k )</td>
<td>0.14998 N/mm²</td>
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<tr>
<td>Design compressive strength</td>
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<td>Design comp stress ( G_k+W_k )</td>
<td>0.17696 N/mm²</td>
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<tr>
<td>Design compressive strength</td>
<td>5.1352 N/mm²</td>
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</tbody>
</table>
Location: Wall on grid line B

Masonry shear wall

These calculations are in accordance with EC6 Part 1-1 General rules for reinforced and unreinforced masonry structures.

Wk = Characteristic wind load

Elevation on shear wall

Length of shear wall \( L = 4 \text{ m} \)
Density of masonry \( \text{dens} = 6 \text{kN/m}^3 \)
Thickness of wall \( t = 100 \text{ mm} \)
between lateral restraints \( h = 2.4 \text{ m} \)
Support Restraint factor \( rf = 1 \)
Mortar designation to Table NA.2 \( \text{mortar} = 3 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ(1)} = 4 \)
Compressive strength of units \( po = 3.6 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1) = 0.75 \)
Normalised mean comp strength \( fbn(1) = 5 \text{ N/mm}^2 \)
Partial safety factor (material) \( \text{gamM} = 2.7 \)
Characteristic permanent load \( Gk = 0 \text{kN} \)
Characteristic variable load \( Qk = 0 \text{kN} \)
Characteristic wind load \( Wk = 6.95 \text{kN} \)
Factor \( \psi_0 \) for variable load \( \psi_0 = 0.5 \)

**DESIGN SUMMARY**

Wall height \( 2.4 \text{ m} \)
Wall length \( 4 \text{ m} \)
Wall thickness \( 100 \text{ mm} \)
Char compressive strength \( 3.5072 \text{ N/mm}^2 \)
Char flexural strength \( 0.25 \text{ N/mm}^2 \)
Design shear force \( 10.425 \text{kN} \)
Design shear resistance \( 24.829 \text{kN} \)
Design tensile stress \( 0.087345 \text{ N/mm}^2 \)
Design tensile strength \( 0.092593 \text{ N/mm}^2 \)
Design comp stress \( (Gk+Qk+Wk) = 0.13292 \text{ N/mm}^2 \)
Design compressive strength \( 1.299 \text{ N/mm}^2 \)
Design comp stress \( (Gk+Wk) = 0.11327 \text{ N/mm}^2 \)
Design compressive strength \( 1.299 \text{ N/mm}^2 \)
Location: Leisure Centre

Single storey building to BS5628

Loadbearing brick fin walls with panels between constructed using cavity construction. Roof plate provides prop at top of fin walls, and transmits wind forces to the flank/end walls.

Spacing of fins Lf=4.05 m
Height to eaves (top of fin) h=7 m

Thickness of outer leaf t(1)=102.5 mm
Density of outer leaf den(1)=20 kN/m³
Thickness of inner leaf t(2)=102.5 mm
Density of inner leaf den(2)=20 kN/m³
Depth of fin D=1115 mm
Thickness of fin t=327 mm
Mortar designation to Table 1 mortar=3
Partial Safety Factor Material gammam=2.5
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=1
Compressive Strength of Units po(1)=40 N/mm²
Water absorption %age of leaf water=7
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(2)=1
Compressive strength of units po(2)=20 N/mm²
Water absorption %age of leaf water=7
Dynamic wind pressure q=0.4 kN/m²
Max Cpe on windward face (+ve) Cpw=0.7
Max Cpe on suction face (-ve) Cps=-0.6
Max Cpe on roof (-ve) Cper=-0.85
Metal decking on roof DL1=0.18 kN/m²
Felt & chippings on roof DL2=0.33 kN/m²
Own weight of roof beams DL3=0.2 kN/m²
Superimposed load LL=0.75 kN/m²
| DESIGN SUMMARY | Wall height | 7 m |
|               | Length between fins | 4.05 m |
| OUTER LEAF    | Thickness | 102.5 mm |
|               | Char flex strength (bed) | 0.4 N/mm² |
|               | Char flex strength (perp) | 1.1 N/mm² |
|               | Char comp strength | 7.4 N/mm² |
| INNER LEAF    | Thickness | 102.5 mm |
|               | Char flex strength (bed) | 0.4 N/mm² |
|               | Char flex strength (perp) | 1.1 N/mm² |
|               | Char comp strength | 5 N/mm² |
| LEAF          | Bending moment between fins | 0.6561 kNm |
|               | Bend strength between fins | 1.5409 kNm |
| FINS (pressure)| Bending stress in tension | 0.001143 N/mm² |
|               | Bending stress in comp | 0.13039 N/mm² |
| (suction)     | Bending stress in tension | -0.019265 N/mm² |
|               | Bending stress in comp | 0.084136 N/mm² |
|               | Bend strength in tension | 0.16 N/mm² |
|               | Bend strength in comp | 3.256 N/mm² |
Location: Leisure Centre

Single storey building to BS5628

Loadbearing brick fin walls with panels between constructed using cavity construction. Roof plate provides prop at top of fin walls, and transmits wind forces to the flank/end walls.

Spacing of fins
Height to eaves (top of fin)
Thicknness of outer leaf
Density of outer leaf
Thicknness of inner leaf
Density of inner leaf
Depth of fin
Thickness of fin
Mortar designation to Table 1
Partial Safety Factor Material
Compressive Strength of Units
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=2
Compressive strength of units
Height
Least horizontal dimension
Dynamic wind pressure
Max Cpe on windward face (+ve)
Max Cpe on suction face (-ve)
Max Cpe on roof (-ve)
Metal decking on roof
Felt & chippings on roof
Own weight of roof beams
Superimposed load
### DESIGN SUMMARY

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<table>
<thead>
<tr>
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<tbody>
<tr>
<td>Wall height</td>
<td>2.5 m</td>
</tr>
<tr>
<td>Length between fins</td>
<td>1 m</td>
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#### OUTER LEAF

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<tbody>
<tr>
<td>Thickness</td>
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<tr>
<td>Char flex strength (bed)</td>
<td>0.3 N/mm²</td>
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<tr>
<td>Char flex strength (perp)</td>
<td>0.9 N/mm²</td>
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<td>Char comp strength</td>
<td>9.24 N/mm²</td>
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#### INNER LEAF

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<tbody>
<tr>
<td>Thickness</td>
<td>305 mm</td>
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<tr>
<td>Char flex strength (bed)</td>
<td>0.25 N/mm²</td>
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<td>Char flex strength (perp)</td>
<td>0.9 N/mm²</td>
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<td>Char comp strength</td>
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#### LEAF

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<tbody>
<tr>
<td>Bending moment between fins</td>
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<td>Bend strength between fins</td>
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#### FINS (pressure)

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<tr>
<td>Bending stress in comp</td>
<td>0.038108 N/mm²</td>
</tr>
<tr>
<td>(suction) Bending stress in tension</td>
<td>-0.11139E-3 N/mm²</td>
</tr>
<tr>
<td>Bending stress in comp</td>
<td>0.02495 N/mm²</td>
</tr>
<tr>
<td>Bend strength in tension</td>
<td>0.12 N/mm²</td>
</tr>
<tr>
<td>Bend strength in comp</td>
<td>4.0656 N/mm²</td>
</tr>
</tbody>
</table>
**Location:** Leisure Centre

**Single storey building to EC6**

Loadbearing brick fin walls with panels between constructed using cavity construction. Roof plate provides prop at top of fin walls and transmits wind forces to the flank/end walls.

Spacing of fins $L_f = 4.05$ m  
Height to eaves (top of fin) $h = 7$ m

Thickness of outer leaf $t(1) = 102.5$ mm  
Density of outer leaf $\text{den}(1) = 20$ kN/m$^3$  
Thickness of inner leaf $t(2) = 102.5$ mm  
Density of inner leaf $\text{den}(2) = 20$ kN/m$^3$

Depth of fin $D = 1115$ mm  
Thickness of fin $t = 327$ mm

Mortar designation to Table NA.2 $\text{mortar} = 3$

Partial safety factor (material) $\gamma_M = 2.3$

Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) $\text{typ}(1) = 1$

Compressive strength of units $\text{po}(1) = 40$ N/mm$^2$

Water absorption %age of leaf $\text{water} = 7$

Height of masonry unit $\text{hu}(1) = 65$ mm

Least horiz. dimension of units $\text{lhd}(1) = 102.5$ mm

Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) $\text{typ}(2) = 1$

Compressive strength of units $\text{po}(2) = 20$ N/mm$^2$

Water absorption %age of leaf $\text{water} = 7$

Constant K from NA Table NA.4 $K(1) = 0.5$

Height of masonry unit $\text{hu}(2) = 65$ mm

Least horiz. dimension of units $\text{lhd}(2) = 100$ mm

Peak velocity pressure $q_p = 0.4$ kN/m$^2$

Max Cpe on windward face (+ve) $C_{pew} = 0.7$

Max Cpe on suction face (-ve) $C_{pes} = -0.6$

Max Cpe on roof (-ve) $C_{per} = -0.85$

Metal decking on roof $G_{k1} = 0.18$ kN/m$^2$

Felt & chippings on roof $G_{k2} = 0.33$ kN/m$^2$

Own weight of roof beams $G_{k3} = 0.2$ kN/m$^2$

Variable load on roof $Q_k = 0.75$ kN/m$^2$
### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height</td>
<td>7 m</td>
</tr>
<tr>
<td>Length between fins</td>
<td>4.05 m</td>
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</table>

### OUTER LEAF

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>102.5 mm</td>
</tr>
<tr>
<td>Char flex strength (bed)</td>
<td>0.4 N/mm²</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
<td>1.1 N/mm²</td>
</tr>
<tr>
<td>Char comp strength</td>
<td>7.1272 N/mm²</td>
</tr>
</tbody>
</table>

### INNER LEAF

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>102.5 mm</td>
</tr>
<tr>
<td>Char flex strength (bed)</td>
<td>0.4 N/mm²</td>
</tr>
<tr>
<td>Char flex strength (perp)</td>
<td>1.1 N/mm²</td>
</tr>
<tr>
<td>Char comp strength</td>
<td>4.4055 N/mm²</td>
</tr>
</tbody>
</table>

### LEAF

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending moment between fins</td>
<td>0.70296 kNm</td>
</tr>
<tr>
<td>Bend strength between fins</td>
<td>1.6749 kNm</td>
</tr>
</tbody>
</table>

### FINS (pressure)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress in tension</td>
<td>-0.0067309 N/mm²</td>
</tr>
<tr>
<td>Bending stress in comp</td>
<td>0.13847 N/mm²</td>
</tr>
</tbody>
</table>

### FINS (suction)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stress in tension</td>
<td>-0.025725 N/mm²</td>
</tr>
<tr>
<td>Bending stress in comp</td>
<td>0.090435 N/mm²</td>
</tr>
<tr>
<td>Bend strength in tension</td>
<td>0.17391 N/mm²</td>
</tr>
<tr>
<td>Bend strength in comp</td>
<td>3.4087 N/mm²</td>
</tr>
</tbody>
</table>

Outer leaf - use Group 1 units (i.e. solid)
Inner leaf - use Group 1 units (i.e. solid)
Location: Leisure Centre

Single storey building to EC6

Loadbearing brick fin walls with panels between constructed using cavity construction. Roof plate provides prop at top of fin walls and transmits wind forces to the flank/end walls.

Spacing of fins Lf=1 m
Height to eaves (top of fin) h=2.5 m

Thickness of outer leaf t(1)=100 mm
Density of outer leaf den(1)=20 kN/m³
Thickness of inner leaf t(2)=305 mm
Density of inner leaf den(2)=5 kN/m³
Depth of fin D=500 mm
Thickness of fin t=200 mm
Mortar designation to Table NA.2 mortar=3
Partial safety factor (material) gamM=2.3
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=2
Compressive strength of units po(1)=60 N/mm²
Constant K from NA Table NA.4 K(1)=0.5
Height of masonry unit hu(1)=65 mm
Least horiz. dimension of units lhd(1)=100 mm
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(2)=4
Compressive strength of units po(2)=22.5 N/mm²
Constant K from NA Table NA.4 K(2)=0.75
Height of masonry unit hu(2)=215 mm
Least horiz. dimension of units lhd(2)=305 mm
Peak velocity pressure qp=0.4 kN/m²
Max Cpe on windward face (+ve) Cpew=0.7
Max Cpe on suction face (-ve) Cpes=-0.6
Max Cpe on roof (-ve) Cper=-0.85
Metal decking on roof Gk1=0.18 kN/m²
Felt & chippings on roof Gk2=0.33 kN/m²
Own weight of roof beams Gk3=0.2 kN/m²
Variable load on roof Qk=0.75 kN/m²
| DESIGN SUMMARY | Wall height | 2.5 m |
|               | Length between fins | 1 m |
| OUTER LEAF    | Thickness | 100 mm |
|               | Char flex strength (bed) | 0.3 N/mm² |
|               | Char flex strength (perp) | 0.9 N/mm² |
|               | Char comp strength | 9.5055 N/mm² |
| INNER LEAF    | Thickness | 305 mm |
|               | Char flex strength (bed) | 0.25 N/mm² |
|               | Char flex strength (perp) | 0.9 N/mm² |
|               | Char comp strength | 8.6774 N/mm² |
| LEAF          | Bending moment between fins | 0.042857 kNm |
|               | Bend strength between fins | 6.719 kNm |
| FINS (pressure) | Bending stress in tension | 0.0028972 N/mm² |
|               | Bending stress in comp | 0.041018 N/mm² |
| (suction)     | Bending stress in tension | -0.0024398 N/mm² |
|               | Bending stress in comp | 0.028057 N/mm² |
|               | Bend strength in tension | 0.13043 N/mm² |
|               | Bend strength in comp | 4.5461 N/mm² |

Outer leaf - use Group 1 units (i.e. solid)
Inner leaf - use Group 1 units (i.e. solid)
Location: Ex1 - Wall on grid line A1-2

Reinforced masonry retaining wall to

BS5628-2:2005

Cavity to be filled with concrete or grout of C25/30 minimum compressive strength. Cavity width to be a minimum of 100 mm and earth face leaf to be ≥ 100 mm thick. Concrete base design is beyond the scope of this proforma.

Outer skin thickness  t1=215 mm
Earth face skin thickness  t2=102 mm
Wall height  H=4.5 m
Cavity width (minimum 100 mm)  cw=100 mm
Ultimate moment at top of base  Mu=20.6 kNm
Ultimate shear at top of base  Vu=27 kN
Mortar designation to Table 1  mortar=2
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=1
Compressive strength of units  po(1)=30 N/mm²
Water absorption %age of leaf  water=7
Partial safety factor (reinft.)  gammam=2
Characteristic tensile strength  fy=500 N/mm²
Diameter of reinforcing bars  diam=12 mm
Char anchorage bond strength  fb=4.1 N/mm²

DESIGN SUMMARY

Ultimate bending moment  20.6 kNm
Ultimate shear force  27 kN
Wall effective depth  265 mm
Wall effective width  1000 mm
Maximum wall height  4.5 m
Tensile strength of reinft.  500 N/mm²
Mortar designation  2
Compressive strength of units  30 N/mm²
Area of reinforcement in wall  565.49 mm²/m
Diameter of reinforcing bars  12 mm

Partial safety factors:
Masonry  2
Reinforcement strength  1.15
Shear strength of masonry  2
Bond strength  1.5

The design of the concrete base is beyond the scope of this proforma. Brick or block could be used for the earth face leaf. Use masonry units of category I manufacturing control and of water absorption ≤ 7%
Location: Ex2 - Brick retaining wall

Reinforced masonry retaining wall to

BS5628-2:2005

Cavity to be filled with concrete or grout of C25/30 minimum compressive strength. Cavity width to be a minimum of 100 mm and earth face leaf to be ≥ 100 mm thick. Concrete base design is beyond the scope of this proforma.

Outer skin thickness \( t_1 = 102 \) mm
Earth face skin thickness \( t_2 = 102 \) mm
Wall height \( H = 2.7 \) m
Cavity width (minimum 100 mm) \( c_w = 100 \) mm
Ultimate moment at top of base \( M_u = 9.5 \) kNm
Ultimate shear at top of base \( V_u = 14.2 \) kN
Mortar designation to Table 1 \( \text{mortar} = 1 \)

Masonry type \( 1=\text{clay} \ 2=\text{cal silicate} \ 3=\text{conc brick} \ 4=\text{block} \) typ(1)=1
Compressive strength of units \( p_o(1) = 30 \) N/mm\(^2\)
Water absorption %age of leaf \( \text{water} = 12 \)
Partial safety factor (reinft.) \( \gamma_{ma} = 2.3 \)
Characteristic tensile strength \( f_y = 500 \) N/mm\(^2\)
Diameter of reinforcing bars \( \text{diam} = 12 \) mm
Char anchorage bond strength \( f_b = 4.1 \) N/mm\(^2\)

DESIGN SUMMARY

Ultimate bending moment 9.5 kNm
Ultimate shear force 14.2 kN
Wall effective depth 152 mm
Wall effective width 1000 mm
Maximum wall height 2.7 m
Tensile strength of reinft. 500 N/mm\(^2\)
Mortar designation 1
Compressive strength of units 30 N/mm\(^2\)
Area of reinforcement in wall 452.39 mm\(^2\)/m
Diameter of reinforcing bars 12 mm

Partial safety factors:
Masonry 2.3
Reinforcement strength 1.15
Shear strength of masonry 2
Bond strength 1.5

The design of the concrete base is beyond the scope of this proforma. Brick or block could be used for the earth face leaf.
Location: Ex3 - Block retaining wall

Reinforced masonry retaining wall to

BS5628-2:2005

Cavity to be filled with concrete or grout of C25/30 minimum compressive strength. Cavity width to be a minimum of 100 mm and earth face leaf to be ≥ 100 mm thick. Concrete base design is beyond the scope of this proforma.

Outer skin thickness t1 = 140 mm
Earth face skin thickness t2 = 100 mm
Wall height H = 3.4 m
Cavity width (minimum 100 mm) cw = 100 mm
Ultimate moment at top of base Mu = 9.5 kNm
Ultimate shear at top of base Vu = 14.2 kN
Mortar designation to Table 1 mortar = 1
Masonry type (1 = clay 2 = cal silicate 3 = conc brick 4 = block) typ(1) = 4
Height of units hb = 215 mm
Least horiz. dimension of units lhd = 140 mm
Compressive strength of units po(1) = 30 N/mm²
Partial safety factor (reinft.) gammam = 2.3
Characteristic tensile strength fy = 500 N/mm²
Diameter of reinforcing bars diam = 12 mm
Char anchorage bond strength fb = 4.1 N/mm²

DESIGN SUMMARY
Ultimate bending moment 9.5 kNm
Ultimate shear force 14.2 kN
Wall effective depth 190 mm
Wall effective width 1000 mm
Maximum wall height 3.4 m
Tensile strength of reinf. 500 N/mm²
Mortar designation 1
Compressive strength of units 30 N/mm²
Area of reinforcement in wall 452.39 mm²/m
Diameter of reinforcing bars 12 mm

Partial safety factors:
Masonry 2.3
Reinforcement strength 1.15
Shear strength of masonry 2
Bond strength 1.5

The design of the concrete base is beyond the scope of this proforma. Brick or block could be used for the earth face leaf.
Location: Ex4 - Outer skin blocks laid flat

Reinforced masonry retaining wall to

BS5628-2:2005

Cavity to be filled with concrete or grout of C25/30 minimum compressive strength. Cavity width to be a minimum of 100 mm and earth face leaf to be ≥ 100 mm thick. Concrete base design is beyond the scope of this proforma.

Outer skin thickness \( t_1 = 215 \text{ mm} \)
Earth face skin thickness \( t_2 = 100 \text{ mm} \)
Wall height \( H = 4.7 \text{ m} \)
Cavity width (minimum 100 mm) \( c_w = 100 \text{ mm} \)
Ultimate moment at top of base \( M_u = 9.5 \text{ kNm} \)
Ultimate shear at top of base \( V_u = 14.2 \text{ kN} \)
Mortar designation to Table 1 \( \text{mortar} = 1 \)
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) \( \text{typ}(1) = 4 \)
Height of units \( h_b = 215 \text{ mm} \)
Least horiz. dimension of units \( l_h d = 215 \text{ mm} \)

WARNING:
The height of blocks needs to be reduced.
Location: Ex1 - Wall on grid line A/1-2

Reinforced masonry retaining wall to EC6

Cavity to be filled with concrete or grout and have a minimum width of 100 mm. Infill to be Grade C25/30 or higher. The earth face leaf must not be less than 100 mm thick. Group 1 units to be used.

Concrete base design is beyond the scope of this proforma.

Outer leaf thickness  
Earth face leaf thickness  
Wall height  
Cavity width (minimum 100 mm)  
Design moment at top of base  
Design shear at top of base  
Mortar designation to Table NA.2  
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=1  
Compressive strength of units  
Constant K from NA Table NA.4  
Height of masonry unit  
Least horiz. dimension of units  
Partial safety factor (masonry)  
Characteristic tensile strength  
Diameter of reinforcing bars  
Char anchorage bond strength

DESIGN SUMMARY

Design bending moment  
Design shear force  
Wall effective depth  
Wall effective width  
Maximum wall height  
Tensile strength (reinft)  
Mortar designation  
Compressive strength of units  
Area of steel provided  
Diameter of reinforcing bars

Partial safety factors:

Group 1 masonry units  
Reinforcement strength  
Shear strength of masonry  
Bond strength

The design of the concrete base is beyond the scope of this proforma. Brick or block Group 1 units could be used for the earth face leaf.

Use masonry units of category I manufacturing.
control and of water absorption ≤ 7%.
Location: Ex2 - Brick retaining wall

Reinforced masonry retaining wall to EC6

Cavity to be filled with concrete or grout and have a minimum width of 100 mm. Infill to be Grade C25/30 or higher. The earth face leaf must not be less than 100 mm thick. Group 1 units to be used. Concrete base design is beyond the scope of this proforma.

Outer leaf thickness t1=102 mm
Earth face leaf thickness t2=102 mm
Wall height H=2.7 m
Cavity width (minimum 100 mm) cw=100 mm
Design moment at top of base Md'=9.5 kNm/m
Design shear at top of base Vd=14.2 kN/m
Mortar designation to Table NA.2 mortar=1
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=1
Compressive strength of units po(1)=30 N/mm²
Constant K from NA Table NA.4 K(1)=0.5
Height of masonry unit hu=65 mm
Least horiz. dimension of units lhd=100 mm
Partial safety factor (masonry) gamM=2.3
Characteristic tensile strength fy=500 N/mm²
Diameter of reinforcing bars diam=12 mm
Char anchorage bond strength fbok=4.1 N/mm²

DESIGN SUMMARY

Design bending moment 9.5 kNm
Design shear force 14.2 kN
Wall effective depth 152 mm
Wall effective width 1000 mm
Maximum wall height 2.7 m
Tensile strength (reinft) 500 N/mm²
Mortar designation 1
Compressive strength of units 30 N/mm²
Area of steel provided 452.39 mm²/m
Diameter of reinforcing bars 12 mm

Partial safety factors:
Group 1 masonry units 2.3
Reinforcement strength 1.15
Shear strength of masonry 2
Bond strength 1.5

The design of the concrete base is beyond the scope of this proforma. Brick or block Group 1 units could be used for the earth face leaf.
Location: Ex3 - Block retaining wall

Reinforced masonry retaining wall to EC6

Cavity to be filled with concrete or grout and have a minimum width of 100 mm. Infill to be Grade C25/30 or higher. The earth face leaf must not be less than 100 mm thick. Group 1 units to be used. Concrete base design is beyond the scope of this proforma.

Outer leaf thickness \( t_1 = 140 \text{ mm} \)
Earth face leaf thickness \( t_2 = 100 \text{ mm} \)
Wall height \( H = 3.4 \text{ m} \)
Cavity width (minimum 100 mm) \( c_w = 100 \text{ mm} \)
Design moment at top of base \( M_d' = 9.5 \text{ kNm/m} \)
Design shear at top of base \( V_d = 14.2 \text{ kN/m} \)
Mortar designation to Table NA.2 mortar=1
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Compressive strength of units \( p_o(1) = 30 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1) = 0.75 \)
Height of masonry unit \( h_u = 215 \text{ mm} \)
Least horiz. dimension of units \( l_{hd} = 140 \text{ mm} \)
Partial safety factor (masonry) \( \gamma_{M} = 2.3 \)
Characteristic tensile strength \( f_y = 500 \text{ N/mm}^2 \)
Diameter of reinforcing bars \( d_{iam} = 12 \text{ mm} \)
Char anchorage bond strength \( f_{bok} = 4.1 \text{ N/mm}^2 \)

DESIGN SUMMARY

Design bending moment 9.5 kNm
Design shear force 14.2 kN
Wall effective depth 190 mm
Wall effective width 1000 mm
Maximum wall height 3.4 m
Tensile strength (reinft) 500 N/mm²
Mortar designation 1
Compressive strength of units 30 N/mm²
Area of steel provided 452.39 mm²/m
Diameter of reinforcing bars 12 mm

Partial safety factors:
Group 1 masonry units 2.3
Reinforcement strength 1.15
Shear strength of masonry 2
Bond strength 1.5

The design of the concrete base is beyond the scope of this proforma. Brick or block Group 1 units could be used for the earth face leaf.
Location: Ex4 - Outer skin blocks laid flat

Reinforced masonry retaining wall to EC6

Cavity to be filled with concrete or grout and have a minimum width of 100 mm. Infill to be Grade C25/30 or higher. The earth face leaf must not be less than 100 mm thick. Group 1 units to be used. Concrete base design is beyond the scope of this proforma.

Outer leaf thickness  t1=215 mm
Earth face leaf thickness  t2=100 mm
Wall height  H=4.7 m
Cavity width (minimum 100 mm)  cw=100 mm
Design moment at top of base  Md'=9.5 kNm/m
Design shear at top of base  Vd=14.2 kN/m
Mortar designation to Table NA.2 mortar=1
Masonry type (1=clay 2=cal silicate 3=conc brick 4=block) typ(1)=4
Compressive strength of units  po(1)=30 N/mm^2
Constant K from NA Table NA.4  K(1)=0.75
Height of masonry unit  hu=100 mm
Least horiz. dimension of units  lhd=215 mm
Partial safety factor (masonry)  gamM=2.3
Characteristic tensile strength  fy=500 N/mm^2
Diameter of reinforcing bars  diam=12 mm
Char anchorage bond strength  fbok=4.1 N/mm^2

DESIGN SUMMARY

Design bending moment  9.5 kNm
Design shear force  14.2 kN
Wall effective depth  265 mm
Wall effective width  1000 mm
Maximum wall height  4.7 m
Tensile strength (reinft)  500 N/mm^2
Mortar designation  1
Compressive strength of units  30 N/mm^2
Area of steel provided  452.39 mm^2/m
Diameter of reinforcing bars  12 mm

Partial safety factors:
Group 1 masonry units  2.3
Reinforcement strength  1.15
Shear strength of masonry  2
Bond strength  1.5

The design of the concrete base is beyond the scope of this proforma. Brick or block Group 1 units could be used for the earth face leaf.
Location: Ex5 - Pocket-Type ret. wall (Ex R1 by J Roberts w. links)

Reinforced masonry pocket-type retaining wall to EC6

Formed vertical voids (pockets) to be filled with concrete or grout. Infill Grade to be C25/30 or higher. Pockets must be at least 100 mm deep formed in Group 1 units. Concrete base design is beyond the scope of this proforma.

Wall thickness $tw=328$ mm
Wall height $H=3.65$ m
Design moment at top of base $Md'=97.24$ kNm/m
Design shear at top of base $Vd=71.85$ kN/m
Pocket depth $pd=113$ mm
Pocket width along wall length $pw=235$ mm
Pocket spacing $ps=900$ mm
Effective depth to be used $d=271.5$ mm
Mortar designation to Table NA.2 $mortar=1$
Compressive strength of units $po(1)=50$ N/mm$^2$
Constant K from NA Table NA.4 $K(1)=0.5$
Height of masonry unit $hu=65$ mm
Least horiz. dimension of units $lhd=100$ mm
Partial safety factor (masonry) $\gamma_M=2$
Characteristic tensile strength $fy=500$ N/mm$^2$
Diameter of reinforcing bars $diam=25$ mm
Spacing (<0.75d & <300mm) $sv=200$ mm
Char tensile strength of links $fy'=200$ N/mm$^2$
Diameter of shear links $Ds=6$ mm
Number of link legs $nleg=2$
Char anchorage bond strength $fbok=3.4$ N/mm$^2$
**DESIGN SUMMARY**

- **Design bending moment**: 97.24 kNm
- **Design shear force**: 71.85 kN
- **Wall effective depth**: 271.5 mm
- **Wall effective width**: 900 mm
- **Maximum wall height**: 3.65 m
- **Tensile strength (reinft)**: 500 N/mm²
- **Mortar designation**: 1
- **Compressive strength of units**: 50 N/mm²
- **Area of steel provided**: 981.75 mm²/m
- **Diameter of reinforcing bars**: 25 mm

**Partial safety factors:**
- **Group 1 masonry units**: 2
- **Reinforcement strength**: 1.15
- **Shear strength of masonry**: 2
- **Bond strength**: 1.5

Provide closed shear links of 6 mm diameter at pocket positions at 200 mm vertical centres.

The design of the concrete base is beyond the scope of this proforma.

Use masonry units of category I manufacturing control and of water absorption ≤ 7%.
Reinforced masonry pocket-type
retaining wall to EC6

Formed vertical voids (pockets) to be filled with concrete or grout. Infill Grade to be C25/30 or higher. Pockets must be at least 100 mm deep formed in Group 1 units. Concrete base design is beyond the scope of this proforma.

Wall thickness: \( t_w = 440 \text{ mm} \)
Wall height: \( H = 4 \text{ m} \)
Design moment at top of base: \( M_d' = 107.62 \text{ kNm/m} \)
Design shear at top of base: \( V_d = 73.63 \text{ kN/m} \)
Pocket depth: \( p_d = 112.5 \text{ mm} \)
Pocket width along wall length: \( p_w = 235 \text{ mm} \)
Pocket spacing: \( p_s = 900 \text{ mm} \)
Effective depth to be used: \( d = 383.75 \text{ mm} \)
Mortar designation to Table NA.2: mortar = 2
Compressive strength of units: \( p_o(1) = 35 \text{ N/mm}^2 \)
Constant K from NA Table NA.4: \( K(1) = 0.5 \)
Height of masonry unit: \( h_u = 65 \text{ mm} \)
Least horiz. dimension of units: \( l_{hd} = 100 \text{ mm} \)
Partial safety factor (masonry): \( \gamma_M = 2.3 \)
Characteristic tensile strength: \( f_y = 500 \text{ N/mm}^2 \)
Diameter of reinforcing bars: \( d_{iam} = 25 \text{ mm} \)
Char anchorage bond strength: \( f_{bok} = 3.4 \text{ N/mm}^2 \)

**DESIGN SUMMARY**

- Design bending moment: 107.62 kNm
- Design shear force: 73.63 kN
- Wall effective depth: 383.75 mm
- Wall effective width: 900 mm
- Maximum wall height: 4 m
- Tensile strength (reinft): 500 N/mm²
- Mortar designation: 2
- Compressive strength of units: 35 N/mm²
- Area of steel provided: 981.75 mm²/m
- Diameter of reinforcing bars: 25 mm

Partial safety factors:
- Group 1 masonry units: 2.3
- Reinforcement strength: 1.15
- Shear strength of masonry: 2
- Bond strength: 1.5

The design of the concrete base is beyond the scope of this proforma.
Reinforced masonry column under vertical load to BS5628-2:2005

Section A-A

- **t** = thickness of column in plane of bending
- **d1** = distance to comp. reinft. from col. face

**Thickness of column**  
**Width of column**  
**Height of column**  
**Support Restraint factor**  
**Ultimate vertical load**  
**Ultimate weight of column**  
**Load eccentricity**  
**Compressive strength of units**  
**Char comp strength of brickwork**  
**Char tensile strength of reinft**  
**Partial safety factor**  
**Diameter of reinforcing bars**  
**Total number of bars to be used**  
**column compression face**

---

**DESIGN SUMMARY**

- **Ultimate vertical load**: 658 kN
- **Ultimate vertical load capacity**: 793.52 kN
- **Column depth**: 440 mm
- **Column width**: 440 mm
- **Column height**: 7000 mm
- **Area of reinft to be provided**: 1963.5 mm²
- **Diameter of reinforcing bars**: 25 mm
- **No of bars to be used**: 4
- **Tensile strength of reinft**: 500 N/mm²
- **Mortar designation**: 2
- **Compressive strength of units**: 20 N/mm²
- **Mortar designation**: 2
- **Partial safety factors:
  - Masonry**: 2.3
  - Reinforcement**: 1.15

---

SCALE 5.48  
Office 1007  
Proforma 538
Reinforced masonry column under vertical load to BS5628-2:2005

Section A-A

- \( t \) = thickness of column in plane of bending
- \( d_1 \) = distance to comp. reinft. from col. face

**Thickens of column**
- \( t = 553 \text{ mm} \)

**Width of column**
- \( b = 553 \text{ mm} \)

**Height of column**
- \( h = 3000 \text{ mm} \)

**Support Restraint factor**
- \( r_f = 1 \)

**Ultimate vertical load**
- \( P = 1100 \text{ kN} \)

**Ultimate weight of column**
- \( C_{WC} = 0 \text{ kN} \)

**Load eccentricity**
- \( e = 0 \text{ mm} \)

**Compressive strength of units**
- \( S_{units} = 20 \text{ N/mm}^2 \)

**Char comp strength of brickwork**
- \( f_k = 5.6 \text{ N/mm}^2 \)

**Char tensile strength of reinft**
- \( f_y = 500 \text{ N/mm}^2 \)

**Partial safety factor**
- \( \gamma_m = 2.3 \)

**Diameter of reinforcing bars**
- \( \text{diam} = 16 \text{ mm} \)

**DESIGN SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate vertical load</td>
<td>1100 kN</td>
</tr>
<tr>
<td>Column depth</td>
<td>553 mm</td>
</tr>
<tr>
<td>Column width</td>
<td>553 mm</td>
</tr>
<tr>
<td>Column height</td>
<td>3000 mm</td>
</tr>
<tr>
<td>Area of reinft to be provided</td>
<td>1005.3 mm²</td>
</tr>
<tr>
<td>Diameter of reinforcing bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>No of bars to be used</td>
<td>5</td>
</tr>
<tr>
<td>Tensile strength of reinft</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Mortar designation</td>
<td>2</td>
</tr>
<tr>
<td>Compressive strength of units</td>
<td>20 N/mm²</td>
</tr>
<tr>
<td>Mortar designation</td>
<td>2</td>
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<tr>
<td>Partial safety factors:</td>
<td></td>
</tr>
<tr>
<td>Masonry</td>
<td>2.3</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>1.15</td>
</tr>
</tbody>
</table>
Reinforced masonry column under vertical load to BS5628-2:2005

Section A-A

\[ t = \text{thickness of column in plane of bending} \]
\[ d_1 = \text{distance to comp. reinft. from col. face} \]

**Design Summary**

- **Ultimate vertical load**: 714 kN
- **Ultimate vertical load capacity**: 924.75 kN
- **Column depth**: 440 mm
- **Column width**: 440 mm
- **Column height**: 6000 mm
- **Area of reinft to be provided**: 1963.5 mm²
- **Diameter of reinforcing bars**: 25 mm
- **No of bars to be used**: 4
- **Tensile strength of reinft**: 500 N/mm²
- **Mortar designation**: 2
- **Compressive strength of units**: 30 N/mm²
- **Mortar designation**: 2

**Partial safety factors:**

- **Masonry**: 2.3
- **Reinforcement**: 1.15
Location: R3 Example 4 by J Roberts (Hollow Concrete Block column)

Reinforced masonry column under vertical load to BS5628-2:2005

First course of bricks is shown with a four bar reinforcement arrangement

Section A-A

- t = thickness of column in plane of bending
- d1 = distance to comp. reinft. from col. face

Thickenss of column: t = 440 mm
Width of column: b = 440 mm
Height of column: h = 6000 mm
Support Restraint factor: rf = 1
Ultimate vertical load: P = 400 kN
Ultimate weight of column: cwc = 0 kN
Load eccentricity: e = 112.5 mm
Compressive strength of units: Sunits = 30 N/mm²
Char comp strength of brickwork: fk = 7.27 N/mm²
Char tensile strength of reinft: fy = 500 N/mm²
Partial safety factor: gammam = 2
Diameter of reinforcing bars: diam = 25 mm
Total number of bars to be used: Bars = 4
Column compression face: d1 = 107.5 mm

DESIGN SUMMARY

- Ultimate vertical load: 400 kN
- Ultimate vertical load capacity: 930.07 kN
- Column depth: 440 mm
- Column width: 440 mm
- Column height: 6000 mm
- Area of reinft to be provided: 1963.5 mm²
- Diameter of reinforcing bars: 25 mm
- No of bars to be used: 4
- Tensile strength of reinft: 500 N/mm²
- Mortar designation: 2
- Compressive strength of units: 30 N/mm²
- Mortar designation: 2
- Partial safety factors:
  - Masonry: 2
  - Reinforcement: 1.15
Reinforced masonry column under vertical load to EC6 Part 1-1

Group 1 units to be used. First course of bricks is shown with a four bar reinforcement arrangement.

Section A-A  

\[ t = \text{thickness of column in plane of bending} \]
\[ d_1 = \text{distance to comp. reinft. from col. face} \]

**Thick of e m the  of column**  \( t = 440 \text{ mm} \)
**Width of column**  \( b = 440 \text{ mm} \)
**Height of column**  \( h = 7000 \text{ mm} \)
**Support restraint factor**  \( r_f = 1 \)
**Permanent vertical load**  \( G_k = 250 \text{ kN} \)
**Variable vertical load**  \( Q_k = 196 \text{ kN} \)
**Column self weight**  \( G_{ksw} = 8 \text{ kN} \)
**Load eccentricity**  \( e = 0 \text{ mm} \)
**Partial safety factor (masonry)**  \( \gamma_{M} = 2.3 \)
**Char tensile strength of reinft.**  \( f_y = 500 \text{ N/mm}^2 \)
**Mortar designation (Table NA.2)**  \( \text{mortar} = 2 \)
**Compressive strength of units**  \( p_o = 20 \text{ N/mm}^2 \)
**Constant K from NA Table NA.4**  \( K(1) = 0.5 \)
**Height of masonry unit**  \( h_u = 65 \text{ mm} \)
**Least horiz. dimension of units**  \( l_{hd} = 100 \text{ mm} \)
**Diameter of reinforcing bars**  \( \text{diam} = 25 \text{ mm} \)
**Total number of bars to be used**  \( \text{Bars} = 4 \)
**Column compression face**  \( d_1 = 130 \text{ mm} \)
**Link diameter**  \( \text{diam}_l = 6 \)

**DESIGN SUMMARY**

- **Design vertical load**  \( 642.3 \text{ kN} \)
- **Design vertical load resistance**  \( 723.1 \text{ kN} \)
- **Column depth**  \( 440 \text{ mm} \)
- **Column width**  \( 440 \text{ mm} \)
- **Column height**  \( 7000 \text{ mm} \)
- **Area of reinft to be provided**  \( 1963.5 \text{ mm}^2 \)
- **Diameter of reinforcing bars**  \( 25 \text{ mm} \)
- **No of bars to be used**  \( 4 \)
- **Tensile strength of reinft**  \( 500 \text{ N/mm}^2 \)
- **Compressive strength of units**  \( 20 \text{ N/mm}^2 \)
- **Mortar designation**  \( 2 \)
- **Partial safety factors:**
  - **Masonry**  \( 2.3 \)
  - **Reinforcement**  \( 1.15 \)
- **Masonry units to be Group 1 (i.e. solid)**
Reinforced masonry column under vertical load to EC6 Part 1-1

Group 1 units to be used. First course of bricks is shown with a four bar reinforcement arrangement.

Section A-A

\[ t = \text{thickness of column in plane of bending} \]
\[ d_1 = \text{distance to comp. reinft. from col. face} \]

Thickmess of column \( t = 553 \text{ mm} \)
Width of column \( b = 553 \text{ mm} \)
Height of column \( h = 3000 \text{ mm} \)
Support restraint factor \( r_f = 1 \)
Permanent vertical load \( G_k = 600 \text{ kN} \)
Variable vertical load \( Q_k = 193 \text{ kN} \)
Column self weight \( G_{ksw} = 0 \text{ kN} \)
Load eccentricity \( e = 0 \text{ mm} \)
Partial safety factor (masonry) \( \gamma_M = 2.3 \)
Char tensile strength of reinft. \( f_y = 500 \text{ N/mm}^2 \)
Mortar designation (Table NA.2) \( \text{mortar}=2 \)
Compressive strength of units \( p_o = 20 \text{ N/mm}^2 \)
Constant K from NA Table NA.4 \( K(1)=0.5 \)
Height of masonry unit \( h_u = 65 \text{ mm} \)
Least horiz. dimension of units \( l_{hd} = 100 \text{ mm} \)
Diameter of reinforcing bars \( \text{diam}\_1 = 16 \text{ mm} \)
Link diameter \( \text{diaml}\_1 = 6 \)

**DESIGN SUMMARY**

Design vertical load \( 1099.5 \text{ kN} \)
Column depth \( 553 \text{ mm} \)
Column width \( 553 \text{ mm} \)
Column height \( 3000 \text{ mm} \)
Area of reinft to be provided \( 804.25 \text{ mm}^2 \)
Diameter of reinforcing bars \( 16 \text{ mm} \)
No of bars to be used \( 4 \)
Tensile strength of reinft \( 500 \text{ N/mm}^2 \)
Compressive strength of units \( 20 \text{ N/mm}^2 \)
Mortar designation \( 2 \)
Partial safety factors:
Masonry \( 2.3 \)
Reinforcement \( 1.15 \)
Masonry units to be Group 1 (i.e. solid)
Reinforced masonry column under vertical load to EC6 Part 1-1

Section A-A

$G_k + Q_k$

Brick column

Group 1 units to be used.
First course of bricks is shown with a four bar reinforcement arrangement.

t = thickness of column in plane of bending
d1 = distance to comp. reinft. from col. face

Thicknes of column $t=440$ mm
Width of column $b=440$ mm
Height of column $h=6000$ mm
Support restraint factor $r_f=1$
Permanent vertical load $G_k=200$ kN
Variable vertical load $Q_k=296$ kN
Column self weight $G_{ksw}=0$ kN
Load eccentricity $e=0$ mm
Partial safety factor (masonry) $\gamma_m=2.3$
Char tensile strength of reinft. $f_y=500$ N/mm$^2$
Mortar designation (Table NA.2) mortar=2
Compressive strength of units $p_o=30$ N/mm$^2$
Constant $K$ from NA Table NA.4 $K(1)=0.5$
Height of masonry unit $h_u=65$ mm
Least horiz. dimension of units $l_{hd}=100$ mm
Diameter of reinforcing bars $d_{iam}=25$ mm
Total number of bars to be used $Bars=4$
column compression face $d_{1}=150$ mm
Link diameter $d_{iam}=6$

DESIGN SUMMARY

Design vertical load 714 kN
Design vertical load resistance 812.55 kN
Column depth 440 mm
Column width 440 mm
Column height 6000 mm
Area of reinft to be provided 1963.5 mm$^2$
Diameter of reinforcing bars 25 mm
No of bars to be used 4
Tensile strength of reinft 500 N/mm$^2$
Compressive strength of units 30 N/mm$^2$
Mortar designation 2
Partial safety factors:
Masonry 2.3
Reinforcement 1.15
Masonry units to be Group 1 (i.e. solid)
Location: R3 Example 4 by J Roberts (Hollow Concrete Block column)

Reinforced masonry column under vertical load to EC6 Part 1-1

Section A-A

Thickness of column \( t = 440 \text{ mm} \)
Width of column \( b = 440 \text{ mm} \)
Height of column \( h = 6000 \text{ mm} \)
Support restraint factor \( rf = 1 \)
Permanent vertical load \( Gk = 0 \text{ kN} \)
Variable vertical load \( Qk = 266.67 \text{ kN} \)
Column self weight \( Gksw = 0 \text{ kN} \)
Load eccentricity \( e = 112.5 \text{ mm} \)
Partial safety factor (masonry) \( \gamma_M = 2 \)
Char tensile strength of reinft. \( f_y = 500 \text{ N/mm}^2 \)
Mortar designation (Table NA.2) \( \text{mortar} = 2 \)
Compressive strength of units \( p_o = 10.4 \text{ N/mm}^2 \)
Percentage of solid masonry \( p_{sm} = 65 \% \)
Diameter of reinforcing bars \( \text{diam} = 25 \text{ mm} \)
Total number of bars to be used \( \text{Bars} = 4 \)
column compression face \( d_1 = 107.5 \text{ mm} \)
Link diameter \( \text{diam}_l = 8 \)
Concrete cover to reinforcement \( \text{cover} = 25 \text{ mm} \)

DESIGN SUMMARY

Design vertical load \( 400.01 \text{ kN} \)
Design vertical load resistance \( 871.89 \text{ kN} \)
Column depth \( 440 \text{ mm} \)
Column width \( 440 \text{ mm} \)
Column height \( 6000 \text{ mm} \)
Area of reinf to be provided \( 1963.5 \text{ mm}^2 \)
Diameter of reinforcing bars \( 25 \text{ mm} \)
No of bars to be used \( 4 \)
Tensile strength of reinf \( 500 \text{ N/mm}^2 \)
Compressive strength of units \( 10.4 \text{ N/mm}^2 \)
Mortar designation \( 2 \)
Partial safety factors:
Masonry \( 2 \)
Reinforcement \( 1.15 \)
Location: Problem 1 in Orton's paper 'Structural Design of Masonry'

**Load distribution to shear walls**

- **Fx**: Resultant wind force in x-direction
- **Fy**: Resultant wind force in y-direction
- **sx**: Shear centre
- **yr**: Distance from origin to centroid of wall in y-dir.

Number of walls resisting movement in x-dirn: \( nx = 4 \)
Number of walls resisting movement in y-dirn: \( ny = 2 \)
Resultant wind force in x-direct. (unfactored) \( F_x = 20.82 \text{ kN} \)
Distance of this force from the origin \( y_m = 3.5 \text{ m} \)
Resultant wind force in y-direct. (unfactored) \( F_y = 0 \text{ kN} \)

**Resistance of walls in x-direction**

**Wall 1**
- Inertia of wall to resist force in x-direction: \( I_{x(i)} = 0.8 \text{ m}^4 \)
- Dist. from origin to centroid of wall in y-dir.: \( y(1) = 7 \text{ m} \)

**Wall 2**
- Inertia of wall to resist force in x-direction: \( I_{x(i)} = 0.3375 \text{ m}^4 \)
- Dist. from origin to centroid of wall in y-dir.: \( y(2) = 0 \text{ m} \)

**Wall 3**
- Inertia of wall to resist force in x-direction: \( I_{x(i)} = 0.0125 \text{ m}^4 \)
- Dist. from origin to centroid of wall in y-dir.: \( y(3) = 4 \text{ m} \)

**Wall 4**
- Inertia of wall to resist force in x-direction: \( I_{x(i)} = 0.0125 \text{ m}^4 \)
- Dist. from origin to centroid of wall in y-dir.: \( y(4) = 4 \text{ m} \)
Resistance of walls in y-direction

Wall 1

Inertia of wall to resist force in y-direction $I_y(i)=0.0125 \, \text{m}^4$
Dist. from origin to centroid of wall in x-dir. $x(i)=0 \, \text{m}$

Wall 2

Inertia of wall to resist force in y-direction $I_y(i)=0.0125 \, \text{m}^4$
Dist. from origin to centroid of wall in x-dir. $x(2)=4 \, \text{m}$

RESULTS

Eccentricity of force $F_x$ from shear centre $e_y= y_m-y_c=-1.4032 \, \text{m}$

Load resistance of individual walls in x-direction

Wall 1:
Load resistance
Proportion of total force resisted by this wall $p_x(1)=f_x(i)/F_x*100$
$=48.788 \%$
Since torsional component reduces calculated load resistance of this wall component, Orton suggests this term be ignored. If this is done, Load resistance $f_x(1)=F_x*I_x(i)/\sigma I_{x_r}=14.328 \, \text{kN}$
Proportion of total force resisted by this wall $p_x(1)=f_x(i)/F_x*100$
$=68.817 \%$

Wall 2:
Load resistance
Proportion of total force resisted by this wall $p_x(2)=f_x(i)/F_x*100$
$=48.792 \%$

Wall 3:
Load resistance
Proportion of total force resisted by this wall $p_x(3)=f_x(i)/F_x*100$
$=1.2101 \%$

Wall 4:
Load resistance
Proportion of total force resisted by this wall $p_x(4)=f_x(i)/F_x*100$
$=1.2101 \%$

Load resistance of individual walls in y-direction

Wall 1:
Load resistance
Wall 2:
Load resistance

NOTE: Negative value indicates force on wall acts in opposite direction to that of applied wind load.
Location: Option 2 with calculation of inertias from wall dimensions

Load distribution to shear walls

\[ F_x = \text{Resultant wind force x-direction} \]
\[ F_y = \text{Resultant wind force y-direction} \]
\[ o = \text{Origin} \]

Number of walls resisting movement in x-dirn \( n_x = 4 \)
Number of walls resisting movement in y-dirn \( n_y = 4 \)

Resultant wind force in x-direct. (unfactored) \( F_x = 50 \text{ kN} \)
Distance of this force from the origin \( y_m = 3 \text{ m} \)
Resultant wind force in y-direct. (unfactored) \( F_y = 30 \text{ kN} \)
Distance of this force from the origin \( x_m = 9 \text{ m} \)

Resistance of walls in x-direction

Wall 1
Inertia of wall to resist force in x-direction \( I_x(i) = 1 \text{ m}^4 \)
Dist. from origin to centroid of wall in y-dir. \( y(1) = 0.5 \text{ m} \)

Wall 2
Inertia of wall to resist force in x-direction \( I_x(i) = 42.667 \text{ m}^4 \)
Dist. from origin to centroid of wall in y-dir. \( y(2) = 6 \text{ m} \)

Wall 3
Inertia of wall to resist force in x-direction \( I_x(i) = 42.667 \text{ m}^4 \)
Dist. from origin to centroid of wall in y-dir. \( y(3) = 11.5 \text{ m} \)

Wall 4
Inertia of wall to resist force in x-direction \( I_x(i) = 1 \text{ m}^4 \)
Dist. from origin to centroid of wall in y-dir. \( y(4) = 11.5 \text{ m} \)
Resistance of walls in y-direction

Wall 1
Inertia of wall to resist force in y-direction $I_y(i)=144 \text{ m}^4$
Dist. from origin to centroid of wall in x-dir. $x(1)=6 \text{ m}$

Wall 2
Inertia of wall to resist force in y-direction $I_y(i)=0.66667 \text{ m}^4$
Dist. from origin to centroid of wall in x-dir. $x(2)=0.5 \text{ m}$

Wall 3
Inertia of wall to resist force in y-direction $I_y(i)=0.66667 \text{ m}^4$
Dist. from origin to centroid of wall in x-dir. $x(3)=11.5 \text{ m}$

Wall 4
Inertia of wall to resist force in y-direction $I_y(i)=144 \text{ m}^4$
Dist. from origin to centroid of wall in x-dir. $x(4)=6 \text{ m}$

RESULTS
Eccentricity of force $F_x$ from shear centre $e_y=y_m-y_c=-5.687 \text{ m}$
Eccentricity of force $F_y$ from shear centre $e_x=x_m-x_c=3 \text{ m}$

Load resistance of individual walls in x-direction

Wall 1 :
Load resistance
Proportion of total force resisted by this wall $p_x(1)=f_x(i)/F_x*100$
=5.3271 %

Wall 2 :
Load resistance
Proportion of total force resisted by this wall $p_x(2)=f_x(i)/F_x*100$
=107.42 %

Wall 3 :
Load resistance
Proportion of total force resisted by this wall $p_x(3)=f_x(i)/F_x*100$
=-12.453 %
Note: Negative value indicates force on wall acts in opposite direction to that of applied wind load.

Wall 4 :
Load resistance
Proportion of total force resisted by this wall $p_x(4)=f_x(i)/F_x*100$
=-0.29187 %
Note: Negative value indicates force on wall acts in opposite direction to that of applied wind load.
Load resistance of individual walls in y-direction

Wall 1:
Load resistance
Proportion of total force resisted by this wall \( py(1) = \frac{f_y(i)}{F_y} \times 100 \)
\[ = 49.77 \% \]

Wall 2:
Load resistance
Proportion of total force resisted by this wall \( py(2) = \frac{f_y(i)}{F_y} \times 100 \)
\[ = 3.3521 \% \]

Wall 3:
Load resistance
Proportion of total force resisted by this wall \( py(3) = \frac{f_y(i)}{F_y} \times 100 \)
\[ = -2.8912 \% \]

NOTE: Negative value indicates force on wall acts in opposite direction to that of applied wind load.

Wall 4:
Load resistance
Proportion of total force resisted by this wall \( py(4) = \frac{f_y(i)}{F_y} \times 100 \)
\[ = 49.77 \% \]
Location: Ex3 - Demonstrates Option 1 input

Load distribution to shear walls

Shear wall r with inertia Ixr
Shear centre

Fx = Resultant wind force x-direction

Fy = Resultant wind force y-direction

o = Origin

Number of walls resisting movement n=4

Wall 1

x-coord. of bottom left-hand corner of wall xs(1)=0 m
y-coord. of bottom left-hand corner of wall ys(1)=0 m
x-coord. of top right-hand corner of wall xe(1)=12 m
y-coord. of top right-hand corner of wall ye(1)=1 m

Wall 2

x-coord. of bottom left-hand corner of wall xs(2)=0 m
y-coord. of bottom left-hand corner of wall ys(2)=2 m
x-coord. of top right-hand corner of wall xe(2)=1 m
y-coord. of top right-hand corner of wall ye(2)=10 m

Wall 3

x-coord. of bottom left-hand corner of wall xs(3)=11 m
y-coord. of bottom left-hand corner of wall ys(3)=2 m
x-coord. of top right-hand corner of wall xe(3)=12 m
y-coord. of top right-hand corner of wall ye(3)=10 m

Wall 4

x-coord. of bottom left-hand corner of wall xs(4)=0 m
y-coord. of bottom left-hand corner of wall ys(4)=11 m
x-coord. of top right-hand corner of wall xe(4)=12 m
y-coord. of top right-hand corner of wall ye(4)=12 m

Resultant wind force in x-direct. (unfactored) Fx=50 kN
Distance of this force from the origin ym=3 m
Resultant wind force in y-direct. (unfactored) Fy=30 kN
Distance of this force from the origin xm=9 m
**Resistance of walls in x-direction**

**Wall 1**

Inertia of wall to resist force in x-direction $I_x(i)=144 \text{ m}^4$
Dist. from origin to centroid of wall in y-dir. $y(i)=0.5 \text{ m}$

**Wall 2**

Inertia of wall to resist force in x-direction $I_x(i)=0.66667 \text{ m}^4$
Dist. from origin to centroid of wall in y-dir. $y(i)=6 \text{ m}$

**Wall 3**

Inertia of wall to resist force in x-direction $I_x(i)=0.66667 \text{ m}^4$
Dist. from origin to centroid of wall in y-dir. $y(i)=6 \text{ m}$

**Wall 4**

Inertia of wall to resist force in x-direction $I_x(i)=144 \text{ m}^4$
Dist. from origin to centroid of wall in y-dir. $y(i)=11.5 \text{ m}$

**Resistance of walls in y-direction**

**Wall 1**

Inertia of wall to resist force in y-direction $I_y(i)=1 \text{ m}^4$
Dist. from origin to centroid of wall in x-dir. $x(i)=6 \text{ m}$

**Wall 2**

Inertia of wall to resist force in y-direction $I_y(i)=42.667 \text{ m}^4$
Dist. from origin to centroid of wall in x-dir. $x(i)=0.5 \text{ m}$

**Wall 3**

Inertia of wall to resist force in y-direction $I_y(i)=42.667 \text{ m}^4$
Dist. from origin to centroid of wall in x-dir. $x(i)=11.5 \text{ m}$

**Wall 4**

Inertia of wall to resist force in y-direction $I_y(i)=1 \text{ m}^4$
Dist. from origin to centroid of wall in x-dir. $x(i)=6 \text{ m}$

**RESULTS**

Eccentricity of force $F_x$ from shear centre $e_y= y_m- y_c = -3 \text{ m}$
Eccentricity of force $F_y$ from shear centre $e_x= x_m- x_c = 3 \text{ m}$
Load resistance of individual walls in x-direction

Wall 1:
Load resistance
Proportion of total force resisted by this wall \( p_x(1) = \frac{f_x(i)}{F_x} \times 100 \)
\[= 58.185 \% \]

Wall 2:
Load resistance
Proportion of total force resisted by this wall \( p_x(2) = \frac{f_x(i)}{F_x} \times 100 \)
\[= 0.23041 \% \]

Wall 3:
Load resistance
Proportion of total force resisted by this wall \( p_x(3) = \frac{f_x(i)}{F_x} \times 100 \)
\[= 0.23041 \% \]

Wall 4:
Load resistance
Proportion of total force resisted by this wall \( p_x(4) = \frac{f_x(i)}{F_x} \times 100 \)
\[= 41.354 \% \]

Since torsional component reduces calculated load resistance of this wall component, Orton suggests this term be ignored. If this is done,
Load resistance
\( f_x(4) = \frac{F_x \times I_x(i)}{\sigma I_{xr}} = 24.885 \text{ kN} \)
Proportion of total force resisted by this wall \( p_x(4) = \frac{f_x(i)}{F_x} \times 100 \)
\[= 49.77 \% \]

Load resistance of individual walls in y-direction

Wall 1:
Load resistance
Proportion of total force resisted by this wall \( p_y(1) = \frac{f_y(i)}{F_y} \times 100 \)
\[= 1.145 \% \]

Wall 2:
Load resistance
Proportion of total force resisted by this wall \( p_y(2) = \frac{f_y(i)}{F_y} \times 100 \)
\[= 53.011 \% \]

Wall 3:
Load resistance
Proportion of total force resisted by this wall \( p_y(3) = \frac{f_y(i)}{F_y} \times 100 \)
\[= 44.699 \% \]

Since torsional component reduces calculated load resistance of this wall component, Orton suggests this term be ignored. If this is done,
Load resistance
\( f_y(3) = \frac{F_y \times I_y(i)}{\sigma I_{yr}} = 14.656 \text{ kN} \)
Proportion of total force resisted by this wall \( p_y(3) = \frac{f_y(i)}{F_y} \times 100 \)
\[= 48.855 \% \]

Wall 4:
Load resistance
Proportion of total force resisted by this wall \( p_y(4) = \frac{f_y(i)}{F_y} \times 100 \)
\[= 1.145 \% \]
Location: Masonry support angle on grid line A/1-2

Reference


![Typical support angle arrangement](image)

- **Thickness of masonry panel**: t=102 mm
- **Nominal cavity width**: c=50 mm
- **Nominal thickness of shims**: s=15 mm
- **Fixing spacing**: fs=500 mm
- **Panel self weight line of action**: e=51 mm
- **Fixing lever arm**: h=75 mm
- **Actual horizontal leg length**: L=110 mm
- **Panel self weight**: W=3.5 kN/m
- **Max allowable bending stress**: fmax=140 N/mm²
- **Modulus of elasticity**: E=195000 N/mm²
- **Poisson's ratio**: v=0.3
- **Maximum allowable deflection**: delta=1.5 mm
- **Actual vertical leg length**: VL=120 mm
- **Actual vertical leg length**: VL=120 mm

**DESIGN SUMMARY**

- **Horizontal leg length**: 110 mm
- **Vertical leg length**: 120 m
- **Thickness**: 5 mm
- **Allowable bending stress**: 140 N/mm²
- **Wp at support**: 5.5373 kN/m
- **Wq at quarter span**: 3.3529 kN/m
- **Wr at midspan**: 1.7568 kN/m
- **Trans bend stress - support**: 114.29 N/mm²
- **Trans bend stress - 1/4 span**: 69.205 N/mm²
- **Trans bend stress at midspan**: 36.26 N/mm²
- **Defln at angle toe (support)**: 1.4646 mm
- **Defln at angle toe (midspan)**: 1.4096 mm
Location: Masonry support angle on grid line A/1-2

Reference


Thickness of masonry panel \( t = 102 \text{ mm} \)
Nominal cavity width \( c = 50 \text{ mm} \)
Nominal thickness of shims \( s = 15 \text{ mm} \)
Fixing spacing \( f_s = 500 \text{ mm} \)
Panel self weight line of action \( e = 51 \text{ mm} \)
Fixing lever arm \( h = 75 \text{ mm} \)
Actual horizontal leg length \( L = 110 \text{ mm} \)
Panel self weight \( W = 3.5 \text{ kN/m} \)
Max allowable bending stress \( f_{\text{max}} = 140 \text{ N/mm}^2 \)
Modulus of elasticity \( E = 195000 \text{ N/mm}^2 \)
Poisson's ratio \( v = 0.3 \)
Maximum allowable deflection \( \Delta = 1.5 \text{ mm} \)
Actual vertical leg length \( V_L = 110 \text{ mm} \)
Actual vertical leg length \( V_L = 110 \text{ mm} \)

DESIGN SUMMARY

- Horizontal leg length \( 110 \text{ mm} \)
- Vertical leg length \( 110 \text{ mm} \)
- Thickness \( 5 \text{ mm} \)
- Allowable bending stress \( 140 \text{ N/mm}^2 \)
- \( W_p \) at support \( 5.6575 \text{ kN/m} \)
- \( W_q \) at quarter span \( 3.3214 \text{ kN/m} \)
- \( W_r \) at midspan \( 1.6997 \text{ kN/m} \)
- Trans bend stress - support \( 116.77 \text{ N/mm}^2 \)
- Trans bend stress - 1/4 span \( 68.554 \text{ N/mm}^2 \)
- Trans bend stress at midspan \( 35.081 \text{ N/mm}^2 \)
- Defln at angle toe (support) \( 1.4964 \text{ mm} \)
- Defln at angle toe (midspan) \( 1.4388 \text{ mm} \)
Location: Masonry support angle on grid line A/1-2

Reference


Typical support angle arrangement

Leg facing up

Thickness of masonry panel \( t = 300 \) mm
Nominal cavity width \( c = 50 \) mm
Nominal thickness of shims \( s = 25 \) mm
Fixing spacing \( f_s = 500 \) mm
Panel self weight line of action \( e = 150 \) mm
Fixing lever arm \( h = 50 \) mm
Actual horizontal leg length \( L = 230 \) mm
Panel self weight \( W = 2 \) kN/m
Max allowable bending stress \( f_{\text{max}} = 350 \) N/mm²
Modulus of elasticity \( E = 195000 \) N/mm²
Poisson's ratio \( v = 0.3 \)
Maximum allowable deflection \( \delta = 25 \) mm
Actual vertical leg length \( V_L = 90 \) mm

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal leg length</td>
<td>230 mm</td>
</tr>
<tr>
<td>Vertical leg length</td>
<td>90 m</td>
</tr>
<tr>
<td>Thickness</td>
<td>3 mm</td>
</tr>
<tr>
<td>Allowable bending stress</td>
<td>350 N/mm²</td>
</tr>
<tr>
<td>Wp at support</td>
<td>2.6666 kN/m</td>
</tr>
<tr>
<td>Wq at quarter span</td>
<td>1.9203 kN/m</td>
</tr>
<tr>
<td>Wr at midspan</td>
<td>1.4928 kN/m</td>
</tr>
<tr>
<td>Trans bend stress - support</td>
<td>311.1 N/mm²</td>
</tr>
<tr>
<td>Trans bend stress - 1/4 span</td>
<td>224.03 N/mm²</td>
</tr>
<tr>
<td>Trans bend stress at midspan</td>
<td>174.16 N/mm²</td>
</tr>
<tr>
<td>Defln at angle toe (support)</td>
<td>20.053 mm</td>
</tr>
<tr>
<td>Defln at angle toe (midspan)</td>
<td>19.302 mm</td>
</tr>
</tbody>
</table>
Location: Ex1 - Masonry support angle on grid line A/1-2

Reference

Typical support angle arrangement
leg facing up

Thickness of masonry panel: t=102 mm
Nominal cavity width: c=50 mm
Nominal thickness of shims: s=15 mm
Fixing spacing: fs=500 mm
Panel self weight line of action: e=51 mm
Fixing lever arm: h=75 mm
Actual horizontal leg length: L=110 mm
Panel self weight (unfactored): W=3.5 kN/m
Modulus of elasticity: E=210000 N/mm²
Poisson's ratio: v=0.3
Maximum allowable deflection: delta=1.5 mm
Actual vertical leg length: VL=120 mm
Actual vertical leg length: VL=120 mm

DESIGN
Support angle horizontal leg length: 110 mm
SUMMARY
Support angle vertical leg length: 120 mm
Thickness: 5 mm
Support angle grade: S 275
Char allowable bending stress: 140 N/mm²
Support load/unit length: 5.5373 kN/m
Quarter span load/unit length: 3.3529 kN/m
Midspan load/unit length: 1.7568 kN/m
Transverse bending stress @ support: 114.29 N/mm²
Transverse bending stress @ 1/4 span: 69.205 N/mm²
Transverse bending stress @ midspan: 36.26 N/mm²
Deflection at angle toe (support): 1.36 mm
Deflection at angle toe (midspan): 1.3089 mm
Location: Ex2 - Masonry support angle on grid line A/1-2

Reference


Typical support angle arrangement
leg facing down

Thickness of masonry panel \( t = 102 \text{ mm} \)
Nominal cavity width \( c = 50 \text{ mm} \)
Nominal thickness of shims \( s = 15 \text{ mm} \)
Fixing spacing \( f_s = 500 \text{ mm} \)
Panel self weight line of action \( e = 51 \text{ mm} \)
Fixing lever arm \( h = 75 \text{ mm} \)
Actual horizontal leg length \( L = 110 \text{ mm} \)
Panel self weight (unfactored) \( W = 3.5 \text{ kN/m} \)
Modulus of elasticity \( E = 210000 \text{ N/mm}^2 \)
Poisson's ratio \( v = 0.3 \)
Maximum allowable deflection \( \delta = 1.5 \text{ mm} \)
Actual vertical leg length \( V_L = 110 \text{ mm} \)

DESIGN

- Support angle horizontal leg length \( 110 \text{ mm} \)
- Support angle vertical leg length \( 110 \text{ mm} \)
- Thickness \( 5 \text{ mm} \)
- Support angle grade \( S 275 \)
- Char allowable bending stress \( 140 \text{ N/mm}^2 \)
- Support load/unit length \( 5.6575 \text{ kN/m} \)
- Quarter span load/unit length \( 3.3214 \text{ kN/m} \)
- Midspan load/unit length \( 1.6997 \text{ kN/m} \)
- Transverse bending stress @ support \( 116.77 \text{ N/mm}^2 \)
- Transverse bending stress @ 1/4 span \( 68.554 \text{ N/mm}^2 \)
- Transverse bending stress @ midspan \( 35.081 \text{ N/mm}^2 \)
- Deflection at angle toe (support) \( 1.3895 \text{ mm} \)
- Deflection at angle toe (midspan) \( 1.336 \text{ mm} \)
Location: Ex3 - Masonry support angle on grid line A/1-2

Reference


Thickness of masonry panel \( t = 300 \text{ mm} \)
Nominal cavity width \( c = 50 \text{ mm} \)
Nominal thickness of shims \( s = 25 \text{ mm} \)
Fixing spacing \( f_s = 500 \text{ mm} \)
Panel self weight line of action \( e = 150 \text{ mm} \)
Fixing lever arm \( h = 50 \text{ mm} \)
Actual horizontal leg length \( L = 230 \text{ mm} \)
Panel self weight (unfactored) \( W = 2 \text{ kN/m} \)
Modulus of elasticity \( E = 210000 \text{ N/mm}^2 \)
Poisson's ratio \( v = 0.3 \)
Maximum allowable deflection \( \delta = 25 \text{ mm} \)
Actual vertical leg length \( V_L = 90 \text{ mm} \)
Actual vertical leg length \( V_L = 90 \text{ mm} \)

DESIGN
Support angle horizontal leg length \( 230 \text{ mm} \)

SUMMARY
Support angle vertical leg length \( 90 \text{ mm} \)
Thickness \( 4 \text{ mm} \)
Support angle grade \( S 355 \)
Char allowable bending stress \( 250 \text{ N/mm}^2 \)
Support load/unit length \( 2.6694 \text{ kN/m} \)
Quarter span load/unit length \( 1.9197 \text{ kN/m} \)
Midspan load/unit length \( 1.4913 \text{ kN/m} \)
Transverse bending stress @ support \( 175.18 \text{ N/mm}^2 \)
Transverse bending stress @ 1/4 span \( 125.98 \text{ N/mm}^2 \)
Transverse bending stress @ midspan \( 97.864 \text{ N/mm}^2 \)
Deflection at angle toe (support) \( 7.8639 \text{ mm} \)
Deflection at angle toe (midspan) \( 7.5686 \text{ mm} \)
Location: Ex1 - Centre arch

Pippard/MEXE analysis for masonry arch bridges (BA 16/97 Section 4)

Analysis for dead load and applied unit live load

Frame analysis:

Number of joints 13
Number of members 12

Radial thickness 0.343 m
Young's modulus 14E6 kN/m²
Shear modulus 5.83E6 kN/m²

Joint Coordinates:

<table>
<thead>
<tr>
<th>Joint No.</th>
<th>X Coordinate (m)</th>
<th>Y Coordinate (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0.58</td>
<td>0.54</td>
</tr>
<tr>
<td>3</td>
<td>1.08</td>
<td>0.83</td>
</tr>
<tr>
<td>4</td>
<td>1.36</td>
<td>0.96</td>
</tr>
<tr>
<td>5</td>
<td>1.58</td>
<td>1.03</td>
</tr>
<tr>
<td>6</td>
<td>2.08</td>
<td>1.14</td>
</tr>
<tr>
<td>7</td>
<td>2.58</td>
<td>1.18</td>
</tr>
<tr>
<td>8</td>
<td>3.08</td>
<td>1.14</td>
</tr>
<tr>
<td>9</td>
<td>3.58</td>
<td>1.03</td>
</tr>
<tr>
<td>10</td>
<td>3.8</td>
<td>0.96</td>
</tr>
<tr>
<td>11</td>
<td>4.08</td>
<td>0.83</td>
</tr>
<tr>
<td>12</td>
<td>4.58</td>
<td>0.54</td>
</tr>
<tr>
<td>13</td>
<td>5.16</td>
<td>0</td>
</tr>
</tbody>
</table>

Coordinates of road surface:

Position 1  x=0 m       y=1.68 m
Position 2  x=4.9 m     y=1.68 m
Axle Load Cases

No. of axle cases analysed 3

<table>
<thead>
<tr>
<th>Axle Load Case</th>
<th>Axle Location From Left Springing</th>
<th>Transverse Load Distribution</th>
<th>Effective Width</th>
<th>No. of Axles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>*</td>
<td></td>
<td>6.2</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>0.5</td>
<td></td>
<td>6.25</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>1.7</td>
<td></td>
<td>5.8</td>
<td>2</td>
</tr>
</tbody>
</table>

* Part axle load disperses behind left springing.

Material densities:

- Arch ring: 21 kN/m³
- Top 100mm surfacing: 24 kN/m³
- Fill material: 19 kN/m³

Partial load factors (BD 21/01 Table 3.1)

<table>
<thead>
<tr>
<th>Material</th>
<th>ULS</th>
<th>SLS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arch ring</td>
<td>1.15</td>
<td>1</td>
</tr>
<tr>
<td>Top 100mm surfacing</td>
<td>1.75</td>
<td>1.2</td>
</tr>
<tr>
<td>Fill material</td>
<td>1.2</td>
<td>1</td>
</tr>
</tbody>
</table>

Characteristic masonry strength $f_k = 5000$ kN/m²

Width factor $F_w$ (Table 3/3) $F_w = 0.9$
Mortar factor $F_m$ (Table 3/4) $F_m = 0.9$
Depth factor $F_d$ (Table 3/5) $F_d = 0.9$

Joint factor $F_j$ (Clause 3.16) $F_j = F_w \times F_d \times F_m = 0.729$
Condition factor $F_{CM}$ (Clause 3.17) $F_{CM} = 0.7$

Partial live load factor $g_{fll} = 3.4$

Allowable single axle loads

<table>
<thead>
<tr>
<th>Axle Load Case</th>
<th>Allowable Axle Load (tonnes)</th>
<th>Member</th>
<th>Member End</th>
<th>Dead Load Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>26.762</td>
<td>1</td>
<td>2</td>
<td>Ultimate</td>
</tr>
<tr>
<td>2</td>
<td>13.883</td>
<td>4</td>
<td>2</td>
<td>Ultimate</td>
</tr>
<tr>
<td>3</td>
<td>14.008</td>
<td>6</td>
<td>1</td>
<td>Ultimate</td>
</tr>
</tbody>
</table>

Axle load case 2

Minimum allowable single axle load 13.883 tonnes
Stresses at End 2 of Member 4
Axle load combined with ultimate dead load.
Span of arch \( L = 4.9 \) m
Centrifugal effect factor \( Fa = 1.0 \)

**DESIGN SUMMARY**

Allowable axle loads are rounded off to the nearest 0.5 tonnes
Rounded values of allowable axle loads - no lift-off case

<table>
<thead>
<tr>
<th>Type of Vehicle</th>
<th>Allowable Axle Load (tonnes) per axle</th>
<th>Max Gross Vehicle Weight (tonnes)</th>
<th>Weight Restriction (tonnes)</th>
<th>Vehicle Permitted</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Single</td>
<td>Double</td>
<td>Triple</td>
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<tr>
<td>HGV axles</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 or 6</td>
<td>11.5</td>
<td>10.0</td>
<td>8.0</td>
<td>40/44</td>
</tr>
<tr>
<td>4</td>
<td>11.5</td>
<td>9.5</td>
<td></td>
<td>32</td>
</tr>
<tr>
<td>3</td>
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<td></td>
<td>26</td>
</tr>
<tr>
<td>2</td>
<td>11.5</td>
<td></td>
<td></td>
<td>18</td>
</tr>
<tr>
<td>2</td>
<td>9.0</td>
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<td></td>
<td>13</td>
</tr>
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<td>10</td>
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<tr>
<td>LGV</td>
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<td></td>
<td>7.5</td>
</tr>
<tr>
<td>Car/Van</td>
<td>2.0</td>
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<td></td>
<td>3</td>
</tr>
<tr>
<td>Fire Eng.</td>
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</tr>
<tr>
<td>Group 1</td>
<td>10.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Group 2</td>
<td>5.0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex1 - Centre arch

Pippard/MEXE analysis for masonry arch bridges (BA 16/97 Section 4)

C & U, AW or User Defined Single Axles and Bogies

Frame analysis:

Number of joints: 13
Number of members: 12
Radial thickness: 0.343 m
Young's modulus: 14E6 kN/m²
Shear modulus: 5.83E6 kN/m²

Joint Coordinates:

<table>
<thead>
<tr>
<th>Joint No.</th>
<th>X Coordinate (m)</th>
<th>Y Coordinate (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0.58</td>
<td>0.54</td>
</tr>
<tr>
<td>3</td>
<td>1.08</td>
<td>0.83</td>
</tr>
<tr>
<td>4</td>
<td>1.36</td>
<td>0.96</td>
</tr>
<tr>
<td>5</td>
<td>1.58</td>
<td>1.03</td>
</tr>
<tr>
<td>6</td>
<td>2.08</td>
<td>1.14</td>
</tr>
<tr>
<td>7</td>
<td>2.58</td>
<td>1.18</td>
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<tr>
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<td>4.58</td>
<td>0.54</td>
</tr>
<tr>
<td>13</td>
<td>5.16</td>
<td>0</td>
</tr>
</tbody>
</table>

Coordinates of road surface:

Position 1: x=0 m, y=1.68 m
Position 2: x=4.9 m, y=1.68 m
Live Load Cases:

No. of live load cases analysed 3

Nominal loads:

<table>
<thead>
<tr>
<th>Live Load Case</th>
<th>Bogie Ref.</th>
<th>No. of axles</th>
<th>O/A Minimum Axle Spread (m)</th>
<th>Gross Bogie Weight (tonnes)</th>
<th>Crit. Axle</th>
<th>Axle 1 Location x1 (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>10.5</td>
<td>1</td>
<td>1.4</td>
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<tr>
<td>2</td>
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<td>2</td>
<td>1.85</td>
<td>20.34</td>
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<td>3</td>
<td>2.7</td>
<td>22.5</td>
<td>1</td>
<td>1.4</td>
</tr>
</tbody>
</table>

where, x1 is the distance to Axle 1 from left springing (Joint 1)

ULS Load factors:

<table>
<thead>
<tr>
<th>Live Load Case</th>
<th>Axle 1</th>
<th>Axle 2</th>
<th>Axle 3</th>
<th>Axle 1</th>
<th>Axle 2</th>
<th>Axle 3</th>
<th>Axle 1</th>
<th>Axle 2</th>
<th>Axle 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BD 21/01 Table 6.2 Factors</td>
<td>BD 21/01 Clause 6.20 Factors</td>
<td>Axle Factors</td>
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</tr>
<tr>
<td></td>
<td>Axle 1</td>
<td>Axle 2</td>
<td>Axle 3</td>
<td>Axle 1</td>
<td>Axle 2</td>
<td>Axle 3</td>
<td>Axle 1</td>
<td>Axle 2</td>
<td>Axle 3</td>
</tr>
<tr>
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<td>1</td>
<td>1.28</td>
<td>0.5</td>
<td>3.4</td>
<td>1.9</td>
<td>1.9</td>
<td>3.4</td>
<td>0.95</td>
<td>4.352</td>
</tr>
<tr>
<td>2</td>
<td>*</td>
<td>0.5</td>
<td>1</td>
<td>1.9</td>
<td>3.4</td>
<td>1.9</td>
<td>4.352</td>
<td>1.9</td>
<td>0.95</td>
</tr>
<tr>
<td>3</td>
<td>*</td>
<td>1.28</td>
<td>1</td>
<td>3.4</td>
<td>1.9</td>
<td>1.9</td>
<td>4.352</td>
<td>1.9</td>
<td>0.95</td>
</tr>
</tbody>
</table>

* Axle lift off case

Transverse load distribution:

<table>
<thead>
<tr>
<th>Live Load Case</th>
<th>Axle 1</th>
<th>Axle 2</th>
<th>Axle 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Effective Width</td>
<td>No. of Axles</td>
<td>Effective Width</td>
</tr>
<tr>
<td>1</td>
<td>6.3</td>
<td>2</td>
<td>6.3</td>
</tr>
<tr>
<td>2</td>
<td>6.2</td>
<td>2</td>
<td>6.3</td>
</tr>
<tr>
<td>3</td>
<td>6.3</td>
<td>2</td>
<td>7.1</td>
</tr>
</tbody>
</table>
Material densities:

Arch ring  21 kN/m³
Top 100mm surfacing  24 kN/m³
Fill material  19 kN/m³

Partial load factors ( BD 21/01 Table 3.1 )

<table>
<thead>
<tr>
<th></th>
<th>ULS</th>
<th>SLS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arch ring</td>
<td>1.15</td>
<td>1</td>
</tr>
<tr>
<td>Top 100mm surfacing</td>
<td>1.75</td>
<td>1.2</td>
</tr>
<tr>
<td>Fill material</td>
<td>1.2</td>
<td>1</td>
</tr>
</tbody>
</table>

Characteristic masonry strength  $f_k=5000$ kN/m²

Factors from MEXE method. ( BA 16/97 Chapter 3 ).
Width factor $F_w$ ( Table 3/3 )  $F_w=0.9$
Mortar factor $F_{mo}$ ( Table 3/4 )  $F_{mo}=0.9$
Depth factor $F_d$ ( Table 3/5 )  $F_d=0.9$
Joint factor $F_j$ ( Clause 3.16 )  $F_j=F_w*F_d*F_{mo}=0.729$
Condition factor $F_{cM}$ ( Clause 3.17 )  $F_{cM}=0.7$
Centrifugal effect factor  $F_a=1.0$

**DESIGN SUMMARY**

**Single axles:**

<table>
<thead>
<tr>
<th>Live Load Case</th>
<th>Bogie Description</th>
<th>Axle Spread (m)</th>
<th>Gross Axle Weight (tonnes)</th>
<th>Capacity Factor</th>
<th>Allowable Axle Load (tonnes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C&amp;U Single axle</td>
<td>-</td>
<td>10.5</td>
<td>1.3382</td>
<td>14.051</td>
</tr>
</tbody>
</table>

**Double axles:**

<table>
<thead>
<tr>
<th>Live Load Case</th>
<th>Bogie Description</th>
<th>Axle Spread (m)</th>
<th>Gross Axle Weight (tonnes)</th>
<th>Capacity Factor</th>
<th>Allowable Axle Load (tonnes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>C&amp;U Double axle</td>
<td>1.85</td>
<td>10.17</td>
<td>1.2188</td>
<td>12.395</td>
</tr>
</tbody>
</table>

**Triple axles:**

<table>
<thead>
<tr>
<th>Live Load Case</th>
<th>Bogie Description</th>
<th>Axle Spread (m)</th>
<th>Gross Axle Weight (tonnes)</th>
<th>Capacity Factor</th>
<th>Allowable Axle Load (tonnes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>C&amp;U Triple axle</td>
<td>2.7</td>
<td>7.5</td>
<td>1.3762</td>
<td>10.322</td>
</tr>
</tbody>
</table>
Location: Ex1 - Centre arch

Pippard/MEXE analysis for masonry arch bridges. BA 16/97 Section 4

User defined vehicles

Frame analysis:

Number of joints 13
Number of members 12
Radial thickness 0.45 m
Young's modulus 14E6 kN/m²
Shear modulus 5.83E6 kN/m²

Joint Coordinates:

<table>
<thead>
<tr>
<th>Joint No.</th>
<th>X Coordinate ( m )</th>
<th>Y Coordinate ( m )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>1.143</td>
<td>0.752</td>
</tr>
<tr>
<td>3</td>
<td>2.286</td>
<td>1.405</td>
</tr>
<tr>
<td>4</td>
<td>4.001</td>
<td>2.196</td>
</tr>
<tr>
<td>5</td>
<td>5.715</td>
<td>2.76</td>
</tr>
<tr>
<td>6</td>
<td>7.43</td>
<td>3.099</td>
</tr>
<tr>
<td>7</td>
<td>9.144</td>
<td>3.212</td>
</tr>
<tr>
<td>8</td>
<td>10.858</td>
<td>3.099</td>
</tr>
<tr>
<td>9</td>
<td>12.573</td>
<td>2.76</td>
</tr>
<tr>
<td>10</td>
<td>14.287</td>
<td>2.196</td>
</tr>
<tr>
<td>11</td>
<td>16.002</td>
<td>1.405</td>
</tr>
<tr>
<td>12</td>
<td>17.145</td>
<td>0.752</td>
</tr>
<tr>
<td>13</td>
<td>18.288</td>
<td>0</td>
</tr>
</tbody>
</table>

Coordinates of road surface:

Position 1 x=0 m y=3.55 m
Position 2 x=18 m y=3.55 m

Live Load Cases

Number of live load cases analysed 1
Live Load Case 1

Number of line loads 1

<table>
<thead>
<tr>
<th>Line Load (kN)</th>
<th>Nominal Load (kN)</th>
<th>Load Factor</th>
<th>Factored Load (kN)</th>
<th>Contact Length (m)</th>
<th>Location From Left Springing</th>
<th>Load Disp.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>40</td>
<td>1.5</td>
<td>60</td>
<td>0</td>
<td>6</td>
<td>No</td>
</tr>
</tbody>
</table>

Live Load Case 1

Number of UD loads 1

<table>
<thead>
<tr>
<th>UD Load</th>
<th>Nominal Load (kN/m)</th>
<th>Load Factor</th>
<th>Factored Load (kN/m)</th>
<th>Contact Length (m)</th>
<th>Location From Left Springing</th>
<th>Load Disp.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>30</td>
<td>1.5</td>
<td>45</td>
<td>9.144</td>
<td>9.144</td>
<td>No</td>
</tr>
</tbody>
</table>

Material densities:

- Arch ring: 21 kN/m³
- Top 100mm surfacing: 24 kN/m³
- Fill material: 19 kN/m³

Partial load factors (BD 21/01 Table 3.1):

- ULS: Arch ring 1.15, Top 100mm surfacing 1.75, Fill material 1.2
- SLS: Arch ring 1, Top 100mm surfacing 1.2, Fill material 1

Characteristic masonry strength \( f_k = 10000 \text{ kN/m}^2 \)

- Width factor \( F_w \) (Table 3/3): \( F_w = 1 \)
- Mortar factor \( F_{mo} \) (Table 3/4): \( F_{mo} = 1 \)
- Pointed joints in good condition.
- Depth factor \( F_d \) (Clause 3.16): \( F_d = 1.0 \)
- Joint factor \( F_j \) (Clause 3.16): \( F_j = F_w * F_d * F_{mo} = 1 \)
- Condition factor \( F_cM \) (Clause 3.17): \( F_cM = 0.9 \)
- Centrifugal effect factor \( F_a = 1.0 \)
<table>
<thead>
<tr>
<th>Live load case 1</th>
<th>Final capacity factor</th>
<th>0.77021</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live load case 1</td>
<td>Minimum final capacity factor</td>
<td>0.77021</td>
</tr>
</tbody>
</table>
**Location:** Ex1 - 8 metre span arch

**Assessment of Masonry Arch Bridges by the Modified MEXE Method**

**Departmental Standard BD 21/01 and Advice Note BA 16/97**

Calculations are in accordance with "The Assessment of Highway Bridges and Structures" as described in the following documents:

- Departmental Standard BD 21/01
- Departmental Advice Note BA 16/97 incorporating Amendment No. 2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span of arch</td>
<td>L= 8 m</td>
</tr>
<tr>
<td>Rise of barrel at crown</td>
<td>rc= 2.95 m</td>
</tr>
<tr>
<td>Rise of barrel at quarter points</td>
<td>rq= 2.14 m</td>
</tr>
<tr>
<td>Thickness of barrel adjacent to keystone</td>
<td>d= 0.305 m</td>
</tr>
<tr>
<td>Average depth of fill between the road surface</td>
<td></td>
</tr>
<tr>
<td>and the arch barrel at the crown</td>
<td>h= 0.3 m</td>
</tr>
<tr>
<td>Span rise factor</td>
<td>Fsr=1</td>
</tr>
<tr>
<td>Profile factor</td>
<td>Fp=1.0</td>
</tr>
<tr>
<td>Barrel factor (Table 3/1)</td>
<td>Fb=1.3</td>
</tr>
<tr>
<td>Width factor (Table 3/3)</td>
<td>Fw=0.8</td>
</tr>
<tr>
<td>Mortar factor (Table 3/4)</td>
<td>Fmo=0.9</td>
</tr>
<tr>
<td>Depth of missing mortar</td>
<td>doms=0.01 m</td>
</tr>
<tr>
<td>Depth factor (3.16)</td>
<td>Fd=1.0</td>
</tr>
<tr>
<td>Joint factor (3.16)</td>
<td>Fj=Fw<em>Fd</em>Fmo=0.72</td>
</tr>
<tr>
<td>Fill factor (Table 3/2)</td>
<td>Ff=0.7</td>
</tr>
<tr>
<td>Material factor</td>
<td>Fm=(Fb<em>dr+Ff</em>h)/(dr+h)=0.99748</td>
</tr>
<tr>
<td>Condition factor (3.17)</td>
<td>FcM=0.9</td>
</tr>
<tr>
<td>Provisional axle load (3.10)</td>
<td>PAL=740*(dr+h)^2/L^1.3=17.549 tonnes</td>
</tr>
<tr>
<td>Modified axle load</td>
<td>MAL=Fsr<em>Fp</em>Fm<em>Fj</em>FcM*PAL=11.343 tonnes</td>
</tr>
<tr>
<td>Centrifugal effect factor</td>
<td>Fa=1.0</td>
</tr>
</tbody>
</table>
DESIGN SUMMARY

Allowable axle loads are rounded off to the nearest 0.5 tonnes
Rounded values of allowable axle loads - no lift-off case

<table>
<thead>
<tr>
<th>Allowable Axle Load (tonnes) per axle</th>
<th>Type of Vehicle</th>
<th>Single</th>
<th>Double</th>
<th>Triple</th>
<th>Max Gross Vehicle Weight (tonnes)</th>
<th>Weight Restriction (tonnes)</th>
<th>Vehicle Permitted</th>
</tr>
</thead>
<tbody>
<tr>
<td>17 tonnes</td>
<td>Single axle</td>
<td></td>
<td></td>
<td>17</td>
<td>40/44</td>
<td>N/A</td>
<td>Yes</td>
</tr>
<tr>
<td>11.5 tonnes</td>
<td>Double axle</td>
<td></td>
<td></td>
<td>11.5</td>
<td>32</td>
<td>33</td>
<td>Yes</td>
</tr>
<tr>
<td>10.5 tonnes</td>
<td>Triple axle</td>
<td></td>
<td></td>
<td>10.5</td>
<td>26</td>
<td>26</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Weight Restrictions and Permitted Vehicles (3.30)

<table>
<thead>
<tr>
<th>Type of Vehicle</th>
<th>Single</th>
<th>Double</th>
<th>Triple</th>
<th>Max Gross Vehicle Weight (tonnes)</th>
<th>Weight Restriction (tonnes)</th>
<th>Vehicle Permitted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fire Eng.</td>
<td>10.0</td>
<td></td>
<td></td>
<td>10.0</td>
<td>10</td>
<td>Yes</td>
</tr>
<tr>
<td>Group 1</td>
<td>5.0</td>
<td></td>
<td></td>
<td>5.0</td>
<td>5.0</td>
<td>Yes</td>
</tr>
</tbody>
</table>
Location: Ex1 - Hand analysis example by Professor MR Horne

Arch geometry and loading

<table>
<thead>
<tr>
<th>X</th>
<th>P</th>
<th>i</th>
<th>e</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td>+</td>
<td>d</td>
<td>+</td>
</tr>
<tr>
<td>+</td>
<td>q</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>/</td>
<td>4</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>+</td>
<td>3</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>/</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>+</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Points 0 to 12 are equally spaced in horizontal projection.

Span of arch \( L = 24 \) m
First intrados trial hinge point \( I_1 = 4 \)
First extrados trial hinge point \( E_1 = 0 \)
Second intrados trial hinge point \( I_2 = 12 \)
Second extrados trial hinge point \( E_2 = 9 \)

Dead load at point 1 \( w(1) = 10 \) kN
Dead load at point 2 \( w(2) = 10 \) kN
Dead load at point 3 \( w(3) = 10 \) kN
Dead load at point 4 \( w(4) = 10 \) kN
Dead load at point 5 \( w(5) = 10 \) kN
Dead load at point 6 \( w(6) = 10 \) kN
Dead load at point 7 \( w(7) = 10 \) kN
Dead load at point 8 \( w(8) = 10 \) kN
Dead load at point 9 \( w(9) = 10 \) kN
Dead load at point 10 \( w(10) = 10 \) kN
Dead load at point 11 \( w(11) = 10 \) kN
Height of intrados at point 0 $H_0=0$ m
Height of intrados at point 1 $H(1)=1.7106$ m
Height of intrados at point 2 $H(2)=2.966$ m
Height of intrados at point 3 $H(3)=3.8739$ m
Height of intrados at point 4 $H(4)=4.4918$ m
Height of intrados at point 5 $H(5)=4.8517$ m
Height of intrados at point 6 $H(6)=4.97$ m
Height of intrados at point 7 $H(7)=4.8517$ m
Height of intrados at point 8 $H(8)=4.4918$ m
Height of intrados at point 9 $H(9)=3.8739$ m
Height of intrados at point 10 $H(10)=2.966$ m
Height of intrados at point 11 $H(11)=1.7106$ m
Height of intrados at point 12 $H(12)=0$ m

Test load $P=100$ kN
Location of test load $X=18$ m
SUMMARY

Line of thrust lies above the intrados and below the extrados.

- Moment at left end of arch: 238.54 kNm
- Horizontal reaction at left end: 55 kN
- Moment at right end of arch: -0.23854E-3 kNm
- Horizontal reaction at right end: 55 kN
- Thickness of arch: 1.1992 m
Location: Ex1 - Hand analysis example by Professor MR Horne

Arch geometry and loading

Span of arch \( L = 24 \text{ m} \)
First intrados trial hinge point \( I_1 = 4 \)
First extrados trial hinge point \( E_1 = 0 \)
Second intrados trial hinge point \( I_2 = 12 \)
Second extrados trial hinge point \( E_2 = 9 \)

Dead load at point 1 \( w(1) = 10 \text{ kN} \)
Dead load at point 2 \( w(2) = 10 \text{ kN} \)
Dead load at point 3 \( w(3) = 10 \text{ kN} \)
Dead load at point 4 \( w(4) = 10 \text{ kN} \)
Dead load at point 5 \( w(5) = 10 \text{ kN} \)
Dead load at point 6 \( w(6) = 10 \text{ kN} \)
Dead load at point 7 \( w(7) = 10 \text{ kN} \)
Dead load at point 8 \( w(8) = 10 \text{ kN} \)
Dead load at point 9 \( w(9) = 10 \text{ kN} \)
Dead load at point 10 \( w(10) = 10 \text{ kN} \)
Dead load at point 11 \( w(11) = 10 \text{ kN} \)
Height of intrados at point 0 \( H(0) = 0 \) m
Height of intrados at point 1 \( H(1) = 1.7106 \) m
Height of intrados at point 2 \( H(2) = 2.966 \) m
Height of intrados at point 3 \( H(3) = 3.8739 \) m
Height of intrados at point 4 \( H(4) = 4.4918 \) m
Height of intrados at point 5 \( H(5) = 4.8517 \) m
Height of intrados at point 6 \( H(6) = 4.97 \) m
Height of intrados at point 7 \( H(7) = 4.8517 \) m
Height of intrados at point 8 \( H(8) = 4.4918 \) m
Height of intrados at point 9 \( H(9) = 3.8739 \) m
Height of intrados at point 10 \( H(10) = 2.966 \) m
Height of intrados at point 11 \( H(11) = 1.7106 \) m
Height of intrados at point 12 \( H(12) = 0 \) m

Test load \( P = 100 \) kN
Location of test load \( X = 18 \) m
SUMMARY

Line of thrust lies above the intrados and below the extrados.

- Moment at left end of arch: 238.54 kNm
- Horizontal reaction at left end: 55 kN
- Moment at right end of arch: -0.23854E-3 kNm
- Horizontal reaction at right end: 55 kN
- Thickness of arch: 1.1992 m

NOTE: All values above are characteristic values.
Location: Ex1 - Masonry dam D1

The following analysis checks the profile of a dam to ensure that the line of thrust comes within the middle third. Stresses at the back and front of the dam are also computed.

Datum \( x(0) \) \( b(0) \) Datum is usually bottom left hand corner of dam cross-section, but need not be. Segments are numbered downwards from top of dam.

\[ p(0) \quad \text{top of dam} \]

Number of segments in dam \( NS=2 \)
Density of masonry \( \gamma=24 \text{ kN/m}^3 \)
Length of dam into paper \( L=1 \text{ m} \)
Horiz.distance from datum at top \( x(0)=0.25 \text{ m} \)
Width of dam at top \( b(0)=2 \text{ m} \)
Normal pressure at top of dam \( p(0)=0 \text{ kN/m}^2 \)

\[ v(i) \quad (i) \text{ is segment number, } i=1 \text{ for top segment } \]
\[ i=2 \text{ for bottom segment} \]

Height of segment 1 \( v(1)=3 \text{ m} \)
Horiz.distance from datum at bottom \( x(1)=0.25 \text{ m} \)
Width of dam at segment bottom \( b(1)=2 \text{ m} \)
Normal pressure at segment bottom \( p(1)=30 \text{ kN/m}^2 \)
Height of segment 2 \( v(2)=5 \text{ m} \)
Horiz.distance from datum at bottom \( x(2)=0 \text{ m} \)
Width of dam at segment bottom \( b(2)=5 \text{ m} \)
Normal pressure at segment bottom \( p(2)=80 \text{ kN/m}^2 \)

**SUMMARY**

<table>
<thead>
<tr>
<th>Segment</th>
<th>Bending moment (kNm)</th>
<th>Left Hand Side Vertical Stress (kN/m²)</th>
<th>Right Hand Side Vertical Stress (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>45</td>
<td>4.5</td>
<td>139.5</td>
</tr>
<tr>
<td>2</td>
<td>415.42</td>
<td>15.85</td>
<td>215.25</td>
</tr>
</tbody>
</table>
The following analysis, checks the profile of a dam to ensure that the line of thrust comes within the middle third. Stresses at the back and front of the dam are also computed.

- Datum $x(0)$, $b(0)$: Datum is usually bottom left hand corner of dam cross-section, but need not be.
- Number of segments in dam $N_S = 2$
- Density of masonry $\gamma = 20 \text{ kN/m}^3$
- Length of dam into paper $L = 1 \text{ m}$
- Horiz. distance from datum at top $x(0) = 0.5 \text{ m}$
- Width of dam at top $b(0) = 4 \text{ m}$
- Normal pressure at top of dam $p(0) = 10 \text{ kN/m}^2$

<table>
<thead>
<tr>
<th>Segment</th>
<th>Bending moment $(\text{kNm})$</th>
<th>Left Hand Side Vertical Stress $(\text{kN/m}^2)$</th>
<th>Right Hand Side Vertical Stress $(\text{kN/m}^2)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>480</td>
<td>-60</td>
<td>300</td>
</tr>
<tr>
<td>2</td>
<td>4283.3</td>
<td>-63.5</td>
<td>450.5</td>
</tr>
</tbody>
</table>

SUMMARY
Location: Ex1 - Masonry dam D1

The following analysis, checks the profile of a dam to ensure that the line of thrust comes within the middle third. Stresses at the back and front of the dam are also computed.

<table>
<thead>
<tr>
<th>Datum x(0)</th>
<th>b(0)</th>
<th>Datum is usually bottom left hand corner of dam cross-section, but need not be.</th>
</tr>
</thead>
<tbody>
<tr>
<td>p(0)</td>
<td></td>
<td>Segments are numbered downwards from top of dam.</td>
</tr>
</tbody>
</table>

Number of segments in dam NS=2
Density of masonry gamma=24 kN/m³
Length of dam into paper L=1 m
Horiz.distance from datum at top x(0)=0.25 m
Width of dam at top b(0)=2 m
Normal pressure at top of dam p(0)=0 kN/m²

<table>
<thead>
<tr>
<th>v(i)</th>
<th>(i) is segment number, i=1 for top segment, i=2 for bottom segment</th>
</tr>
</thead>
<tbody>
<tr>
<td>p(i)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>x(i)</th>
<th>b(i)</th>
</tr>
</thead>
</table>

Height of segment 1 v(1)=3 m
Horiz.distance from datum at bottom x(1)=0.25 m
Width of dam at segment bottom b(1)=2 m
Normal pressure at segment bottom p(1)=30 kN/m²
Height of segment 2 v(2)=5 m
Horiz.distance from datum at bottom x(2)=0 m
Width of dam at segment bottom b(2)=5 m
Normal pressure at segment bottom p(2)=80 kN/m²

**SUMMARY**

<table>
<thead>
<tr>
<th>Segment</th>
<th>Bending moment (kNm)</th>
<th>Left Hand Side Vertical Stress (kN/m²)</th>
<th>Right Hand Side Vertical Stress (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>45</td>
<td>4.5</td>
<td>139.5</td>
</tr>
<tr>
<td>2</td>
<td>415.42</td>
<td>15.85</td>
<td>215.25</td>
</tr>
</tbody>
</table>

**NOTE:** All values above are characteristic values.
The following analysis, checks the profile of a dam to ensure that the line of thrust comes within the middle third. Stresses at the back and front of the dam are also computed.

<table>
<thead>
<tr>
<th>Datums</th>
<th>x(0)</th>
<th>b(0)</th>
<th>Datum is usually bottom left hand corner of dam cross-section, but need not be. Segments are numbered downwards from top of dam.</th>
</tr>
</thead>
<tbody>
<tr>
<td>p(0)</td>
<td></td>
<td></td>
<td>Number of segments in dam NS=2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Density of masonry gamma=20 kN/m³</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Length of dam into paper L=1 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Horiz.distance from datum at top x(0)=0.5 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Width of dam at top b(0)=4 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Normal pressure at top of dam p(0)=10 kN/m²</td>
</tr>
</tbody>
</table>

Height of segment 1 v(1)=6 m
Horiz.distance from datum at bottom x(1)=0.5 m
Width of dam at segment bottom b(1)=4 m
Normal pressure at segment bottom p(1)=60 kN/m²
Height of segment 2 v(1)=10 m
Horiz.distance from datum at bottom x(2)=0 m
Width of dam at segment bottom b(2)=10 m
Normal pressure at segment bottom p(2)=160 kN/m²

**SUMMARY**

<table>
<thead>
<tr>
<th>Segment</th>
<th>Bending moment (kNm)</th>
<th>Left Hand Side Vertical Stress (kN/m²)</th>
<th>Right Hand Side Vertical Stress (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>480</td>
<td>-60</td>
<td>300</td>
</tr>
<tr>
<td>2</td>
<td>4283.3</td>
<td>-63.5</td>
<td>450.5</td>
</tr>
</tbody>
</table>

**NOTE:** All values above are characteristic values.
Location: Wall on grid line A/1-2

Steps are numbered down from the top

h(a) = height at step (a) from the top
\( t(a) = \text{thickness at step (a)} \)
\( Vo = \text{vertical point load on wall} \)

As a masonry wall has limited capacity the design will assume the backfill is drained i.e. no water pressure behind the wall.

Weight/unit volume of soil & wall \( w_e = 18 \text{ kN/m}^3 \)
Internal angle of friction (fill) \( \phi = 35^\circ \)
Angle of backfill \( \beta = 0^\circ \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
\( \text{typ(1)} = 4 \)
Height \( h_b = 215 \text{ mm} \)
Least horizontal dimension \( l_h = 100 \text{ mm} \)
Compressive strength of blocks \( p_o = 10.5 \text{ N/mm}^2 \)
Partial safety factor material \( \gamma_m = 3.5 \)
Applied moment at top of wall \( x_m = 0.32 \text{ kNm} \)
Applied vertical load on the wall \( V_o = 2 \text{ kN} \)
Applied shear at top of wall \( x_v = 0.36 \text{ kN} \)
Weight/unit area of surcharge \( w_s = 5 \text{ kN/m}^2 \)
Load factor external loads at top \( \gamma_x = 1.6 \)
Load factor surcharge \( \gamma_s = 1.6 \)
Load factor earth pressure \( \gamma_e = 1.4 \)
Load factor self weight \( \gamma_f = 0.9 \)
Proposed height at step \( h(l) = 0.45 \text{ m} \)
Proposed thickness at step \( t(l) = 0.325 \text{ m} \)
Width of toe \( t_t = 0.15 \text{ m} \)
Width of heel \( t_h = 0.15 \text{ m} \)
Point of application of Vo from A \( eVo = 0.3125 \text{ m} \)
Depth of base \( d_b = 0.15 \text{ m} \)
Density of base \( d_{eb} = 24 \text{ kN/m}^3 \)
Angle of friction (u/s of base) \( \phi_{ib} = 40^\circ \)
Soil cohesion \( c_u = 0 \text{ kN/m}^2 \)
### DESIGN SUMMARY

Wall height 1.8 m

<table>
<thead>
<tr>
<th>Step height (m)</th>
<th>Step thickness (m)</th>
<th>Ultimate Moments below are in kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.45</td>
<td>0.325</td>
<td>1.562</td>
</tr>
<tr>
<td>0.9</td>
<td>0.45</td>
<td>3.19</td>
</tr>
<tr>
<td>1.35</td>
<td>0.675</td>
<td>7.629</td>
</tr>
<tr>
<td>1.8</td>
<td>0.9</td>
<td>14.46</td>
</tr>
</tbody>
</table>

Stability check summary:
- Factor of safety overturning: 3.088
- Factor of safety sliding: 1.783
- Maximum base pressure: 68.22 kN/m²
Location: Wall on grid line B/1-2

Constant wall thickness

- \( h(1) \) = height of wall
- \( t(1) \) = thickness of wall
- \( Vo \) = vert point load on wall

As a masonry wall has limited capacity the design assumes the backfill is drained i.e. no water pressure behind the wall.

Weight/unit volume of soil & wall \( w_e = 18 \text{ kN/m}^3 \)
Internal angle of friction (fill) \( \phi_i = 35^\circ \)
Angle of backfill \( \beta = 0^\circ \)
Mortar designation to Table 1 \( \text{mortar} = 3 \)
Compressive strength of units \( p_o = 10.5 \text{ N/mm}^2 \)
Partial safety factor material \( \gamma_{m} = 3.5 \)
Applied moment at top of wall \( x_m = 0.25 \text{ kNm} \)
Applied vertical load on the wall \( Vo = 0 \text{ kN} \)
Applied shear at top of wall \( x_v = 0.25 \text{ kN} \)
Weight/unit area of surcharge \( w_s = 5 \text{ kN/m}^2 \)
Load factor external loads at top \( \gamma_{mx} = 1.6 \)
Load factor surcharge \( \gamma_{ms} = 1.6 \)
Load factor earth pressure \( \gamma_{me} = 1.4 \)
Load factor self weight \( \gamma_{mf} = 0.9 \)
Proposed height of wall \( h(1) = 1.25 \text{ m} \)
Proposed thickness of wall \( t(1) = 0.625 \text{ m} \)
Width of toe \( t_t = 0.15 \text{ m} \)
Width of heel \( t_h = 0.15 \text{ m} \)
Point of application of \( Vo \) from A \( e_Vo = 0.4625 \text{ m} \)
Depth of base \( d_b = 0.15 \text{ m} \)
Density of base \( d_{eb} = 24 \text{ kN/m}^3 \)
Angle of friction (u/s of base) \( \phi_b = 40^\circ \)
Soil cohesion \( c_u = 0 \text{ kN/m}^2 \)

WARNING:
Factor of safety for sliding is less than 1.5 and therefore changes are required.
Location: Wall on grid line C/1-2

Steps are numbered down from the top

- $h(a)$ = height at step (a) from the top
- $t(a)$ = thickness at step (a)
- $V_o$ = vertical point load on wall

As a masonry wall has limited capacity, the design will assume the backfill is drained i.e. no water pressure behind the wall.

Weight/unit volume of soil & wall $w_e=18$ kN/m³
Internal angle of friction (fill) $\phi_i=35^\circ$
Angle of backfill $\beta=30^\circ$
Mortar designation to Table 1 $\text{mortar}=3$
$typ(1)=1$
Compressive strength of units $p_o=10.5$ N/mm²
Water absorption %age of leaf $\text{water}=7$
Partial safety factor material $\gamma_{ma}=3.5$
Applied moment at top of wall $x_m=0.32$ kNm
Applied vertical load on the wall $V_o=0$ kN
Applied shear at top of wall $x_v=0.36$ kN
Weight/unit area of surcharge $w_s=5$ kN/m²
Load factor external loads at top $g_{mx}=1.6$
Load factor surcharge $g_{ms}=1.6$
Load factor earth pressure $g_{me}=1.4$
Load factor self weight $g_{mf}=0.9$
Proposed height at step $h(1)=0.45$ m
Proposed thickness at step $t(1)=0.325$ m
Width of toe $t_t=0.15$ m
Width of heel $t_h=0.15$ m
Point of application of $V_o$ from A $e_{V_o}=0.3125$ m
Depth of base $d_b=0.15$ m
Density of base $d_{en}=24$ kN/m³
Angle of friction (u/s of base) $\phi_{ib}=40^\circ$
Soil cohesion $c_u=0$ kN/m²

WARNING:
Factor of safety for sliding is less than 1.5 and therefore changes are required.
Location: Ex1 - Surface water sewer example

Design of surface water sewer

Area drained
= Ar m²

h m head of drain

L m length of drain

Effective area to be drained Ar=10000 m²
Impermeability factor P=0.875
Height of the drain (head) h=1.2 m
Length of the drain L=200 m

1. Using Chezy formula

Diameter to be adopted D=375 mm

2. Using Escritt’s formulae (see Drainage & Sewerage, published by the Clay Pipe Development Association)

Q = 0.00035D^2.62/(1/i)^0.5
V = 26.738D^0.62/(1/i)^0.5

Diameter to be adopted D=375 mm
Velocity V=26.738*D^0.62/(SQR(1/i)*60)
=1.3613 m/s

3. To check for drain running partially full; see hydraulic design of drains using Colebrook-White equation in Hydraulics Research Papers No 2 and No 4

Diameter of drain to be used D=375 mm

Depth of flow as proportion of D x=0.67098
Velocity of flow V=1.5424 m/s

Velocity V = -2Zlog(ks/3.7D+2.51v/DZ) for full pipes where
Velocity running full V=1.3984 m/s
### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design discharge</td>
<td>0.12153 m³/s</td>
</tr>
<tr>
<td>Gradient of pipe</td>
<td>1 in 166.67 or 0.006</td>
</tr>
<tr>
<td>(a) Using Chezy formula:</td>
<td></td>
</tr>
<tr>
<td>Diameter of pipe</td>
<td>375 mm</td>
</tr>
<tr>
<td>Velocity of flow</td>
<td>1.3282 m/s</td>
</tr>
<tr>
<td>(b) Using Escritt's formulae:</td>
<td></td>
</tr>
<tr>
<td>Diameter of pipe</td>
<td>375 mm</td>
</tr>
<tr>
<td>Velocity of flow</td>
<td>1.3613 m/s</td>
</tr>
<tr>
<td>(c) Using Colebrook-White formula:</td>
<td></td>
</tr>
<tr>
<td>Diameter of pipe</td>
<td>375 mm</td>
</tr>
<tr>
<td>Velocity of flow</td>
<td>1.5424 m/s</td>
</tr>
<tr>
<td>Proportional depth</td>
<td>0.67098</td>
</tr>
<tr>
<td>Capacity of pipe</td>
<td>0.15445 m³/s</td>
</tr>
<tr>
<td>Velocity running full</td>
<td>1.3984 m/s</td>
</tr>
</tbody>
</table>
Location: Ex2 - Foul sewer example

Design of foul sewer

Houses discharging Q' l/s foul sewage

\[ \begin{align*} \text{h m head of drain} & \quad \text{L m length of drain} \\ \text{Height of the drain (head)} & = h = 1.4 \text{ m} \\ \text{Length of the drain} & = L = 100 \text{ m} \end{align*} \]

1. Using Chezy formula

<table>
<thead>
<tr>
<th>Diameter to be adopted</th>
<th>D=150 mm</th>
</tr>
</thead>
</table>

2. Using Escritt's formulae (see Drainage & Sewerage, published by the Clay Pipe Development Association)

\[ \begin{align*} Q & = 0.00035D^{2.62}/(1/i)^{0.5} \\ V & = 26.738D^{0.62}/(1/i)^{0.5} \end{align*} \]

<table>
<thead>
<tr>
<th>Diameter to be adopted</th>
<th>D=150 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Velocity</td>
<td>$V=26.738\times D^{0.62}/(\sqrt{1/i})\times 60$ $=1.1782 \text{ m/s}$</td>
</tr>
</tbody>
</table>

3. To check for drain running partially full; see hydraulic design of drains using Colebrook-White equation in Hydraulics Research Papers No 2 and No 4

<table>
<thead>
<tr>
<th>Diameter of drain to be used</th>
<th>D=150 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of flow as proportion of D</td>
<td>x=0.63287</td>
</tr>
<tr>
<td>Velocity of flow</td>
<td>$V=1.1306 \text{ m/s}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Velocity</th>
<th>$V=-2\times \log(ks/3.7D+2.51v/DZ)$ for full pipes where</th>
</tr>
</thead>
<tbody>
<tr>
<td>Velocity running full</td>
<td>$V=1.0363 \text{ m/s}$</td>
</tr>
</tbody>
</table>
### DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design discharge</td>
<td>0.01333 m³/s</td>
</tr>
<tr>
<td>Gradient of pipe</td>
<td>1 in 71.429 or 0.014</td>
</tr>
</tbody>
</table>

(a) Using Chezy formula:
- Diameter of pipe: 150 mm
- Velocity of flow: 1.4103 m/s

(b) Using Escritt's formulae:
- Diameter of pipe: 150 mm
- Velocity of flow: 1.1782 m/s

(c) Using Colebrook-White formula:
- Diameter of pipe: 150 mm
- Velocity of flow: 1.1306 m/s
- Proportional depth: 0.63287
- Capacity of pipe: 0.018313 m³/s
- Velocity running full: 1.0363 m/s
Location: Ex3 - Combined example

Design of combined sewer

Houses discharging
Q' l/s foul sewage
Area drained =Ar m²

<table>
<thead>
<tr>
<th>h m head of drain</th>
<th>L m length of drain</th>
</tr>
</thead>
</table>

Effective area to be drained       Ar=10000 m²
Impermeability factor             P=0.875
Height of the drain (head)        h=1.2 m
Length of the drain               L=200 m

1. Using Chezy formula

Diameter to be adopted            D=375 mm

2. Using Escritt's formulae (see Drainage & Sewerage, published by
the Clay Pipe Development Association)

\[ Q = 0.00035D^{2.62}/(1/i)^{0.5} \]
\[ V = 26.738D^{0.62}/(1/i)^{0.5} \]

Diameter to be adopted            D=375 mm
Velocity                           V=26.738*D^{0.62}/(SQR(1/i)*60)
                                      =1.3613 m/s

3. To check for drain running partially full; see hydraulic design
of drains using Colebrook-White equation in Hydraulics Research

Papers No 2 and No 4

Diameter of drain to be used       D=375 mm
Depth of flow as proportion of D   x=0.80943
Velocity of flow                   V=1.4081 m/s

Velocity V = \(-2Z\log(ks/3.7D+2.51v/DZ)\) for full pipes where
Velocity running full              V=1.2401 m/s
4. Check for self cleansing velocity under dry weather conditions

Depth of flow as proportion of D x=0.13462
Velocity of flow V=0.60059 m/s

DESIGN SUMMARY

Design discharge 0.13486 m³/s
Gradient of pipe 1 in 166.67 or 0.006

(a) Using Chezy formula:
   Diameter of pipe 375 mm
   Velocity of flow 1.3282 m/s

(b) Using Escritt's formulae:
   Diameter of pipe 375 mm
   Velocity of flow 1.3613 m/s

(c) Using Colebrook-White formula:
   Diameter of pipe 375 mm
   Velocity of flow 1.4081 m/s
   Proportional depth 0.80943
   Capacity of pipe 0.13696 m³/s
   Velocity running full 1.2401 m/s

Under dry weather conditions:
   Design discharge 0.005332 m³/s
   Velocity of flow 0.60059 m/s
   Proportional depth 0.13462
Location: Flat roof with half-round eaves gutter

Roof drainage gutters - to BS EN 12056-3:2000 (with Amd. No. 17041)

Width of roof (gutter to ridge) \( Br = 8.65 \) m
Length of roof to be drained \( L_r = 6.45 \) m
Rainfall intensity \( r = 0.021 \) l/s/m²
The runoff coefficient \( C = 1 \)
Depth of water below designed water line \( W = 75 \) mm
Length of gutter \( L = 6.45 \) m
Gradient of gutter \( G_{grad} = 3 \) mm/m
Diameter of circular downpipe \( D = 75 \) mm

DESIGN

Gutter is a true half round gutter

SUMMARY

Width of gutter \( 150 \) mm
Capacity of gutter is \( 2.04 \) l/s
Rate of flow of water \( 1.1716 \) l/s
Diameter of outlet pipe \( 75 \) mm
Location: Single pitched roof with half-round eaves gutter

Roof drainage gutters - to BS EN 12056-3:2000 (with Amd. No. 17041)

Ridge

Eaves gutter

Br

Hr

Single pitched roof

Width of roof (gutter to ridge) \( Br = 8.65 \) m

Height of roof (gutter to ridge) \( Hr = 5 \) m

Length of roof to be drained \( Lr = 6.45 \) m

Minimum rainfall intensity rate \( r = 0.021 \) l/s/m²

Risk factor \( rf = 1 \)

The runoff coefficient \( C = 1 \)

Depth of water below designed water line \( W = 75 \) mm

Length of gutter \( L = 6.45 \) m

Gradient of gutter \( G_{\text{grad}} = 2 \) mm/m

Diameter of circular downpipe \( D = 75 \) mm

**DESIGN**

Gutter is a true half round gutter

**SUMMARY**

Width of gutter \( 150 \) mm

Capacity of gutter is \( 2.04 \) l/s

Rate of flow of water \( 1.5103 \) l/s

Diameter of outlet pipe \( 75 \) mm
Location: Double pitched roof with valley gutter

Roof drainage gutters - to BS EN 12056-3:2000 (with Amd. No. 17041)

valley gutter

Double pitched roof

Width from valley to ridge        Br1=6 m
Width from valley to ridge        Br2=15 m
Ridge to valley fall              Hr1=2 m
Valley to ridge rise              Hr2=4 m
Length of roof to be drained      Lr=30 m
Area of wall draining to roof     Awall=50 m²
Minimum rainfall intensity rate   r=0.0208 l/s/m²
Risk factor                       rf=1
The runoff coefficient            C=1

Valley gutter

W = depth below designed water line
T = width of gutter
S = width of sole
Z = total depth of gutter

Width of gutter                   T=500 mm
Depth of water                    W=179 mm
Width of sole (min 300mm)         S=300 mm
Gutter depth including freeboard  Z=255 mm
Cross sectional area of gutter    Aw=71600 mm²
Length of gutter                  L=30 m
Gradient of gutter                Ggrad=5 mm/m
Discharge from second gutter      Q2=3.3 l/s
Critical depth for second gutter  yc2=90 mm
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of chute</td>
<td>Lw=500 mm</td>
</tr>
<tr>
<td>Width of box</td>
<td>Bb=450 mm</td>
</tr>
<tr>
<td>Length of box</td>
<td>Lb=500 mm</td>
</tr>
<tr>
<td>Assumed diameter of outlet</td>
<td>D=200 mm</td>
</tr>
<tr>
<td>Depth of box below gutter</td>
<td>hb=200 mm</td>
</tr>
</tbody>
</table>

**DESIGN**  
Trapezoidal valley gutter

**SUMMARY**
- Width of gutter: 500 mm
- Total depth of gutter: 255 mm
- Width of sole: 300 mm
- Capacity of gutter is: 36.068 l/s
- Rate of flow of water: 17.548 l/s

**Sluice/Chute**
- Width of chute in parapet: 500 mm

**Box receiver**
- Length of box receiver: 500 mm
- Width of box receiver: 450 mm
- Depth of box: 200 mm
- Diameter of outlet pipe: 200 mm
Location: Single pitched roof with trapezoidal eaves gutter

Roof drainage gutters - to BS EN 12056-3:2000 (with Amd. No. 17041)

Width of roof (gutter to ridge) \( Br = 6 \text{ m} \)
Length of roof to be drained \( Lr = 20 \text{ m} \)
Area of wall draining to roof \( A_{wall} = 50 \text{ m}^2 \)
Minimum rainfall intensity rate \( r = 0.021 \text{ l/s/m}^2 \)
Risk factor \( rf = 1 \)
The runoff coefficient \( C = 1 \)

Eaves gutter

\[ W = \text{depth below designed water line} \]
\[ T = \text{width of gutter} \]
\[ S = \text{width of sole} \]
\[ Z = \text{total depth of gutter} \]

Width of gutter \( T = 150 \text{ mm} \)
Depth of water \( W = 150 \text{ mm} \)
Width of sole \( S = 150 \text{ mm} \)
Depth of gutter including freeboard \( Z = 150 \text{ mm} \)
Cross sectional area of gutter \( A_e = 22500 \text{ mm}^2 \)
Length of gutter \( L = 20 \text{ m} \)
Gradient of gutter \( G_{grad} = 1 \text{ mm/m} \)
Width of chute \( L_w = 400 \text{ mm} \)
Width of box \( B_b = 450 \text{ mm} \)
Length of box \( L_b = 500 \text{ mm} \)
Assumed diameter of outlet \( D = 200 \text{ mm} \)
Depth of box below gutter \( h_b = 200 \text{ mm} \)

**DESIGN**

Gutter is trapezoidal

**SUMMARY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of gutter</td>
<td>150 mm</td>
</tr>
<tr>
<td>Total depth of gutter</td>
<td>150 mm</td>
</tr>
<tr>
<td>Width of sole</td>
<td>150 mm</td>
</tr>
<tr>
<td>Capacity of gutter is</td>
<td>7.6526 l/s</td>
</tr>
<tr>
<td>Rate of flow of water</td>
<td>3.045 l/s</td>
</tr>
<tr>
<td>Sluice/Chute</td>
<td></td>
</tr>
<tr>
<td>-------------</td>
<td>----------------</td>
</tr>
<tr>
<td>Width of chute in parapet</td>
<td>400 mm</td>
</tr>
<tr>
<td>Box receiver</td>
<td></td>
</tr>
<tr>
<td>Length of box receiver</td>
<td>500 mm</td>
</tr>
<tr>
<td>Width of box receiver</td>
<td>450 mm</td>
</tr>
<tr>
<td>Depth of box</td>
<td>200 mm</td>
</tr>
<tr>
<td>Diameter of outlet pipe</td>
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</tr>
</tbody>
</table>
Location: Double pitched roof with trapezoidal valley gutter

Roof drainage gutters - to BS EN 12056-3:2000 (with Amd. No. 17041)

Width from valley to ridge \( Br1 = 6 \text{ m} \)
Width from valley to ridge \( Br2 = 15 \text{ m} \)
Length of roof to be drained \( Lr = 30 \text{ m} \)
Area of wall draining to roof \( A_{wall} = 50 \text{ m}^2 \)
Minimum rainfall intensity rate \( r = 0.021 \text{ l/s/m}^2 \)
Risk factor \( rf = 1 \)
The runoff coefficient \( C = 1 \)

Valley gutter

\( W = \) depth below designed water line
\( T = \) width of gutter
\( S = \) width of sole
\( Z = \) total depth of gutter

Width of gutter \( T = 500 \text{ mm} \)
Depth of water \( W = 178 \text{ mm} \)
Width of sole (min 300mm) \( S = 300 \text{ mm} \)
Gutter depth including freeboard \( Z = 254 \text{ mm} \)
Cross sectional area of gutter \( A_w = 71200 \text{ mm}^2 \)
Length of gutter \( L = 30 \text{ m} \)
Gradient of gutter \( G_{grad} = 1 \text{ mm/m} \)
Width of chute \( L_w = 400 \text{ mm} \)
### Width of box

<table>
<thead>
<tr>
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<th>Value</th>
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</thead>
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<tr>
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### Length of box

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### Assumed diameter of outlet

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### Depth of box below gutter

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</table>

### Design

- Trapezoidal valley gutter

### Summary

- **Width of gutter**: 500 mm
- **Total depth of gutter**: 254 mm
- **Width of sole**: 300 mm
- **Capacity of gutter is**: 26.343 l/s
- **Rate of flow of water**: 13.755 l/s
- **Sluice/Chute**
  - **Width of chute in parapet**: 400 mm
- **Box receiver**
  - **Length of box receiver**: 500 mm
  - **Width of box receiver**: 450 mm
  - **Depth of box**: 200 mm
  - **Diameter of outlet pipe**: 200 mm
Location: Flat roof with trapezoidal parapet gutter

Roof drainage gutters - to BS EN 12056-3:2000 (with Amd. No. 17041)

Width of roof (gutter to ridge) \( Br = 8.65 \) m
Length of roof to be drained \( L_r = 6.45 \) m
Rainfall intensity \( r = 0.021 \) l/s/m²
The runoff coefficient \( C = 1 \)

Parapet gutter

\[ W = \text{depth below designed water line} \]
\[ T = \text{width of gutter} \]
\[ S = \text{width of sole} \]
\[ Z = \text{total depth of gutter} \]

Width of gutter \( T = 400 \) mm
Depth of water \( W = 150 \) mm
Width of sole (min 300mm) \( S = 300 \) mm
Gutter depth including freeboard \( Z = 214 \) mm
Cross sectional area of gutter \( A_w = 52500 \) mm²
Length of gutter \( L = 6.45 \) m
Diameter of circular downpipe \( D = 75 \) mm

**DESIGN**

**SUMMARY**

<table>
<thead>
<tr>
<th>Trapezoidal parapet gutter</th>
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<tbody>
<tr>
<td>Width of gutter</td>
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<td>Total depth of gutter</td>
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<tr>
<td>Width of sole</td>
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<tr>
<td>Capacity of gutter is</td>
</tr>
<tr>
<td>Rate of flow of water</td>
</tr>
<tr>
<td>Diameter of outlet pipe</td>
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</tbody>
</table>
Location: Single pitched roof with trapezoidal parapet gutter

Roof drainage gutters – to BS EN 12056-3:2000 (with Amd. No. 17041)

Width of roof (gutter to ridge) \( \text{Br}=8.65 \text{ m} \)
Height of roof (gutter to ridge) \( \text{Hr}=5 \text{ m} \)
Length of roof to be drained \( \text{Lr}=6.45 \text{ m} \)
Minimum rainfall intensity rate \( \text{r}=0.021 \text{ l/s/m}^2 \)
Risk factor \( \text{rf}=1 \)
The runoff coefficient \( \text{C}=1 \)

Parapet gutter

\[ \begin{align*}
\text{W} &= \text{depth below designed water line} \\
\text{T} &= \text{width of gutter} \\
\text{S} &= \text{width of sole} \\
\text{Z} &= \text{total depth of gutter}
\end{align*} \]

\[ \begin{align*}
\text{Width of gutter} &= \text{T}=400 \text{ mm} \\
\text{Depth of water} &= \text{W}=150 \text{ mm} \\
\text{Width of sole (min 300mm)} &= \text{S}=300 \text{ mm} \\
\text{Gutter depth including freeboard} &= \text{Z}=214 \text{ mm} \\
\text{Cross sectional area of gutter} &= \text{Aw}=52500 \text{ mm}^2 \\
\text{Length of gutter} &= \text{L}=6.45 \text{ m} \\
\text{Diameter of circular downpipe} &= \text{D}=75 \text{ mm}
\end{align*} \]

**DESIGN**

**SUMMARY**

Trapezoidal parapet gutter

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<th>Value</th>
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<td>Width of gutter</td>
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<tr>
<td>Total depth of gutter</td>
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<tr>
<td>Width of sole</td>
<td>300 mm</td>
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<tr>
<td>Capacity of gutter is</td>
<td>21.988 l/s</td>
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<tr>
<td>Rate of flow of water</td>
<td>1.5103 l/s</td>
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<tr>
<td>Diameter of outlet pipe</td>
<td>75 mm</td>
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</table>
Location: Road alignment; vertical curve example

Road alignment vertical curve


\[ y = -\frac{(q-p)x^2}{2L} \]

Note as shown here

p is positive (ascending)
q is negative (descending)

Gradient p (as a percentage) \( p = 6 \)
Gradient q (as a percentage) \( q = -3 \)
Design speed \( V = 50 \text{ km/h} \)
K-Value for curve \( K = 6.5 \text{ m} \)
Sight distance required \( sd = 50 \text{ m} \)
Driver's eye height \( h1 = 1.05 \text{ m} \)
Object height \( h2 = 0.26 \text{ m} \)
Selected curve length \( L = 60 \text{ m} \)

Chainage of intersection point \( PI_{ch} = 120 \text{ m} \)
Level of intersection point \( PIL\ell = 55.5 \text{ m} \)
Location: Road Alignment Horizontal Transition Curve

Degrees in angle                  d=18°
Minutes in angle                  m=36 '
Seconds in angle                  s=0"
Circular curve radius             R=450 m
Design speed                      V=70 km/h
Rate of change of radial accln.   a=0.3 m/s³
Chainage of Intersection point    IC=2524.2 m
Main transition chord length      x=10 m
Main chord length                 c=20 m

Balancing percent of radial force per=45 %
Carriageway width                 B=7.6 m
Location: Example 9.6 from Bannister, Raymond & Baker - 7th Edition

Cutting or embankment

Formation width of road \( b = 12.5 \text{ m} \)

X-fall existing ground \( \text{1 in k} \) \( k = 12 \)

Side slopes \( \text{1 in m} \) \( m = 2 \)

Centre line height \( h = 3.1 \text{ m} \)

Side width \( w_1 = (b/2 + m*h)*(k/(k-m)) = 14.94 \text{ m} \)

Side width \( w_2 = (b/2 + m*h)*(k/(k+m)) = 10.671 \text{ m} \)

Area of cutting or embankment \( \text{area} = 1/(2*m)*((b/2+m*h)*(w_1+w_2) - b^2)/2 = 60.184 \text{ m}^2 \)
Location: Example 9.7 from Bannister, Raymond & Baker - 7th Edition

Sections part in cut and part in fill
Formation width of road \( b = 9.5 \text{ m} \)
X-fall existing ground (1 in k) \( k = 5 \)
Side slopes in fill (1 in m) \( m = 3 \)
Side slopes in cut (1 in n) \( n = 1 \)
Centre line height \( h = 0.5 \text{ m} \)
Side width \( w_1 = (k/(k-n)) \times (b/2+n \times h) = 6.5625 \text{ m} \)
Side width \( w_2 = (k/(k-m)) \times (b/2-m \times h) = 8.125 \text{ m} \)
Area of fill \( \text{areaf} = 1/2 \times (b/2-k \times h)^2/(k-m) = 1.2656 \text{ m}^2 \)
Area of cut \( \text{areac} = 1/2 \times (b/2+k \times h)^2/(k-n) = 6.5703 \text{ m}^2 \)
Location: Example 9.7 from Bannister, Raymond & Baker – with -ve 'h'

Sections part in cut and part in fill
Formation width of road \( b = 9.5 \text{ m} \)
X-fall existing ground (1 in k) \( k = 5 \)
Side slopes in fill (1 in m) \( m = 1 \)
Side slopes in cut (1 in n) \( n = 3 \)
Centre line height \( h = -0.5 \text{ m} \)
Side width \( w_1 = (k/(k-n)) \times (b/2+n \times h) = 8.125 \text{ m} \)
Side width \( w_2 = (k/(k-m)) \times (b/2-m \times h) = 6.5625 \text{ m} \)
Area of fill \( \text{areaf} = 1/2 \times (b/2-k \times h)^2/(k-m) = 6.5703 \text{ m}^2 \)
Area of cut \( \text{areac} = 1/2 \times (b/2+k \times h)^2/(k-n) = 1.2656 \text{ m}^2 \)
Location: Example 9.6 from Bannister, Raymond & Baker - 7th Edition

Embankment on three level section

Formation width of road \( b = 12.5 \text{ m} \)

X-fall left of centre (1 in \( k \)) \( k = 12 \)

X-fall right of centre (1 in \( l \)) \( l = 12 \)

Side slopes (1 in \( m \)) \( m = 2 \)

Centre line height \( h = 3.1 \text{ m} \)

Side width \( w_1 = \frac{b}{2} + m \cdot h \cdot \frac{k}{k - m} = 14.94 \text{ m} \)

Side width \( w_2 = \frac{b}{2} + m \cdot h \cdot \frac{l}{l + m} = 10.671 \text{ m} \)

Area of cross section

\[
\text{area} = \frac{1}{2m} \left( (w_1 + w_2) \cdot (m \cdot h + b/2) - b^2 / 2 \right) = 60.184 \text{ m}^2
\]
Volumes by prismoidal formula
The volume of a prismoid is given by  \( V = \frac{D}{6} (A_1 + 4M + A_2) \)
where \( A_1 \) & \( A_2 \) are the areas of the two end faces distance \( D \) apart,  
\( M \) is is the area of the section mid-way between (and not the average of \( A_1 \) & \( A_2 \), see Bannister & Raymond.

Area \( A_1 \) \( \text{m}^2 \)  
Area \( A_2 \) \( \text{m}^2 \)  
Area of sectn midway betwn \( A_1 \& A_2 \) \( \text{m}^2 \)  
Distance between two end faces \( \text{m} \)  
Volume of the prismoid \( \text{m}^3 \)

\[ \begin{align*}
A_1 &= 72 \text{ m}^2 \\
A_2 &= 139.84 \text{ m}^2 \\
M &= 103.36 \text{ m}^2 \\
D &= 40 \text{ m} \\
V &= \frac{D}{6} (A_1 + 4M + A_2) = 4168.5 \text{ m}^3
\end{align*} \]
Location: Volumes from spot levels; example from Bannister & Raymond

No. of levels (or corners) N=9
General formation level FLG=120 m
Area of each rectangle A=2000 m²
Level number 1 y(1)=130.24 m
No. of rectangles where it occurs n(1)=1
Formation level f(1)=120 m
Level number 2 y(2)=132.15 m
No. of rectangles where it occurs n(2)=2
Formation level f(2)=120 m
Level number 3 y(3)=134.21 m
No. of rectangles where it occurs n(3)=1
Formation level f(3)=120 m
Level number 4 y(4)=132.92 m
No. of rectangles where it occurs n(4)=2
Formation level f(4)=120 m
Level number 5 y(5)=135.75 m
No. of rectangles where it occurs n(5)=4
Formation level f(5)=120 m
Level number 6 y(6)=136.32 m
No. of rectangles where it occurs n(6)=2
Formation level f(6)=120 m
Level number 7 y(7)=136.98 m
No. of rectangles where it occurs n(7)=1
Formation level f(7)=120 m
Level number 8 y(8)=140.03 m
No. of rectangles where it occurs n(8)=2
Formation level f(8)=120 m
Level number 9 y(9)=135.34 m
No. of rectangles where it occurs n(9)=1
Formation level f(9)=120 m

<table>
<thead>
<tr>
<th>Station Number</th>
<th>Depth of excavation (hn)</th>
<th>Number of rectangles in which it occurs (n)</th>
<th>Product hn x n</th>
</tr>
</thead>
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<td>1</td>
<td>10.24</td>
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</tr>
</tbody>
</table>

\[ \Sigma hn \times n = 242.61 \]

Therefore volume \[ V = A \times \sigma / 4 = 121305 \text{ m}^3 \]
Location: Simpson's rule for volumes

Simpson's rule for volumes:

\[
V = \frac{D}{3} \left[ A(1) + 4A(2) + 2A(3) + 4A(4) + 2A(n-2) + 4A(n-1) + A(n) \right]
\]

where, \( n \) is number of sections having area \( A(1), A(2) \ldots A(n) \),
(n being odd) equally spaced at uniform distance between them = \( D \).

Number of sections (must be odd) \( n=3 \)
Distance between each section \( D=100 \) m
Section number 1 \( A(1)=416.04 \) m²
Section number 2 \( A(2)=563.7 \) m²
Section number 3 \( A(3)=916.1 \) m²

Accumulate areas

Area coefficient for section 1 \( ac=1 \)
\[
\text{Acum}=\text{Acum}+ac\times A(i)=416.04 \text{ m}^2
\]

Area coefficient for section 2 \( ac=4 \)
\[
\text{Acum}=\text{Acum}+ac\times A(i)=2670.8 \text{ m}^2
\]

Area coefficient for section 3 \( ac=1 \)
\[
\text{Acum}=\text{Acum}+ac\times A(i)=3586.9 \text{ m}^2
\]

Therefore volume \( V=D\times \text{Acum}/3=119565 \text{ m}^3 \)
Location: Example of setting out group of 6 piles

Number of theodolite stations  N=3
X-coordinate                      x(1)=2.88 m
Y-coordinate                      y(1)=2.88 m
X-coordinate                      x(2)=17.38 m
Y-coordinate                      y(2)=20.35 m
X-coordinate                      x(3)=26.76 m
Y-coordinate                      y(3)=2.92 m
Number of angles to be computed   M=6
Set up theodolite station number  t(1)=1
First sight theodolite station    f(1)=2
Set up theodolite station number  t(2)=1
First sight theodolite station    f(2)=3
Set up theodolite station number  t(3)=2
First sight theodolite station    f(3)=1
Set up theodolite station number  t(4)=2
First sight theodolite station    f(4)=3
Set up theodolite station number  t(5)=3
First sight theodolite station    f(5)=1
Set up theodolite station number  t(6)=3
First sight theodolite station    f(6)=2
Number of piles to be set-out     P=6
Pile No. 1
X-coordinate of pile              px(1)=20.9 m
Y-coordinate of pile              py(1)=18.08 m
Pile diameter                     pd(1)=0.75 m
Pile No. 2
X-coordinate of pile              px(2)=23.86 m
Y-coordinate of pile              py(2)=18.08 m
Pile diameter                     pd(2)=0.75 m
Pile No. 3
X-coordinate of pile              px(3)=26.76 m
Y-coordinate of pile              py(3)=18.08 m
Pile diameter                     pd(3)=0.75 m
Pile No. 4
X-coordinate of pile              px(4)=20.9 m
Y-coordinate of pile              py(4)=15.15 m
Pile diameter                     pd(4)=0.75 m
Pile No. 5
X-coordinate of pile              px(5)=23.86 m
Y-coordinate of pile              py(5)=15.15 m
Pile diameter                     pd(5)=0.75 m
Pile No. 6
X-coordinate of pile              px(6)=26.76 m
Y-coordinate of pile              py(6)=15.15 m
Pile diameter                     pd(6)=0.75 m
<table>
<thead>
<tr>
<th>Station</th>
<th>Station</th>
<th>Pile No</th>
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<th>Sight to Centre of Pile</th>
<th>Angle from Pile Centre</th>
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<td>50.4</td>
<td>1</td>
</tr>
<tr>
<td>1 2</td>
<td>6</td>
<td>23</td>
<td>6</td>
<td>45.2</td>
<td>0</td>
</tr>
<tr>
<td>1 3</td>
<td>6</td>
<td>27</td>
<td>5</td>
<td>56.4</td>
<td>0</td>
</tr>
<tr>
<td>2 1</td>
<td>6</td>
<td>100</td>
<td>41</td>
<td>23.1</td>
<td>2</td>
</tr>
<tr>
<td>2 3</td>
<td>6</td>
<td>32</td>
<td>42</td>
<td>36.9</td>
<td>2</td>
</tr>
<tr>
<td>3 1</td>
<td>6</td>
<td>90</td>
<td>5</td>
<td>45.5</td>
<td>1</td>
</tr>
<tr>
<td>3 2</td>
<td>6</td>
<td>28</td>
<td>17</td>
<td>13.3</td>
<td>1</td>
</tr>
</tbody>
</table>
Location: Wall setting out

Co-ordinate Geometry

Define a Point

<table>
<thead>
<tr>
<th>Point number</th>
<th>Co-ordinates</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>y</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1000</td>
<td>2000</td>
</tr>
</tbody>
</table>

Define a Point

<table>
<thead>
<tr>
<th>Point number</th>
<th>Co-ordinates</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>y</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>6000</td>
<td>2000</td>
</tr>
</tbody>
</table>

Define a Point by Angle and Distance

<table>
<thead>
<tr>
<th>Point number</th>
<th>jj(1)</th>
<th>kj(1)</th>
<th>lj(1)</th>
<th>dj(1)</th>
<th>aj(1)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>J</td>
<td>2</td>
<td>1</td>
<td>3</td>
<td>3000</td>
<td>120°</td>
<td>SOP 3</td>
</tr>
</tbody>
</table>

Description of Point 3

Line runs from Point 2 to Point 1
Distance 3000 along a line defined by a clockwise angle 120° locates Point 3

<table>
<thead>
<tr>
<th>Point number</th>
<th>Co-ordinates</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>y</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>7500</td>
<td>4598.1</td>
</tr>
</tbody>
</table>

Generate a Parallel Line

<table>
<thead>
<tr>
<th>Point number</th>
<th>jk(1)</th>
<th>kk(1)</th>
<th>lk(1)</th>
<th>mk(1)</th>
<th>dk(1)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>J</td>
<td>1</td>
<td>2</td>
<td>6</td>
<td>11</td>
<td>-1000</td>
<td>TEMP 11</td>
</tr>
</tbody>
</table>

| Description of Point 6 | SOP 6 |
| Description of Point 11 |

SCALE 5.48 Office 1007 Proforma 581
First line runs from Point 1 to Point 2
Parallel line off-set by -1000 from first line
Second line runs from Point 6 to Point 11

<table>
<thead>
<tr>
<th>Point number</th>
<th>Co-ordinates</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>1000 3000</td>
<td>SOP 6</td>
</tr>
<tr>
<td>11</td>
<td>6000 3000</td>
<td>TEMP 11</td>
</tr>
</tbody>
</table>

Generate a Parallel Line

<table>
<thead>
<tr>
<th>Point number</th>
<th>Co-ordinates</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>J</td>
<td>JK(2)=2</td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>KK(2)=3</td>
<td></td>
</tr>
<tr>
<td>L</td>
<td>LK(2)=12</td>
<td></td>
</tr>
<tr>
<td>M</td>
<td>MK(2)=4</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>DK(2)=-750</td>
<td></td>
</tr>
<tr>
<td>Description</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Intersection of Two Lines

<table>
<thead>
<tr>
<th>Point number</th>
<th>Co-ordinates</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>J</td>
<td>JH(1)=6</td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>KH(1)=11</td>
<td></td>
</tr>
<tr>
<td>L</td>
<td>LH(1)=12</td>
<td></td>
</tr>
<tr>
<td>M</td>
<td>MH(1)=4</td>
<td></td>
</tr>
<tr>
<td>N</td>
<td>NH(1)=5</td>
<td></td>
</tr>
</tbody>
</table>

Intersection Point 5 of line from Point 6 to Point 11 with line from Point 12 to Point 4

<table>
<thead>
<tr>
<th>Point number</th>
<th>Co-ordinates</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>5711.3 3000</td>
<td>SOP 5</td>
</tr>
</tbody>
</table>
Table of Points

<table>
<thead>
<tr>
<th>Point number</th>
<th>Co-ordinates</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>y</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1000</td>
<td>2000</td>
</tr>
<tr>
<td>2</td>
<td>6000</td>
<td>2000</td>
</tr>
<tr>
<td>3</td>
<td>7500</td>
<td>4598.1</td>
</tr>
<tr>
<td>4</td>
<td>6850.5</td>
<td>4973.1</td>
</tr>
<tr>
<td>5</td>
<td>5711.3</td>
<td>3000</td>
</tr>
<tr>
<td>6</td>
<td>1000</td>
<td>3000</td>
</tr>
</tbody>
</table>

Distance Between Two Points

Point number J  \( je(1) = 1 \)
Point number K  \( ke(1) = 2 \)
Distance from Point 1 to Point 2 is 5000

Distance Between Two Points

Point number J  \( je(2) = 2 \)
Point number K  \( ke(2) = 3 \)
Distance from Point 2 to Point 3 is 3000

Distance Between Two Points

Point number J  \( je(3) = 3 \)
Point number K  \( ke(3) = 4 \)
Distance from Point 3 to Point 4 is 750

Distance Between Two Points

Point number J  \( je(4) = 4 \)
Point number K  \( ke(4) = 5 \)
Distance from Point 4 to Point 5 is 2278.3

Distance Between Two Points

Point number J  \( je(5) = 5 \)
Point number K  \( ke(5) = 6 \)
Distance from Point 5 to Point 6 is 4711.3
Distance Between Two Points

Point number J $\text{je}(6)=6$
Point number K $\text{ke}(6)=1$

Distance from Point 6 to Point 1 is 1000
Location: Example from 'Surveying' by Bannister & Raymond

Number of stations

N=7

X coordinate of station 1

x(1)=1000 m

Y coordinate of station 1

y(1)=2000 m

Degrees in angle

d(1)=187°

Minutes in angle

m(1)=22 '

Seconds in angle

s(1)=20"

Length

l(1)=310.207 m

Degrees in angle

d(2)=178°

Minutes in angle

m(2)=19 '

Seconds in angle

s(2)=0"

Length

l(2)=471.762 m

Degrees in angle

d(3)=118°

Minutes in angle

m(3)=21 '

Seconds in angle

s(3)=45"

Length

l(3)=430.087 m

Degrees in angle

d(4)=94°

Minutes in angle

m(4)=42 '

Seconds in angle

s(4)=25"

Length

l(4)=507.243 m

Degrees in angle

d(5)=158°

Minutes in angle

m(5)=7 '

Seconds in angle

s(5)=30"

Length

l(5)=530.206 m

Degrees in angle

d(6)=89°

Minutes in angle

m(6)=3 '

Seconds in angle

s(6)=55"

Length

l(6)=331.801 m

Degrees in angle

d(7)=167°

Minutes in angle

m(7)=15 '

Seconds in angle

s(7)=50"

Length

l(7)=422.493 m

Degrees in angle

d(8)=94°

Minutes in angle

m(8)=10 '

Seconds in angle

s(8)=0"

SUMMARY

<table>
<thead>
<tr>
<th>Station number</th>
<th>X coordinate (m)</th>
<th>Y coordinate (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1000</td>
<td>2000</td>
</tr>
<tr>
<td>2</td>
<td>960.15421</td>
<td>1692.377</td>
</tr>
<tr>
<td>3</td>
<td>913.334765</td>
<td>1222.96762</td>
</tr>
<tr>
<td>4</td>
<td>1269.62781</td>
<td>982.185927</td>
</tr>
<tr>
<td>5</td>
<td>1587.09201</td>
<td>1377.77935</td>
</tr>
<tr>
<td>6</td>
<td>1740.97017</td>
<td>1885.17674</td>
</tr>
<tr>
<td>7</td>
<td>1421.88825</td>
<td>1976.34081</td>
</tr>
</tbody>
</table>
Location: Tell-tales located on NE corner of store

Crack movement by tell-tales

Tell-tales are located at positions A, B & C. Tell tales A & B are approximately parallel to the crack denoted thus: \(\|\). The dimensions \(a, b \& c\) are measured by micrometer at various time intervals.

From the initial readings the angle at A is computed by trigonometry.

Let \(s = (a+b+c)/2\) then:

\[
\sin(A) = \frac{2}{bc} \sqrt{s(s-a)(s-b)(s-c)}
\]

then compute: \(x = b \cdot \sin(A)\) & \(y = b \cdot \cos(A)\).

Repeat for final condition and the difference between the two sets gives the movements normal and parallel to the crack.

Initial dimensions

\[
a = 120 \text{ mm} \\
b = 103.92 \text{ mm} \\
c = 60 \text{ mm}
\]

Final dimensions

\[
a' = 119.18 \text{ mm} \\
b' = 108.34 \text{ mm}
\]

Crack movements

Change in dimension normal to AB 4.0073 mm (+ve denotes opening)
parallel to AB -9.4421 mm (+ve in direction of A)
Location: Table for checking beam deflections

Elastic modulus for beam \( E_m = 205 \text{ kN/mm}^2 \)
Smallest span to be considered \( s_{\text{min}} = 6 \text{ m} \)
Largest span to be considered \( s_{\text{max}} = 8 \text{ m} \)
Increment in length of span \( s_{\text{inc}} = 2 \text{ m} \)
Smallest service load considered \( W_{\text{min}} = 50 \text{ kN} \)
Largest service load considered \( W_{\text{max}} = 300 \text{ kN} \)
Increment in service load \( W_{\text{inc}} = 50 \text{ kN} \)
Smallest inertia to be considered \( I_{\text{min}} = 2000 \text{ cm}^4 \)
Largest inertia to be considered \( I_{\text{max}} = 20000 \text{ cm}^4 \)
Increment in inertia \( I_{\text{inc}} = 2000 \text{ cm}^4 \)

Table of maximum deflections for 6 m span (Span/300 = 20 mm)
Table of maximum deflections for 8 m span (Span/300 = 27 mm)
Location: Shears, moments and deflections of beams

Shear, moment and deflection formulas for beams generally taken from 'Formulas for Stress and Strain' by Roark, published by McGraw-Hill with additions from 'Steel Designer's Manual'.

Inertia of beam $I = 0.294 \times 10^{-3}$ m$^4$

Elastic modulus for beam $E = 205$ GPa

--

Length of beam $l = 3$ m

Load $W = 10$ kN

Distance from end C to load $a = 3$ m

Maximum shear $V = -W = -10$ kN (B to C)

Maximum bending moment $M = -W \cdot a = -30$ kNm at C

Maximum deflection $y = -\left(\frac{W}{6Ei}\right) \cdot (3a^2 - a^3) = -0.0014933$ m
Location: Beams under combined axial and transverse loading

Formulas for beams under combined axial & transverse loading, Table VI from 'Formulas for Stress and Strain' by Roark, published McGraw-Hill.

- Young's modulus for beam \( E = 205 \times 10^6 \text{ kN/m}^2 \)
- Inertia of beam \( I = 4.77 \times 10^{-6} \text{ m}^4 \)
- Length of beam \( l = 3 \text{ m} \)
- Axial load \( P = 150 \text{ kN} \)

Moments are positive when clockwise, negative when anti-clockwise.
Deflections in y direction are positive when upwards, negative when downwards. Beam slope with horizontal (theta) positive when upward to the right.

**Cantilever beam under axial compression & transverse end load**

![Cantilever beam diagram]

- PL on cantilever end (+ve down) \( W = 10 \text{ kN} \)
- Maximum moment \( M = -W \times j \times \tan(U) = -61.101 \text{ kNm} \) (at \( x = l \))
- Maximum deflection \( y = -W \times (j \times \tan(U) - l) / P = -0.20734 \text{ m} \) (at \( x = 0 \))
- Maximum slope \( \theta = W \times (1 - \cos(U)) / \cos(U) / P = 0.10624 \text{ radians} \)
Location: Beams under combined axial and transverse loading

Formulas for beams under combined axial & transverse loading, Table VI from 'Formulas for Stress and Strain' by Roark, published McGraw-Hill.

- Young's modulus for beam \( E = 1.6 \times 10^6 \) kN/m²
- Inertia of beam \( I = 13.5 \) m⁴
- Length of beam \( l = 100 \) m
- Axial load \( P = 10000 \) kN

Moments are positive when clockwise, negative when anti-clockwise. Deflections in y direction are positive when upwards, negative when downwards. Beam slope with horizontal (theta) positive when upward to the right.

**Simply supported beam under axial compression & uniform transverse load**

UDL on SS beam (+ve down) \( w = 3 \) kN/m
Cross-sectional area of the strut \( A = 18 \) m²
Dist. from n.a. to outside fibre \( h = 1.5 \) m
Yield stress \( \sigma_y = 5000 \) kN/m²

Maximum moment
\[
M = c_1 (c_2 - 1) = 7162 \text{ kNm} \quad (at \ x = l/2)
\]

Maximum deflection
\[
y = -c_1 (c_2 - 1 - U^2/8)/P = -0.3412 \text{ m} \quad (at \ x = l/2)
\]

Maximum slope \[ \theta = -(w^2/2)(-U/2+c_3)/P \]
\[ = -0.01083 \text{ radians} \quad (at \ x = 0) \]
Location: Beams under combined axial and transverse loading

Formulas for beams under combined axial & transverse loading, Table VI from 'Formulas for Stress and Strain' by Roark, published McGraw-Hill.

Young's modulus for beam \( E = 30.464E^6 \) kN/m²
Inertia of beam \( I = 0.018565 \) m⁴
Length of beam \( l = 60 \) m
Axial load \( P = 500 \) kN

Moments are positive when clockwise, negative when anti-clockwise. Deflections in y direction are positive when upwards, negative when downwards. Beam slope with horizontal (theta) positive when upward to the right.

Simply supported beam under axial compression & uniform transverse load

UDL on SS beam (+ve down) \( w = 0.83 \) kN/m
Cross-sectional area of the strut \( A = 0.16608 \) m²
Dist. from n.a. to outside fibre \( h = 0.5 \) m
Yield stress \( p_y = 17920 \) kN/m²

Maximum moment \( M = c_1(c_2 - 1) = 556.47 \) kNm (at \( x = l/2 \))
Maximum deflection \( y = -c_1(c_2 - 1 - U^2/8)/P \)
\( = -0.36594 \) m (at \( x = l/2 \))
Maximum slope \( \theta = -w*j*(-U/2+c_3)/P \)
\( = -0.01941 \) radians (at \( x = 0 \))
**Location:** Two span continuous beam without cantilevers

<table>
<thead>
<tr>
<th>LH end</th>
<th>RH end</th>
<th>If no cantilever at left hand or right hand end, then cantilever moment is zero, clockwise FEM's are positive.</th>
</tr>
</thead>
<tbody>
<tr>
<td>![span 1]</td>
<td>![span n etc.]</td>
<td>![Inertia I(1)]</td>
</tr>
<tr>
<td>No. of spans excluding cantilevers n=2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- DL FEM for cantilever at LHE | DLlhcm=0 kNm |
- LL FEM for cantilever at LHE | LLlhcm=0 kNm |
- DL FEM for cantilever at RHE -ve | DLrhcm=0 kNm |
- LL FEM for cantilever at RHE | LLrhcm=0 kNm |
- Total No. of concentrated loads | Nc=1 |
- Total number of UDL's | Nu=1 |
- Span number containing load | Sc(1)=2 |
- Distance from left hand end | Lc(1)=6 m |
- Characteristic dead load | DLC(1)=60 kN |
- Characteristic live load | LLC(1)=60 kN |
- Span number containing UDL | Su(1)=1 |
- Distance from left end to start | Lau(1)=0 m |
- Distance from left end to end | Lbu(1)=12 m |
- Characteristic dead load | DLu(1)=10 kN/m |
- Characteristic live load | LLu(1)=20 kN/m |
- Partial safety factor for DL | gdl=1.4 |
- Partial safety factor for LL | gll=1.6 |
### Location: Three span beam with cantilevers

<table>
<thead>
<tr>
<th>LH end</th>
<th>span 1</th>
<th>span n etc.</th>
<th>RH end</th>
<th>If no cantilever at left hand or right hand end, then cantilever moment is zero, Clockwise FEM's are positive.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inertia I(1)</td>
<td>Inertia I(n)</td>
<td>Length l(1)</td>
<td>Length l(n)</td>
<td></td>
</tr>
</tbody>
</table>

No. of spans excluding cantilevers n=3

| DL FEM for cantilever at LHE | DLlhcm=112 kNm |
| LL FEM for cantilever at LHE | LLlhcm=103 kNm |
| DL FEM for cantilever at RHE -ve | DLrhcm=-112 kNm |
| LL FEM for cantilever at RHE | LLrhcm=-103 kNm |
| Total No. of concentrated loads | Nc=3 |
| Total number of UDL's | Nu=4 |
| Span number containing load | Sc(1)=1 |
| Distance from left hand end | Lc(1)=6 m |
| Characteristic dead load | Dlc(1)=30 kN |
| Characteristic live load | Llc(1)=35 kN |
| Span number containing load | Sc(2)=2 |
| Distance from left hand end | Lc(2)=3.5 m |
| Characteristic dead load | Dlc(2)=45 kN |
| Characteristic live load | Llc(2)=49 kN |
| Span number containing load | Sc(3)=3 |
| Distance from left hand end | Lc(3)=5.5 m |
| Characteristic dead load | Dlc(3)=45 kN |
| Characteristic live load | Llc(3)=49 kN |
| Span number containing UDL | Su(1)=1 |
| Distance from left end to start | Lau(1)=0 m |
| Distance from left end to end | Lbu(1)=12 m |
| Characteristic dead load | Dlu(1)=10 kN/m |
| Characteristic live load | Llu(1)=20 kN/m |
| Span number containing UDL | Su(2)=2 |
| Distance from left end to start | Lau(2)=0 m |
| Distance from left end to end | Lbu(2)=3.5 m |
| Characteristic dead load | Dlu(2)=23 kN/m |
| Characteristic live load | Llu(2)=28 kN/m |
| Span number containing UDL | Su(3)=2 |
| Distance from left end to start | Lau(3)=3.5 m |
| Distance from left end to end | Lbu(3)=12 m |
| Characteristic dead load | Dlu(3)=10 kN/m |
| Characteristic live load | Llu(3)=12 kN/m |
| Span number containing UDL | Su(4)=3 |
| Distance from left end to start | Lau(4)=0 m |
| Distance from left end to end | Lbu(4)=9 m |
| Characteristic dead load | Dlu(4)=9 kN/m |
| Characteristic live load | Llu(4)=7 kN/m |
| Partial safety factor for DL | gdl=1.4 |
| Partial safety factor for LL | gll=1.6 |
**Location: Four span beam with cantilever on right - unfactored loads**

<table>
<thead>
<tr>
<th>LH end</th>
<th>RH end</th>
<th>If no cantilever at left hand or right hand end, then cantilever moment is zero, Clockwise FEM's are positive.</th>
</tr>
</thead>
<tbody>
<tr>
<td>span 1</td>
<td>span n etc.</td>
<td></td>
</tr>
<tr>
<td>Inertia I(1)</td>
<td>Inertia I(n)</td>
<td></td>
</tr>
<tr>
<td>Length l(1)</td>
<td>Length l(n)</td>
<td></td>
</tr>
</tbody>
</table>

No. of spans excluding cantilevers n=4

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL FEM for cantilever at LHE</td>
<td>DLLhcm=0 kNm</td>
</tr>
<tr>
<td>LL FEM for cantilever at LHE</td>
<td>LLlhcm=0 kNm</td>
</tr>
<tr>
<td>DL FEM for cantilever at RHE -ve</td>
<td>DLrhcm=-112 kNm</td>
</tr>
<tr>
<td>LL FEM for cantilever at RHE</td>
<td>LLrhcm=-103 kNm</td>
</tr>
<tr>
<td>Total No. of concentrated loads</td>
<td>Nc=4</td>
</tr>
<tr>
<td>Total number of UDL's</td>
<td>Nu=4</td>
</tr>
<tr>
<td>Span number containing load</td>
<td>Sc(1)=1</td>
</tr>
<tr>
<td>Distance from left hand end</td>
<td>Lc(1)=6 m</td>
</tr>
<tr>
<td>Characteristic dead load</td>
<td>DLc(1)=30 kN</td>
</tr>
<tr>
<td>Characteristic live load</td>
<td>LLC(1)=35 kN</td>
</tr>
<tr>
<td>Span number containing load</td>
<td>Sc(2)=1</td>
</tr>
<tr>
<td>Distance from left hand end</td>
<td>Lc(2)=7 m</td>
</tr>
<tr>
<td>Characteristic dead load</td>
<td>DLc(2)=30 kN</td>
</tr>
<tr>
<td>Characteristic live load</td>
<td>LLC(2)=35 kN</td>
</tr>
<tr>
<td>Span number containing load</td>
<td>Sc(3)=1</td>
</tr>
<tr>
<td>Distance from left hand end</td>
<td>Lc(3)=8 m</td>
</tr>
<tr>
<td>Characteristic dead load</td>
<td>DLc(3)=30 kN</td>
</tr>
<tr>
<td>Characteristic live load</td>
<td>LLC(3)=35 kN</td>
</tr>
<tr>
<td>Span number containing load</td>
<td>Sc(4)=1</td>
</tr>
<tr>
<td>Distance from left hand end</td>
<td>Lc(4)=9 m</td>
</tr>
<tr>
<td>Characteristic dead load</td>
<td>DLc(4)=30 kN</td>
</tr>
<tr>
<td>Characteristic live load</td>
<td>LLC(4)=35 kN</td>
</tr>
<tr>
<td>Span number containing UDL</td>
<td>Su(1)=1</td>
</tr>
<tr>
<td>Distance from left end to start</td>
<td>Lau(1)=0 m</td>
</tr>
<tr>
<td>Distance from left end to end</td>
<td>Lbu(1)=12 m</td>
</tr>
<tr>
<td>Characteristic dead load</td>
<td>DLu(1)=10 kN/m</td>
</tr>
<tr>
<td>Characteristic live load</td>
<td>LLu(1)=20 kN/m</td>
</tr>
<tr>
<td>Span number containing UDL</td>
<td>Su(2)=2</td>
</tr>
<tr>
<td>Distance from left end to start</td>
<td>Lau(2)=0 m</td>
</tr>
<tr>
<td>Distance from left end to end</td>
<td>Lbu(2)=8 m</td>
</tr>
<tr>
<td>Characteristic dead load</td>
<td>DLu(2)=23 kN/m</td>
</tr>
<tr>
<td>Characteristic live load</td>
<td>LLu(2)=28 kN/m</td>
</tr>
<tr>
<td>Span number containing UDL</td>
<td>Su(3)=3</td>
</tr>
<tr>
<td>Distance from left end to start</td>
<td>Lau(3)=0 m</td>
</tr>
<tr>
<td>Distance from left end to end</td>
<td>Lbu(3)=5 m</td>
</tr>
<tr>
<td>Characteristic dead load</td>
<td>DLu(3)=23 kN/m</td>
</tr>
<tr>
<td>Characteristic live load</td>
<td>LLu(3)=28 kN/m</td>
</tr>
<tr>
<td>Span number containing UDL</td>
<td>Su(4)=4</td>
</tr>
<tr>
<td>Distance from left end to start</td>
<td>Lau(4)=0 m</td>
</tr>
<tr>
<td>Distance from left end to end</td>
<td>Lbu(4)=6 m</td>
</tr>
<tr>
<td>Characteristic dead load</td>
<td>DLu(4)=23 kN/m</td>
</tr>
<tr>
<td>Characteristic live load</td>
<td>LLu(4)=28 kN/m</td>
</tr>
</tbody>
</table>
Location: Beam at A/1

Cantilever has 'n' segments with inertias $i_z(1)-i_z(n)$ and spans $s(1)-s(n)$ in order, left to right.

Number of segments in cantilever $n=2$
End point load $p=1$ kN
Young's modulus $e=1E6$ kN/m²
Length of segment 1 $s(i)=1.2$ m (left to right)
Inertia of segment 1 $i_z(i)=0.01$ m⁴ (left to right)
Length of segment 2 $s(i)=1.8$ m (left to right)
Inertia of segment 2 $i_z(i)=0.005$ m⁴ (left to right)
Total length of cantilever $3$ m
Maximum moment $m_m=p\cdot l=3$ kNm
Area of bmd rectangular segment $a_r=p\cdot l\cdot s(i)=2.16$ kNm²
Lever arm of rectangular segment $l_r=l+s(i)/2=2.4$ m
Area of bmd triangular segment $a_t=p\cdot s(i)^2/2=0.72$ kNm²
Lever arm of triangular segment $l_t=l+2\cdot s(i)/3=2.6$ m
Cumulate rotation components $\theta=\theta+(a_r+a_t)/(e\cdot i_z(i))$
$=0.288E-3$ radians
Cumulate deflection components $\delta=\delta+(a_r\cdot l_r+a_t\cdot l_t)/(e\cdot i_z(i))$
$=0.7056E-3$ m
Area of bmd rectangular segment $a_r=p\cdot l\cdot s(i)=-0.39968E-15$ kNm²
Lever arm of rectangular segment $l_r=l+s(i)/2=0.9$ m
Area of bmd triangular segment $a_t=p\cdot s(i)^2/2=1.62$ kNm²
Lever arm of triangular segment $l_t=l+2\cdot s(i)/3=1.2$ m
Cumulate rotation components $\theta=\theta+(a_r+a_t)/(e\cdot i_z(i))$
$=0.612E-3$ radians
Cumulate deflection components $\delta=\delta+(a_r\cdot l_r+a_t\cdot l_t)/(e\cdot i_z(i))$
$=0.0010944$ m
Location: Simply supported beam

![Beam diagram]

Elastic modulus \( E = 205 \times 10^6 \) kN/m²
Beam span \( L = 3 \) m
Inertia about major axis \( I = 4.77 \times 10^{-6} \) m⁴
Dead load factor \( \gamma_{d} = 1.4 \)
Live load factor \( \gamma_{l} = 1.6 \)

All loads are positive downwards, reactions are positive upwards, sagging moments are positive.

\[
\begin{array}{c|c}
Lc & DLc \\
\hline
& Characteristic dead point load \\
LLc & Characteristic live point load \\
\end{array}
\]

Distances are measured from left hand support

Distance from left support \( Lc(1) = 1.2 \) m
Characteristic dead load \( DLc(1) = 2 \) kN
Characteristic live load \( LLc(1) = 2.4 \) kN

\[
\begin{array}{c|c}
Lbu & DLu \\
\hline
& Characteristic dead UDL \\
Lau & LLu Characteristic live UDL \\
\end{array}
\]

Distances are measured from left hand support

Distance from left support to start \( Lau(1) = 0 \) m
Distance from left support to end \( Lbu(1) = 1.2 \) m
Characteristic dead load \( DLu(1) = 0.8 \) kN/m
Characteristic live load \( LLu(1) = 0.4 \) kN/m

\[
\begin{array}{c|c|c|c|c|c}
0 & 1.1374 & 2.2352 & 3.2935 & 4.3121 & 5.2911 \\
6.2305 & 7.1303 & 7.9906 & 7.7925 & 7.5093 \\
7.1411 & 6.6878 & 6.1495 & 5.5262 & 4.8178 \\
4.0243 & 3.1458 & 2.1823 & 1.1337 & 0 \\
\end{array}
\]

Factored maximum span bending moment 7.9906 kNm (the factors are 1.4 & 1.6)

BM at 20th points, from left to right (sagging is positive)

End shears

Shear force at left hand end \( 7.7148 \) kN
Shear force at right hand end \( 7.8412 \) kN

Unfactored dead shear at LHE \( 2.562 \) kN
Unfactored live shear at LHE \( 2.58 \) kN
Unfactored dead shear at RHE \( 2.378 \) kN
Unfactored live shear at RHE   2.82 kN

**Midspan deflections**

- Unfactored dead load deflection  0.0021615 m
- Unfactored live load deflection  0.0024433 m
- Total dead & live deflections    0.0046048 m
- Span:defln ratio for dead load    1387.9
- Span:defln ratio for live load    1227.9
- Span:defln ratio for total load    651.5
- Shear area for beam              $Ay=0.5334E-3 \text{ m}^2$
- Total unfactored dead & live load applied as a UDL, additional defl due to shear $dsu=WT'*L/(8*Ay*G)=92.018E-6 \text{ m}^2$
- Approximate defl due to shear $ds=dsu*M/((gamd+gami)/2)/(WT'*L/8)$ $=0.12642E-3 \text{ m}$
- Total dead + live + shear defl $ dt=dld+ild+ds=0.0047312 \text{ m}$
Location: SS beam with UDL, point & triangular loads

---

Elastic modulus $E=7.9E6$ kN/m²
Beam span $L=4$ m
Inertia about major axis $I=65.104E-6$ m⁴
Dead load factor $g_{ma}=1.4$
Live load factor $g_{mi}=1.6$
All loads are positive downwards, reactions are positive upwards, sagging moments are positive.

---

<table>
<thead>
<tr>
<th>Lc</th>
<th>DLc Characteristic dead point load</th>
</tr>
</thead>
<tbody>
<tr>
<td>LLc</td>
<td>Characteristic live point load</td>
</tr>
</tbody>
</table>

---

Distances are measured from left hand support

Distance from left support $Lc(1)=2$ m
Characteristic dead load $DLc(1)=2$ kN
Characteristic live load $LLc(1)=3$ kN

---

<table>
<thead>
<tr>
<th>Lbu</th>
<th>DLu Characteristic dead UDL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lau</td>
<td>LLu Characteristic live UDL</td>
</tr>
</tbody>
</table>

---

Distances are measured from left hand support

Dist. from left support to start $Lau(1)=0$ m
Dist. from left support to end $Lbu(1)=4$ m
Characteristic dead load $DLu(1)=0.5$ kN/m
Characteristic live load $LLu(1)=0.3$ kN/m

---

Loads are measured at centre of span in kN/m units where:
$DLt$ Characteristic dead kN/m
$LLt$ Characteristic live kN/m

---

Characteristic dead ld at centre $DLt(1)=3$ kN/m
Characteristic live ld at centre $LLt(1)=1$ kN/m

---

Loads are measured at right support in kN/m units where:
$DLr$ Characteristic dead kN/m
$LLr$ Characteristic live kN/m

---
Characteristic dead load at right DLr(1)=1 kN/m
Characteristic live load at right LLr(1)=1 kN/m

**BMs at 20th points, from left to right (sagging is positive)**

<p>| | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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<th></th>
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<tbody>
<tr>
<td>0</td>
<td>2.7635</td>
<td>5.4507</td>
<td>8.0322</td>
<td>10.479</td>
</tr>
<tr>
<td>19.867</td>
<td>18.718</td>
<td>17.264</td>
<td>15.523</td>
<td>13.512</td>
</tr>
<tr>
<td>11.247</td>
<td>8.7462</td>
<td>6.0267</td>
<td>3.1055</td>
<td>0</td>
</tr>
</tbody>
</table>

Factored maximum span bending moment 20.693 kNm
(the factors are 1.4 & 1.6)

**End shears**

Shear force at left hand end 13.96 kN
Shear force at right hand end 15.96 kN

Unfactored dead shear at LHE 5.6667 kN
Unfactored live shear at LHE 3.7667 kN
Unfactored dead shear at RHE 6.3333 kN
Unfactored live shear at RHE 4.4333 kN

**Midspan deflections**

Unfactored dead load deflection 0.024109 m
Unfactored live load deflection 0.017111 m
Total dead & live deflections 0.041219 m
Span:defln ratio for dead load 165.91
Span:defln ratio for live load 233.78
Span:defln ratio for total load 97.04
Shear area for beam $Ay=0.0125$ m$^2$
Total unfactored dead & live $WT'=20.2$ kN
If total unfactored dead & live load applied as a UDL,
additional defln due to shear $dsu=WT'*L/(8*Ay*G)=0.0016365$ m
Approximate defln due to shear $ds=dsu*M/((gamd+gami)/2)/(WT'*L/8)=0.0022352$ m
Total dead + live + shear defln $dt=dld+ild+ds=0.043455$ m
Location: Lean-to type 1

Length of non-sloping member \( a = 4 \) m  
Horizontal projection of sloping member \( b = 3 \) m  
Sloping member projection on vertical \( c = 2 \) m

Moment of inertia of member 1-2 \( I_{12} = 8.38 \times 10^{-6} \) m\(^4\)  
Moment of inertia of member 2-3 \( I_{23} = 4.77 \times 10^{-6} \) m\(^4\)

Notation 1->2 denotes: at joint 1 in the member joining joints 1 & 2  
Loads and reactions are positive in the directions shown.

Vertical load per unit length \( w_1 = 2 \) kN/m  
Moment at 2 (+ve hogging) \( M = \frac{w_1 \cdot r^2}{8} \cdot \frac{I_{12}}{a} \cdot \frac{I_{12}}{a + I_{23}} \)  
\( = 1.6575 \) kNm

Horizontal react at 1 (+ve right) \( H_1 = M/a = 0.41437 \) kN  
Horizontal react at 3 (+ve left) \( H_3 = 0.41437 \) kN

Vertical reaction at 3 (+ve up) \( V_3 = (w_1 \cdot r \cdot b / 2 - H_1 \cdot (a + c)) / b = 2.7768 \) kN

Vertical reaction at 1 (+ve up) \( V_1 = w_1 \cdot r - V_3 = 4.4343 \) kN

Axial force 1->2 (+ve compression) \( F_1 = V_1 = 4.4343 \) kN  
Axial force 2->3 (+ve compression) \( F_2 = V_1 \cdot \sin(\alpha) + H_1 \cdot \cos(\alpha) = 2.8045 \) kN

Shear force 2->3 \( S_2 = V_1 \cdot \cos(\alpha) - H_1 \cdot \sin(\alpha) = 3.4597 \) kN
Plan load per unit length \( w_2 = 0.8 \text{ kN/m} \)

Moment at 2 (+ve hogging) \( M = \frac{(w'r^2/8)(I_{12}/a)(I_{12}/a+I_{23}/r)}{I_{12}/a+I_{23}/r} \)
\( = 0.55164 \text{ kNm} \)

Horizontal react at 1 (+ve right) \( H_1 = M/a = 0.13791 \text{ kN} \)

Horizontal react at 3 (+ve left) \( H_3 = 0.13791 \text{ kN} \)

Vertical reaction at 3 (+ve up) \( V_3 = \frac{(w_2'b^2/2-H_1(a+c))/b}{0.92418 \text{ kN}} \)

Vertical reaction at 1 (+ve up) \( V_1 = w_2'b - V_3 = 0.92418 \text{ kN} \)

Axial force 1→2 (+ve compressn) \( F_1 = V_1 = 1.4758 \text{ kN} \)

Axial force 2→3 (+ve compressn) \( F_2 = V_1 \times \sin(\alpha) + H_1 \times \cos(\alpha) \)
\( = 0.93339 \text{ kN} \)

Shear force 2→3 \( S_2 = V_1 \times \cos(\alpha) - H_1 \times \sin(\alpha) \)
\( = 1.1515 \text{ kN} \)

Suction on rafter (+ve) \( w_3 = 0.75 \text{ kN/m} \)

Moment at 2 (+ve hogging) \( M = \frac{(w'r^2/8)(I_{12}/a)(I_{12}/a+I_{23}/r)}{I_{12}/a+I_{23}/r} \)
\( = -0.74702 \text{ kNm} \)

Horizontal react at 1 (+ve right) \( H_1 = M/a = -0.18675 \text{ kN} \)

Horizontal react at 3 (+ve left) \( H_3 = H_1 - w_3'r \times \sin(\alpha) = -1.6868 \text{ kN} \)

Vertical reaction at 3 (+ve up) \( V_3 = \frac{(-w_3'r^2/2-M-H_3^c)/b}{0.25149 \text{ kN}} \)

Vertical reaction at 1 (+ve up) \( V_1 = -w_3'r \times \cos(\alpha) - V_3 = -1.9985 \text{ kN} \)

Axial force 1→2 (+ve compressn) \( F_1 = V_1 = -1.9985 \text{ kN} \)

Axial force 2→3 (+ve compressn) \( F_2 = V_1 \times \sin(\alpha) + H_1 \times \cos(\alpha) \)
\( = -1.264 \text{ kN} \)

Shear force 2→3 \( S_2 = V_1 \times \cos(\alpha) - H_1 \times \sin(\alpha) \)
\( = -1.5593 \text{ kN} \)
Wind on side (+ve to left) $w_4 = 0.8 \text{ kN/m}$
Moment at 2 (+ve hogging) 
$$M = \frac{(w_4 \cdot a^2/2)(I_{23}/r)}{(I_{12}/a+I_{23}/r)} = 0.6193 \text{ kNm}$$

Horizontal react at 1 (+ve right) $H_1 = \frac{(M-w_4 \cdot a^2/2)}{a} = -1.4452 \text{ kN}$
Horizontal react at 3 (+ve left) $H_3 = H_1 + w_4 \cdot a = 1.7548 \text{ kN}$
Vertical reaction at 3 (+ve up) $V_3 = \frac{(-M-H_3 \cdot c)}{b} = -1.3763 \text{ kN}$
Vertical reaction at 1 (+ve up) $V_1 = -V_3 = 1.3763 \text{ kN}$

Axial force 1→2 (+ve compressn) $F_1 = V_1 = 1.3763 \text{ kN}$
Shear force at top of vertical $S = H_1 + w_4 \cdot a = 1.7548 \text{ kN}$
Axial force 2→3 (+ve compressn) $F_2 = V_1 \cdot \sin(\alpha) + S \cdot \cos(\alpha) = 2.2235 \text{ kN}$
Shear force 2→3 $S_2 = V_1 \cdot \cos(\alpha) - S \cdot \sin(\alpha) = 0.17176 \text{ kN}$

Vertical load at joint 2 $W_{2v} = 1.7 \text{ kN}$
Horizontal load at joint 2 $W_{2h} = 0.6 \text{ kN}$

Vertical load at joint 2 $W_{2v} = 1.7 \text{ kN}$
Horizontal load at joint 2 $W_{2h} = 0.6 \text{ kN}$

Axial force 1→2 (+ve compressn) $F_1 = V_1 = 1.3763 \text{ kN}$
Shear force at top of vertical $S = H_1 + w_4 \cdot a = 1.7548 \text{ kN}$
Axial force 2→3 (+ve compressn) $F_2 = V_1 \cdot \sin(\alpha) + S \cdot \cos(\alpha) = 2.2235 \text{ kN}$
Shear force 2→3 $S_2 = V_1 \cdot \cos(\alpha) - S \cdot \sin(\alpha) = 0.17176 \text{ kN}$

Vertical reaction at 1 (+ve up) $V_1 = 2.1 \text{ kN}$
Vertical reaction at 3 (+ve up) $V_3 = W_{2v} - V_1 = -0.4 \text{ kN}$
**Location: Lean-to type 2**

- Alpha may be 90 degrees but not less than 60 degrees
- Length of non-sloping member: \( a = 1.8 \) m
- Horizontal projection of sloping member: \( b = 0.6 \) m
- Sloping member projection on vertical: \( c = 3.2 \) m
- Moment of inertia of member 1-2: \( I_{12} = 8.38 \times 10^{-6} \) m\(^4\)
- Moment of inertia of member 2-3: \( I_{23} = 4.77 \times 10^{-6} \) m\(^4\)

Notation 1->2 denotes: at joint 1 in the member joining joints 1 & 2

Loads and reactions are positive in the directions shown.

- Vertical load per unit length: \( w_1 = 2 \) kN/m
- Moment at 2:
  \[
  M = \frac{(w_1a^2/2)(I_{12}/r)}{(I_{12}/r+I_{23}/a)}
  = 0.3991 \text{ kNm}
  \]

- Vertical reaction at 3 (+ve up):
  \[
  V_3 = \frac{(w_1a^2/2-M)}{a} = 1.5783 \text{ kN}
  \]

- Vertical reaction at 1 (+ve up):
  \[
  V_1 = w_1a - V_3 = 2.0217 \text{ kN}
  \]

- Horizontal reaction at 1 (+ve right):
  \[
  H_1 = \frac{(V_1b+M)}{c} = 0.50379 \text{ kN}
  \]

- Horizontal reaction at 3 (+ve left):
  \[
  H_3 = 0.50379 \text{ kN}
  \]

- Axial force 1->2 (+ve compress):
  \[
  F_1 = V_1 \sin(\alpha) + H_1 \cos(\alpha) = 2.0799 \text{ kN}
  \]

- Shear force 1->2:
  \[
  S_1 = V_1 \cos(\alpha) - H_1 \sin(\alpha) = -0.12258 \text{ kN}
  \]
Vertical load per unit length \( w_2 = 0.8 \text{ kN/m} \)
Moment at 2
\[
M = (w \cdot r^2 / 8) \cdot (I_{23}/a) / (I_{12}/r + I_{23}/a)
\]
\( = 0.099096 \text{ kNm} \)
Vertical reaction at 3 (+ve up) \( V_3 = -M/a = -0.055053 \text{ kN} \)
Vertical reaction at 1 (+ve up) \( V_1 = w_2 \cdot r - V_3 = 2.6597 \text{ kN} \)
Horizontal react at 1 (+ve right) \( H_1 = (V_1 \cdot b + M - w_2 \cdot r \cdot b / 2) / c = 0.28547 \text{ kN} \)
Horizontal react at 3 (+ve left) \( H_3 = 0.28547 \text{ kN} \)
Axial force 1->2 (+ve compressn) \( F_1 = V_1 \cdot \sin(\alpha) + H_1 \cdot \cos(\alpha) = 2.6667 \text{ kN} \)
Shear force 1->2 \( S_1 = V_1 \cdot \cos(\alpha) - H_1 \cdot \sin(\alpha) = 0.20956 \text{ kN} \)

Plan load per unit length \( w_3 = 0.3 \text{ kN/m} \)
Moment at 2
\[
M = (w \cdot r^2 / 8) \cdot (I_{23}/a) / (I_{12}/r + I_{23}/a)
\]
\( = 0.0068483 \text{ kNm} \)
Vertical reaction at 3 (+ve up) \( V_3 = -M/a = -0.0038046 \text{ kN} \)
Vertical reaction at 1 (+ve up) \( V_1 = w_3 \cdot b - V_3 = 0.1838 \text{ kN} \)
Horizontal react at 3 (+ve left) \( H_3 = (w_3 \cdot b^2 / 2 - V_3 \cdot (a+b)) / c = 0.019728 \text{ kN} \)
Horizontal react at 1 (+ve right) \( H_1 = 0.019728 \text{ kN} \)
Axial force 1->2 (+ve compressn) \( F_1 = V_1 \cdot \sin(\alpha) + H_1 \cdot \cos(\alpha) = 0.18429 \text{ kN} \)
Shear force 1->2 \( S_1 = V_1 \cdot \cos(\alpha) - H_1 \cdot \sin(\alpha) = 0.014483 \text{ kN} \)
Suction on rafter (+ve) 

\[ w_4 = 0.75 \text{ kN/m} \]

Moment at 2 

\[ M = \left( \frac{w \times r^2}{8} \right) \left( I_{23}/a \right) / \left( I_{12}/r + I_{23}/a \right) \]

\[ = -0.50411 \text{ kNm} \]

Vertical reaction at 3 (+ve up) 

\[ V_3 = -M/a = 0.28006 \text{ kN} \]

Vertical reaction at 1 (+ve up) 

\[ V_1 = -w_4 \times r \times \cos(\alpha) - V_3 = -0.73006 \text{ kN} \]

Horizontal react at 1 (+ve right) 

\[ H_1 = (V_1 \times b + M + w_4 \times r^2/2)/c = 0.94777 \text{ kN} \]

Horizontal react at 3 (+ve left) 

\[ H_3 = H_1 - w_4 \times r \times \sin(\alpha) = -1.4522 \text{ kN} \]

Axial force 1->2 (+ve compressn) 

\[ F_1 = V_1 \times \sin(\alpha) + H_1 \times \cos(\alpha) = -0.5429 \text{ kN} \]

Shear force 1->2 

\[ S_1 = V_1 \times \cos(\alpha) - H_1 \times \sin(\alpha) = -1.0661 \text{ kN} \]

Vertical load at joint 2 

\[ W_{2v} = 1.7 \text{ kN} \]

Horizontal load at joint 2 

\[ W_{2h} = 0.6 \text{ kN} \]

For small displacements, the horizontal load \( W_{2h} \) causes compression in member 2-3 only; but the vertical force \( W_{2v} \) causes compression in members 1-2 and 2-3. Resolving vertically at joint 2:

Vertical reaction at 3 (+ve up) 

\[ V_3 = 0 \text{ kN} \]

Vertical reaction at 1 (+ve up) 

\[ V_1 = W_{2v} = 1.7 \text{ kN} \]

Axial force 1->2 (+ve compressn) 

\[ F_1 = W_{2v} / \sin(\alpha) = 1.7296 \text{ kN} \]

Shear force 1->2 

\[ S_1 = 0 \text{ kN} \]

Horizontal react at 1 (+ve right) 

\[ H_1 = V_1 \times b / c = 0.31875 \text{ kN} \]

Horizontal react at 3 (+ve left) 

\[ H_3 = H_1 + W_{2h} = 0.91875 \text{ kN} \]

Axial force 2->3 (+ve compressn) 

\[ F_2 = H_3 = 0.91875 \text{ kN} \]

Shear force 2->3 

\[ S_2 = 0 \text{ kN} \]
Location: Influence lines for two span beam

The procedure sets up and reduces a stiffness matrix in which each beam element contributes the submatrices. The matrix is banded 4 elements wide. For each influence line, forces are applied to the appropriate element. The deflected form of the beam is the influence line by the Muller-Breslau principle. Areas in each span are computed by Simpson's rule.

Influence lines for shear are given for all points in the nominated span. The values for shear are those due to a unit load applied just to the right of the position to which the value is referenced. Therefore subtract the value from unity for shears when the unit load is applied just to the left of the reference position.

Number of spans $s_1=2$

Spans in order left to right
(or span ratios):

L₁ = 10 m
L₂ = 10 m

Uniform inertia set in L₁₁ et seq.

Influence lines in order now follow:

Influence line for support number x(1)=1

Influence line for shear in span x(2)=2

Influence line for mmt at point x(3)=2

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Location: Influence lines for eight span beam

The procedure sets up and reduces a stiffness matrix in which each beam element contributes the submatrices. The matrix is banded 4 elements wide. For each influence line, forces are applied to the appropriate element. The deflected form of the beam is the influence line by the Muller-Breslau principle. Areas in each span are computed by Simpson's rule.

Influence lines for shear are given for all points in the nominated span. The values for shear are those due to a unit load applied just to the right of the position to which the value is referenced. Therefore subtract the value from unity for shears when the unit load is applied just to the left of the reference position.

Number of spans $s_1=8$

Spans in order left to right (or span ratios):

$$
\begin{align*}
L(1) &= 10 \text{ m} \\
L(2) &= 8 \text{ m} \\
L(3) &= 10 \text{ m} \\
L(4) &= 8 \text{ m} \\
L(5) &= 7 \text{ m} \\
L(6) &= 6 \text{ m} \\
L(7) &= 7 \text{ m} \\
L(8) &= 8 \text{ m}
\end{align*}
$$

Inertias in order left to right (or inertia ratios) stored in $L(11), L(12),...$

$$
\begin{align*}
L(11) &= 1 \text{ m}^4 \\
L(12) &= 1.2 \text{ m}^4 \\
L(13) &= 1 \text{ m}^4 \\
L(14) &= 1.2 \text{ m}^4 \\
L(15) &= 1 \text{ m}^4 \\
L(16) &= 2 \text{ m}^4 \\
L(17) &= 2.1 \text{ m}^4 \\
L(18) &= 2.2 \text{ m}^4
\end{align*}
$$

Influence lines in order now follow:

Influence line for support number $x(1)=5$

Influence line for shear in span $x(2)=7$
Influence lines

Ordinates of influence lines in order — left to right

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Location: Slab with fixed edges supporting central concentrated load

Slab moments due to concentrated loads using Pigeaud's coefficients

Concrete slab with concentrated load symmetrical about both axes

Total load = F kN
All dimensions in metres

Poisson's ratio nu = 0.2
Shorter span lx = 3 m
Longer span ly = 3.75 m

Load details
Load length in lx direction ax = 0.525 m
Load length in ly direction ay = 1.05 m
Individual concentrated load F = 200 kN

Continuity conditions
Two-way slab with all edges continuous.

Calculation of bending moments

Bending moment in x-direction
Mx = F * L * Factor * (alfx1 + nu * alfy1)
= 41.094 kNm/m

Bending moment in y-direction
My = F * L * Factor * (nu * alfx1 + alfy1)
= 31.47 kNm/m

Taking account of suggested allowances for continuity in x-direction:
Bending moment at end support Mx1 = -0.25 * Mx = -10.274 kNm/m
Maximum moment in end span Mx2 = 0.85 * Mx = 34.93 kNm/m
Moment at penultimate support Mx3 = -0.85 * Mx = -34.93 kNm/m
Maximum moment in interior span Mx4 = 0.70 * Mx = 28.766 kNm/m
Moment at interior supports Mx5 = -0.90 * Mx = -36.985 kNm/m

Taking account of suggested allowances for continuity in y-direction:
Bending moment at end support My1 = -0.25 * My = -7.8675 kNm/m
Maximum moment in end span My2 = 0.85 * My = 26.75 kNm/m
Moment at penultimate support My3 = -0.85 * My = -26.75 kNm/m
Maximum moment in interior span My4 = 0.70 * My = 22.029 kNm/m
Moment at interior supports My5 = -0.90 * My = -28.323 kNm/m
Effective width in direction of shorter span \( lx \)

Effective width of slab \( ly_{Eff} = ay + 2lx^2 = 2.85 \) m

Effective width in direction of longer span \( ly \)

Effective width of slab \( lx_{Eff} = ax + 2ly^2 = 2.775 \) m

Approximate shearing forces from each load \( F \)

Shear force at centre of \( ax \) \( V_{ax} = F/(3ay) = 63.492 \) kN/m

Shear force at centre of \( ay \) \( V_{ay} = F/(2ay+ax) = 76.19 \) kN/m
Location: Square fixed-edge slab supporting twin loads

Concrete slab with concentrated loads symmetrical about both axes

Each individual total load = F kN

All dimensions in metres

Poisson's ratio nu=0.2
Shorter span lx=5 m
Longer span ly=5 m

Load details
Load length in lx direction ax=2.5 m
Start of load from nearer support x=0 m
Load length in ly direction ay=1.05 m
Individual concentrated load F=450 kN

Continuity conditions
Two-way slab with all edges continuous.

Calculation of bending moments

Bending moment in x-direction
Mx=F*Factor*(alfx1+nu*alfy1)
=67.302 kNm/m

Bending moment in y-direction
My=F*Factor*(nu*alfx1+alfy1)
=86.814 kNm/m

Taking account of suggested allowances for continuity in x-direction:
Bending moment at end support Mx1=-0.25*Mx=-16.826 kNm/m
Maximum moment in end span Mx2=0.85*Mx=57.207 kNm/m
Moment at penultimate support Mx3=-0.85*Mx=-57.207 kNm/m
Maximum moment in interior span Mx4=0.70*Mx=47.111 kNm/m
Moment at interior supports Mx5=-0.90*Mx=-60.572 kNm/m

Taking account of suggested allowances for continuity in y-direction:
Bending moment at end support My1=-0.25*My=-21.704 kNm/m
Maximum moment in end span My2=0.85*My=73.792 kNm/m
Moment at penultimate support My3=-0.85*My=-73.792 kNm/m
Maximum moment in interior span My4=0.70*My=60.77 kNm/m
Moment at interior supports My5=-0.90*My=-78.133 kNm/m
Effective width in direction of shorter span $lx$

Effective width of slab $ly_{Eff} = ay + 2lx^2 = 3.3 \text{ m}$

Effective width in direction of longer span $ly$

Effective width is equal to total slab width $lx$.

Effective width of slab $5 \text{ m}$

Approximate shearing forces from each load $F$

Shear force at centre of $ax$ $V_{ax} = F / (2ax + ay) = 74.38 \text{ kN/m}$

Shear force at centre of $ay$ $V_{ay} = F / (3ax) = 60 \text{ kN/m}$
Location: Slab w. two adjacent edges continuous & unsymmetrical load

Slab moments due to concentrated loads using Pigeaud's coefficients

Concentrated load is unsymmetrical about both axes

\[
\begin{align*}
\text{Total load} & = F \text{ kN} \\
\text{All dimensions} & \text{ in metres} \\
\end{align*}
\]

Poisson's ratio \( \nu = 0.2 \)
Shorter span \( l_x = 4 \text{ m} \)
Longer span \( l_y = 7 \text{ m} \)

Load details

- Load length in \( l_x \) direction \( a_x = 2 \text{ m} \)
- Start of load from nearer support \( x = 0 \text{ m} \)
- Load length in \( l_y \) direction \( a_y = 3.5 \text{ m} \)
- Start of load from nearer support \( y = 0 \text{ m} \)
- Individual concentrated load \( F = 500 \text{ kN} \)

Continuity conditions

Two-way slab with two adjacent edges continuous.

WARNING:
Specified combination of slab dimensions and fixity conditions falls outside validity of Pigeaud's tables.
Location: To give table failure

Slab moments due to concentrated loads using Pigeaud's coefficients

Concentrated load is unsymmetrical about both axes

Poisson's ratio \( \nu = 0.2 \)
Shorter span \( l_x = 1 \, \text{m} \)
Longer span \( l_y = 20 \, \text{m} \)

Load details
Load length in \( l_x \) direction \( a_x = 0.5 \, \text{m} \)
Start of load from nearer support \( x = 0 \, \text{m} \)
Load length in \( l_y \) direction \( a_y = 10 \, \text{m} \)
Start of load from nearer support \( y = 0 \, \text{m} \)
Individual concentrated load \( F = 0 \, \text{kN} \)

Continuity conditions
Two-way slab with two adjacent edges continuous.

WARNING:
Specified combination of slab dimensions and fixity conditions falls outside validity of Pigeaud's tables.
Location: Slab with fixed edges supporting central concentrated load

Slab moments due to concentrated loads using Pigeaud's coefficients

![Concrete slab with concentrated load symmetrical about both axes]

- Poisson's ratio: $\nu=0.2$
- Shorter span: $l_x=3\,\text{m}$
- Longer span: $l_y=3.75\,\text{m}$

**Load details**

- Load length in $l_x$ direction: $a_x=0.525\,\text{m}$
- Load length in $l_y$ direction: $a_y=1.05\,\text{m}$
- Individual concentrated load: $F=200\,\text{kN}$

**Continuity conditions**

Two-way slab with all edges continuous.

**Calculation of bending moments**

- Bending moment in $x$-direction: $M_x=F\times LF_{\text{Factor}}\times (a_{lfx}+\nu\times a_{lfy})$
  
  \[=41.094\,\text{kNm/m}\]

- Bending moment in $y$-direction: $M_y=F\times LF_{\text{Factor}}\times (\nu\times a_{lfx}+a_{lfy})$
  
  \[=31.47\,\text{kNm/m}\]

Taking account of suggested allowances for continuity in $x$-direction:

- Bending moment at end support: $M_{x1}=-0.25\times M_x=-10.274\,\text{kNm/m}$
- Maximum moment in end span: $M_{x2}=0.85\times M_x=34.93\,\text{kNm/m}$
- Moment at penultimate support: $M_{x3}=-0.85\times M_x=-34.93\,\text{kNm/m}$
- Maximum moment in interior span: $M_{x4}=0.70\times M_x=28.766\,\text{kNm/m}$
- Moment at interior supports: $M_{x5}=-0.90\times M_x=-36.985\,\text{kNm/m}$

Taking account of suggested allowances for continuity in $y$-direction:

- Bending moment at end support: $M_{y1}=-0.25\times M_y=-7.8675\,\text{kNm/m}$
- Maximum moment in end span: $M_{y2}=0.85\times M_y=26.75\,\text{kNm/m}$
- Moment at penultimate support: $M_{y3}=-0.85\times M_y=-26.75\,\text{kNm/m}$
- Maximum moment in interior span: $M_{y4}=0.70\times M_y=22.029\,\text{kNm/m}$
- Moment at interior supports: $M_{y5}=-0.90\times M_y=-28.323\,\text{kNm/m}$
Effective width in direction of shorter span $l_x$

Effective width of slab $l_{yEff} = a_y + 2l_x^2 = 2.85 \text{ m}$

Effective width in direction of longer span $l_y$

Effective width of slab $l_{xEff} = a_x + 2l_y^2 = 2.775 \text{ m}$

Approximate shearing forces from each load $F$

Shear force at centre of $a_x$ $V_{ax} = F/(3a_y) = 63.492 \text{ kN/m}$

Shear force at centre of $a_y$ $V_{ay} = F/(2a_y + a_x) = 76.19 \text{ kN/m}$
Location: Square fixed-edge slab supporting twin loads

Slab moments due to concentrated loads using Pigeaud’s coefficients

Concrete slab with concentrated loads symmetrical about both axes

Each individual total load = F kN
All dimensions in metres

Poisson's ratio nu=0.2
Shorter span lx=5 m
Longer span ly=5 m

Load details

Load length in lx direction ax=2.5 m
Start of load from nearer support x=0 m
Load length in ly direction ay=1.05 m
Individual concentrated load F=450 kN

Continuity conditions

Two-way slab with all edges continuous.

Calculation of bending moments

Bending moment in x-direction

\[ M_x = F \times L \text{Factor} \times (\alpha_f x + \nu \alpha_y l) \]

\[ = 67.302 \text{ kNm/m} \]

Bending moment in y-direction

\[ M_y = F \times L \text{Factor} \times (\nu \alpha_f y + \alpha_y l) \]

\[ = 86.814 \text{ kNm/m} \]

Taking account of suggested allowances for continuity in x-direction:

Bending moment at end support

\[ M_{x1} = -0.25 \times M_x = -16.826 \text{ kNm/m} \]

Maximum moment in end span

\[ M_{x2} = 0.85 \times M_x = 57.207 \text{ kNm/m} \]

Moment at penultimate support

\[ M_{x3} = -0.85 \times M_x = -57.207 \text{ kNm/m} \]

Maximum moment in interior span

\[ M_{x4} = 0.70 \times M_x = 47.111 \text{ kNm/m} \]

Moment at interior supports

\[ M_{x5} = -0.90 \times M_x = -60.572 \text{ kNm/m} \]

Taking account of suggested allowances for continuity in y-direction:

Bending moment at end support

\[ M_{y1} = -0.25 \times M_y = -21.704 \text{ kNm/m} \]

Maximum moment in end span

\[ M_{y2} = 0.85 \times M_y = 73.792 \text{ kNm/m} \]

Moment at penultimate support

\[ M_{y3} = -0.85 \times M_y = -73.792 \text{ kNm/m} \]

Maximum moment in interior span

\[ M_{y4} = 0.70 \times M_y = 60.77 \text{ kNm/m} \]

Moment at interior supports

\[ M_{y5} = -0.90 \times M_y = -78.133 \text{ kNm/m} \]
Effective width in direction of shorter span $l_x$

Effective width of slab $l_y^{\text{eff}} = a_y + 2l_x^2 = 3.3 \text{ m}$

Effective width in direction of longer span $l_y$

Effective width is equal to total slab width $l_x$.

Effective width of slab $5 \text{ m}$

Approximate shearing forces from each load $F$

Shear force at centre of $a_x$ $V_{ax} = F/(2a_x + a_y) = 74.38 \text{ kN/m}$

Shear force at centre of $a_y$ $V_{ay} = F/(3a_x) = 60 \text{ kN/m}$
Location: Slab w. two adjacent edges continuous & unsymmetrical load

Slab moments due to concentrated loads using Pigeaud's coefficients

Concentrated load is unsymmetrical about both axes

---

Poisson's ratio \( \nu = 0.2 \)
Shorter span \( l_x = 4 \text{ m} \)
Longer span \( l_y = 7 \text{ m} \)

**Load details**

- Load length in \( l_x \) direction \( ax = 2 \text{ m} \)
- Start of load from nearer support \( x = 0 \text{ m} \)
- Load length in \( l_y \) direction \( ay = 3.5 \text{ m} \)
- Start of load from nearer support \( y = 0 \text{ m} \)
- Individual concentrated load \( F = 500 \text{ kN} \)

**Continuity conditions**

Two-way slab with two adjacent edges continuous.

**WARNING:**
Specified combination of slab dimensions and fixity conditions falls outside validity of Pigeaud's tables.
Location: To give table failure

Slab moments due to concentrated loads using Pigeaud's coefficients

Concentrated load is unsymmetrical about both axes

Concentrated load is unsymmetrical about both axes

Poisson's ratio \( \nu = 0.2 \)
Shorter span \( l_x = 1 \text{ m} \)
Longer span \( l_y = 20 \text{ m} \)

Load details

Load length in \( l_x \) direction \( a_x = 0.5 \text{ m} \)
Start of load from nearer support \( x = 0 \text{ m} \)
Load length in \( l_y \) direction \( a_y = 10 \text{ m} \)
Start of load from nearer support \( y = 0 \text{ m} \)
Individual concentrated load \( F = 0 \text{ kN} \)

Continuity conditions

Two-way slab with two adjacent edges continuous.

WARNING:

Specified combination of slab dimensions and fixity conditions falls outside validity of Pigeaud's tables.
Location: Demonstration of input of all loading types

Design moments and shears in box culverts

The formulae employed and their basis are as given in Table 186 of the 10th Edition of the Reinforced Concrete Designer’s Handbook.

![Diagram of box culvert]

Bending moment and shearing force diagrams are symmetrical about vertical centre-line of section (i.e., MB = MA, MD = MC, etc.).

Values of moment and shear are given at quarter-points along walls, floor and roof of culvert, as shown. h and l are measured between centres of walls and slabs.

Moments and shear in culvert floor are only approximate.

Overall height of culvert \( h_0 = 2.5 \) m
Overall width of culvert \( l_0 = 3.5 \) m
Floor and roof thickness \( h_s = 500 \) mm
Wall thickness \( h_w = 400 \) mm
Degree of compressibility of soil \( \text{comp} = 0.5 \)

Central concentrated (or line) load on culvert roof

Concentrated load \( F(1) = 500 \) kN/m

With non-compressible soil:
- Moment at corner A \( MA_n = -F(1)(l/4)K_2 = -118.86 \) kNm/m
- Moment at corner C \( MC_n = -MA_n/2 = 59.431 \) kNm/m

With highly-compressible soil:
- Moment at corner A \( MA_c = -F(1)K_4/(24K_1K_3) = -94.179 \) kNm/m
- Moment at corner C \( MC_c = MA_c K_6/K_4 = -48.699 \) kNm/m

For soil condition considered:
- Moment at corner A \( MA = MA_c \text{comp} + MA_n (1-\text{comp}) = -106.52 \) kNm/m
- Moment at corner C \( MC = MC_c \text{comp} + MC_n (1-\text{comp}) = 5.366 \) kNm/m

Bending moments at intermediate points:

In roof:
- at midspan \( Mr_m = F(1)/4 + MA = 280.98 \) kNm/m
- at 1/4-point \( Mr_q = (MA + Mr_m)/2 = 87.229 \) kNm/m

In wall:
- at 0.75h above base \( Mw_u = (3*MA + MC)/4 = -78.549 \) kNm/m
- at 0.50h \( Mw_m = (MA + MC)/2 = -50.577 \) kNm/m
- at 0.25h \( Mw_l = (MA + 3*MC)/4 = -22.606 \) kNm/m

In floor:
- at midspan \( Mf_f = (F(1)/8 + MC_c) \text{comp} = 72.525 \) kNm/m
- at 1/4-point \( Mf_q = (3*F(1)/32 + MC_c) \text{comp} = 48.307 \) kNm/m
### Moments and shears due to action of current load

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<td></td>
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<td></td>
<td></td>
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<tr>
<td>(in kNm per metre run)</td>
<td>(in kN per metre run)</td>
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</tbody>
</table>

Moments and shears are symmetrical about vertical centreline through section.

### Uniform load on culvert roof

Uniform load

$q(2)=20 \text{ kN/m}^2$

With non-compressible soil:
- Moment at corner A: $M_{An}=-qL^2/(6K^2)=-9.8259 \text{ kNm/m}$
- Moment at corner C: $M_{Cn}=M_{An}/2=4.913 \text{ kNm/m}$

With highly-compressible soil:
- Moment at corner A: $M_{Ac}=-qL^2/(12K^1)=-7.0868 \text{ kNm/m}$
- Moment at corner C: $M_{Cc}=M_{Ac}=-7.0868 \text{ kNm/m}$

For soil condition considered:
- Moment at corner A: $M_{A}=M_{Ac}\text{comp}+M_{An}(1-\text{comp})=-8.4564 \text{ kNm/m}$
- Moment at corner C: $M_{C}=M_{Cc}\text{comp}+M_{Cn}(1-\text{comp})=-1.0869 \text{ kNm/m}$

Bending moments at intermediate points:

**In roof:** at midspan $M_{rm}=qL^2/8+M_{A}=15.569 \text{ kNm/m}$
- at 1/4-point $M_{rq}=3qL^2/32+M_{A}=9.5624 \text{ kNm/m}$

**In wall:** at 0.75h above base $M_{wu}=(3M_{A}+M_{C})/4=6.614 \text{ kNm/m}$
- at 0.50h $M_{wm}=(M_{A}+M_{C})/2=-4.7716 \text{ kNm/m}$
- at 0.25h $M_{wl}=(M_{A}+3M_{C})/4=-2.9293 \text{ kNm/m}$

**In floor:** at midspan $M_{fm}=(qL^2/8+M_{Cc})\text{comp}=8.4691 \text{ kNm/m}$
- at 1/4-point $M_{fq}=(3qL^2/32+M_{Cc})\text{comp}=5.466 \text{ kNm/m}$
Moments and shears due to action of current load

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BENDING MOMENTS
(in kNm per metre run)
SHEARING FORCES
(in kN per metre run)
Moments and shears are symmetrical about vertical centreline through section.

Self-weight of culvert walls

Unit weight of concrete \( D_c(3) = 24 \text{ kN/m}^3 \)

Moments and shears due to action of current load

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</table>

Lateral earth pressure on culvert walls

Earth pressure at bottom of wall \( q_{ep}(4) = 20 \text{ kN/m}^2 \)
Let \( q_{ep} = q_{ep}(cycle) = 20 \)

With non-compressible soil:
- Moment at corner A: \( M_{An} = -q_{ep}h^2k/(30K2) = -1.0307 \text{ kNm/m} \)
- Moment at corner C: \( M_{Cn} = M_{An}K8/(2K) = -4.818 \text{ kNm/m} \)

With highly-compressible soil:
- Moment at corner A: \( M_{Ac} = -q_{ep}h^2kK7/(60K1K3) = -1.6613 \text{ kNm/m} \)
- Moment at corner C: \( M_{Cc} = M_{Ac}K8/K7 = -2.0557 \text{ kNm/m} \)

For soil condition considered:
- Moment at corner A: \( M_{A} = M_{Ac} \text{ comp} + M_{An} \text{ (1-comp)} = -1.346 \text{ kNm/m} \)
Moment at corner C  
\[ MC = MC_c \cdot \text{comp} + MC_n \cdot (1 - \text{comp}) = -3.4368 \text{ kNm/m} \]

Bending moments at intermediate points:

- **In roof:**
  - At midspan: \[ Mr_m = MA = -1.346 \text{ kNm/m} \]
  - At 1/4-point: \[ Mr_q = MA = -1.346 \text{ kNm/m} \]

- **In wall:**
  - At 0.75h above base: \[ M_w = q_e \cdot h^2 \cdot 5/128 + (3 \cdot MA + MC)/4 = 1.2563 \text{ kNm/m} \]
  - At 0.50h: \[ M_w = q_e \cdot h^2 / 16 + (MA + MC)/2 = 2.6086 \text{ kNm/m} \]
  - At 0.25h: \[ M_w = q_e \cdot h^2 \cdot 7/128 + (3 \cdot MA + MC)/4 = 1.4609 \text{ kNm/m} \]

- **In floor:**
  - At midspan: \[ M_f = MC_c \cdot \text{comp} = -1.0278 \text{ kNm/m} \]
  - At 1/4-point: \[ M_f = MC_c \cdot \text{comp} = -1.0278 \text{ kNm/m} \]

Moments and shears due to action of current load:

- BENDING MOMENTS:
  - Table:
    - Line 1: -1.346, -1.346, -1.346, 0, 0, 0, -5.6213
    - Line 2: 
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      - 2.6086
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      - 1.4609
      - 
    - Line 3: -1.0278, -1.0278, -3.4368, 0, 0, 0, 14.379

- SHEARING FORCES:
  - Table:
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    - Line 2: 
      - -5.6213
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      - 5.6287
    - Line 3: -1.0278, -1.0278, -3.4368, 0, 0, 0, 14.379

Moments and shears are symmetrical about vertical centreline through section.

Uniform earth pressure on walls due to surcharge, etc

- Uniform earth pressure: \( q_{el} = 20 \text{ kN/m}^2 \)
- Let: \( q_{el} = q_{el}(\text{cycle}) = 20 \)

- With non-compressible soil:
  - Moment at corner A: \( MA_n = -q_{el} \cdot h^2 \cdot k / (12 \cdot K2) = -2.5768 \text{ kNm/m} \)
  - Moment at corner C: \( MC_n = MA_n \cdot K3 / k = -8.7116 \text{ kNm/m} \)

- With highly-compressible soil:
  - Moment at corner A: \( MA_c = -q_{el} \cdot h^2 \cdot k / (12 \cdot K1) = -3.7169 \text{ kNm/m} \)
  - Moment at corner C: \( MC_c = MA_c = -3.7169 \text{ kNm/m} \)

- For soil condition considered:
  - Moment at corner A: \( MA = MA_c \cdot \text{comp} + MA_n \cdot (1 - \text{comp}) = -3.1469 \text{ kNm/m} \)
  - Moment at corner C: \( MC = MC_c \cdot \text{comp} + MC_n \cdot (1 - \text{comp}) = -6.2143 \text{ kNm/m} \)

Bending moments at intermediate points:

- **In roof:**
  - At midspan: \[ Mr_m = MA = -3.1469 \text{ kNm/m} \]
  - At 1/4-point: \[ Mr_q = MA = -3.1469 \text{ kNm/m} \]

- **In wall:**
  - At 0.75h above base: \[ M_w = q_e \cdot h^2 \cdot 2/7 + (MA + 3 \cdot MC)/4 = 3.5863 \text{ kNm/m} \]
  - At 0.50h: \[ M_w = q_e \cdot h^2 / 8 + (MA + MC)/2 = 5.3194 \text{ kNm/m} \]
  - At 0.25h: \[ M_w = q_e \cdot h^2 / 32 + (3 \cdot MA + 3 \cdot MC)/4 = 2.0526 \text{ kNm/m} \]
In floor: at midspan \[ M_{fm} = MC_{c} \cdot \text{comp} = -1.8585 \text{ kNm/m} \]

at 1/4-point \[ M_{fq} = MC_{c} \cdot \text{comp} = -1.8585 \text{ kNm/m} \]

### Moments and shears due to action of current load

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**BENDING MOMENTS**

(in kNm per metre run)

**SHEARING FORCES**

(in kN per metre run)

Moments and shears are symmetrical about vertical centreline through section.

### Internal hydrostatic pressure on walls and floor

Internal pressure at base of wall \( q_{ip}(6) = 20 \text{ kN/m}^2 \)

Let \( q_{ip} = q_{ip}(\text{cycle}) = 20 \text{ kN/m}^2 \)

**With non-compressible soil:**

- Moment at corner A: \( M_{An} = q_{ip} \cdot h^2 \cdot k / (30 \cdot K2) = 1.0307 \text{ kNm/m} \)
- Moment at corner C: \( M_{Cn} = M_{An} \cdot K8 / (2 \cdot k) = 4.818 \text{ kNm/m} \)

**With highly-compressible soil:**

- Moment at corner A: \( M_{Ac} = q_{ip} \cdot h^2 \cdot k \cdot k7 / (60 \cdot K1 \cdot K3) = 1.6613 \text{ kNm/m} \)
- Moment at corner C: \( M_{Cc} = M_{Ac} \cdot K8 / K7 = 2.0557 \text{ kNm/m} \)

**For soil condition considered:**

- Moment at corner A: \( M_{A} = M_{Ac} \cdot \text{comp} + M_{An} \cdot (1 - \text{comp}) = 1.346 \text{ kNm/m} \)
- Moment at corner C: \( M_{C} = M_{Cc} \cdot \text{comp} + M_{Cn} \cdot (1 - \text{comp}) = 3.4368 \text{ kNm/m} \)

Bending moments at intermediate points:

**In roof:**
- at midspan: \( M_{rm} = M_{A} = 1.346 \text{ kNm/m} \)
- at 1/4-point: \( M_{rq} = M_{A} = 1.346 \text{ kNm/m} \)

**In wall:**
- at 0.75h above base: \( M_{wu} = -q_{ip} \cdot h^2 \cdot 5 / 128 + (3 \cdot M_{A} + M_{C}) / 4 = -1.2563 \text{ kNm/m} \)
- at 0.50h: \( M_{wm} = -q_{ip} \cdot h^2 / 16 + (M_{A} + M_{C}) / 2 = -2.6086 \text{ kNm/m} \)
- at 0.25h: \( M_{wl} = -q_{ip} \cdot h^2 \cdot 7 / 128 + (M_{A} + 3 \cdot M_{C}) / 4 = -1.4609 \text{ kNm/m} \)

**In floor:**
- at midspan: \( M_{fm} = M_{Cc} \cdot \text{comp} = 1.0278 \text{ kNm/m} \)
- at 1/4-point: \( M_{fq} = M_{Cc} \cdot \text{comp} = 1.0278 \text{ kNm/m} \)
Moments and shears due to action of current load

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BENDING MOMENTS
SHEARING FORCES
(in kNm per metre run) (in kN per metre run)

Moments and shears are symmetrical about vertical centreline through section.

Moments and shears due to action of all loads considered

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BENDING MOMENTS
SHEARING FORCES
(in kNm per metre run) (in kN per metre run)

Moments and shears are symmetrical about vertical centreline through section.
**Location:** Deep narrow culvert with earth pressure on walls

**Design moments and shears in box culverts**

The formulae employed and their basis are as given in Table 186 of the 10th Edition of the Reinforced Concrete Designer's Handbook.

![Diagram](image)

Bending moment and shearing force diagrams are symmetrical about vertical centre-line of section (i.e. MB=MA, MD=MC, etc).

Values of moment and shear are given at quarter-points along walls, floor and roof of culvert, as shown.

h and l are measured between centres of walls and slabs. Moments and shear in culvert floor are only approximate.

Overall height of culvert $h_0=3.25$ m

Overall width of culvert $l_0=2$ m

Floor and roof thickness $h_s=500$ mm

Wall thickness $h_w=825$ mm

Degree of compressibility of soil $comp=0.25$

**Lateral earth pressure on culvert walls**

Earth pressure at bottom of wall $q_{ep(1)}=22$ kN/m²

Let $q_{ep}=q_{ep(cycle)}=22$

With non-compressible soil:

- Moment at corner A $MA_n=-q_{ep}h^2k/(30K2)=-1.1461$ kNm/m
- Moment at corner C $MC_n=MA_nK8/(2k)=-10.519$ kNm/m

With highly-compressible soil:

- Moment at corner A $MA_c=-q_{ep}h^2kK7/(60K1K3)$
  $=-2.1694$ kNm/m
- Moment at corner C $MC_c=MA_cK8/K7=-2.5797$ kNm/m

For soil condition considered:

- Moment at corner A $MA=MA_ccomp+MA_n(1-comp)=-1.402$ kNm/m
- Moment at corner C $MC=MC_ccomp+MC_n(1-comp)=-8.5339$ kNm/m

Bending moments at intermediate points:

In roof: at midspan $Mr_m=MA=-1.402$ kNm/m

- at 1/4-point $Mr_q=MA=-1.402$ kNm/m

In wall: at 0.75h above base $Mw_u=q_{ep}h^2/128+(3*MA+MC)/4$

- at 0.50h $Mw_m=q_{ep}h^2/16+(MA+MC)/2=5.4305$ kNm/m
- at 0.25h $Mw_l=q_{ep}h^2/7/128+(MA+3MC)/4$
  $=2.3477$ kNm/m

In floor: at midspan $Mf_m=MC_ccomp=-0.64494$ kNm/m

- at 1/4-point $Mf_q=MC_ccomp=-0.64494$ kNm/m
## Moments and shears due to action of current load

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**BENDING MOMENTS**

(in kNm per metre run)

**SHEARING FORCES**

(in kN per metre run)

Moments and shears are symmetrical about vertical centreline through section.
Location: Central concentrated load on wide shallow culvert

Design moments and shears in box culverts

The formulae employed and their basis are as given in Table 186 of the 10th Edition of the Reinforced Concrete Designer's Handbook.

Bending moment and shearing force diagrams are symmetrical about vertical centre-line of section (i.e. MB=MA, MD=MC, etc).

Values of moment and shear are given at quarter-points along walls, floor and roof of culvert, as shown.

h and l are measured between centres of walls and slabs. Moments and shear in culvert floor are only approximate.

Overall height of culvert $h_0=1.75$ m
Overall width of culvert $l_0=8$ m
Floor and roof thickness $h_s=750$ mm
Wall thickness $h_w=350$ mm
Degree of compressibility of soil $c_{mp}=0.8$

Central concentrated (or line) load on culvert roof

Concentrated load $F(1)=750$ kN/m

With non-compressible soil:
- Moment at corner A $M_{A_{n}}=-F*l/(4*K^2)=-436.48$ kNm/m
- Moment at corner C $M_{C_{n}}=-M_{A_{n}}/2=218.24$ kNm/m

With highly-compressible soil:
- Moment at corner A $M_{A_{c}}=-F*l*K^4/(24*K_{1}*K_{3})=-345.08$ kNm/m
- Moment at corner C $M_{C_{c}}=M_{A_{c}}*K_{6}/K_{4}=-177.75$ kNm/m

For soil condition considered:
- Moment at corner A $M_{A}=M_{A_{c}}*c_{mp}+M_{A_{n}}*(1-c_{mp})=-363.36$ kNm/m
- Moment at corner C $M_{C}=M_{C_{c}}*c_{mp}+M_{C_{n}}*(1-c_{mp})=-98.555$ kNm/m

Bending moments at intermediate points:

In roof: at midspan $M_{r_{m}}=F*l/4+M_{A}=1071$ kNm/m
- at 1/4-point $M_{r_{q}}=(M_{A}+M_{r_{m}})/2=353.83$ kNm/m

In wall: at 0.75h above base $M_{w_{u}}=(3*MA+MC)/4=-297.16$ kNm/m
- at 0.50h $M_{w_{m}}=(MA+MC)/2=-230.96$ kNm/m
- at 0.25h $M_{w_{l}}=(MA+3*MC)/4=-164.76$ kNm/m

In floor: at midspan $M_{f_{m}}=(F*l/8+MC_{c})*c_{mp}=431.55$ kNm/m
- at 1/4-point $M_{f_{q}}=(3*F*l/32+MC_{c})*c_{mp}=288.11$ kNm/m
### Moments and shears due to action of current load

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BENDING MOMENTS  SHEARING FORCES
(in kNm per metre run)  (in kN per metre run)

Moments and shears are symmetrical about vertical centreline through section.
**Location: Deflection at the centre of a circular plate**

### Circular plate with simple support carrying a UDL

- Radius of circular plate: $a = 1 \text{ m}$
- Thickness of circular plate: $t = 0.01 \text{ m}$
- Distance from centre to point: $r = 0 \text{ m}$
- Uniformly distributed load: $w = 5 \text{ kN/m}$
- Modulus of elasticity: $E = 205 \times 10^6 \text{ kN/m}^2$
- Poisson's ratio: $v = 0.3$
- Reciprocal of Poisson's ratio: $m = 1/v = 3.3333$
- Total load on plate: $W = w \pi a^2 = 15.708 \text{ kN}$
- Maximum moment at centre: $M = (3+v)w a^2/16 = 1.0313 \text{ kNm}$
- Radial stress: $sr = -3W((3m+1)(1-r^2/a^2))/(8\pi mt^2) = -61875 \text{ kN/m}^2$
- Tangential stress: $st = -3W((3m+1)-(m+3)r^2/a^2)/(8\pi mt^2) = -61875 \text{ kN/m}^2$
- Deflection constant: $dc = (5m+1)a^2/(2(m+1))+r^4/2a^2-(3m+1)r^2/(m+1) = 2.0385$
- Deflection: $y = -3W(m^2-1)*dc/(8\pi E m^2 t^3) = -0.016966 \text{ m}$
Location: Rectangular plate with simple supports carrying a UDL

Rectangular plate supporting a UDL - this calculation computes deflections and bending moments for a rectangular plate in accordance with "Theory of Plates & Shells" by Timoshenko & Woinowsky-Krieger.

Width of plate \( a = 1 \text{ m} \)
Length of plate \( b = 1.5 \text{ m} \)
Thickness of plate \( t = 0.01 \text{ m} \)
Uniformly distributed load \( q = 5 \text{ kN/m}^2 \)
Modulus of elasticity \( E = 205E6 \text{ kN/m}^2 \)
Max deflection \( w_{max} = \alpha q a^4/D = 0.0020562 \text{ m} \)
Bending moment/unit length \( M_{max} = \beta q a^2 = 0.406 \text{ kNm/m} \)
Shearing force/unit length \( Q_{max} = \gamma q a = 2.12 \text{ kN/m} \)
Support reaction for edge \( x \) \( V_{xmax} = \delta q a = 2.43 \text{ kN/m} \)
Support reaction for edge \( y \) \( V_{ymax} = \delta q a = 2.4 \text{ kN/m} \)
Corner tiedown reactions \( R = \eta q a^2 = 0.425 \text{ kN} \)

From Timoshenko's table 6, \( M_x \) at position \( y=0 \text{ m i.e along x axis.} \)

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From Timoshenko's table 6, \( M_y \) at position \( y=0 \text{ m i.e along x axis.} \)

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\( \text{SCALE 5.48} \) \( \text{Office 1007} \) \( \text{Proforma 615} \)
and \( x = 0.2a = 0.2 \, \text{m} \)

\[
\beta_1' = \text{TABLE 6 for } b'a = 1.5, \text{ column } 7 \\
= 0.0312
\]

\[
M_y = \beta_1' \times q \times a^2 = 0.156 \, \text{kNm/m}
\]

and \( x = 0.3a = 0.3 \, \text{m} \)

\[
\beta_1' = \text{TABLE 6 for } b'a = 1.5, \text{ column } 8 \\
= 0.0415
\]

\[
M_y = \beta_1' \times q \times a^2 = 0.2075 \, \text{kNm/m}
\]

and \( x = 0.4a = 0.4 \, \text{m} \)

\[
\beta_1' = \text{TABLE 6 for } b'a = 1.5, \text{ column } 9 \\
= 0.0478
\]

\[
M_y = \beta_1' \times q \times a^2 = 0.239 \, \text{kNm/m}
\]

and \( x = 0.5a = 0.5 \, \text{m} \)

\[
\beta_1' = \text{TABLE 6 for } b'a = 1.5, \text{ column } 10 \\
= 0.0498
\]

\[
M_y = \beta_1' \times q \times a^2 = 0.249 \, \text{kNm/m}
\]

From Timoshenko's table 7, \( M_x \) at position \( x = a/2 = 0.5 \, \text{m} \) i.e. centre

and \( y = 0.4b = 0.6 \, \text{m} \)

\[
\beta_1'' = \text{TABLE 7 for } b'a = 1.5, \text{ column } 1 \\
= 0.0302
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\[
M_x = \beta_1'' \times q \times a^2 = 0.151 \, \text{kNm/m}
\]

and \( y = 0.3b = 0.45 \, \text{m} \)

\[
\beta_1'' = \text{TABLE 7 for } b'a = 1.5, \text{ column } 2 \\
= 0.0532
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\[
M_x = \beta_1'' \times q \times a^2 = 0.266 \, \text{kNm/m}
\]

and \( y = 0.2b = 0.3 \, \text{m} \)

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\beta_1'' = \text{TABLE 7 for } b'a = 1.5, \text{ column } 3 \\
= 0.069
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\[
M_x = \beta_1'' \times q \times a^2 = 0.345 \, \text{kNm/m}
\]

and \( y = 0.1b = 0.15 \, \text{m} \)

\[
\beta_1'' = \text{TABLE 7 for } b'a = 1.5, \text{ column } 4 \\
= 0.0781
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\[
M_x = \beta_1'' \times q \times a^2 = 0.3905 \, \text{kNm/m}
\]

and \( y = 0 \, \text{m} \)

\[
\beta_1'' = \text{TABLE 7 for } b'a = 1.5, \text{ column } 5 \\
= 0.0812
\]

\[
M_x = \beta_1'' \times q \times a^2 = 0.406 \, \text{kNm/m}
\]

From Timoshenko's table 7, \( M_y \) at position \( x = a/2 = 0.5 \, \text{m} \) i.e. centre.

and \( y = 0.4b = 0.6 \, \text{m} \)

\[
\beta_1''' = \text{TABLE 7 for } b'a = 1.5, \text{ column } 6 \\
= 0.0275
\]

\[
M_x = \beta_1''' \times q \times a^2 = 0.1375 \, \text{kNm/m}
\]

and \( y = 0.3b = 0.45 \, \text{m} \)

\[
\beta_1''' = \text{TABLE 7 for } b'a = 1.5, \text{ column } 7 \\
= 0.041
\]

\[
M_x = \beta_1''' \times q \times a^2 = 0.205 \, \text{kNm/m}
\]

and \( y = 0.2b = 0.3 \, \text{m} \)

\[
\beta_1''' = \text{TABLE 7 for } b'a = 1.5, \text{ column } 8 \\
= 0.047
\]

\[
M_x = \beta_1''' \times q \times a^2 = 0.235 \, \text{kNm/m}
\]

and \( y = 0.1b = 0.15 \, \text{m} \)

\[
\beta_1''' = \text{TABLE 7 for } b'a = 1.5, \text{ column } 9 \\
= 0.0493
\]

\[
M_x = \beta_1''' \times q \times a^2 = 0.2465 \, \text{kNm/m}
\]

and \( y = 0 \, \text{m} \)

\[
\beta_1''' = \text{TABLE 7 for } b'a = 1.5, \text{ column } 10 \\
= 0.0498
\]

\[
M_x = \beta_1''' \times q \times a^2 = 0.249 \, \text{kNm/m}
\]
Location: Rectangular plate with three S.S. edges and one free edge

Rectangular plate supporting a UDL - this calculation computes deflections and bending moments for a rectangular plate in accordance with "Theory of Plates & Shells" by Timoshenko & Woinowsky-Krieger.

Width of plate \( a = 1 \text{ m} \)
Length of plate \( b = 1.5 \text{ m} \)
Thickness of plate \( t = 0.01 \text{ m} \)
Uniformly distributed load \( q = 5 \text{ kN/m}^2 \)
Modulus of elasticity \( E = 205E6 \text{ kN/m}^2 \)
At \( x = a/2 = 0.5 \text{ m} \) and \( y = b = 1.5 \text{ m} \)
Max deflection \( w_{max} = \alpha q a^4/D = 0.0038939 \text{ m} \)
Bending moment/unit length \( M_{xmax} = \beta q a^2 = 0.64 \text{ kNm/m} \)
At \( x = a/2 = 0.5 \text{ m} \) and \( y = b/2 = 0.75 \text{ m} \)
Bending moment/unit length \( M_x = \beta q a^2 = 0.505 \text{ kNm/m} \)
Location: Rectangular plate with built-in edges carrying a UDL

Rectangular plate supporting a UDL - this calculation computes deflections and bending moments for a rectangular plate in accordance with "Theory of Plates & Shells" by Timoshenko & Woinowsky-Krieger.

Width of plate: $a = 1$ m  
Length of plate: $b = 1.5$ m  
Thickness of plate: $t = 0.01$ m  
Uniformly distributed load: $q = 5$ kN/m$^2$  
Modulus of elasticity: $E = 205 \times 10^6$ kN/m$^2$

At $x=0$, $y=0$ i.e. centre  
Max deflection: $w_{max} = \alpha q a^4/D = 0.58595 \times 10^{-3}$ m  
Bending moment/unit length: $M_x = \beta q a^2 = 0.3785$ kNm/m  
Bending moment/unit length: $M_y = \beta_1 q a^2 = 0.285$ kNm/m

At $x=a/2=0.5$ m and $y=0$ m  
Bending moment/unit length: $M_x = \beta q a^2 = 0.3785$ kNm/m  
Bending moment/unit length: $M_y = \beta_1 q a^2 = 0.285$ kNm/m

At $x=0$ and $y=b/2=0.75$ m  
Bending moment/unit length: $M_x = \beta q a^2 = 0.184$ kNm/m  
Bending moment/unit length: $M_y = \beta_1 q a^2 = 0.1015$ kNm/m
Location: Subframe on grid A1-A4

Clockwise fixed end moments are positive.

From inertias and lengths of beams and columns and FEM's from beams 1, 2 & 3; computes end moments for beam 2.

Take 75% of inertia for pin ended members.

Take zero inertia and unit length for missing members.

Inertia of beam 1: $I(1) = 0.0054 \text{ m}^4$
Length of beam 1: $l(1) = 6 \text{ m}$
Inertia of beam 2: $I(2) = 0.0054 \text{ m}^4$
Length of beam 2: $l(2) = 6 \text{ m}$
Inertia of beam 3: $I(3) = 0.0054 \text{ m}^4$
Length of beam 3: $l(3) = 6 \text{ m}$
Inertia of column 4: $I(4) = 0.675 \times 10^{-3} \text{ m}^4$
Length of column 4: $l(4) = 2.7 \text{ m}$
Inertia of column 5: $I(5) = 0.675 \times 10^{-3} \text{ m}^4$
Length of column 5: $l(5) = 2.7 \text{ m}$
Inertia of column 6: $I(6) = 0.675 \times 10^{-3} \text{ m}^4$
Length of column 6: $l(6) = 2.7 \text{ m}$
Inertia of column 7: $I(7) = 0.675 \times 10^{-3} \text{ m}^4$
Length of column 7: $l(7) = 2.7 \text{ m}$

Total No. of concentrated loads: $N_c = 1$
Total number of UDL's: $N_u = 2$

Span No. with concentrated load: $S_c(1) = 1$
Distance from left hand end: $L_c(1) = 3 \text{ m}$
Dead load: $DLC(1) = 50 \text{ kN}$
Live load: $LLC(1) = 60 \text{ kN}$
Span number containing UDL: $S_u(1) = 2$
Distance from left end to start: $Lau(1) = 0 \text{ m}$
Distance from left end to end: $Lbu(1) = 6 \text{ m}$
Dead load: $DLu(1) = 10 \text{ kN/m}$
Live load: $LLu(1) = 20 \text{ kN/m}$
Span number containing UDL: $S_u(2) = 3$
Distance from left end to start: $Lau(2) = 3 \text{ m}$
Distance from left end to end: $Lbu(2) = 6 \text{ m}$
Dead load: $DLu(2) = 15 \text{ kN/m}$
Live load: $LLu(2) = 25 \text{ kN/m}$
All spans loaded with factored DL + LL

Factored FEM at end of span 1  \( Mfa' = 1.4 \times dle(1) + 1.6 \times lle(1) = 124.5 \text{ kNm} \)
Factored FEM at start of span 2  \( Mfa = 1.4 \times dl(2) + 1.6 \times ll(2) = -138 \text{ kNm} \)
Factored FEM at end of span 2  \( Mfb = 1.4 \times dle(2) + 1.6 \times lle(2) = 138 \text{ kNm} \)
Factored FEM at start of span 3  \( Mfb' = 1.4 \times dl(3) + 1.6 \times ll(3) = -57.188 \text{ kNm} \)
Sum of FEM's at A  \( \text{sig}Mfa = Mfa' + Mfa = -13.5 \text{ kNm} \)
Sum of FEM's at B  \( \text{sig}Mfb = Mfb + Mfb' = 80.813 \text{ kNm} \)
Rotation at A (for unit E)  \( \theta_a = k1 \times \text{sig}Mfa + k2 \times \text{sig}Mfb = -13251 \)
Rotation at B (for unit E)  \( \theta_b = k3 \times \text{sig}Mfa + k4 \times \text{sig}Mfb = 37729 \)
Beam (2) moment at A  \( M_a = Mfa - (I(2)/l(2)) \times (\theta_a + 0.5 \times \theta_b) = -143.05 \text{ kNm} \)
Beam (2) moment at B  \( M_b = Mfb - (I(2)/l(2)) \times (\theta_b + 0.5 \times \theta_a) = 104.01 \text{ kNm} \)

BM at 20th points along span 2 (Sagging is positive)

<table>
<thead>
<tr>
<th>Distance from start</th>
<th>Bending moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 m</td>
<td>-143.05 kNm</td>
</tr>
<tr>
<td>0.3 m</td>
<td>-102.07 kNm</td>
</tr>
<tr>
<td>0.6 m</td>
<td>-65.227 kNm</td>
</tr>
<tr>
<td>0.9 m</td>
<td>-32.525 kNm</td>
</tr>
<tr>
<td>1.2 m</td>
<td>-9.629 kNm</td>
</tr>
<tr>
<td>1.5 m</td>
<td>20.459 kNm</td>
</tr>
<tr>
<td>1.8 m</td>
<td>40.742 kNm</td>
</tr>
<tr>
<td>2.1 m</td>
<td>56.884 kNm</td>
</tr>
<tr>
<td>2.4 m</td>
<td>68.886 kNm</td>
</tr>
<tr>
<td>2.7 m</td>
<td>76.748 kNm</td>
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<tr>
<td>3 m</td>
<td>80.47 kNm</td>
</tr>
<tr>
<td>3.3 m</td>
<td>80.053 kNm</td>
</tr>
<tr>
<td>3.6 m</td>
<td>75.495 kNm</td>
</tr>
<tr>
<td>3.9 m</td>
<td>66.797 kNm</td>
</tr>
<tr>
<td>4.2 m</td>
<td>53.959 kNm</td>
</tr>
<tr>
<td>4.5 m</td>
<td>36.982 kNm</td>
</tr>
<tr>
<td>4.8 m</td>
<td>15.864 kNm</td>
</tr>
<tr>
<td>5.1 m</td>
<td>-9.394 kNm</td>
</tr>
<tr>
<td>5.4 m</td>
<td>-38.792 kNm</td>
</tr>
<tr>
<td>5.7 m</td>
<td>-72.33 kNm</td>
</tr>
<tr>
<td>6 m</td>
<td>-110.01 kNm</td>
</tr>
</tbody>
</table>

Centre span loaded with factored DL + LL, others with DL only

Factored FEM at end of span 1  \( Mfa' = 1.0 \times dle(1) + 0.0 \times lle(1) = 37.5 \text{ kNm} \)
Factored FEM at start of span 2  \( Mfa = 1.4 \times dl(2) + 1.6 \times ll(2) = -138 \text{ kNm} \)
Factored FEM at end of span 2  \( Mfb = 1.4 \times dle(2) + 1.6 \times lle(2) = 138 \text{ kNm} \)
Factored FEM at start of span 3  \( Mfb' = 1.0 \times dl(3) + 0.0 \times ll(3) = -104.063 \text{ kNm} \)
Sum of FEM's at A  \( \text{sig}Mfa = Mfa' + Mfa = -100.5 \text{ kNm} \)
Sum of FEM's at B  \( \text{sig}Mfb = Mfb + Mfb' = 123.94 \text{ kNm} \)
Rotation at A (for unit E)  \( \theta_a = k1 \times \text{sig}Mfa + k2 \times \text{sig}Mfb = -56397 \)
Rotation at B (for unit E)  \( \theta_b = k3 \times \text{sig}Mfa + k4 \times \text{sig}Mfb = 64920 \)
Beam (2) moment at A  \( M_a = Mfa - (I(2)/l(2)) \times (\theta_a + 0.5 \times \theta_b) = -116.46 \text{ kNm} \)
Beam (2) moment at B  \( M_b = Mfb - (I(2)/l(2)) \times (\theta_b + 0.5 \times \theta_a) = 104.95 \text{ kNm} \)
<table>
<thead>
<tr>
<th>Distance from start</th>
<th>Bending moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 m</td>
<td>-116.46 kNm</td>
</tr>
<tr>
<td>0.3 m</td>
<td>-76.551 kNm</td>
</tr>
<tr>
<td>0.6 m</td>
<td>-40.786 kNm</td>
</tr>
<tr>
<td>0.9 m</td>
<td>-9.1605 kNm</td>
</tr>
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<td>1.2 m</td>
<td>18.325 kNm</td>
</tr>
<tr>
<td>1.5 m</td>
<td>41.67 kNm</td>
</tr>
<tr>
<td>1.8 m</td>
<td>60.875 kNm</td>
</tr>
<tr>
<td>2.1 m</td>
<td>75.941 kNm</td>
</tr>
<tr>
<td>2.4 m</td>
<td>86.866 kNm</td>
</tr>
<tr>
<td>2.7 m</td>
<td>93.651 kNm</td>
</tr>
<tr>
<td>3 m</td>
<td>96.296 kNm</td>
</tr>
<tr>
<td>3.3 m</td>
<td>94.802 kNm</td>
</tr>
<tr>
<td>3.6 m</td>
<td>89.167 kNm</td>
</tr>
<tr>
<td>3.9 m</td>
<td>79.392 kNm</td>
</tr>
<tr>
<td>4.2 m</td>
<td>65.478 kNm</td>
</tr>
<tr>
<td>4.5 m</td>
<td>47.423 kNm</td>
</tr>
<tr>
<td>4.8 m</td>
<td>25.228 kNm</td>
</tr>
<tr>
<td>5.1 m</td>
<td>-1.1066 kNm</td>
</tr>
<tr>
<td>5.4 m</td>
<td>-31.581 kNm</td>
</tr>
<tr>
<td>5.7 m</td>
<td>-66.196 kNm</td>
</tr>
<tr>
<td>6 m</td>
<td>-104.95 kNm</td>
</tr>
</tbody>
</table>
Location: Subframe with zero columns over and no right hand beam

Clockwise fixed end moments are positive.
From inertias and lengths of beams and columns and FEM's from beams 1, 2 & 3; computes end moments for beam 2.

Take 75% of inertia for pin ended members.
Take zero inertia and unit length for missing members.

Inertia of beam 1  I(1)=0.0054 m^4
Length of beam 1  l(1)=6 m
Inertia of beam 2  I(2)=0.0054 m^4
Length of beam 2  l(2)=6 m
Inertia of beam 3  I(3)=0 m^4
Length of beam 3  l(3)=1 m
Inertia of column 4  I(4)=0 m^4
Length of column 4  l(4)=1 m
Inertia of column 5  I(5)=0 m^4
Length of column 5  l(5)=1 m
Inertia of column 6  I(6)=0.675E-3 m^4
Length of column 6  l(6)=2.7 m
Inertia of column 7  I(7)=0.675E-3 m^4
Length of column 7  l(7)=2.7 m

Total No. of concentrated loads  Nc=2
Total number of UDL's  Nu=2

Span No. with concentrated load  Sc(1)=1
Distance from left hand end  Lc(1)=3 m
Dead load  DLC(1)=50 kN
Live load  LLC(1)=60 kN
Span No. with concentrated load  Sc(2)=2
Distance from left hand end  Lc(2)=3 m
Dead load  DLC(2)=50 kN
Live load  LLC(2)=60 kN
Span number containing UDL  Su(1)=1
Distance from left end to start  Lau(1)=0 m
Distance from left end to end  Lbu(1)=6 m
Dead load  DLu(1)=10 kN/m
Live load  LLu(1)=20 kN/m
Span number containing UDL  Su(2)=2
Distance from left end to start  Lau(2)=3 m
Distance from left end to end  Lbu(2)=6 m
Dead load  DLu(2)=15 kN/m
Live load  LLu(2)=25 kN/m
**All spans loaded with factored DL + LL**

- **Factored FEM at end of span 1**  
  \[ M_{fa}' = 1.4 \times d_{le}(1) + 1.6 \times d_{le}(1) = 262.5 \text{ kNm} \]
- **Factored FEM at start of span 2**  
  \[ M_{fa} = 1.4 \times d_{ls}(2) + 1.6 \times d_{ls}(2) = 181.69 \text{ kNm} \]
- **Factored FEM at end of span 2**  
  \[ M_{fb} = 1.4 \times d_{le}(2) + 1.6 \times d_{le}(2) = 250.31 \text{ kNm} \]
- **Factored FEM at start of span 3**  
  \[ M_{fb}' = 1.4 \times d_{ls}(3) + 1.6 \times d_{ls}(3) = 0 \text{ kNm} \]
- **Sum of FEM's at A**  
  \[ \text{sig}_{M_{fa}} = M_{fa}' + M_{fa} = 80.813 \text{ kNm} \]
- **Sum of FEM's at B**  
  \[ \text{sig}_{M_{fb}} = M_{fb} + M_{fb}' = 250.31 \text{ kNm} \]
- **Rotation at A (for unit E)**  
  \[ \theta_{A} = k_{1} \times \text{sig}_{M_{fa}} + k_{2} \times \text{sig}_{M_{fb}} = -9144.4 \text{ kNm} \]
- **Rotation at B (for unit E)**  
  \[ \theta_{B} = k_{3} \times \text{sig}_{M_{fa}} + k_{4} \times \text{sig}_{M_{fb}} = 221241 \text{ kNm} \]
- **Beam (2) moment at A**  
  \[ M_{a} = M_{fa} - \left( \frac{I_{(2)}}{l_{(2)}} \right) \times \left( \theta_{A} + 0.5 \times \theta_{B} \right) = -273.02 \text{ kNm} \]
- **Beam (2) moment at B**  
  \[ M_{b} = M_{fb} - \left( \frac{I_{(2)}}{l_{(2)}} \right) \times \left( \theta_{B} + 0.5 \times \theta_{A} \right) = 55.31 \text{ kNm} \]

**BM's at 20th points along span 2**  
(Sagging is positive)

<table>
<thead>
<tr>
<th>Distance from start</th>
<th>Bending moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 m</td>
<td>-273.02 kNm</td>
</tr>
<tr>
<td>0.3 m</td>
<td>-223.51 kNm</td>
</tr>
<tr>
<td>0.6 m</td>
<td>-174 kNm</td>
</tr>
<tr>
<td>0.9 m</td>
<td>-124.49 kNm</td>
</tr>
<tr>
<td>1.2 m</td>
<td>-74.975 kNm</td>
</tr>
<tr>
<td>1.5 m</td>
<td>-25.465 kNm</td>
</tr>
<tr>
<td>1.8 m</td>
<td>24.046 kNm</td>
</tr>
<tr>
<td>2.1 m</td>
<td>73.556 kNm</td>
</tr>
<tr>
<td>2.4 m</td>
<td>123.07 kNm</td>
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<tr>
<td>2.7 m</td>
<td>172.58 kNm</td>
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<tr>
<td>3.6 m</td>
<td>210.53 kNm</td>
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<tr>
<td>3.9 m</td>
<td>196.51 kNm</td>
</tr>
<tr>
<td>4.2 m</td>
<td>177.01 kNm</td>
</tr>
<tr>
<td>4.5 m</td>
<td>152.01 kNm</td>
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<tr>
<td>4.8 m</td>
<td>121.53 kNm</td>
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<td>85.554 kNm</td>
</tr>
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<td>44.089 kNm</td>
</tr>
<tr>
<td>5.7 m</td>
<td>-2.8656 kNm</td>
</tr>
<tr>
<td>6 m</td>
<td>-55.31 kNm</td>
</tr>
</tbody>
</table>

**Centre span loaded with factored DL + LL, others with DL only**

- **Factored FEM at end of span 1**  
  \[ M_{fa}' = 1.0 \times d_{le}(1) + 0.0 \times d_{le}(1) = 67.5 \text{ kNm} \]
- **Factored FEM at start of span 2**  
  \[ M_{fa} = 1.4 \times d_{ls}(2) + 1.6 \times d_{ls}(2) = 181.69 \text{ kNm} \]
- **Factored FEM at end of span 2**  
  \[ M_{fb} = 1.4 \times d_{le}(2) + 1.6 \times d_{le}(2) = 250.31 \text{ kNm} \]
- **Factored FEM at start of span 3**  
  \[ M_{fb}' = 1.0 \times d_{ls}(3) + 0.0 \times d_{ls}(3) = 0 \text{ kNm} \]
- **Sum of FEM's at A**  
  \[ \text{sig}_{M_{fa}} = M_{fa}' + M_{fa} = -114.19 \text{ kNm} \]
- **Sum of FEM's at B**  
  \[ \text{sig}_{M_{fb}} = M_{fb} + M_{fb}' = 250.31 \text{ kNm} \]
- **Rotation at A (for unit E)**  
  \[ \theta_{A} = k_{1} \times \text{sig}_{M_{fa}} + k_{2} \times \text{sig}_{M_{fb}} = -113205 \text{ kNm} \]
- **Rotation at B (for unit E)**  
  \[ \theta_{B} = k_{3} \times \text{sig}_{M_{fa}} + k_{4} \times \text{sig}_{M_{fb}} = 261961 \text{ kNm} \]
- **Beam (2) moment at A**  
  \[ M_{a} = M_{fa} - \left( \frac{I_{(2)}}{l_{(2)}} \right) \times \left( \theta_{A} + 0.5 \times \theta_{B} \right) = -197.69 \text{ kNm} \]
- **Beam (2) moment at B**  
  \[ M_{b} = M_{fb} - \left( \frac{I_{(2)}}{l_{(2)}} \right) \times \left( \theta_{B} + 0.5 \times \theta_{A} \right) = 65.49 \text{ kNm} \]
### BMs at 20th points along span 2
(Sagging is positive)

<table>
<thead>
<tr>
<th>Distance from start</th>
<th>Bending moment (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 m</td>
<td>-197.69</td>
</tr>
<tr>
<td>0.3 m</td>
<td>-152.45</td>
</tr>
<tr>
<td>0.6 m</td>
<td>-107.22</td>
</tr>
<tr>
<td>0.9 m</td>
<td>-61.981</td>
</tr>
<tr>
<td>1.2 m</td>
<td>-16.746</td>
</tr>
<tr>
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<td>28.488</td>
</tr>
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<tr>
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</tr>
<tr>
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<td>254.66</td>
</tr>
<tr>
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<td>247.35</td>
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<tr>
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<td>234.55</td>
</tr>
<tr>
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<td>216.26</td>
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<td>163.21</td>
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<tr>
<td>4.8 m</td>
<td>128.45</td>
</tr>
<tr>
<td>5.1 m</td>
<td>88.201</td>
</tr>
<tr>
<td>5.4 m</td>
<td>42.46</td>
</tr>
<tr>
<td>5.7 m</td>
<td>-8.7699</td>
</tr>
<tr>
<td>6 m</td>
<td>-65.49</td>
</tr>
</tbody>
</table>
Location: Plane truss analysis for triangulated frameworks

Number of known forces (1-4)  \( nkf = 1 \)
Number of unknown forces (1-2)  \( nuf = 2 \)

First known force  \( f_kf = 40 \text{ kN} \)
Angle of first known force  \( \alpha_{kf} = 270^\circ \)
Angle of first unknown force  \( \alpha_{uf} = 0^\circ \)
Angle of second unknown force  \( \alpha_{sf} = 45^\circ \)
First unknown force  -40 \text{ kN tension}
Second unknown force  56.569 \text{ kN compression}
Location: Example from 'Computer Methods in Solid Mechanics'

Plane truss analysis

Elastic modulus for truss members $E=205\times 10^6$ kN/m²
Number of joints $NJ=4$
Number of members $NM=5$
Joint number 1 must always be a pinned support, the highest joint number (4 for this case) must always be a roller. Joints must be numbered consecutively from 1 such that only two unknown member forces occur at each joint. No more than five members may meet at a joint.

X coordinate for joint 1 $X(1)=0$ m
Y coordinate for joint 1 $Y(1)=0$ m
X coordinate for joint 2 $X(2)=2$ m
Y coordinate for joint 2 $Y(2)=1$ m
X coordinate for joint 3 $X(3)=2$ m
Y coordinate for joint 3 $Y(3)=2$ m
X coordinate for joint 4 $X(4)=3$ m
Y coordinate for joint 4 $Y(4)=1$ m

The order of numbering of members is significant. These are numbered consecutively counterclockwise about each joint starting with joint 1 and working through the joints in order.

Joint number at start of member $JS(1)=1$
Joint number at end of member $JE(1)=2$
Joint number at start of member $JS(2)=1$
Joint number at end of member $JE(2)=3$
Joint number at start of member $JS(3)=2$
Joint number at end of member $JE(3)=4$
Joint number at start of member $JS(4)=2$
Joint number at end of member $JE(4)=3$
Joint number at start of member $JS(5)=3$
Joint number at end of member $JE(5)=4$
X-sectional area of member $A(1)=0.001$ m²
X-sectional area of member $A(2)=0.001$ m²
X-sectional area of member $A(3)=0.001$ m²
X-sectional area of member $A(4)=0.001$ m²
X-sectional area of member $A(5)=0.001$ m²

Loaded joint number $LJ(1)=2$
Horizontal load (+ve to right) $H(2)=0$ kN
Vertical load (positive up) $V(2)=20$ kN
Loaded joint number $LJ(2)=3$
Horizontal load (+ve to right) $H(3)=6$ kN
Vertical load (positive up) $V(3)=10$ kN
Loaded joint number $LJ(3)=0$

Support reactions

Positive is up and to the right.
Vertical reaction at joint 1 $-14$ kN
Horizontal reaction at joint 1 $-6$ kN
Vertical reaction at joint 4 $-16$ kN
Horizontal reaction at joint 4 $0$ kN
Σ vertical forces applied $30$ kN
Σ horizontal forces applied $6$ kN
Member forces

<table>
<thead>
<tr>
<th>Member number</th>
<th>Forces in member</th>
<th>Positive denotes tension</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-17.889 kN</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>31.113 kN</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>-16 kN</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>-28 kN</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>22.627 kN</td>
<td></td>
</tr>
</tbody>
</table>

Joint deflections

Positive is up and to the right.

Vertical defln at joint 3  0.79488E-3 m
Horizontal defln at joint 3 -0.18781E-3 m
Plane truss analysis

Elastic modulus for truss members $E=205E6$ kN/m²
Number of joints $NJ=13$
Number of members $NM=23$

Joint number 1 must always be a pinned support, the highest joint number (13 for this case) must always be a roller. Joints must be numbered consecutively from 1 such that only two unknown member forces occur at each joint. No more than five members may meet at a joint.

| X coordinate for joint 1: $X(1)=0$ m | Y coordinate for joint 1: $Y(1)=0$ m |
| X coordinate for joint 2: $X(2)=1$ m | Y coordinate for joint 2: $Y(2)=0.45$ m |
| X coordinate for joint 3: $X(3)=2$ m | Y coordinate for joint 3: $Y(3)=0$ m |
| X coordinate for joint 4: $X(4)=3$ m | Y coordinate for joint 4: $Y(4)=0.45$ m |
| X coordinate for joint 5: $X(5)=4$ m | Y coordinate for joint 5: $Y(5)=0$ m |
| X coordinate for joint 6: $X(6)=5$ m | Y coordinate for joint 6: $Y(6)=0.45$ m |
| X coordinate for joint 7: $X(7)=6$ m | Y coordinate for joint 7: $Y(7)=0$ m |
| X coordinate for joint 8: $X(8)=7$ m | Y coordinate for joint 8: $Y(8)=0.45$ m |
| X coordinate for joint 9: $X(9)=8$ m | Y coordinate for joint 9: $Y(9)=0$ m |
| X coordinate for joint 10: $X(10)=9$ m | Y coordinate for joint 10: $Y(10)=0.45$ m |
| X coordinate for joint 11: $X(11)=10$ m | Y coordinate for joint 11: $Y(11)=0$ m |
| X coordinate for joint 12: $X(12)=11$ m | Y coordinate for joint 12: $Y(12)=0.45$ m |
| X coordinate for joint 13: $X(13)=12$ m | Y coordinate for joint 13: $Y(13)=0$ m |

The order of numbering of members is significant. These are numbered consecutively counterclockwise about each joint starting with joint 1 and working through the joints in order.

Joint number at start of member $JS(1)=1$
Joint number at end of member $JE(1)=3$
Joint number at start of member $JS(2)=1$
Joint number at end of member $JE(2)=3$
Joint number at start of member $JS(3)=2$
Joint number at end of member $JE(3)=3$
Joint number at start of member $JS(4)=2$
Joint number at end of member $JE(4)=4$
Joint number at start of member $JS(5)=3$
Joint number at end of member $JE(5)=5$
Joint number at start of member $JS(6)=3$
Joint number at end of member $JE(6)=4$
Joint number at start of member $JS(7)=4$
Joint number at end of member $JE(7)=5$
Joint number at start of member \( JS(8)=4 \)
Joint number at end of member \( JE(8)=6 \)
Joint number at start of member \( JS(9)=5 \)
Joint number at end of member \( JE(9)=7 \)
Joint number at start of member \( JS(10)=5 \)
Joint number at end of member \( JE(10)=6 \)
Joint number at start of member \( JS(11)=6 \)
Joint number at end of member \( JE(11)=7 \)
Joint number at start of member \( JS(12)=6 \)
Joint number at end of member \( JE(12)=8 \)
Joint number at start of member \( JS(13)=7 \)
Joint number at end of member \( JE(13)=9 \)
Joint number at start of member \( JS(14)=7 \)
Joint number at end of member \( JE(14)=8 \)
Joint number at start of member \( JS(15)=8 \)
Joint number at end of member \( JE(15)=9 \)
Joint number at start of member \( JS(16)=8 \)
Joint number at end of member \( JE(16)=10 \)
Joint number at start of member \( JS(17)=9 \)
Joint number at end of member \( JE(17)=11 \)
Joint number at start of member \( JS(18)=9 \)
Joint number at end of member \( JE(18)=10 \)
Joint number at start of member \( JS(19)=10 \)
Joint number at end of member \( JE(19)=11 \)
Joint number at start of member \( JS(20)=10 \)
Joint number at end of member \( JE(20)=12 \)
Joint number at start of member \( JS(21)=11 \)
Joint number at end of member \( JE(21)=13 \)
Joint number at start of member \( JS(22)=11 \)
Joint number at end of member \( JE(22)=12 \)
Joint number at start of member \( JS(23)=12 \)
Joint number at end of member \( JE(23)=13 \)

X-sectional area of member \( A(1)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(2)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(3)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(4)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(5)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(6)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(7)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(8)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(9)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(10)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(11)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(12)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(13)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(14)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(15)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(16)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(17)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(18)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(19)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(20)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(21)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(22)=0.001 \text{ m}^2 \)
X-sectional area of member \( A(23)=0.001 \text{ m}^2 \)
Loaded joint number  
Horizontal load (+ve to right)  
Vertical load (positive up)  
Loaded joint number  
Horizontal load (+ve to right)  
Vertical load (positive up)  
Loaded joint number  
Horizontal load (+ve to right)  
Vertical load (positive up)  
Loaded joint number  
Horizontal load (+ve to right)  
Vertical load (positive up)  
Loaded joint number  
Horizontal load (+ve to right)  
Vertical load (positive up)  
Loaded joint number  
Horizontal load (+ve to right)  
Vertical load (positive up)  
Loaded joint number  
Horizontal load (+ve to right)  
Vertical load (positive up)  
_Loaded joint number  
Support reactions  
Positive is up and to the right.  
Vertical reaction at joint 1  
Horizontal reaction at joint 1  
Vertical reaction at joint 13  
Horizontal reaction at joint 13  
\[ \Sigma \text{ vertical forces applied} \]  
\[ \Sigma \text{ horizontal forces applied} \]  
Member forces  
Member number  Forces in member  
Positive denotes tension  
1  100.89 kN  
2  -110.63 kN  
3  80.416 kN  
4  -174.22 kN  
5  247.56 kN  
6  -80.416 kN  
7  40.208 kN  
8  -284.22 kN  
9  320.89 kN  
10  -40.208 kN  
11  0 kN  
12  -320.89 kN  
13  320.89 kN  
14  0 kN  
15  -40.208 kN  
16  -284.22 kN  
17  247.56 kN
Joint deflections

Positive is up and to the right.

Vertical defln at joint 7 0.061167 m
Horizontal defln at joint 7 -0.0065301 m
Location: Multi-storey & multi-bay frames by moment distribution

The method used follows that given in 'Computer Methods in Solid Mechanics' by Gennaro. The effects of axial and shear deformations are ignored, as is traditional in moment distribution. Loading data comprises: horizontal loads applied at each floor level, vertical point loads and full or partial UDL's applied to beams.

Levels are numbered from top down to ground floor (from 1 to imax).

Column rows are numbered from left to right (from 1 to jmax).

Column bases may be either fixed (jond=1) or pinned (jond=0).

Number of levels                  imax=4
Number of column rows             jmax=5
Fixity of base                    jond=0

Spans in order, left to right, from 1 to 4 spans.

Sp(1)=18.75 m
Sp(2)=22 m
Sp(3)=24 m
Sp(4)=20.25 m

Storey heights in order, top to bottom, from 1 to 3 storeys.

Ht(1)=15 m
Ht(2)=16 m
Ht(3)=18.75 m
Beam inertias in order, left to right & top to bottom, 1 to 12

\[ \begin{align*}
    b_i(1) &= 100 \text{ cm}^4 \\
    b_i(2) &= 100 \text{ cm}^4 \\
    b_i(3) &= 0 \text{ cm}^4 \\
    b_i(4) &= 0 \text{ cm}^4 \\
    b_i(5) &= 200 \text{ cm}^4 \\
    b_i(6) &= 200 \text{ cm}^4 \\
    b_i(7) &= 400 \text{ cm}^4 \\
    b_i(8) &= 0 \text{ cm}^4 \\
    b_i(9) &= 300 \text{ cm}^4 \\
    b_i(10) &= 360 \text{ cm}^4 \\
    b_i(11) &= 425 \text{ cm}^4 \\
    b_i(12) &= 300 \text{ cm}^4
\end{align*}\]

Column inertias in order, left to right & top to bottom 1 to 15

\[ \begin{align*}
    c_i(1) &= 100 \text{ cm}^4 \\
    c_i(2) &= 100 \text{ cm}^4 \\
    c_i(3) &= 100 \text{ cm}^4 \\
    c_i(4) &= 0 \text{ cm}^4 \\
    c_i(5) &= 0 \text{ cm}^4 \\
    c_i(6) &= 200 \text{ cm}^4 \\
    c_i(7) &= 300 \text{ cm}^4 \\
    c_i(8) &= 300 \text{ cm}^4 \\
    c_i(9) &= 250 \text{ cm}^4 \\
    c_i(10) &= 0 \text{ cm}^4 \\
    c_i(11) &= 300 \text{ cm}^4 \\
    c_i(12) &= 400 \text{ cm}^4 \\
    c_i(13) &= 400 \text{ cm}^4 \\
    c_i(14) &= 375 \text{ cm}^4 \\
    c_i(15) &= 250 \text{ cm}^4
\end{align*}\]

Horizontal (sway) loads applied to each floor level, top to bottom in order. Positive loads act from left to right (3 : reqd)

\[ \begin{align*}
    p_h(1) &= 0 \text{ kN} \\
    p_h(2) &= 0 \text{ kN} \\
    p_h(3) &= 0 \text{ kN}
\end{align*}\]

Total No. of vertical UDL's \( M = 6 \)

Level number containing UDL \( u_l(1) = 1 \)

Span number containing UDL \( u_j(1) = 1 \)

Magnitude of UDL (+ve upwards) \( u_w(1) = -2 \text{ kN/m} \)

Dist to start of UDL from LH col \( u_a(1) = 0 \text{ m} \)
Dist to end of UDL from LH column $ub(1)=18.75$ m
Level number containing UDL $ui(2)=1$
Span number containing UDL $uj(2)=2$
Magnitude of UDL (+ve upwards) $uw(2)=-2$ kN/m
Dist to start of UDL from LH col $ua(2)=0$ m
Dist to end of UDL from LH column $ub(2)=22$ m
Level number containing UDL $ui(3)=2$
Span number containing UDL $uj(3)=1$
Magnitude of UDL (+ve upwards) $uw(3)=-4$ kN/m
Dist to start of UDL from LH col $ua(3)=0$ m
Dist to end of UDL from LH column $ub(3)=18.75$ m
Level number containing UDL $ui(4)=2$
Span number containing UDL $uj(4)=3$
Magnitude of UDL (+ve upwards) $uw(4)=-4$ kN/m
Dist to start of UDL from LH col $ua(4)=0$ m
Dist to end of UDL from LH column $ub(4)=24$ m
Level number containing UDL $ui(5)=3$
Span number containing UDL $uj(5)=1$
Magnitude of UDL (+ve upwards) $uw(5)=-4$ kN/m
Dist to start of UDL from LH col $ua(5)=0$ m
Dist to end of UDL from LH column $ub(5)=18.75$ m
Level number containing UDL $ui(6)=3$
Span number containing UDL $uj(6)=3$
Magnitude of UDL (+ve upwards) $uw(6)=-4.25$ kN/m
Dist to start of UDL from LH col $ua(6)=0$ m
Dist to end of UDL from LH column $ub(6)=24$ m
Total No of vertical point loads $N=8$
Level no. containing point load $pi(1)=1$
Span number containing point load $pj(1)=1$
Magnitude of load (+ve upwards) $pv(1)=-10$ kN
Distance from start of span $pa(1)=18.75$ m
Level no. containing point load $pi(2)=1$
Span number containing point load $pj(2)=2$
Magnitude of load (+ve upwards) $pv(2)=5$ kN
Distance from start of span $pa(2)=5$ m
Level no. containing point load $pi(3)=1$
Span number containing point load $pj(3)=2$
Magnitude of load (+ve upwards) $pv(3)=-5$ kN
Distance from start of span $pa(3)=10$ m
Level no. containing point load $pi(4)=2$
Span number containing point load $pj(4)=3$
Magnitude of load (+ve upwards) $pv(4)=-10$ kN
Distance from start of span $pa(4)=4$ m
Level no. containing point load $pi(5)=2$
Span number containing point load $pj(5)=3$
Magnitude of load (+ve upwards) $pv(5)=-2$ kN
Distance from start of span $pa(5)=11.5$ m
Level no. containing point load $pi(6)=3$
Span number containing point load $pj(6)=3$
Magnitude of load (+ve upwards) $pv(6)=-20$ kN
Distance from start of span $pa(6)=7.125$ m
Level no. containing point load $pi(7)=3$
Span number containing point load $pj(7)=4$
Magnitude of load (+ve upwards) $pv(7)=-5$ kN
Distance from start of span $pa(7)=11$ m
Level no. containing point load   \( pi(8)=3 \)
Span number containing point load  \( pj(8)=4 \)
Magnitude of load (+ve upwards)   \( pv(8)=1 \text{ kN} \)
Distance from start of span       \( pa(8)=13 \text{ m} \)
Test increment                    \( \epsilon=0.01 \text{ kNm} \)

**Beam moments at 20th points**

The sign convention given above is that generally used in the computer analysis of structures. The traditional convention of sagging positive and hogging negative cannot apply to vertical members. Because the traditional convention is still in general use for the design of beams, it is used in the following tables of beam moments at 20th points.

<table>
<thead>
<tr>
<th>Level 1 span 1 BMs at 20th points (+ve sagging) left to right kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>-31.12</td>
</tr>
<tr>
<td>26.65</td>
</tr>
<tr>
<td>26.44</td>
</tr>
<tr>
<td>-17.71</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Level 1 span 2 BMs at 20th points (+ve sagging) left to right kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>-88.88</td>
</tr>
<tr>
<td>24.59</td>
</tr>
<tr>
<td>62.93</td>
</tr>
<tr>
<td>23.78</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Level 2 span 1 BMs at 20th points (+ve sagging) left to right kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>-94.83</td>
</tr>
<tr>
<td>50.65</td>
</tr>
<tr>
<td>75.19</td>
</tr>
<tr>
<td>11.85</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Level 2 span 2 BMs at 20th points (+ve sagging) left to right kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>-3</td>
</tr>
<tr>
<td>-13.73</td>
</tr>
<tr>
<td>-31.61</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Level 2 span 3 BMs at 20th points (+ve sagging) left to right kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>-185.9</td>
</tr>
<tr>
<td>106.5</td>
</tr>
<tr>
<td>155.1</td>
</tr>
<tr>
<td>51.04</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Level 3 span 1 BMs at 20th points (+ve sagging) left to right kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>-87.68</td>
</tr>
<tr>
<td>53.89</td>
</tr>
<tr>
<td>75.18</td>
</tr>
<tr>
<td>8.589</td>
</tr>
</tbody>
</table>
Level 3 span 2 BMs at 20th points (+ve sagging) left to right kNm

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 3 span 2 BMs at 20th points (+ve sagging) left to right kNm</td>
<td></td>
<td></td>
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</tbody>
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<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-26.67</td>
<td>-29.05</td>
<td>-31.44</td>
<td>-33.82</td>
<td>-36.2</td>
</tr>
<tr>
<td></td>
<td>-38.59</td>
<td>-40.97</td>
<td>-43.35</td>
<td>-45.73</td>
<td>-48.12</td>
</tr>
</tbody>
</table>

Level 3 span 3 BMs at 20th points (+ve sagging) left to right kNm

<table>
<thead>
<tr>
<th></th>
<th>-233.1</th>
<th>-157</th>
<th>-87.04</th>
<th>-23.21</th>
<th>34.51</th>
<th>86.11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 3 span 3 BMs at 20th points (+ve sagging) left to right kNm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>130.1</th>
<th>145.4</th>
<th>154.7</th>
<th>157.8</th>
<th>154.8</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>145.7</td>
<td>130.4</td>
<td>109</td>
<td>81.56</td>
<td>47.96</td>
</tr>
<tr>
<td></td>
<td>8.235</td>
<td>-37.61</td>
<td>-89.57</td>
<td>-147.7</td>
<td>-211.9</td>
</tr>
</tbody>
</table>

Level 3 span 4 BMs at 20th points (+ve sagging) left to right kNm

<table>
<thead>
<tr>
<th></th>
<th>-52.36</th>
<th>-47.63</th>
<th>-42.91</th>
<th>-38.18</th>
<th>-33.45</th>
<th>-28.72</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 3 span 4 BMs at 20th points (+ve sagging) left to right kNm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-1.031</td>
<td>-1.365</td>
<td>-1.536</td>
<td>-0.8564</td>
<td>-0.1773</td>
</tr>
<tr>
<td></td>
<td>0.5018</td>
<td>1.181</td>
<td>1.86</td>
<td>2.539</td>
<td>3.218</td>
</tr>
</tbody>
</table>
**Location: Multi-storey frame by Naylor's Method**

Number of storeys (1 to 4) \( ns = 4 \)
Span of frame \( L = 20 \) m

Analysis of 1 bay x 4 storey frame using Naylor's method; Ref: the Structural Engineer April 1950.

Inertia of columns:
- Ground floor \( I_g = 40 \) m
- First floor \( I_1 = 30 \) m
- Second floor \( I_2 = 20 \) m
- Third floor \( I_3 = 10 \) m

Inertia of beams \( I_b = 20 \) m

Horizontal force at first floor \( H_c = -50 \) kN
Horizontal force at second floor \( H_e = 100 \) kN
Horizontal force at third floor \( H_g = 100 \) kN
Horizontal force at fourth floor \( H_j = 0 \) kN

**Moment distribution:**

<table>
<thead>
<tr>
<th>A</th>
<th>C</th>
<th>E</th>
<th>G</th>
<th>J</th>
</tr>
</thead>
<tbody>
<tr>
<td>-375</td>
<td>-375</td>
<td>0</td>
<td>-500</td>
<td>-500</td>
</tr>
</tbody>
</table>

**Distribution factors:**

| 0  | 0.308 | 0.462 | 0.231 | 0.273 | 0.545 | 0.182 | 0.222 | 0.667 | 0.111 | 0.143 | 0.857 |

**Fixed end moments:**

<table>
<thead>
<tr>
<th>D denotes distributed moments</th>
<th>C denotes carry over moments</th>
<th>F denotes final moments</th>
</tr>
</thead>
<tbody>
<tr>
<td>D 0  269  202  205  136  55.6  27.8  0</td>
<td>C -269  0  -205  -202  -55.6  -136  0</td>
<td>G  94.4  140  90.9  23.8</td>
</tr>
<tr>
<td>D 0  62.9  47.2  70.2  46.8  30.3  15.2  3.97</td>
<td>C -62.9  0  -70.2  -47.2  -30.3  -46.8  -3.97  -15.2</td>
<td></td>
</tr>
<tr>
<td>D 0  32.4  42.3  33.9  13</td>
<td>C -21.6  0  -21.1  -16.2  -11.3  -14.1  -2.16  -5.64</td>
<td></td>
</tr>
<tr>
<td>D 9.76  15  10.8  4.84</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>0</td>
<td>6.5</td>
</tr>
<tr>
<td>C</td>
<td>-6.5</td>
<td>0</td>
</tr>
<tr>
<td>D</td>
<td>3.46</td>
<td>4.63</td>
</tr>
<tr>
<td>D</td>
<td>2.31</td>
<td>1.73</td>
</tr>
<tr>
<td>C</td>
<td>-2.31</td>
<td>0</td>
</tr>
<tr>
<td>D</td>
<td>1.07</td>
<td>1.65</td>
</tr>
<tr>
<td>D</td>
<td>0.712</td>
<td>0.534</td>
</tr>
<tr>
<td>C</td>
<td>-2.31</td>
<td>0</td>
</tr>
<tr>
<td>D</td>
<td>1.07</td>
<td>1.65</td>
</tr>
<tr>
<td>D</td>
<td>0.712</td>
<td>0.534</td>
</tr>
<tr>
<td>C</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>F</td>
<td>545</td>
<td>614</td>
</tr>
<tr>
<td>F</td>
<td>-739</td>
<td>-11.4</td>
</tr>
</tbody>
</table>

**SCALE 5.48**  
Office 1007  
Proforma 624
Location: Example of Vierendeel girders by Naylor's Method

![Diagram of Vierendeel girders]

Inertia of column AB \( I' = 10 \) m\(^2\)
Inertia of column CD \( I_1 = 10 \) m\(^2\)
Inertia of column EF \( I_2 = 10 \) m\(^2\)
Inertia of column GH \( I_3 = 10 \) m\(^2\)
Vertical force at C \( V_c = 100 \) kN
Vertical force at E \( V_e = 50 \) kN
Height of Vierendeel girder \( L = 10 \) m
Width of bay B \( \rightarrow \) D \( a = 10 \) m
Width of bay D \( \rightarrow \) F \( b = 10 \) m
Width of bay F \( \rightarrow \) H \( c = 10 \) m
Inertia of beams in bay B \( \rightarrow \) D \( I_{b1} = 10 \) m\(^2\)
Inertia of beams in bay D \( \rightarrow \) F \( I_{b2} = 10 \) m\(^2\)
Inertia of beams in bay F \( \rightarrow \) H \( I_{b3} = 10 \) m\(^2\)

Moment distribution:

<table>
<thead>
<tr>
<th></th>
<th>A</th>
<th>C</th>
<th>E</th>
<th>G</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>179</td>
<td>125</td>
<td>-156</td>
<td>-143</td>
</tr>
<tr>
<td>C</td>
<td>29.8</td>
<td>20.8</td>
<td>20.8</td>
<td>-26</td>
</tr>
<tr>
<td>D</td>
<td>-20.8</td>
<td>-29.8</td>
<td>26</td>
<td>-20.8</td>
</tr>
<tr>
<td>D</td>
<td>17.9</td>
<td>2.79</td>
<td>-2.23</td>
<td>-22.3</td>
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<tr>
<td>C</td>
<td>2.98</td>
<td>0.465</td>
<td>0.465</td>
<td>-0.372</td>
</tr>
<tr>
<td>D</td>
<td>-0.465</td>
<td>-2.98</td>
<td>0.372</td>
<td>-0.465</td>
</tr>
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<td>-0.319</td>
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<tr>
<td>C</td>
<td>197</td>
<td>130</td>
<td>-161</td>
<td>-166</td>
</tr>
<tr>
<td>C</td>
<td>-197</td>
<td>-220</td>
<td>90.1</td>
<td>-6.72</td>
</tr>
</tbody>
</table>

D denotes distributed moments - 1st line of each pair is column moment
C denotes carry over moments
F denotes final moments
**Location: Example using notation of Arthur Morley**

- **Weight supported by shear legs**: \( W = 5000 \text{ kN} \)
- **Length of each shear leg**: \( L = 8 \text{ m} \)
- **Distance between leg supports**: \( d_c = 5 \text{ m} \)
- **Horizontal distance between A & E**: \( a_e = 10 \text{ m} \)
- **Horizontal distance between E & F**: \( e_f = 3 \text{ m} \)
- **Distance between B & E**: \( b_e = \sqrt{L^2 - (d_c/2)^2} = 7.5993 \text{ m} \)
- **Vertical distance between B & F**: \( b_f = \sqrt{b_e^2 - e_f^2} = 6.9821 \text{ m} \)
- **Length of guy**: \( a_f = a_e + e_f = 13 \text{ m} \)
- **Angle of guy with ground**: \( \alpha = \text{ATN}(b_f/a_f) = 0.49287 \text{ rad} \) \( \alpha' = \text{DEG}(\alpha) = 28.24^\circ \)
- **Angle of BE with ground**: \( \beta = \text{ATN}(b_f/e_f) = 1.165 \text{ rad} \) \( \beta' = \text{DEG}(\beta) = 66.748^\circ \)
- **Tension in guy AB**: \( T = \frac{W}{\sin(\alpha) - \cos(\alpha) \sin(\beta)/\cos(\beta)} = -3170.2 \text{ kN} \)
- **Compression in shear legs**: \( C = -T \cos(\alpha)/\cos(\beta)/(2 \cdot b_e/L) = 3723.8 \text{ kN} \)
Location: Ex1 - Rectangular pinned base portal – after Kleinlogel

Sign conventions:

The direction of the load is considered to be positive.

The moments causing tension on the inside faces of the frame are considered to be positive.

Upward vertical and inward horizontal reactions are positive.

Height of portal $h=4.88$ m
Length of portal $L=9.75$ m
Inertia of columns $I_1=1$ m$^4$
Inertia of beam $I_2=1$ m$^4$
Uniformly distributed load $w=2.05$ kN/m
Moment at B $M_b=(w\cdot h^2/4)\cdot(-k/(2*N)+1)=11.441$ kNm
Moment at C $M_c=(w\cdot h^2/4)\cdot(-k/(2*N)-1)=-12.968$ kNm
Vertical reaction at A $V_a=-w\cdot h^2/(2*L)=-2.5036$ kN
Vertical reaction at D $V_d=-V_a=2.5036$ kN
Horizontal reaction at D $H_d=-M_c/h=2.6574$ kN
Horizontal reaction at A $H_a=-(w\cdot h-H_d)=-7.3466$ kN
**Location:** Ridged pinned base portal – after Kleinlogel

---

*Sign conventions:*

- The direction of the load is considered to be positive.
- The moments causing tension on the inside faces of the frame are considered to be positive.
- Upward vertical and inward horizontal reactions are positive.

**Dimensions:**

- Height to eaves: \( h = 4.88 \) m
- Height eaves to ridge: \( f = 1.6 \) m
- Span of portal: \( L = 9.75 \) m
- Inertia of columns: \( I_1 = 1 \) m\(^4\)
- Inertia of rafters: \( I_2 = 1 \) m\(^4\)

**Uniformly distributed load:** \( w = 2.05 \) kN/m

**Moments:**

- Moment at D: \( M_D = -w h^2/2 + M_D = -11.33 \) kNm
- Moment at B: \( M_B = w h^2/2 + M_D = 13.08 \) kNm
- Moment at C: \( M_C = w h^2/4 + M_D = -2.8396 \) kNm

**Vertical reactions:**

- Vertical reaction at A: \( V_A = -w h^2/(2 L) = -2.5036 \) kN
- Vertical reaction at E: \( V_E = V_A = 2.5036 \) kN

**Horizontal reactions:**

- Horizontal reaction at E: \( H_E = -M_D/h = 2.3217 \) kN
- Horizontal reaction at A: \( H_A = -(w h - H_E) = -7.6823 \) kN
Location: Two bay ridged pinned base portal

Side uniformly distributed load  \( w = 2.05 \text{ kN/m} \)

Moment at A  
\[
M_a = w h^2 \xi (-n_{11} - n) / 4 + w h^2 (y_{11} + y_{13}) / 2 = 8.739 \text{ kNm}
\]

Moment at A'  
\[
M_a' = w h^2 \xi (-n_{11} + n) / 4 + w h^2 (y_{11} - y_{13}) / 2 = -6.6323 \text{ kNm}
\]

Moment at C in L.H. rafter  
\[
M_{c1} = w h^2 \xi (n_{12} - n) / 4 + w h^2 (y_{12} - y_{14}) / 2 = -2.3038 \text{ kNm}
\]

Moment at C in R.H. rafter  
\[
M_{c2} = w h^2 \xi (n_{12} + n) / 4 + w h^2 (y_{12} + y_{14}) / 2 = 6.7347 \text{ kNm}
\]

Moment at B  
\[
M_b = -w h^2 \phi / 2 + \frac{M_a (1 + 2 \phi) + M_{c1}}{2} = -1.9203 \text{ kNm}
\]

Moment at B'  
\[
M_b' = \frac{M_{c2} + M_a (1 + 2 \phi) + M_{c1}}{2} = -2.1233 \text{ kNm}
\]

Moment at C in centre column  
\[
M_{cii} = M_{c1} - M_{c2} = -9.0385 \text{ kNm}
\]

Vertical reaction at D  
\[
V_d = \frac{-M_a + M_{c1}}{L} = -1.1326 \text{ kN}
\]

Vertical reaction at E  
\[
V_e = \frac{M_{cii}}{h} = -1.8521 \text{ kN}
\]

Shear forces in the rafters (numbered from left to right):  
First rafter  
\[
T_1 = V_1 \cos \theta + (H_d - w h) \sin \theta = -2.0775 \text{ kN}
\]

Second rafter  
\[
T_2 = V_2 \cos \theta + (H_d - w h) \sin \theta = -0.8788 \text{ kN}
\]

Third rafter  
\[
T_3 = V_3 \cos \theta + (H_d - w h) \sin \theta = 3.4043 \text{ kN}
\]

Axial thrusts in the rafters (numbered from left to right):  
First rafter  
\[
N_1 = V_1 \sin \theta - (H_d - w h) \cos \theta = 2.6979 \text{ kN}
\]

Second rafter  
\[
N_2 = V_2 \sin \theta - (H_d - w h) \cos \theta = 0.86378 \text{ kN}
\]
Fourth rafter

\[ N'2 = -V2 \sin \theta - H'd \cos \theta = 1.7188 \text{ kN} \]
Location: Couple roof by Kleinlogel

Sign conventions:

The direction of the load is considered to be positive.

The moments causing tension on the inside faces of the frame are considered to be positive.

Upward vertical and inward horizontal reactions are positive.

Height to ridge \( h = 5.6 \) m
Length of couple roof \( L = 9.75 \) m
Horizontal length A to B \( a = 6.75 \) m
Inertia of left rafter \( I_1 = 0.1E^{-3} \) m\(^4\)
Inertia of right rafter \( I_2 = 0.2E^{-3} \) m\(^4\)

Uniformly distributed load \( w = 4 \) kN/m
Moment at B \( M_B = -w \cdot h^2 \cdot 2 \cdot k / (8 \cdot N) = -4.169 \) kNm
Vertical reaction at A \( V_A = w \cdot h^2 / (2 \cdot L) = 6.4328 \) kN
Vertical reaction at C \( V_C = -V_A = -6.4328 \) kN
Horizontal reaction at A \( H_A = w \cdot h \cdot a / 2 - M_B / h = 8.4983 \) kN
Horizontal reaction at C \( H_C = -(w \cdot h - H_A) = -13.902 \) kN
Location: Circular arch — after Roark (Fourth Edition)

O is centre of circle of radius R, of which arch is a segment

th' is angle (degrees) subtended by half arch segment

R = radius of arch segment
i.e. distance between centre O and support points 1 & 2

Forces are positive when as shown in the diagrams.

Elastic modulus of elasticity $E = 9 \times 10^6 \text{kN/m}^2$
Area of X section of arch $A = 1.44 \text{m}^2$
Moment of inertia of arch X sectn $I = 0.1728 \text{m}^4$
Radius of arch $R = 15 \text{m}$
Angle $\theta' = 40^\circ$
Point load $W = 20 \text{kN}$
Angle $\phi' = 15^\circ$
Horizontal force $H = \frac{\mu u}{d e} = 16.748 \text{kN}$
Vertical reaction $V_1 = 0.5 \times W \times (s+n)/s = 14.027 \text{kN}$
Vertical reaction $V_2 = W - V_1 = 5.9735 \text{kN}$
Location: Example on page 386 of Designer's Handbook

Analysis of helical stair with fixed supports

---

**Summary of analysis**

<table>
<thead>
<tr>
<th>Angle deg.</th>
<th>Mv kNm</th>
<th>Mh kNm</th>
<th>Mt kNm</th>
<th>Vv kN</th>
<th>Vh kN</th>
<th>Hh kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>-120</td>
<td>-10.33</td>
<td>47.95</td>
<td>-0.5821</td>
<td>-31.67</td>
<td>-16.82</td>
<td>46.91</td>
</tr>
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<tr>
<td>0</td>
<td>-3.803</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>33.64</td>
<td>0</td>
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<td>1.136</td>
<td>-48.19</td>
<td>0.0665</td>
<td>9.676</td>
<td>16.82</td>
<td>-36.66</td>
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<tr>
<td>120</td>
<td>-10.33</td>
<td>-47.95</td>
<td>0.5821</td>
<td>31.67</td>
<td>-16.82</td>
<td>-46.91</td>
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</tbody>
</table>
Location: Stair turning through more than a complete circle

Analysis of helical stair with fixed supports

<table>
<thead>
<tr>
<th>Angle deg.</th>
<th>Mv kNm</th>
<th>Mh kNm</th>
<th>Mt kNm</th>
<th>Vv kN</th>
<th>Vh kN</th>
<th>Hh kN</th>
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<tbody>
<tr>
<td>-225</td>
<td>-231</td>
<td>-48.36</td>
<td>121.4</td>
<td>-155.8</td>
<td>-53.46</td>
<td>10.83</td>
</tr>
<tr>
<td>-175</td>
<td>-50.96</td>
<td>18.95</td>
<td>14.21</td>
<td>-101.6</td>
<td>-75.31</td>
<td>52.43</td>
</tr>
<tr>
<td>-125</td>
<td>37.47</td>
<td>127.9</td>
<td>20.93</td>
<td>-49.26</td>
<td>-43.36</td>
<td>89.72</td>
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<tr>
<td>-75</td>
<td>4.389</td>
<td>160.7</td>
<td>46.85</td>
<td>-14.97</td>
<td>19.57</td>
<td>86.6</td>
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<tr>
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<td>-61.01</td>
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<td>68.52</td>
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<td>1.896</td>
<td>68.52</td>
<td>-35.82</td>
</tr>
<tr>
<td>75</td>
<td>4.389</td>
<td>-160.7</td>
<td>-46.85</td>
<td>14.97</td>
<td>19.57</td>
<td>-86.6</td>
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<td>49.26</td>
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<td>-10.83</td>
</tr>
</tbody>
</table>

SCALE 5.48          Office 1007            Proforma 639
Location: Steep narrow semi-circular stairway

Analysis of helical stair with fixed supports

<table>
<thead>
<tr>
<th>Angle deg.</th>
<th>Mv [kNm]</th>
<th>Mh [kNm]</th>
<th>Mt [kNm]</th>
<th>Vv [kN]</th>
<th>Vh [kN]</th>
<th>Hh [kN]</th>
</tr>
</thead>
<tbody>
<tr>
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<td>-2.408</td>
<td>20.44</td>
<td>-0.8511</td>
<td>-14.15</td>
<td>0</td>
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<tr>
<td>-45</td>
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<td>-0.5159</td>
<td>-5.28</td>
<td>13.04</td>
<td>17.58</td>
</tr>
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<td>0</td>
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</tr>
<tr>
<td>45</td>
<td>-0.3564</td>
<td>-14.5</td>
<td>0.5159</td>
<td>5.28</td>
<td>13.04</td>
<td>-17.58</td>
</tr>
<tr>
<td>90</td>
<td>-2.408</td>
<td>-20.44</td>
<td>0.8511</td>
<td>14.15</td>
<td>0</td>
<td>-28.41</td>
</tr>
</tbody>
</table>

Breadth of stair $B=1000\ mm$
Thickness of stair $H=135\ mm$
Dead load partial safety factor $gf=1.4$
Imposed-load safety factor $qf=1.6$
Analysis of helical stair with fixed supports

Breadth of stair  
Thickness of stair  
Perm load partial safety factor  
Variable load safety factor

Summary of analysis

<table>
<thead>
<tr>
<th>Angle deg.</th>
<th>Mv</th>
<th>Mh</th>
<th>Mt</th>
<th>Vv</th>
<th>Vh</th>
<th>Hh</th>
</tr>
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<tbody>
<tr>
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<td>1.083</td>
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<td>16.04</td>
<td>-34.96</td>
</tr>
<tr>
<td>120</td>
<td>-9.851</td>
<td>-45.73</td>
<td>0.5552</td>
<td>30.2</td>
<td>-16.04</td>
<td>-44.74</td>
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</table>
Location: Stair turning through more than a complete circle

Analysis of helical stair with fixed supports

<table>
<thead>
<tr>
<th>Angle deg.</th>
<th>Mv kNm</th>
<th>Mh kNm</th>
<th>Mt kNm</th>
<th>Vv kN</th>
<th>Vh kN</th>
<th>Hh kN</th>
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</thead>
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<tr>
<td>-225</td>
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<tr>
<td>-175</td>
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</table>

Breadth of stair B=1500 mm
Thickness of stair H=150 mm
Perm load partial safety factor gamG=1.35
Variable load safety factor gamQ=1.5
Location: Steep narrow semi-circular stairway

Analysis of helical stair with fixed supports

<table>
<thead>
<tr>
<th>Angle deg.</th>
<th>Mv kNm</th>
<th>Mh kNm</th>
<th>Mt kNm</th>
<th>Vv kN</th>
<th>Vh kN</th>
<th>Hh kN</th>
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<td>19.45</td>
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<tr>
<td>45</td>
<td>-0.3391</td>
<td>-13.8</td>
<td>0.491</td>
<td>5.025</td>
<td>12.41</td>
<td>-16.73</td>
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<td>90</td>
<td>-2.292</td>
<td>-19.45</td>
<td>0.8099</td>
<td>13.46</td>
<td>0</td>
<td>-27.04</td>
</tr>
</tbody>
</table>

Summary of analysis

Breadth of stair: B=1000 mm
Thickness of stair: H=135 mm
Perm load partial safety factor: gamG=1.35
Variable load safety factor: gamQ=1.5

Thickness of stair 'H' is measured at the waist.
In beams of small span/depth ratio, the shear stresses are likely to be high, and the deflection due to shear must be added to the deflection due to bending. Roark gives expressions for computing the deflection due to shear for common cases of loading.

For extremely short beams (span/depth ratios < 3), the assumption of linear stress distribution on which the simple theory of flexure is based is no longer valid. Actual extreme fibre stresses, and horizontal shear stresses are significantly higher than those computed by the simple theory. Roark gives a table of values by which stresses from the simple theory must be multiplied to obtain values found from strain-gauge measurements and photoelastic studies, and these values are used below.

### Beams of relatively great depth

- Uniform load over entire length $w=200 \text{ kN/m}$
- Span taken as length of beam $l=3 \text{ m}$
- Depth of beam $d=1 \text{ m}$
- Thickness of beam $t=0.4 \text{ m}$
- Span of beam $\text{span}=l\times23/24=2.875 \text{ m}$
- Total load on beam $W=w\times l=600 \text{ kN}$
- End shears $V=W/2=300 \text{ kN}$
- Maximum bending moment $M=W\times \text{span}/8=215.63 \text{ kNm}$
- Total deflection $y=y_b+y_s=0.00199 \text{ m}$
Location: Roof trusses – after Dorman Long Handbook 1964

Span of truss
L=10 m
UDL on plan
w1=3 kN/m
UDL suction on L.H. rafter
w2=1.5 kN/m

3 kN/m (on plan) =30 kN total load

Forces due to plan loading (positive indicates compression)
Force in member a-b
fab=1.006*W1=30.18 kN
Force in member b-d
fbd=0.894*W1=26.82 kN
Force in member d-e
fde=0.894*W1=26.82 kN
Force in member e-g
feg=0.671*W1=20.13 kN
Force in member g-h
fgh=0.671*W1=20.13 kN
Force in member a-c
fac=(-0.9)*W1=-27 kN
Force in member c-f
fcf=(-0.7)*W1=-21 kN
Force in member f-j
ffj=(-0.5)*W1=-15 kN
Force in member b-c
fbc=0.112*W1=3.36 kN
Force in member c-d
fcd=0.100*W1=3 kN
Force in member e-f
fef=0.180*W1=5.4 kN
Force in member f-g
ffg=0.100*W1=3 kN
Force in member c-e
fce=(-0.18)*W1=-5.4 kN
Force in member f-h
ffh=(-0.269)*W1=-8.07 kN

Vertical reaction at both ends
fyb=0.5*W1=15 kN
Horizontal reaction at both ends
fhb=0.0*W1=0 kN

Forces due to wind suction on L.H. rafter (+ve indicates compression)
Force in member a-b
fab=(-1.175)*W2=-9.8527 kN
Force in member b-d
fbd=(-1.025)*W2=-8.5949 kN
Force in member d-e
fde=(-1.125)*W2=-9.4334 kN
Force in member e-g
feg=(-0.725)*W2=-6.0793 kN
Force in member g-h
fgh=(-0.825)*W2=-6.9178 kN
Force in member a-c
fac=1.230*W2=10.314 kN
Force in member c-f
fcf=-0.735*W2=-6.5657 kN
Force in member f-j
ffj=-0.335*W2=-2.8091 kN
Force in member b-c
fbc=(-0.250)*W2=-2.0963 kN
Force in member c-d
fcd=(-0.224)*W2=-1.8783 kN
Force in member e-f
fef=(-0.403)*W2=-3.3793 kN
Force in member f-g
ffg=(-0.224)*W2=-1.8783 kN
Force in member c-e
fce=0.403*W2=3.3793 kN
Force in member f-h \( f_{fh} = 0.602 \times W^2 = 5.0479 \text{ kN} \)
Vertical reaction at a \( f_{va} = -0.615 \times W^2 = -5.1569 \text{ kN} \)
Vertical reaction at other end \( f_{vo} = -0.280 \times W^2 = -2.3479 \text{ kN} \)
Horizontal reaction at both ends \( f_{hb} = 0.224 \times W^2 = 1.8783 \text{ kN} \)
Depth of section                  $h=1.0361\ m$
Breadth of section                $b=0.3085\ m$
Web thickness                     $tw=0.03\ m$
Flange thickness                  $tf=0.0541\ m$
Root radius                       $r=0.03\ m$
Steel grade (235,275,355,460)     $stg=275\ N/mm^2$
Axial force at ULS                $N_{ed}=7903\ kN$
Bending moment about yy axis      $M_{yyed}=2958\ kNm$
Bending moment about zz axis      $M_{zzed}=354\ kNm$
Analysis by traditional methods
Section design for UB's UC's SHS's RHS's & CHS's

Depth of section                  h=0.127 m
Breadth of section                b=0.076 m
Web thickness                     tw=0.004 m
Flange thickness                  tf=0.0076 m
Root radius                       r=0.0076 m
Steel grade (235,275,355,460)     stg=275 N/mm²
Axial force at ULS                Ned=208 kN
Bending moment about yy axis      Myed=10 kNm
Bending moment about zz axis      Mzed=2.8 kNm
Depth of section                  h=0.5 m
Breadth of section                b=0.3 m
Wall thickness                    t=0.02 m
Outside corner radius             R=0.025 m
Inside corner radius              r=0.02 m
Steel grade (235,275,355,460)     stg=275 N/mm²
Axial force                       Ned=6353 kN
Bending moment about yy axis      Myed=208 kNm
Bending moment about zz axis      Mzed=145 kNm
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Depth of section</td>
<td>h = 0.05 m</td>
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<tr>
<td>Breadth of section</td>
<td>b = 0.03 m</td>
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<tr>
<td>Wall thickness</td>
<td>t = 0.0032 m</td>
</tr>
<tr>
<td>Outside corner radius</td>
<td>R = 0.004 m</td>
</tr>
<tr>
<td>Inside corner radius</td>
<td>r = 0.0032 m</td>
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<tr>
<td>Steel grade (235, 275, 355, 460)</td>
<td>stg = 275 N/mm²</td>
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<tr>
<td>Axial force</td>
<td>Ned = 50 kN</td>
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<tr>
<td>Bending moment about yy axis</td>
<td>Myed = 0.81 kNm</td>
</tr>
<tr>
<td>Bending moment about zz axis</td>
<td>Mzed = 0.56 kNm</td>
</tr>
</tbody>
</table>
Diameter of section         \( d = 0.508 \text{ m} \)
Wall thickness             \( t = 0.02 \text{ m} \)
Steel grade (235,275,355,460) \( \text{stg} = 275 \text{ N/mm}^2 \)
Axial force                \( \text{Ned} = 32 \text{ kN} \)
Bending moment about yy axis \( \text{Myed} = 0.25 \text{ kNm} \)
Bending moment about zz axis \( \text{Mzed} = 0.25 \text{ kNm} \)
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<td>Axial force</td>
<td>Ned=32 kN</td>
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<td>Bending moment about yy axis</td>
<td>Myed=0.25 kNm</td>
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<tr>
<td>Bending moment about zz axis</td>
<td>Mzed=0.25 kNm</td>
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Location: Plastic section properties for I section

Plastic section properties of I Section:

\[ \begin{align*}
\text{First moment of I Section about } zz &= DZ \cdot TY \cdot (DY - TY) + \frac{(DY - 2 \cdot TY)^2 \cdot TZ}{4} \\
\text{First moment of I Section about } yy &= \frac{2 \cdot DZ^2 \cdot TY}{4} + \frac{TZ^2 \cdot (DY - 2 \cdot TY)}{4} \\
\text{Plastic torque constant} &= \frac{DZ \cdot TY^2}{2} + \frac{DY \cdot TZ^2}{4} 
\end{align*} \]

Depth of section: \( DY = 200 \text{ mm} \)
Width of section: \( DZ = 100 \text{ mm} \)
Thickness in y direction: \( TY = 12 \text{ mm} \)
Thickness in z direction: \( TZ = 6 \text{ mm} \)
First moment of area about zz: \( S_{zz} = DZ \cdot TY \cdot (DY - TY) + (DY - 2 \cdot TY)^2 \cdot TZ / 4 = 272064 \text{ mm}^3 \)
First moment of area about yy: \( S_{yy} = DZ^2 \cdot TY / 2 + (DY - 2 \cdot TY) \cdot TZ^2 / 4 = 61584 \text{ mm}^3 \)
Plastic torque constant: \( S_{xx} = DZ \cdot TY^2 + DY \cdot TZ^2 / 2 = 18000 \text{ mm}^3 \)
Location: Reduced plastic moment due to shear & thrust

Effect of simultaneous shear force and axial thrust on $M_p$, see Example 3.4. in "Plastic Design of Low-Rise Frames" by MR Horne & LJ Morris, Constrado Monographs, Published: Collins, London.

- **Depth of section**: $D_Y = 257.2$ mm
- **Width of section**: $D_Z = 101.9$ mm
- **Thickness in y direction**: $T_Y = 8.4$ mm
- **Thickness in z direction**: $T_Z = 6$ mm
- **Axial load**: $f_x = 150$ kN
- **Shear force**: $f_y = 180$ kN
- **Design strength of steel**: $p_v = 240$ N/mm²

Let neutral axis position for combined bending and axial thrust be at a distance 'y' from centroidal axis $z_z$, then:

$$y = \frac{f_x \times 1E3}{(2 \times t' \times p_v)} = 84.608 \text{ mm}$$

Reduction in plastic modulus due to thrust $r_p t = 2 \times y^2 \times t' / 4 = 13220$ mm³

Hence reduced plastic modulus $z_{pr} = z_{pr} - r_p t = 248287$ mm³

Reduced plastic moment $M_{pr} = z_{pr} \times p_v / 1E6 = 59.589$ kNm
Interaction formulae for I section subjected to axial load, twisting moment and bending about the Y & Z axes, ref. Prof. Michael R Horne.

Depth of section \( D_Y = 0.921 \text{ m} \)

Width of section \( D_Z = 0.4205 \text{ m} \)

Flange thickness in y direction \( T_Y = 0.0366 \text{ m} \)

Web thickness in z direction \( T_Z = 0.0214 \text{ m} \)

Axial load \( F_x = 2648 \text{ kN} \)

Ultimate axial load \( F_{xp} = 11497 \text{ kN} \)

Twisting moment \( M_x = 0.274 \text{ kNm} \)

Plastic twisting moment \( M_{xp} = 105.04 \text{ kNm} \)

Bending moment about minor axis \( M_y = 745.37 \text{ kNm} \)

Ultimate BM about minor axis \( M_{yp} = 783.23 \text{ kNm} \)

Bending moment about major axis \( M_z = 223.71 \text{ kNm} \)

Ultimate BM about major axis \( M_{zp} = 4151.3 \text{ kNm} \)

For \( n \leq \alpha \) then major axis \( M_{zp} = t \times M_{zp} \times \left(1 - \frac{n^2}{\alpha \times (2-\alpha)}\right) = 3809 \text{ kNm} \)

For \( n \leq \alpha \) then minor axis \( M_{yp} = t \times M_{yp} = 783.23 \text{ kNm} \)

Then with \( M_{zp} \) and \( M_{yp} \) use \( U_{mrh} = (M_z/M_{zp})^2 + M_y/M_{yp} = 0.95511 \)
Location: Properties of typical channel section

Sectional properties of channel

Depth of section                  D=200 mm
Width of section                  B=200 mm
Thickness of flanges              T=10 mm
Thickness of web                  t=10 mm

Part area                         az=2*B*T=4000 mm²
Part area                         az'=(D-2*T)*t=1800 mm²
Area of section                   AX=az+az'=5800 mm²
Shear area in Y-direction         AY=D*t=2000 mm²
Shear area in Z-direction         AZ=az*5/6=3333.3 mm²
Neutral-axis depth                n=(az*B/2+az'*(t/2))/AX=70.517 mm
Inertia about YY-axis             IY=(D*n²+2*T*(B-n)^3-(n-t)^3*(D-2 *T))/3=24.552E6 mm⁴
Inertia about ZZ-axis             IZ=B*D³/12-(B-t)*(D-2*T)³/12 =40.993E6 mm⁴
Torsional constant for flange     ix'=B*T³*(1/3-0.21*T/B)=64567 mm⁴
Torsional constant for web        ix''=D*t³*(1/3-0.21*t/D)=64567 mm⁴
Torsional constant for section    IX=2*ix'+ix''=193700 mm⁴
Location: Properties of typical I-beam

Depth of section \( D = 200 \text{ mm} \)
Width of section \( B = 200 \text{ mm} \)
Thickness in Y-direction \( T = 10 \text{ mm} \)
Thickness in Z-direction \( t = 10 \text{ mm} \)

compute \( D_1 = D - 2T = 180 \text{ mm} \)
compute \( D_2 = B - 2t = 180 \text{ mm} \)
Area of section \( A_X = D B - D_1 D_2 = 7600 \text{ mm}^2 \)
Inertia about YY-axis \( I_Y = \frac{D B^3}{12} - \frac{D_1 D_2^3}{12} = 45.853 \times 10^6 \text{ mm}^4 \)
Inertia about ZZ-axis \( I_Z = \frac{B D^3}{12} - \frac{D_2 D_1^3}{12} = 45.853 \times 10^6 \text{ mm}^4 \)
Torsion constant:
\[
I_X = 2T t (D-T)^2 (B-t)^2 / (D+T+B+t-T^2-t^2) = 68.59 \times 10^6 \text{ mm}^4
\]
compute \( D_2 = D/2 = 100 \text{ mm} \)
compute \( D_1 = D_2 - T = 90 \text{ mm} \)
Shape factor:
\[
F = (1+3(D_2^2-D_1^2) * D_1 / (2*D_2^3)(0.5*B/t-1)) * (4*D_2^2*AX)/(10*I_Z) = 2.1935
\]
Shear area in Y-direction \( A_Y = AX/F = 3464.8 \text{ mm}^2 \)
compute \( D_2 = B/2 = 100 \text{ mm} \)
compute \( D_1 = D_2 - t = 90 \text{ mm} \)
Shape factor:
\[
F = (1+3(D_2^2-D_1^2) * D_1 / (2*D_2^3)(0.5*D/T-1)) * (4*D_2^2*AX)/(10*I_Y) = 2.1935
\]
Shear area in Z-direction \( A_Z = AX/F = 3464.8 \text{ mm}^2 \)
Location: Square solid section analysed using general method

Properties of a solid plane section

The section is defined by coordinates of corner points taken in anti-clockwise order round the section. The cross section is kept to the left of the edge running from a previous point to the next point. The section is closed when the original point is specified again.

Coordinates of points in order:
Point 1 : X-coordinate          x(1)=0 mm
        Y-coordinate          y(1)=0 mm
Point 2 : X-coordinate          x(2)=400 mm
        Y-coordinate          y(2)=0 mm
Point 3 : X-coordinate          x(3)=400 mm
        Y-coordinate          y(3)=400 mm
Point 4 : X-coordinate          x(4)=0 mm
        Y-coordinate          y(4)=400 mm
Point 5 : X-coordinate          x(5)=0 mm
        Y-coordinate          y(5)=0 mm

Cross-sectional area              160000 mm²
Second moments of area (inertias) Ixx=2.1333E9 mm⁴
                                           Iyy=2.1333E9 mm⁴
Product of inertia ³dA.xy         Ixy=-1.9073E-6 mm⁴
Distance of centroid from origin  X=200 mm
                                           Y=200 mm
X and Y are principal axes.

Angle of rotation of section      thetaD=45°

Second moments of area (inertia) about redefined axes:
IX'=(Ixx*(COS(thetaR))^2+Iyy*(SIN(thetaR))^2-Ixy*SIN(2*thetaR))
         =2.1333E9 mm⁴
IY'=(Iyy*(COS(thetaR))^2+Ixx*(SIN(thetaR))^2+Ixy*SIN(2*thetaR))
         =2.1333E9 mm⁴
IXy'=(Ixx-Iyy)/2*SIN(2*thetaR)+Ixy*COS(2*thetaR)=-0.11679E-21 mm⁴
Plot of section
Properties of a hollow plane section

The section is defined by coordinates of corner points taken in anti-clockwise order round the section. The cross section is kept to the left of the edge running from a previous point to the next point. The section is closed when the original point is specified again.

Coordinates of points in order:
Point 1 : X-coordinate          x(1)=0 mm
         Y-coordinate          y(1)=0 mm
Point 2 : X-coordinate          x(2)=400 mm
         Y-coordinate          y(2)=0 mm
Point 3 : X-coordinate          x(3)=400 mm
         Y-coordinate          y(3)=400 mm
Point 4 : X-coordinate          x(4)=0 mm
         Y-coordinate          y(4)=400 mm
Point 5 : X-coordinate          x(5)=0 mm
         Y-coordinate          y(5)=360 mm
Point 6 : X-coordinate          x(6)=360 mm
         Y-coordinate          y(6)=360 mm
Point 7 : X-coordinate          x(7)=360 mm
         Y-coordinate          y(7)=40 mm
Point 8 : X-coordinate          x(8)=40 mm
         Y-coordinate          y(8)=40 mm
Point 9 : X-coordinate          x(9)=40 mm
         Y-coordinate          y(9)=360 mm
Point 10: X-coordinate          x(10)=0 mm
         Y-coordinate          y(10)=360 mm
Point 11: X-coordinate          x(11)=0 mm
         Y-coordinate          y(11)=0 mm

Cross-sectional area              57600 mm²
Second moments of area (inertias) Ixx=1.2595E9 mm⁴
         Iyy=1.2595E9 mm⁴
Product of inertia  dA.xy         Ixy=-1.4305E-6 mm⁴
Distance of centroid from origin  X=200 mm
         Y=200 mm
X and Y are principal axes.
Angle of rotation of section        thetaD=45°
Second moments of area (inertia) about redefined axes:
         Ix'=(Ixx-2*Ixy/SIN(2*thetaR))*2+Iyy*SIN(2*thetaR)-Ixy*COS(2*thetaR) =1.2595E9 mm⁴
         Iy'=(Iyy-2*Ixy/SIN(2*thetaR))*2+Ixx*SIN(2*thetaR)-Ixy*COS(2*thetaR) =1.2595E9 mm⁴
         Ixy'=(Ixx-Iyy)/2*SIN(2*thetaR)+Ixy*COS(2*thetaR)=-87.594E-24 mm⁴
Plot of section
Properties of a hollow plane section

The section is defined by coordinates of corner points taken in anti-clockwise order round the section. The cross section is kept to the left of the edge running from a previous point to the next point. The section is closed when the original point is specified again.

Coordinates of points in order:
Point 1 : X-coordinate          x(1)=1 mm  
        Y-coordinate          y(1)=0 mm  
Point 2 : X-coordinate          x(2)=4 mm  
        Y-coordinate          y(2)=0 mm  
Point 3 : X-coordinate          x(3)=5 mm  
        Y-coordinate          y(3)=1 mm  
Point 4 : X-coordinate          x(4)=5 mm  
        Y-coordinate          y(4)=4 mm  
Point 5 : X-coordinate          x(5)=4 mm  
        Y-coordinate          y(5)=5 mm  
Point 6 : X-coordinate          x(6)=1 mm  
        Y-coordinate          y(6)=5 mm  
Point 7 : X-coordinate          x(7)=0 mm  
        Y-coordinate          y(7)=4 mm  
Point 8 : X-coordinate          x(8)=0 mm  
        Y-coordinate          y(8)=2 mm  
Point 9 : X-coordinate          x(9)=1 mm  
        Y-coordinate          y(9)=2 mm  
Point 10: X-coordinate          x(10)=1 mm  
        Y-coordinate          y(10)=3 mm  
Point 11: X-coordinate          x(11)=2 mm  
        Y-coordinate          y(11)=3 mm  
Point 12: X-coordinate          x(12)=2 mm  
        Y-coordinate          y(12)=4 mm  
Point 13: X-coordinate          x(13)=3 mm  
        Y-coordinate          y(13)=4 mm  
Point 14: X-coordinate          x(14)=3 mm  
        Y-coordinate          y(14)=3 mm  
Point 15: X-coordinate          x(15)=4 mm  
        Y-coordinate          y(15)=3 mm  
Point 16: X-coordinate          x(16)=4 mm  
        Y-coordinate          y(16)=2 mm  
Point 17: X-coordinate          x(17)=3 mm  
        Y-coordinate          y(17)=2 mm  
Point 18: X-coordinate          x(18)=3 mm  
        Y-coordinate          y(18)=1 mm  
Point 19: X-coordinate          x(19)=2 mm  
        Y-coordinate          y(19)=1 mm  
Point 20: X-coordinate          x(20)=2 mm  
        Y-coordinate          y(20)=2 mm  
Point 21: X-coordinate          x(21)=0 mm  
        Y-coordinate          y(21)=2 mm  
Point 22: X-coordinate          x(22)=0 mm  
        Y-coordinate          y(22)=1 mm
Point 23 : X-coordinate  
  x(23)=1 mm  
  Y-coordinate  
  y(23)=0 mm  

Cross-sectional area  
  18 mm²  

Second moments of area (inertias)  
  Ixx=40.167 mm⁴  
  Iyy=40.167 mm⁴  

Product of inertia  
  Ixy=0 mm⁴  

Distance of centroid from origin  
  X=2.5 mm  
  Y=2.5 mm  

X and Y are principal axes.  

Angle of rotation of section  
  thetaD=45°  

Second moments of area (inertia) about redefined axes:  
  Ix'=(Ixx*(COS(thetaR))^2)+Iyy*(SIN(thetaR))^2-Ixy*SIN(2*thetaR)  
  =40.167 mm⁴  
  Iy'=(Iyy*(COS(thetaR))^2)+Ixx*(SIN(thetaR))^2+Ixy*SIN(2*thetaR)  
  =40.167 mm⁴  
  Ixy'=(Ixx-Iyy)/2*SIN(2*thetaR)+Ixy*COS(2*thetaR)=7.1054E-15 mm⁴
Plot of section
Location: Properties of typical channel section

**Sectional properties of channel**

Depth of section \( h = 200 \text{ mm} \)
Width of section \( b = 200 \text{ mm} \)
Thickness of flanges \( t_f = 10 \text{ mm} \)
Thickness of web \( t_w = 10 \text{ mm} \)

Part area \( a_z = 2 * b * t_f = 4000 \text{ mm}^2 \)
Part area \( a_z' = (h - 2 * t_f) * t_w = 1800 \text{ mm}^2 \)
Area of section \( A_x = a_z + a_z' = 5800 \text{ mm}^2 \)
Shear area in z-direction \( A_z = h * t_w = 2000 \text{ mm}^2 \)
Shear area in y-direction \( A_y = a_z * 5/6 = 3333.3 \text{ mm}^2 \)
Neutral-axis depth \( n = (a_z * b / 2 + a_z' * t_w / 2) / A_x = 70.517 \text{ mm} \)
Inertia about zz-axis \( I_z = (h * n^3 + 2 * t_f * (b - n) * 3 - (n - t_w) * 3 * (h - 2 * t_f)) / 3 = 24.552E6 \text{ mm}^4 \)
Inertia about yy-axis \( I_y = b * h^3 / 12 - (b - t_w) * (h - 2 * t_f) * 3 / 12 = 40.993E6 \text{ mm}^4 \)
Torsional constant for flange \( I_{t1} = b * t_f^3 / (1/3 - 0.21 * t_f / b) = 64567 \text{ mm}^4 \)
Torsional constant for web \( I_{t2} = h * t_w^3 / (1/3 - 0.21 * t_w / h) = 64567 \text{ mm}^4 \)
Torsional constant for section \( I_t = 2 * I_{t1} + I_{t2} = 193700 \text{ mm}^4 \)
Location: Properties of typical I-beam

Depth of section \( h = 200 \text{ mm} \)
Width of section \( b = 200 \text{ mm} \)
Thickness in z-direction \( t_f = 10 \text{ mm} \)
Thickness in y-direction \( t_w = 10 \text{ mm} \)

```
compute \( D_1 = h - 2 \cdot t_f = 180 \text{ mm} \)
compute \( D_2 = b - 2 \cdot t_w = 180 \text{ mm} \)
Area of section \( A_x = h \cdot b \cdot 3/12 - D_1 \cdot D_2 \cdot 3/12 = 45.853E6 \text{ mm}^4 \)
Inertia about zz-axis \( I_z = h \cdot b \cdot 3/12 - D_1 \cdot D_2 \cdot 3/12 = 45.853E6 \text{ mm}^4 \)
Inertia about yy-axis \( I_y = b \cdot h \cdot 3/12 - D_2 \cdot D_1 \cdot 3/12 = 45.853E6 \text{ mm}^4 \)
Torsion constant:
\[
I_t = 2 \cdot t_f \cdot t_w \cdot (h - t_f)^2 \cdot (b - t_w)^2 / (h \cdot t_f + b \cdot t_w - t_f^2 - t_w^2)
\]
\[
= 68.59E6 \text{ mm}^4
\]
compute \( D_2 = h/2 = 100 \text{ mm} \)
compute \( D_1 = D_2 - t_f = 90 \text{ mm} \)
Shape factor:
\[
F = (1 + 3 \cdot (D_2^2 - D_1^2) \cdot D_1 / (2 \cdot D_2^3) \cdot (0.5 \cdot b / t_w - 1)) \cdot (4 \cdot D_2^2 \cdot A_x) / (10 \cdot I_y) = 2.1935
\]
Shear area in z-direction \( A_z = A_x / F = 3464.8 \text{ mm}^2 \)
compute \( D_2 = h/2 = 100 \text{ mm} \)
compute \( D_1 = D_2 - t_w = 90 \text{ mm} \)
Shape factor:
\[
F = (1 + 3 \cdot (D_2^2 - D_1^2) \cdot D_1 / (2 \cdot D_2^3) \cdot (0.5 \cdot h / t_f - 1)) \cdot (4 \cdot D_2^2 \cdot A_x) / (10 \cdot I_z) = 2.1935
\]
Shear area in y-direction \( A_y = A_x / F = 3464.8 \text{ mm}^2 \)
Properties of any plane section

The section is defined by coordinates of corner points taken in an anticlockwise order round the section. The cross section is kept to the left of the edge running from a previous point to the next point. The section is closed when the original point is specified again.

Coordinates of points in order:
Point 1 : X-coordinate $x(1)=0$ mm
         Y-coordinate $y(1)=0$ mm
Point 2 : X-coordinate $x(2)=400$ mm
         Y-coordinate $y(2)=0$ mm
Point 3 : X-coordinate $x(3)=400$ mm
         Y-coordinate $y(3)=400$ mm
Point 4 : X-coordinate $x(4)=0$ mm
         Y-coordinate $y(4)=400$ mm
Point 5 : X-coordinate $x(5)=0$ mm
         Y-coordinate $y(5)=0$ mm

Cross-sectional area $160000$ mm²

Second moments of area (inertias) $I_{xx}=2.1333E9$ mm⁴
          $I_{yy}=2.1333E9$ mm⁴

Product of inertia $dA.xy$ $I_{xy}=-1.9073E-6$ mm⁴

Distance of centroid from origin $X=200$ mm
                               $Y=200$ mm

X and Y are principal axes.

Angle of rotation of section $\theta_D=45^\circ$

Second moments of area (inertia) about redefined axes:
$I_{x'}=I_{xx}*(\text{COS}(\theta_R))^2+I_{yy}*(\text{SIN}(\theta_R))^2-I_{xy}*(\text{SIN}(2*\theta_R))$
$=2.1333E9$ mm⁴
$I_{y'}=I_{yy}*(\text{COS}(\theta_R))^2+I_{xx}*(\text{SIN}(\theta_R))^2+I_{xy}*(\text{SIN}(2*\theta_R))$
$=2.1333E9$ mm⁴
$I_{xy'}=(I_{xx}-I_{yy})/2*\text{SIN}(2*\theta_R)+I_{xy}*(\text{COS}(2*\theta_R))=-0.11679E-21$ mm⁴
Plot of section
Location: Square box-section analysed using general method

Properties of any plane-section

The section is defined by coordinates of corner points taken in anticlockwise order round the section. The cross section is kept to the left of the edge running from a previous point to the next point. The section is closed when the original point is specified again.

Coordinates of points in order:

Point 1 : X-coordinate $x(1)=0$ mm  
          Y-coordinate $y(1)=0$ mm

Point 2 : X-coordinate $x(2)=400$ mm  
          Y-coordinate $y(2)=0$ mm

Point 3 : X-coordinate $x(3)=400$ mm  
          Y-coordinate $y(3)=400$ mm

Point 4 : X-coordinate $x(4)=0$ mm  
          Y-coordinate $y(4)=400$ mm

Point 5 : X-coordinate $x(5)=0$ mm  
          Y-coordinate $y(5)=360$ mm

Point 6 : X-coordinate $x(6)=360$ mm  
          Y-coordinate $y(6)=360$ mm

Point 7 : X-coordinate $x(7)=360$ mm  
          Y-coordinate $y(7)=40$ mm

Point 8 : X-coordinate $x(8)=40$ mm  
          Y-coordinate $y(8)=40$ mm

Point 9 : X-coordinate $x(9)=40$ mm  
          Y-coordinate $y(9)=360$ mm

Point 10 : X-coordinate $x(10)=0$ mm  
           Y-coordinate $y(10)=360$ mm

Point 11 : X-coordinate $x(11)=0$ mm  
           Y-coordinate $y(11)=0$ mm

Cross-sectional area 57600 mm²

Second moments of area (inertias) $I_{xx}=1.2595E9$ mm⁴  
$I_{yy}=1.2595E9$ mm⁴

Product of inertia $\int dA \cdot xy$ $I_{xy}=-1.4305E-6$ mm⁴

Distance of centroid from origin $X=200$ mm  
$Y=200$ mm

X and Y are principal axes.

Angle of rotation of section $\theta_D=45^\circ$

Second moments of area (inertia) about redefined axes:

$I'_{x}=I_{xx}*(\text{COS}(\theta_D))^2+I_{yy}*(\text{SIN}(\theta_D))^2-2*I_{xy}*(\text{SIN}(2*\theta_D))$  
$=1.2595E9$ mm⁴

$I'_{y}=I_{yy}*(\text{COS}(\theta_D))^2+I_{xx}*(\text{SIN}(\theta_D))^2+2*I_{xy}*(\text{SIN}(2*\theta_D))$  
$=1.2595E9$ mm⁴

$I'_{xy}=\frac{I_{xx}-I_{yy}}{2}*(\text{SIN}(2*\theta_D))+I_{xy}*(\text{COS}(2*\theta_D))=-87.594E-24$ mm⁴
Plot of section
Location: Circular voided edge member

Properties of a plane section

Number of items in section \( n_{item} = 2 \)

**Item Number 1**

Item is defined by co-ordinates of corner points taken in anti-clockwise order around the item. The item is closed when the co-ordinates of the first point input are re-specified.

Item weighting \( i_{w}(1) = 1 \)

Coordinates of points in order:

\[
\begin{align*}
&x(1) = 500 \\
&y(1) = 1000 \\
&x(2) = 1500 \\
&y(2) = 1000 \\
&x(3) = 1500 \\
&y(3) = 1500 \\
&x(4) = 1750 \\
&y(4) = 1500 \\
&x(5) = 1750 \\
&y(5) = 2000 \\
&x(6) = 500 \\
&y(6) = 2000 \\
&x(7) = 500 \\
&y(7) = 1000
\end{align*}
\]

Location of centroid

\[
\begin{align*}
X_c &= 1069.4 \\
Y_c &= 1527.8
\end{align*}
\]

Properties of item are weighted

Area of item \( A_W = 1.125E6 \)

First moment of area about X axis \( M_X = 1.7188E9 \)

First moment of area about Y axis \( M_Y = 1.2031E9 \)

Second moment of area about X axis:

\( I_{XX} = 2.7188E12 \)

Second moment of area about Y axis:

\( I_{YY} = 1.4141E12 \)

Product second moment of area about XY axes:

\( I_{XY} = 1.8555E12 \)
Plot of Item Number 1
Item Number 2

Circle diameter \( cd(2)=600 \)
Location of centroid \( Xc(2)=1000 \)
\( Yc(2)=1500 \)
Weighting of item \( iw(2)=-1 \)
Second moment of area about X axis:
\[
IXX=(\pi*cd(2)^4/64)*iw(2)+AW *Xc(2)^2=-642.53E9
\]
Second moment of area about Y axis:
\[
IYY=(\pi*cd(2)^4/64)*iw(2)+AW *Yc(2)^2=-289.11E9
\]
Product second moment of area about XY axes
\[
IXY=AW*Xc(2)*Yc(2)=-424.12E9
\]

Weighted sectional properties

x,y - centroidal axes
X,Y - global axes
u,v - principal axes

Cross-sectional area 842257
Second moments of area (inertias)  
\( Ixx=86.125E9 \)
\( Iyy=119.3E9 \)
\( Ixy=16.648E9 \)

Distance of centroid from origin  
\( X=1092.7 \)
\( Y=1537.2 \)

Principal second moments of area  
\( Iu=79.211E9 \)
\( Iv=126.21E9 \)

Angle of principal u axis counter-clockwise from global X axis 22.553 Degrees
**Location:** Edge beam

**Section properties of a plate girder with welded connections**

- Number of bottom flange plates: nobfp=1
- Number of top flange plates: notfp=1
- Bottom flange plate:
  - Breadth: bbf(1)=435 mm
  - Depth: dbf(1)=24 mm
- Web plate:
  - Breadth: bw=12 mm
  - Depth: dw=475 mm
- Top flange plate:
  - Breadth: btf(1)=435 mm
  - Depth: dtf(1)=24 mm

**SECTION SUMMARY - gross values**

**Gross section elastic properties**

- Depth of girder: dob=Db(n)=523 mm
- Area: 26580 mm²
- 1st moment of area about soffit: 6.9507E6 mm⁴
- Centroid of section from soffit: CGga=Mga/Ga=261.5 mm
- Second moments of area about centroidal axes:
  - About x-x axis: 1.408E9 mm⁶
  - About y-y axis: 329.32E6 mm⁶
- Elastic section moduli:
  - Top: Ztg=Igxx/(dob-CGga)=5.3842E6 mm⁴
  - Bottom: Zbg=Igxx/CGga=5.3842E6 mm³

**Gross section plastic properties**

- Half gross area: hga=Ga/2=13290 mm²
- Equal area line is in web
- Distance from beam soffit: eal=Db(m)-(ca-hga)/b(m)=261.5 mm
- Plastic modulus: 5.8864E6 mm³

**Bottom flange plate**

- Total number of holes: nh(1)=4
- Diameter of hole: di(1)=22 mm
- Hole centre-line from y-y axis: dis(1)=56 mm

**Top flange plate**

- Total number of holes: nh(2)=0

**Web**

- Total number of holes: nh(3)=0
SECTION SUMMARY - net values

Net section elastic properties

Area 24468 mm²
1st moment of area about soffit 6.9253E6 mm³
Centroid of girder from soffit CGna=Mna/Na=283.04 mm
Second moments of area about centroidal axes
About x-x axis 1.265E9 mm⁴
About y-y axis 298.21E6 mm⁴
Elastic section moduli
Top Ztn=Inxx/(dob-CGna)=5.2718E6 mm³
Bottom Zbn=Inxx/CGna=4.4695E6 mm³

Net section plastic properties

Distance to equal area line from beam soffit
Distance eal=Dbn(p)-(ca-hna)/bn(p)=349.5 mm
Plastic modulus 5.2666E6 mm³
Location: Edge girder

Section properties of a plate girder with angle connections

| Number of bottom flange plates | nobfp=1 |
| Number of top flange plates    | notfp=1 |
| Bottom flange plate            |         |
| Breadth                        | bbf(1)=435 mm |
| Depth                          | dbf(1)=24 mm |
| Web plate                      |         |
| Breadth                        | bw=12 mm |
| Depth                          | dw=475 mm |
| Top flange plate               |         |
| Breadth                        | btf(1)=435 mm |
| Depth                          | dtf(1)=24 mm |

SECTION SUMMARY - gross values

Gross section elastic properties

| Depth of girder                | dob=Db(n)=523 mm |
| Area                           | 35604 mm²        |
| 1st moment of area about soffit| 9.3104E6 mm⁴     |
| Centroid of girder from soffit | CGga=Mga/Ga=261.5 mm |
| Second moments of area about centroidal axes | |
| About x-x axis                 | 1.8071E9 mm⁴     |
| About y-y axis                 | 349.03E6 mm⁴     |
| Elastic section moduli         |                |
| Top                            | Ztg=Igxx/(dob-CGga)=6.9106E6 mm³ |
| Bottom                         | Zbg=Igxx/CGga=6.9106E6 mm³ |

Gross section plastic properties

| Half gross area                | hga=Ga/2=17802 mm² |
| Equal area line is in web between web legs of angles | |
| Distance to girder soffit      | eal=Db(m)-(ca-hga)/b(m)=261.5 mm |
| Plastic modulus                | 7.7643E6 mm³      |

Plastic modulus of bottom flange plates and angles.

| Equal area line is in bottom flange | |
| Distance to girder soffit           | eal=Db(m)-(ca-htabg)/bm(m)=17.186 mm |
| Plastic modulus                     | 237756 mm³       |

Plastic modulus of top flange plates and angles.

| Equal area line is in top flange    | |
| Distance to girder soffit           | eal=Db(m)-(ca-htatg)/bm(m)=505.81 mm |
| Plastic modulus                     | 237756 mm³       |

Allowance for holes through section

<p>| Bottom flange plate                | |
| Total number of holes              | nh(1)=4            |</p>
<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of hole</td>
<td>(d_i(1)=22) mm</td>
</tr>
<tr>
<td>Hole centre-line from y-y axis</td>
<td>(d_{is}(1)=56) mm</td>
</tr>
<tr>
<td>Diameter of hole</td>
<td>(d_i(2)=22) mm</td>
</tr>
<tr>
<td>Hole centre-line from y-y axis</td>
<td>(d_{is}(2)=162) mm</td>
</tr>
<tr>
<td>Bottom angle flange leg</td>
<td></td>
</tr>
<tr>
<td>Total number of holes</td>
<td>(n_{h}(2)=2)</td>
</tr>
<tr>
<td>Diameter of hole</td>
<td>(d_i(51)=22) mm</td>
</tr>
<tr>
<td>Hole centre-line from y-y axis</td>
<td>(d_{is}(51)=56) mm</td>
</tr>
<tr>
<td>Bottom angle web leg and included web</td>
<td></td>
</tr>
<tr>
<td>Total number of holes</td>
<td>(n_{h}(3)=1)</td>
</tr>
<tr>
<td>Diameter of hole</td>
<td>(d_i(101)=22) mm</td>
</tr>
<tr>
<td>Hole centre from soffit of girder</td>
<td>(d_{is}(101)=74) mm</td>
</tr>
<tr>
<td>Web not included with angles</td>
<td></td>
</tr>
<tr>
<td>Total number of holes</td>
<td>(n_{h}(4)=0)</td>
</tr>
<tr>
<td>Top angle web leg and included web</td>
<td></td>
</tr>
<tr>
<td>Total number of holes</td>
<td>(n_{h}(5)=0)</td>
</tr>
<tr>
<td>Top angle flange leg</td>
<td></td>
</tr>
<tr>
<td>Total number of holes</td>
<td>(n_{h}(6)=0)</td>
</tr>
<tr>
<td>Top flange plate</td>
<td></td>
</tr>
<tr>
<td>Total number of holes</td>
<td>(n_{h}(7)=0)</td>
</tr>
</tbody>
</table>
SECTION SUMMARY - net values

Net section elastic properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area</td>
<td>32172 mm²</td>
</tr>
<tr>
<td>1st moment of area about soffit</td>
<td>9.2107E6 mm³</td>
</tr>
<tr>
<td>Centroid of girder from soffit</td>
<td>CGna=Mna/Na=286.29 mm</td>
</tr>
<tr>
<td>Second moments of area about centroidal axes</td>
<td></td>
</tr>
<tr>
<td>About x-x axis</td>
<td>1.5996E9 mm⁴</td>
</tr>
<tr>
<td>About y-y axis</td>
<td>316.16E6 mm⁴</td>
</tr>
<tr>
<td>Elastic section moduli</td>
<td></td>
</tr>
<tr>
<td>Top</td>
<td>Ztn=Inxx/(dob-CGna)=6.7578E6 mm³</td>
</tr>
<tr>
<td>Bottom</td>
<td>Zbn=Inxx/CGna=5.5873E6 mm³</td>
</tr>
</tbody>
</table>

Net section plastic properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Half net area</td>
<td>hna=Na/2=16086 mm²</td>
</tr>
<tr>
<td>Distance to equal area line from girder soffit</td>
<td></td>
</tr>
<tr>
<td>Distance</td>
<td>eal=DBn(p)-(ca-hna)/bn(p)=400.83 mm</td>
</tr>
<tr>
<td>Plastic modulus</td>
<td>6.7215E6 mm³</td>
</tr>
<tr>
<td>Plastic modulus of bottom flange plates and angles.</td>
<td></td>
</tr>
<tr>
<td>Distance</td>
<td>eal=DBn(p)-(ca-htabn)/bz(p)=16.98 mm</td>
</tr>
<tr>
<td>Plastic modulus bottom flange and angles 185939 mm³</td>
<td></td>
</tr>
<tr>
<td>Plastic modulus of top flange plates and angles.</td>
<td></td>
</tr>
<tr>
<td>Distance</td>
<td>eal=DBn(p)-(ca-htatn)/bz(p)=505.81 mm</td>
</tr>
<tr>
<td>Plastic modulus top flange and angles 237756 mm³</td>
<td></td>
</tr>
</tbody>
</table>
Location: Zone A

Influence surfaces of elastic plates - Pucher Chart 17a

Bending moment at the restrained edge of a cantilever

Cantilever span length \( \text{span}=3.5 \)

Number of applied loads \( n0=4 \)

Total patch load \( pl(1)=10.5 \)

Load position \( x' \) coordinate \( x'(1)=0 \)

\( y' \) coordinate \( y'(1)=0 \)

Load spread dimension \( dx' \) \( dx'(1)=0.74 \)

\( dy' \) coordinate \( dy'(1)=1 \)

Point load \( pl(2)=10.5 \)

Load position \( x' \) coordinate \( x'(2)=0 \)

\( y' \) coordinate \( y'(2)=1.6 \)

Point load \( pl(3)=7 \)

Load position \( x' \) coordinate \( x'(3)=3 \)

\( y' \) coordinate \( y'(3)=0 \)

Point load \( pl(4)=7 \)

Load position \( x' \) coordinate \( x'(4)=3 \)

\( y' \) coordinate \( y'(4)=1.6 \)

\[y\]

\[x-y \text{ global axes}\]

\[
\begin{array}{c}
\text{Restrained edge} \\
\begin{array}{c}
\text{Hog} \\
\text{Range of analysis } x = \pm 9.1 \ (2.6 \text{ times effective span }) \\
\text{Loads within } x = \pm 9.1 \ (2.6 \text{ times span }) \text{ result in hog at A.} \\
\text{Loads in the vicinity of A should be applied as patch loads.} \\
\text{Moment per unit length and associated position of load group.} \\
\text{Hogging moment - negative.} \\
\end{array}
\end{array}
\]

\[
\begin{array}{c}
\text{Moment } m_y \\
\text{Position of Load Group} \\
x \quad \text{y} \\
-10.021 \quad 0 \quad 0.5
\end{array}
\]
Location: Zone A

Influence surfaces of elastic plates - Pucher Charts 18A and 19A

Bending moments at the centre of a cantilever

| Total patch load | $p_l(1)=10.5$ |
| Load position x' coordinate | $x'(1)=0$ |
| y' coordinate | $y'(1)=0$ |
| Load spread dimension dx' | $dx'(1)=0.74$ |
| dimension dy' | $dy'(1)=1$ |
| Point load | $p_l(2)=10.5$ |
| Load position x' coordinate | $x'(2)=0$ |
| y' coordinate | $y'(2)=1.6$ |
| Point load | $p_l(3)=7$ |
| Load position x' coordinate | $x'(3)=3$ |
| y' coordinate | $y'(3)=0$ |
| Point load | $p_l(4)=7$ |
| Load position x' coordinate | $x'(4)=3$ |
| y' coordinate | $y'(4)=1.6$ |

Range of analysis $x = \pm 9.1$ (2.6 times effective span).
Loads within $x = \pm 2.1$ and $x = \pm 1.4$ result in sag at A.
Loads outside $x = \pm 2.1$ and $x = \pm 1.4$ result in hog at A.
Loads in the immediate vicinity of A should be applied as patch loads.
Range of analysis  \( x = \pm 9.1 \) ( 2.6 times effective span ).
Loads with \( y > 1.4 \) and within \( x = \pm 2.8 \) and \( x = \pm 0.7 \)
result in sag at A.
Loads outside this region result in hog at A.
Loads in the vicinity of A should be applied as patch loads.

Moments per unit length and associated positions of load group.
Sagging moment - positive  Hogging moment - negative.

<table>
<thead>
<tr>
<th>Moment</th>
<th>Moment</th>
<th>Position of Load Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>( mx )</td>
<td>( my )</td>
<td>( x )  ( y )</td>
</tr>
<tr>
<td>1.3355</td>
<td>0.73378</td>
<td>0  1.75</td>
</tr>
</tbody>
</table>
Location: Zone A

Influence surfaces of elastic plates - Pucher Chart 20A

Bending moment at the free edge of a cantilever

<table>
<thead>
<tr>
<th>Cantilever span length</th>
<th>span=3.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of applied loads</td>
<td>nol=4</td>
</tr>
<tr>
<td>Total patch load</td>
<td>pl(1)=10.5</td>
</tr>
<tr>
<td>Load position x' coordinate</td>
<td>x'(1)=0</td>
</tr>
<tr>
<td></td>
<td>y'(1)=0</td>
</tr>
<tr>
<td>Load spread</td>
<td>dx'(1)=0.74</td>
</tr>
<tr>
<td></td>
<td>dy'(1)=1</td>
</tr>
<tr>
<td>Point load</td>
<td>pl(2)=10.5</td>
</tr>
<tr>
<td>Load position x' coordinate</td>
<td>x'(2)=0</td>
</tr>
<tr>
<td></td>
<td>y'(2)=1.6</td>
</tr>
<tr>
<td>Point load</td>
<td>pl(3)=7</td>
</tr>
<tr>
<td>Load position x' coordinate</td>
<td>x'(3)=3</td>
</tr>
<tr>
<td></td>
<td>y'(3)=0</td>
</tr>
<tr>
<td>Point load</td>
<td>pl(4)=7</td>
</tr>
<tr>
<td>Load position x' coordinate</td>
<td>x'(4)=3</td>
</tr>
<tr>
<td></td>
<td>y'(4)=1.6</td>
</tr>
</tbody>
</table>

Range of analysis  x = ± 9.1  ( 2.6 times effective span ).
Loads within x = ± 2.1  ( 0.6 times span ) result in sag at A.
Loads outside x = ± 2.1  ( 0.6 times span ) result in hog at A.
Loads in the immediate vicinity of A should be applied as patch loads.

Moment per unit length and associated positions of load group.
Sagging moment - positive  Hogging moment - negative.

<table>
<thead>
<tr>
<th>mx</th>
<th>Position of Load Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5643</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>0</td>
</tr>
</tbody>
</table>

Moment Mx at A about y axis

x - y global axes

Restrained edge
Location: Zone A

Influence surfaces of elastic plates - Pucher Charts 1 and 2

Bending moment at the centre of a plate strip with

two supported edges

Span of plate \( \text{span} = 3.5 \)

Number of applied loads \( \text{nol} = 4 \)

Total patch load \( \text{pl}(1) = 10.5 \)

Load position \( x' \) coordinate \( x'(1) = 0 \)
\( y' \) coordinate \( y'(1) = 0 \)

Load spread dimension \( dx' \) \( dx'(1) = 0.74 \)
\( dy' \) coordinate \( dy'(1) = 1 \)

Point load \( \text{pl}(2) = 10.5 \)

Load position \( x' \) coordinate \( x'(2) = 0 \)
\( y' \) coordinate \( y'(2) = 1.6 \)

Point load \( \text{pl}(3) = 7 \)

Load position \( x' \) coordinate \( x'(3) = 3 \)
\( y' \) coordinate \( y'(3) = 0 \)

Point load \( \text{pl}(4) = 7 \)

Load position \( x' \) coordinate \( x'(4) = 3 \)
\( y' \) coordinate \( y'(4) = 1.6 \)

\[
\begin{array}{l}
\text{Sag} \\
\hline
\text{Hog} \\
\hline
\text{A} \\
\hline
\text{Hog} \\
\end{array}
\]

Moment \( M_x \) at A

\[
\begin{array}{l}
1.05 \\
\hline
1.05 \\
\hline
4.2 \\
\hline
4.2 \\
\end{array}
\]

Range of analysis \( x = \pm 4.2 \) (1.2 times effective span).

Loads within \( x = \pm 1.05 \) (0.3 times span) result in sag at A.

Loads outside \( x = \pm 1.05 \) (0.3 times span) result in hog at A.

Loads in the vicinity of A should be applied as patch loads.
Range of analysis $x = \pm 4.2$ (1.2 times effective span).
Loads within $x = \pm 4.2$ (1.2 times span) result in sag at A.
Loads in the vicinity of A should be applied as patch loads.

Moments per unit length and associated positions of load group.

<table>
<thead>
<tr>
<th>Moment</th>
<th>Moment</th>
<th>Position of Load Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m_x$</td>
<td>$m_y$</td>
<td>$x$</td>
</tr>
<tr>
<td>1.2372</td>
<td>2.4674</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.75</td>
</tr>
</tbody>
</table>
Location: Zone A

Influence surfaces of elastic plates - Pucher Charts 3 and 4

Bending moment at quarter point of a plate strip with two supported edges

Span of plate: \( \text{span} = 3.5 \)

Number of applied loads: \( nol = 4 \)

Total patch load: \( pl(1) = 10.5 \)

Load position:
- \( x' \) coordinate: \( x'(1) = 0 \)
- \( y' \) coordinate: \( y'(1) = 0 \)

Load spread:
- \( dx' \) dimension: \( dx'(1) = 0.74 \)
- \( dy' \) dimension: \( dy'(1) = 1 \)

Point load: \( pl(2) = 10.5 \)

Load position:
- \( x' \) coordinate: \( x'(2) = 0 \)
- \( y' \) coordinate: \( y'(2) = 1.6 \)

Point load: \( pl(3) = 7 \)

Load position:
- \( x' \) coordinate: \( x'(3) = 3 \)
- \( y' \) coordinate: \( y'(3) = 0 \)

Point load: \( pl(4) = 7 \)

Load position:
- \( x' \) coordinate: \( x'(4) = 3 \)
- \( y' \) coordinate: \( y'(4) = 1.6 \)

Range of analysis: \( x = \pm 4.2 \) (1.2 times effective span).

Loads within \( x = \pm 1.4 \) (0.4 times span) and within \( x = \pm 0.7 \) (0.2 times span) result in sag at A.

Loads outside these limits result in hog at A.

Loads in the vicinity of A should be applied as patch loads.
Range of analysis \( x = \pm 4.2 \) (1.2 times effective span).

Loads within \( x = \pm 4.2 \) (1.2 times span) result in sag at A. Loads in the vicinity of A should be applied as patch loads.

Moments per unit length and associated positions of load group.

- Sagging moment - positive
- Hogging moment - negative

<table>
<thead>
<tr>
<th>Moment ( mx )</th>
<th>Moment ( my )</th>
<th>Position of Load Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2581</td>
<td>2.3184</td>
<td>0</td>
</tr>
</tbody>
</table>
Location: Zone A

Influence surfaces of elastic plates - Pucher Charts 12 and 13

Bending moment at the centre of a plate strip with two restrained edges

Span of plate \( \text{span}=3.5 \)

Number of applied loads \( nol=4 \)

Total patch load \( pl(1)=10.5 \)

Load position \( x' \) coordinate \( x'(1)=0 \)

\( y' \) coordinate \( y'(1)=0 \)

Load spread dimension \( dx' \) \( dx'(1)=0.74 \)

dimension \( dy' \) \( dy'(1)=1 \)

Point load \( pl(2)=10.5 \)

Load position \( x' \) coordinate \( x'(2)=0 \)

\( y' \) coordinate \( y'(2)=1.6 \)

Point load \( pl(3)=7 \)

Load position \( x' \) coordinate \( x'(3)=3 \)

\( y' \) coordinate \( y'(3)=0 \)

Point load \( pl(4)=7 \)

Load position \( x' \) coordinate \( x'(4)=3 \)

\( y' \) coordinate \( y'(4)=1.6 \)

\( y \)

\( x - y \) global axes

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\\[\\]
Range of analysis \( x = \pm 4.2 \) (1.2 times effective span).

Loads within \( x = \pm 4.2 \) (1.2 times span) result in sag at A.

Loads in the vicinity of A should be applied as patch loads.

Moments per unit length and associated positions of load group.
Sagging moment - positive  Hogging moment - negative.

<table>
<thead>
<tr>
<th>Moment ( mx )</th>
<th>Moment ( my )</th>
<th>Position of Load Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.96017</td>
<td>1.3949</td>
<td>0</td>
</tr>
</tbody>
</table>
Location: Zone A

Influence surfaces of elastic plates - Pucher Charts 14 and 15

Bending moment at quarter point of a plate strip with two restrained edges

Span of plate  \(\text{span}=3.5\)

Number of applied loads  \(nol=4\)

Total patch load  \(pl(1)=10.5\)

Load position  \(x'\) coordinate  \(x'(1)=0\)

\(y'\) coordinate  \(y'(1)=0\)

Load spread  dimension \(dx'\)  \(dx'(1)=0.74\)

\(\text{dimension } dy'\)  \(dy'(1)=1\)

Point load  \(pl(2)=10.5\)

Load position  \(x'\) coordinate  \(x'(2)=0\)

\(y'\) coordinate  \(y'(2)=1.6\)

Point load  \(pl(3)=7\)

Load position  \(x'\) coordinate  \(x'(3)=3\)

\(y'\) coordinate  \(y'(3)=0\)

Point load  \(pl(4)=7\)

Load position  \(x'\) coordinate  \(x'(4)=3\)

\(y'\) coordinate  \(y'(4)=1.6\)

---

Range of analysis  \(x = \pm 4.2 \)  (1.2 times effective span).

Loads within \(x = \pm 1.225\)  (0.35 times span) and within \(x = \pm 0.525\)  (0.15 times span) result in sag at A.

Loads outside these limits result in hog at A.

Loads in the vicinity of A should be applied as patch loads.
Range of analysis $x = \pm 4.2$ (1.2 times effective span).
Loads with $y < 1.575$ (0.45 times span) and with
$y < 2.1$ (0.60 times span) result in sag at A.
Loads outside these limits result in hog at A.
Loads in the vicinity of A should be applied as patch loads.

<table>
<thead>
<tr>
<th>Moment $m_x$</th>
<th>Moment $m_y$</th>
<th>Position of Load Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.79537</td>
<td>0.67223</td>
<td>x</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0</td>
</tr>
</tbody>
</table>
Location: Zone A

Influence surfaces of elastic plates - Pucher Chart 16

Bending moment for the edge of a plate strip with two restrained edges

Span of plate \( \text{span}=3.5 \)

Number of applied loads \( \text{nol}=4 \)

Total patch load \( \text{pl}(1)=10.5 \)

Load position \( x' \) coordinate \( x'(1)=0 \)
\( y' \) coordinate \( y'(1)=0 \)

Load spread dimension \( dx' \) \( dx'(1)=0.74 \)
\( dy' \) coordinate \( dy'(1)=1 \)

Point load \( \text{pl}(2)=10.5 \)

Load position \( x' \) coordinate \( x'(2)=0 \)
\( y' \) coordinate \( y'(2)=1.6 \)

Point load \( \text{pl}(3)=7 \)

Load position \( x' \) coordinate \( x'(3)=3 \)
\( y' \) coordinate \( y'(3)=0 \)

Point load \( \text{pl}(4)=7 \)

Load position \( x' \) coordinate \( x'(4)=3 \)
\( y' \) coordinate \( y'(4)=1.6 \)

\[ y \quad \text{Restrained edge} \]

\[ \frac{\text{Hog}}{\text{3.5}} \]

\[ x - y \text{ global axes} \]

Moment \( M_y \) at A about x axis

Range of analysis \( x = \pm 4.2 \) (1.2 times effective span).

Loads in the vicinity of A should be applied as patch loads.

Moment per unit length and associated positions of load group.

Hogging moment - negative.

<table>
<thead>
<tr>
<th>Moment ( \text{my} )</th>
<th>Position of Load Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>(-2.3277)</td>
<td>( x ) 0 ( y ) 3</td>
</tr>
</tbody>
</table>
### Location: Yield line analysis - Example 8.04 by LL Jones

**Rectangular slab, simply supported along its edges**

Subjected to a UDL of \( w \) kN/m\(^2\)

- Length of slab: \( L = 6.096 \) m
- Width of slab: \( A = 4.572 \) m
- Ultimate UDL: \( w = 15.848 \) kN/m\(^2\)

<table>
<thead>
<tr>
<th>Moment ratio (( \mu ))</th>
<th>Ultimate mmt (m)</th>
<th>( \mu ).m</th>
<th>Distance (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>22.654 kNm/m</td>
<td>11.327 kNm/m</td>
<td>2.0708 m</td>
</tr>
<tr>
<td>0.6</td>
<td>21.427 kNm/m</td>
<td>12.856 kNm/m</td>
<td>2.2062 m</td>
</tr>
<tr>
<td>0.7</td>
<td>20.367 kNm/m</td>
<td>14.257 kNm/m</td>
<td>2.3233 m</td>
</tr>
<tr>
<td>0.8</td>
<td>19.435 kNm/m</td>
<td>15.548 kNm/m</td>
<td>2.4262 m</td>
</tr>
<tr>
<td>0.9</td>
<td>18.605 kNm/m</td>
<td>16.744 kNm/m</td>
<td>2.5178 m</td>
</tr>
<tr>
<td>1.0</td>
<td>17.859 kNm/m</td>
<td>17.859 kNm/m</td>
<td>2.6002 m</td>
</tr>
</tbody>
</table>

**Conventions:**

- Positive: a/b
- \( \mu \).m: d/c
- m: Length of slab

Steel in bottom of slab produces positive moment. The side key indicates that the ultimate moment/unit length along a positive yield line in the direction ab is \( m \), and the ultimate moment per unit length along a positive yield-line in the direction bc is \( \mu \).m. This implies that the steel to produce the moment \( m \) is at right angles to this direction, that is in direction bc, while the steel to produce the moment \( \mu \).m is in the direction ab.
Location: Ex 5 from 'The analysis of engineering structures'

Suspension bridge with three-pinned stiffening girder

- Span of suspension bridge: \( L = 106.68 \text{ m} \)
- Dip in cable at centre of bridge: \( d = 10.668 \text{ m} \)
- Total DW of bridge as UDL: \( w_0 = 32.69 \text{ kN/m} \)
- Point load on stiffening girders: \( W = 3487.4 \text{ kN} \)
- Distance of \( W \) from left hand end: \( x_1 = 26.67 \text{ m} \)
- Total sectional area of cables: \( A = 0.076129 \text{ m}^2 \)
- Total number of suspension rods: \( N = 22 \)
- Equivalent UDL due to conc.load: \( w = 4 * W * x_1 / L^2 = 32.69 \text{ kN/m} \)
- Horizontal force at supports: \( H = w * L^2 / (8 * d) = 4359.3 \text{ kN} \)

Moment and shears in girders at 20'th points due to concentrated load

Convention: Sagging moment is positive; hogging moment is negative.

<table>
<thead>
<tr>
<th>Distance from A (m)</th>
<th>Moment (kNm)</th>
<th>Shear (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>871.85</td>
</tr>
<tr>
<td>5.334</td>
<td>5115.5</td>
<td>1046.2</td>
</tr>
<tr>
<td>10.668</td>
<td>11161</td>
<td>1220.6</td>
</tr>
<tr>
<td>16.002</td>
<td>18137</td>
<td>1395</td>
</tr>
<tr>
<td>21.336</td>
<td>26043</td>
<td>1569.3</td>
</tr>
<tr>
<td>26.67</td>
<td>34878</td>
<td>1743.7</td>
</tr>
<tr>
<td>32.004</td>
<td>26043</td>
<td>-1569.3</td>
</tr>
<tr>
<td>37.338</td>
<td>18137</td>
<td>-1395</td>
</tr>
<tr>
<td>42.672</td>
<td>11161</td>
<td>-1220.6</td>
</tr>
<tr>
<td>48.006</td>
<td>5115.5</td>
<td>-1046.2</td>
</tr>
<tr>
<td>53.34</td>
<td>0</td>
<td>-871.85</td>
</tr>
<tr>
<td>58.674</td>
<td>-4185.4</td>
<td>-697.48</td>
</tr>
<tr>
<td>64.008</td>
<td>-7440.7</td>
<td>-523.11</td>
</tr>
<tr>
<td>69.342</td>
<td>-9765.9</td>
<td>-348.74</td>
</tr>
<tr>
<td>74.676</td>
<td>-11161</td>
<td>-174.37</td>
</tr>
<tr>
<td>80.01</td>
<td>-11626</td>
<td>0.46456E-12</td>
</tr>
<tr>
<td>85.344</td>
<td>-11161</td>
<td>174.37</td>
</tr>
<tr>
<td>90.678</td>
<td>-9765.9</td>
<td>348.74</td>
</tr>
<tr>
<td>96.012</td>
<td>-7440.7</td>
<td>523.11</td>
</tr>
<tr>
<td>101.35</td>
<td>-4185.4</td>
<td>697.48</td>
</tr>
<tr>
<td>106.68</td>
<td>0</td>
<td>871.85</td>
</tr>
</tbody>
</table>
Combining equivalent UDL due to conc load with UDL from dead weight

Total UDL on cables \( wt = w_0 + w = 65.38 \text{ kN/m} \)
Vertical force at each support \( V = \frac{wt \times L}{2} = 3487.4 \text{ kN} \)
Horizontal force at each support \( H = \frac{wt \times L^2}{8 \times d} = 8718.5 \text{ kN} \)
Total tension in cables at centre \( T = H \times \sqrt{1 + \frac{16 \times d^2}{L^2}} = 9390.1 \text{ kN} \)
Force in each suspension rod \( F_{rod} = \frac{wt \times L}{N} = 317.03 \text{ kN} \)
The method of analysis deals with the two dimensional cable problem in which only horizontal and vertical loads are considered, and these act in the plane of the cable. A Newton iteration procedure is used to satisfy 'equilibrium' and 'compatibility'.

Cable data

Unstretched length of cable    UCL=1100
Cross-sectional area of cable  A=0.055556
Elastic modulus of cable       E=2.88E9
Unit weight of cable           SW=27
Coefficient of thermal expansion ALPHA=6.5E-6

Supports

X coordinate of right support  XR=1000
Y coordinate of right support  YR=-200
Hor. flexibility of left support FLH=0
Vert. flexibility of left support FLV=0
Sloping flex. of left support  FLS=0
Hor. flexibility of right support FRH=0
Vert flexibility of right support FRV=0
Sloping flex. of right support  FRS=0

Loading details

No. of loads at initially fixed positions NSL=0
No. of loads at constant X coord NXL=0
Temperature rise TR=0
No. of cable elements (typ. 100) NS=100
Initial tension at left end TG=30000
Initial slope at left end THETA=30

Tension=30000  Angle=30  Misclose=216.29
Tension=14779  Angle=33.729  Misclose=195.6
Tension=19574  Angle=30.926  Misclose=35.998
Tension=21644  Angle=27.882  Misclose=6.6914
Tension=21999  Angle=27.615  Misclose=0.080237
Tension=22003  Angle=27.613  Misclose=0.68297E-3
Tension=22003  Angle=27.613  Misclose=8.6308E-6
Tension=22003  Angle=27.613  Misclose=95.697E-9
L.H. tension=22003  Slope angle=27.613°
R.H. tension=27367  Slope angle=44.568°
Total stretched length =1100.1
Maximum depression =95.397 at node No. 36
Dip =169.2 at node No. 36

Node  X-coordinate  Y-coordinate  Element  Tension
2     9.7484       5.0992       1        22003
3     19.558       10.081       2        21867
4     29.427       14.942       3        21734
5     39.355       19.682       4        21604
6     49.341       24.297       5        21478
7     59.385       28.786       6        21355
8     69.486       33.146       7        21236

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Natural frequency of multi-storey frames

For frames in which the mass can be assumed to be lumped at floor levels, and where compression of columns and rotation of column heads may be considered negligible in comparison to sway effects, the period of the fundamental mode is given by:

\[ T = \frac{2\pi \sqrt{\sum m_i \cdot x_i^2}}{\sqrt{\sum g \cdot m_i \cdot x_i}} \]

Raleigh's formula

The method:

a) compute the sum of DL + 20% of imposed load for each floor and roof
b) apply these as a set of horizontal forces to the structure
c) substitute computed deflections and masses in Raleigh's formula to find T.

Number of levels \( n=3 \)

\[
\begin{array}{c}
\text{m3} \\
| \quad x_3 \\
\text{m2} \\
| \quad x_2 \\
\text{m1} \\
| \quad x_1 \\
\end{array}
\]

Ex.: if for a three storey frame, \( x_1 \), \( x_2 \) & \( x_3 \) are the displacements corresponding to the set of masses \( m_1 \), \( m_2 \) & \( m_3 \), then period is given by

\[
T = 2\pi \sqrt{\frac{m_1 \cdot x_1^2 + m_2 \cdot x_2^2 + m_3 \cdot x_3^2}{g(m_1 \cdot x_1 + m_2 \cdot x_2 + m_3 \cdot x_3)}}
\]

where \( g \) is 9.8067 m/s²

Lumped mass at this level \( m(1)=24.91 \text{ kN} \)
Corresponding displacement \( x(1)=0.017502 \text{ m} \)
Lumped mass at this level \( m(2)=23.216 \text{ kN} \)
Corresponding displacement \( x(2)=0.079553 \text{ m} \)
Lumped mass at this level \( m(3)=4.1849 \text{ kN} \)
Corresponding displacement \( x(3)=0.11911 \text{ m} \)
Period \( T=2\pi \sqrt{\frac{\text{sigmx}^2}{g \cdot \text{sigmx}}}) =0.55645 \text{ sec} \)
Natural frequency \( f=\frac{1}{T}=1.7971 \text{ hertz} \)
Location: Elastic critical load of column with end load

Formulas taken from 'Formulas for Stress and Strain' by Roark.
Elastic modulus of bar $E=210\, \text{E6 kN/m}^2$
Moment of inertia of bar $I=0.001 \, \text{m}^4$
Length of bar $L=5 \, \text{m}$

Uniform straight bar under end load, one end free, other end fixed.

$P'=\pi^2 E I / (4 \times L^2) = 20726 \, \text{kN}$

Uniform straight bar under end load, both ends pinned.

$P'=\pi^2 E I / L^2 = 82905 \, \text{kN}$

Uniform straight bar under end load, one end fixed, other end hinged and horizontally constrained over fixed end.

$P'=\pi^2 E I / (0.7 \times L)^2 = 169193 \, \text{kN}$

Uniform straight bar under end load, both ends fixed.

$P'=4 \times \pi^2 E I / L^2 = 331619 \, \text{kN}$
Elastic modulus $E = 0.021328 \, \text{kN/m}^2$

Poisson's ratio $\nu = 0.3$

Plate thickness $t = 8 \, \text{m}$

Plate dimension $a = 0.2 \, \text{m}$

$b = 0.2 \, \text{m}$

$c = 0.25 \, \text{m}$

$d = 0.25 \, \text{m}$

Downward deflection at position 1 $w(1) = 0.62072 \, \text{m}$

Downward deflection at position 2 $w(2) = 0.48404 \, \text{m}$

Downward deflection at position 3 $w(3) = 0.093208 \, \text{m}$

Downward deflection at position 4 $w(4) = 2.1104 \, \text{m}$

Downward deflection at position 5 $w(5) = 1.9428 \, \text{m}$

Downward deflection at position 6 $w(6) = 1.4589 \, \text{m}$

Downward deflection at position 7 $w(7) = 3.2902 \, \text{m}$

Downward deflection at position 8 $w(8) = 3.1015 \, \text{m}$

Downward deflection at position 9 $w(9) = 2.5547 \, \text{m}$
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<td>c=0.1 m</td>
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<tr>
<td></td>
<td>d=0.25 m</td>
</tr>
<tr>
<td>Downward deflection at position 1</td>
<td>w(1)=0.59863 m</td>
</tr>
<tr>
<td>Downward deflection at position 2</td>
<td>w(2)=0.48404 m</td>
</tr>
<tr>
<td>Downward deflection at position 3</td>
<td>w(3)=0.093208 m</td>
</tr>
<tr>
<td>Downward deflection at position 4</td>
<td>w(4)=2.0834 m</td>
</tr>
<tr>
<td>Downward deflection at position 5</td>
<td>w(5)=1.9428 m</td>
</tr>
<tr>
<td>Downward deflection at position 6</td>
<td>w(6)=1.4589 m</td>
</tr>
<tr>
<td>Downward deflection at position 7</td>
<td>w(7)=2.6041 m</td>
</tr>
<tr>
<td>Downward deflection at position 8</td>
<td>w(8)=2.4554 m</td>
</tr>
<tr>
<td>Downward deflection at position 9</td>
<td>w(9)=1.9428 m</td>
</tr>
</tbody>
</table>
Elastic modulus $\quad E = 0.021328 \text{kN/m}^2$
Poisson's ratio $\quad \nu = 0.3$
Plate thickness $\quad t = 8 \text{m}$
Plate dimension $\quad a = 0.12 \text{m}$
$\quad b = 0.12 \text{m}$
$\quad c = 0.1 \text{m}$
$\quad d = 0.1 \text{m}$

Downward deflection at position 1 $\quad w(1) = 1.5123 \text{m}$
Downward deflection at position 2 $\quad w(2) = 1.3811 \text{m}$
Downward deflection at position 3 $\quad w(3) = 1.1434 \text{m}$
Downward deflection at position 4 $\quad w(4) = 2.0834 \text{m}$
Downward deflection at position 5 $\quad w(5) = 1.9428 \text{m}$
Downward deflection at position 6 $\quad w(6) = 1.6876 \text{m}$
Downward deflection at position 7 $\quad w(7) = 2.6041 \text{m}$
Downward deflection at position 8 $\quad w(8) = 2.4554 \text{m}$
Downward deflection at position 9 $\quad w(9) = 2.1852 \text{m}$
Sample output for SCALE Proforma 674. (ans=4)
Pre+post processors for structural analysis
Derives moments and principal stresses from plate
Copyright 1986-2019 Fitzroy Systems Ltd.

Elastic modulus E = 205E6 kN/m²
Poisson's ratio v = 0.3
Plate thickness t = 1 m
Plate dimension a = 1 m
b = 1 m
c = 1 m
d = 1 m

Downward deflection at position 1 w(1) = 13.447E-6 m
Downward deflection at position 2 w(2) = 13.714E-6 m
Downward deflection at position 3 w(3) = 13.447E-6 m
Downward deflection at position 4 w(4) = 13.714E-6 m
Downward deflection at position 5 w(5) = 13.983E-6 m
Downward deflection at position 6 w(6) = 13.714E-6 m
Downward deflection at position 7 w(7) = 13.447E-6 m
Downward deflection at position 8 w(8) = 13.714E-6 m
Downward deflection at position 9 w(9) = 13.447E-6 m
Elastic modulus \( E = 205 \times 10^6 \text{ kN/m}^2 \)

Poisson's ratio \( v = 0.3 \)

Plate thickness \( t = 0.01 \text{ m} \)

Plate dimension
\[
\begin{align*}
  a &= 0.1 \text{ m} \\
  b &= 0.1 \text{ m} \\
  c &= 0.15 \text{ m} \\
  d &= 0.15 \text{ m}
\end{align*}
\]

Downward deflection at position 1 \( w(1) = 0.019001 \text{ m} \)
Downward deflection at position 2 \( w(2) = 0.019929 \text{ m} \)
Downward deflection at position 3 \( w(3) = 0.019001 \text{ m} \)
Downward deflection at position 4 \( w(4) = 0.019813 \text{ m} \)
Downward deflection at position 5 \( w(5) = 0.020782 \text{ m} \)
Downward deflection at position 6 \( w(6) = 0.019813 \text{ m} \)
Downward deflection at position 7 \( w(7) = 0.019001 \text{ m} \)
Downward deflection at position 8 \( w(8) = 0.019929 \text{ m} \)
Downward deflection at position 9 \( w(9) = 0.019001 \text{ m} \)
Polar to Cartesian coordinate conversion

+ j start joint number
radius R /
/ theta (+ve anticlockwise)

Y comes out
of the paper

Z

Radius  R=1.45 m
Initial angle  theta'=90°
Angle of each tread  alpha'=-20°
Number of treads  ntread=18
Initial joint number  j=1
Increment in joint number  inc=2
Initial level  y=0 m
Location: Coordinates of circle in space, rotated about Y, Z & X axes

Radius                           R=1 m
X final ordinate of centre       Xc=1 m
Y final ordinate of centre       Yc=1 m
Z final ordinate of centre       Zc=1 m
Rotation about X axis (deg)      beta=90°
Rotation about Y axis (deg)      psi1=90°
Rotation about Z axis (deg)      psi2=90°
Number of segments in circle     nseg=36
Angle subtended at centre        alpha=0.17453 radians
Joint 1 has maximum initial X ordinate, joints set out in anti-clockwise direction on plan (i.e. clockwise when looking in the Y direction). The centre of the circle is initially located at the origin and the coordinates of each joint computed for the rotations about the Y, Z & X axis in order. The coordinates are
Location: Example of horizontal I section rotated anticlockwise 45°

This procedure computes the angle 'beta' between the vertical plane through coordinates xj,yj,zj and xk,yk,zk and the plane defined by points J, K & P. X and Z axes are as shown Y comes out of the paper.

X coordinate of point J: xj = 0 m
Y coordinate of point J: yj = -7.0711 m
Z coordinate of point J: zj = 10 m
X coordinate of point K: xk = 10 m
Y coordinate of point K: yk = -7.0711 m
Z coordinate of point K: zk = 0 m
X coordinate of point P: xp = 0 m
Y coordinate of point P: yp = 0 m
Z coordinate of point P: zp = 0 m
Plane frame member stiffness

Matrix for member axes

Three degrees of freedom at each end: displacements in x & y directions & rotation about z axis.

Modulus of elasticity $E = 9 \times 10^6$ kN/m²
Modulus of rigidity (shearing) $G = 562500$ kN/m²
Cross sectional area of member $A_x = 0.08$ m²
Shear area (Ax*5/6 for rectangle) $A_y = 0.066667$ m²
Inertia about z axis $I_z = 0.0010667$ m⁴
Length of member $L = 5$ m
Spring stiffness at member start $K_s = 5$ kNm/r
Spring stiffness at member end $K_e = 0$ kNm/r

Member stiffness matrix for plane frame:

\[
\begin{bmatrix}
144000 & 0 & 0 & -144000 & 0 & 0 \\
0 & 0.19982 & 0.99911 & 0 & -0.19982 & 0 \\
0 & 0.99911 & 4.9955 & 0 & -0.99911 & 0 \\
-144000 & 0 & 0 & 144000 & 0 & 0 \\
0 & -0.19982 & -0.99911 & 0 & 0.19982 & 0 \\
0 & 0 & 0 & 0 & 0 & 0
\end{bmatrix}
\]
Sample output for SCALE Proforma 686. (ans=1)
Page: 1
Pre & post processors for structural analysis
Made by: IFB
Loss of stiffness due to partial plasticity
Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.
Ref No: SC686
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
Elastic-plastic analysis of compression members
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In rigid-frame (non-triangulated) structures, bending moments arise as
'primary moments', whereas in rigid-jointed triangulated frames, loaded
only at the joints, any bending moments arise only as 'secondary
moments'. Results obtained from 'unit shape factor analysis', in which
plastic deformation is assumed to be confined to discrete plastic
'hinges', have been found, both analytically and experimentally, to be
of fully sufficient accuracy for rigid frames. For triangulated
frames, in which the structural action of the members lies essentially
in their resistance to axial forces, unit shape factor analysis may
give results which are not sufficiently reliable. The reasons are as
follows.
a) The presence of initial imperfections and residual stresses has an
appreciable influence on the carrying capacities of members loaded in
axial compression, whereas for members loaded primarily in bending
(even when some compressive axial loading is also present) their effect
explored analytically is small, and certainly overshadowed by the many
variable effects due to other factors which arise in practice and
experimentally.
b) Compression members may, at their maximum capacity load, have
attained a state of only limited plasticity, without the formation
anywhere of a plastic hinge. This is because partially penetrating
plastic zones have so reduced the stiffness of the members of a frame
that a state of critical buckling has been reached.
The following calculation derives the imperfection e to be applied at
the centre of a member using Professor Horne's theory, and computes the
coordinates at quarter points for a member having the imperfection e.
Coordinates of '3' points on a member having imperfection 'e'
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
(xm,ym) M _ _ _ _ _ _ _ _ _ _ _ _ _ _ _ (x2,y2)
/|\
_ / | \e
_ Member joints points
/
| \
_ (x1,y1) and (x2,y2).
/
D|___\ _ -__)theta
Undisplaced mid point
/
_ -C
of member is at C.
/
_ (xc,yc)
Displaced mid point is
/ _ M with coords xm,ym.
(x1,y1)/Ä
Length DC=e*sin(theta)
X coordinate of start
Y coordinate of start
X coordinate of end
Y coordinate of end
Modulus of elasticity
Yield stress
Cross sectional area
Inertia of section
Robertson constant

of
of
of
of

member
member
member
member

x1=0 mm
y1=0 mm
x2=3000 mm
y2=3000 mm
E=205000 N/mmý
py=350 N/mmý
A=3710 mmý
I=16.3E6 mm®
an=3

ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
SCALE 5.48
Office 1007
Proforma 686


For e positive in local y direction as shown above:

Displaced 1/4 point coordinate
\[ x_q = \frac{(x_1 + x_c)}{2} - 0.75 \times e \times \frac{(y_2 - y_1)}{L} \]
\[ = 744.78 \text{ mm} \]

Displaced 1/4 point coordinate
\[ y_q = \frac{(y_1 + y_c)}{2} + 0.75 \times e \times \frac{(x_2 - x_1)}{L} \]
\[ = 755.22 \text{ mm} \]

Displaced mid-point coordinate
\[ x_m = \frac{(x_2 + x_1)}{2} - e \times \frac{(y_2 - y_1)}{L} = 1493 \text{ mm} \]

Displaced mid-point coordinate
\[ y_m = \frac{(y_2 + y_1)}{2} + e \times \frac{(x_2 - x_1)}{L} = 1507 \text{ mm} \]

Displaced 3/4 point coordinate
\[ x_t = \frac{(x_c + x_2)}{2} - 0.75 \times e \times \frac{(y_2 - y_1)}{L} = 2244.8 \text{ mm} \]

Displaced 3/4 point coordinate
\[ y_t = \frac{(y_c + y_2)}{2} + 0.75 \times e \times \frac{(x_2 - x_1)}{L} = 2255.2 \text{ mm} \]

For e negative in local y direction:

Displaced 1/4 point coordinate
\[ x_q = \frac{(x_1 + x_c)}{2} + 0.75 \times e \times \frac{(y_2 - y_1)}{L} = 755.22 \text{ mm} \]

Displaced 1/4 point coordinate
\[ y_q = \frac{(y_1 + y_c)}{2} - 0.75 \times e \times \frac{(x_2 - x_1)}{L} = 744.78 \text{ mm} \]

Displaced mid-point coordinate
\[ x_m = \frac{(x_2 + x_1)}{2} + e \times \frac{(y_2 - y_1)}{L} = 1507 \text{ mm} \]

Displaced mid-point coordinate
\[ y_m = \frac{(y_2 + y_1)}{2} - e \times \frac{(x_2 - x_1)}{L} = 1493 \text{ mm} \]

Displaced 3/4 point coordinate
\[ x_t = \frac{(x_c + x_2)}{2} + 0.75 \times e \times \frac{(y_2 - y_1)}{L} = 2255.2 \text{ mm} \]

Displaced 3/4 point coordinate
\[ y_t = \frac{(y_c + y_2)}{2} - 0.75 \times e \times \frac{(x_2 - x_1)}{L} = 2244.8 \text{ mm} \]
Equilibrium and compatibility check of plane frame/truss members

X coordinate at start of member xcs=0
Y coordinate at start of member ycs=2000
X coordinate at end of member xce=0
Y coordinate at end of member yce=4000
X displacement at start (GLOBAL) Xds=78.13413
Y displacement at start (GLOBAL) Yds=-1.54724
Z rotation at start (GLOBAL) Zrs=-0.0318884
X displacement at end (GLOBAL) Xde=113.47263
Y displacement at end (GLOBAL) Yde=-1.879371
Z rotation at end (GLOBAL) Zre=0
X force at start of member fxs=991.0094
Y force at start of member fys=517.5643
Z moment at start (GLOBAL) mzs=-1.0768567E6
X force at end of member fxe=-991.0094
Y force at end of member fye=-517.5643
Z moment at end of member mze=2.1119853E6

Elastic modulus e=100000
Shearing modulus g=0
Cross sectional area of member ax=1000
Shear area of member ay=0
Moment of inertia about Z axis iz=1E6

Deflection in local x direction xds=Cx*Xds+Cy*Yds=-0.16641324
xde=Cx*Xde+Cy*Yde=0.12591862

Deflection in local y direction yds=-Cy*Xds+Cx*Yds=-78.149271
yde=-Cy*Xde+Cx*Yde=-113.48812

Equilibrium

Final length l=SQR((xce+Xde-xcs-Xds)^2+(yce+Yde-ycs-Yds)^2)=1999.9801
Net moment M=mzs+mze=1.0351286E6
Should balance start shear x length M'=fys*l=1.0351183E6
Percentage error p=100*(M-M')/M=0.99505629E-3 %

Flexure

From buildup of stiffness matrix in NL-STRESS Reference manual, taking into account shear deformation: mzs=EI/L + 3EI.S/L for unit rotation at the start, and mze=EI/L+3EI.S/L for unit rotation at the end, where

\[
1 + \frac{12EI}{L^2 G A_y}
\]

the shear deformation coefficient, S = \[
\frac{1}{1 + \frac{12EI}{L^2 G A_y}}
\]

For zero shear area take s=1
Percentage error p=100*(theta-theta')/theta
= -0.93234687E-3 %
Net length

Net length should be \( L - PL/AE \)
Where \( L \) is original length \( L = \sqrt{(x_{ce} - x_{cs})^2 + (y_{ce} - y_{cs})^2} = 2000 \)
Thus \( \text{netl} = L - f_{xs} * L / (a * e) = 1999.9802 \)
Analysis gives \( \text{netl}' = \sqrt{(x_{ce} + X_{de} - x_{cs} - X_{ds})^2 + (y_{ce} + Y_{de} - y_{cs} - Y_{ds})^2} = 1999.9801 \)
Percentage error \( p = 100 * (\text{netl} - \text{netl}') / \text{netl} = 4.0469251E-6 \% \)
Equilibrium and compatibility check of plane grid members

X coordinate at start of member xcs=12.006
Y coordinate at start of member ycs=0
X coordinate at end of member xce=12.006
Y coordinate at end of member yce=1.7393
X rotation at start of member Xrs=-0.011072826
Y rotation at start of member Yrs=0.005295661
Z displacement at start of member Zds=-0.15362974
X rotation at end of member Xre=-0.014049463
Y rotation at end of member Yre=0.006476447
Z displacement at end of member Zde=-0.17480877
X moment at start of member mxs=-10.1319
Y moment at start of member mys=-67.9809
Z force at start of member fzs=-62.8811
X moment at end of member mxe=10.1319
Y moment at end of member mye=177.3501
Z force at end of member fze=62.8811

Elastic modulus e=9.3333333E6
Shearing modulus g=1.8666667E6
Torsion constant of member ix=0.0079951805
Moment of inertia about Y axis iy=0.0076795238
Shear area of member az=0.20638333

Length of segment L=SQR((xce-xcs)^2+(yce-ycs)^2)=1.7393
Direction cosines Cx=(xce-xcs)/L=0
Cy=(yce-ycs)/L=1
Rotation about local x axis xrs=Cx*Xrs+Cy*Yrs=0.005295661
xre=Cx*Xre+Cy*Yre=0.006476447
Rotation about local y axis yrs=-Cy*Xrs+Cx*Yrs=0.011072826
yre=-Cy*Xre+Cx*Yre=0.014049463
Deflection in local z direction zds=Zds=-0.15362974
zde=Zde=-0.17480877

Equilibrium

Final length l=SQR((xce-xcs)^2+(yce-ycs)^2)=1.7393
Net moment M=mys+mye=109.3692
Should balance end shear times length M'=fze*l=109.3691
Percentage error p=100*(M-M')/M=93.966126E-6 %

Flexure

From buildup of stiffness matrix in NL-STRESS Reference manual, taking into account shear deformation: 

\[
mys = EI/L + 3EI.S/L \quad \text{for unit rotation at the start, and} \quad mye = -EI/L + 3EI.S/L \quad \text{for unit rotation at the end, where} \quad \text{1} \]

the shear deformation coefficient, \( S = \left( \frac{1 + \frac{12EI}{L^2GAz}}{L^2GAz} \right) \)

Shear deformation coefficient \( s=1/(1+12*\text{e}*\text{iy}/(1^2*\text{g}^*\text{az}))=0.57537086 \)
Percentage error \( p=100*(\theta-\theta')/\theta \)
\(-2.5711772E-6 \) %
Net rotation

Net rotation should be netr=T*L/GIx, where T is torque and Ix is torsion constant, length
Thus netr=mxs*L/(ix*g)=-0.0011807837
Analysis gives netr'=xrs-xre=-0.001180786
Percentage error p=100*(netr-netr')/netr
=-0.19721958E-3 %
Equilibrium and compatibility check of space frame/truss members

X coordinate at start of member \( x_{cs} = 240 \)
Y coordinate at start of member \( y_{cs} = 120 \)
Z coordinate at start of member \( z_{cs} = 0 \)
X coordinate at end of member \( x_{ce} = 360 \)
Y coordinate at end of member \( y_{ce} = 0 \)
Z coordinate at end of member \( z_{ce} = 120 \)

X displacement at start (GLOBAL) \( X_{ds} = 0.22201994 \)
Y displacement at start (GLOBAL) \( Y_{ds} = -0.48118948 \)
Z displacement at start (GLOBAL) \( Z_{ds} = -0.70160623 \)

X displacement at end (GLOBAL) \( X_{de} = 0 \)
Y displacement at end (GLOBAL) \( Y_{de} = 0 \)
Z displacement at end (GLOBAL) \( Z_{de} = 0 \)

X rotation at start (GLOBAL) \( X_{rs} = -0.008024871 \)
Y rotation at start (GLOBAL) \( Y_{rs} = 0.001007657 \)
Z rotation at start (GLOBAL) \( Z_{rs} = -0.00434716 \)

X rotation at end (GLOBAL) \( X_{re} = 0 \)
Y rotation at end (GLOBAL) \( Y_{re} = 0 \)
Z rotation at end (GLOBAL) \( Z_{re} = 0 \)

X force at start of member \( f_{xs} = 1.4696 \)
Y force at start of member \( f_{ys} = -0.7149 \)
Z force at start of member \( f_{zs} = -0.4798 \)

X force at end of member \( f_{xe} = -1.4696 \)
Y force at end of member \( f_{ye} = 0.7149 \)
Z force at end of member \( f_{ze} = 0.4798 \)

X moment at start of member \( m_{xs} = -37.0171 \)
Y moment at start of member \( m_{ys} = 15.6888 \)
Z moment at start of member \( m_{zs} = -53.2791 \)

X moment at end of member \( m_{xe} = 37.0171 \)
Y moment at end of member \( m_{ye} = 84.0397 \)
Z moment at end of member \( m_{ze} = -95.3189 \)

Elastic modulus \( e = 30000 \)
Shearing modulus \( g = 12000 \)
Cross sectional area of member \( a_x = 11 \)
Shear area area of member \( a_y = 0 \)
Shear area area of member \( a_z = 0 \)
Torsion constant for member \( i_x = 83 \)
Moment of inertia about Y axis \( i_y = 56 \)
Moment of inertia about Z axis \( i_z = 56 \)

Deflection in local x
\[ \text{xds}=C_x\times X_{ds}+C_y\times Y_{ds}+C_z\times Z_{ds} = -0.0027939167 \]
\[ xde=C_x\times X_{de}+C_y\times Y_{de}+C_z\times Z_{de} = 0 \]

Deflection in local y
\[ \text{yds}=-C_x\times C_y\times X_{ds}/c+C_y\times Y_{ds}+C_z\times Z_{ds}/c = -0.59012373 \]
\[ yde=-C_x\times C_y\times X_{de}/c+C_y\times Y_{de}+C_z\times Z_{de}/c = 0 \]
\[ zds=-C_x\times X_{ds}/c+C_y\times Y_{ds}+C_z\times Z_{ds}/c = -0.65179504 \]
\[ zde=-C_x\times X_{de}/c+C_y\times Y_{de}+C_z\times Z_{de}/c = 0 \]

Rotation about local x
\[ \text{xrs}=C_x\times X_{rs}+C_y\times Y_{rs}+C_z\times Z_{rs} = -0.0077285358 \]
\[ xre=C_x\times X_{re}+C_y\times Y_{re}+C_z\times Z_{re} = 0 \]

Rotation about local y
\[ \text{yrs}=-C_x\times C_y\times X_{rs}/c+C_y\times Y_{rs}+C_z\times Z_{rs}/c = -0.0042004278 \]
\[ yre=-C_x\times C_y\times X_{re}/c+C_y\times Y_{re}+C_z\times Z_{re}/c = 0 \]

Rotation about local z
\[ \text{zrs}=-C_x\times X_{rs}/c+C_y\times Y_{rs}+C_z\times Z_{rs}/c = 0.0026341153 \]
Net length

Net length should be \( L - PL/AE \)
Thus \( \text{netl}=L-fxs*L/(ax*e)=207.84517 \)
Analysis gives
\[
\text{netl}'=\sqrt{(xce+Xde-xcs-Xds)^2+(yce+Yde-ycs-Yds)^2+(zce+Zde-zcs-Zds)^2} = 207.84703
\]
Percentage error \( p=100*(\text{netl}-\text{netl}')/\text{netl} = -0.89478292E-3 \% \)

Net rotation

Net rotation should be \( \text{netr}=T*L/GIx \), where \( T \) is torque and \( Ix \) is torsion constant, length \( L=\text{netl}'=207.84703 \)
Thus \( \text{netr}=mxs*L/(ix*g)=-0.0077247935 \)
Analysis gives \( \text{netr}'=xrs-xre=-0.0077285358 \)
Percentage error \( p=100*(\text{netr}-\text{netr}')/\text{netr}=-0.048445108 \% \)

Equilibrium

Net moment about y axis \( M=mys+mye=99.7285 \)
Should balance end shear x length \( M'=fze*l=99.725006 \)
Percentage error \( p=100*(M-M')/M=0.0035040092 \% \)
Net moment about z axis \( M=mzs+mze=-148.598 \)
Should balance start shear x len \( M'=fys*l=-148.58984 \)
Percentage error \( p=100*(M-M')/M=0.0054896383 \% \)

Flexure about y axis

From buildup of stiffness matrix in NL-STRESS Reference manual, taking into account shear deformation: \( mys=EI/L + 3EI.S/L \) for unit rotation at the start, and \( mye=-EI/L+3EI.S/L \) for unit rotation at the end, where
\[
1 \quad \text{the shear deformation coefficient, } S = \frac{1 + \frac{12EI}{L^2.G.Az}}{1}
\]
For zero shear area take \( s=1 \)
Percentage error \( p=100*(\theta-\theta')/\theta = 0.65529167 \% \)

Flexure about z axis

From buildup of stiffness matrix in NL-STRESS Reference manual, taking into account shear deformation: \( mzs=EI/L + 3EI.S/L \) for unit rotation at the start, and \( mze=-EI/L+3EI.S/L \) for unit rotation at the end, where
\[
1 \quad \text{the shear deformation coefficient, } S = \frac{1 + \frac{12EI}{L^2.G.Ay}}{1}
\]
For zero shear area take \( s=1 \)
Percentage error \( p=100*(\theta-\theta')/\theta = -1.2907055 \% \)
Location: Support displacements to model for guy pretension

Support displacements to model for guy pretension

Guy joins joints i and j with start of guy at joint i.

Projection of guy onto horizontal $H=6$ m
Projection of guy onto vertical $V=5$ m
Cross sectional area of guy $A=18.4E-6$ m$^2$
Elastic modulus of guy $E=210E6$ kN/m$^2$
Tension applied to end i of guy $T_i=1$ kN
Original length of guy $L_o=L-e=7.8083$ m
Impose horizontal displacement $D_h=(L_o-L)\times H/L=-0.0015309$ m
Impose vertical displacement $D_v=(L_o-L)\times V/L=-0.0012757$ m

Therefore to model for average tension 1.0045 kN and vertical load 0.0018 kN/m run on the guy, apply joint displacements of $-0.0015309$ m in the horizontal direction and $-0.0012757$ m in the vertical direction assuming the guy was initially straight between i and j.
Location: Support displacements to model for guy pretension

Analysis of guyed structures using Non Linear STRESS

The analysis of structures such as guyed offshore platforms presents a special problem of modelling flexible members (guys) so that they may be analysed by the stiffness method.

NL-STRESS permits the effects of finite displacements to be taken into account in the analysis when METHOD SWAY is selected; NL-STRESS can therefore be used to analyse structures with attached guys. The stiff part of the structure is modelled in the normal way, but setting the NUMBER OF INCREMENTS to typically 20 (to apply the load in 20 increments) and the NUMBER OF SEGMENTS to 4 also (to divide each member in the structure — cables included — into 4 segments and thereby cater for 'within-member stability' effects).

Because NL-STRESS uses the stiffness method of analysis, each member must have some stiffness. Guys have small diameter in comparison to their length and stiffness is introduced by a guy diameter modelling (or magnification) factor denoted 's', typically having a value of 40. The elastic modulus for the guy is divided by $s^2$ so that the guy behaves in tension just as it would if unmodified.

It will be apparent that the introduction of stiffness to the guys will affect the tensile forces in them. The effects of bending stiffness are nullified by applying a compensating transverse UDL and balancing end point loads to each guy. A further calculation computes these compensating loads — the method was derived by Michael Horne.

The foot of guys are pretensioned to some value when the guy is subject only to its own self weight. The pretension is best modelled by applying JOINT DISPLACEMENTS at the foot of each guy.

Simple guy theory

If a cable is suspended between two points $i$ and $j$, and subjected to a load transverse to the chord $ij$ and evenly distributed along the cable length $iOj$ then when the curve is flat the loading closely approximates to a U.D.L. and the curve to a parabola.

\[
\text{Assuming a parabolic shape, } y = a \cdot x^2
\]
Limitation on s in terms of tension and guy properties

Projection of guy onto horizontal $H=5 \text{ m}$
Projection of guy onto vertical $V=5 \text{ m}$
Cross sectional area of guy $A=18.4E-6 \text{ m}^2$
Elastic modulus of guy $E=210E6 \text{ kN/m}^2$
Percentage error for 1st analysis $p=5\%$
Lowest expected tension in guy $T_{\text{min}}=1 \text{ kN}$

Support displacements to model for guy pretension

Guy joins joints i and j with start of guy at joint i.

Projection of guy onto horizontal $H=5 \text{ m}$
Projection of guy onto vertical $V=5 \text{ m}$
Cross sectional area of guy $A=18.4E-6 \text{ m}^2$
Elastic modulus of guy $E=210E6 \text{ kN/m}^2$
Tension applied to end i of guy $T_i=1 \text{ kN}$
Original length of guy $L_0=L_e=7.0693 \text{ m}$
Impose horizontal displacement $D_h=(L_0-L)\times H/L=-0.0012831 \text{ m}$
Impose vertical displacement $D_v=(L_0-L)\times V/L=-0.0012831 \text{ m}$

Therefore to model for average tension 1.0045 kN and vertical load 0.0018 kN/m run on the guy, apply joint displacements of -0.0012831 m in the horizontal direction and -0.0012831 m in the vertical direction assuming the guy was initially straight between i and j.

Guy projection on X axis (+ or -) $X=3 \text{ m}$
Guy projection on Z axis (+ or -) $Z=4 \text{ m}$
Impose displacement in X directn $D_x=D_h\times X/P=-0.76986E-3 \text{ m}$
Impose displacement in Z directn $D_y=D_h\times Z/P=-0.0010265 \text{ m}$
Location: Guy mark 1-3-5-7

Guy loading to model for artificial guy stiffness

\[ \Delta d_Y = \frac{d_X}{d_X} \Delta d_Y \]
\[ \Delta d_Z = \frac{d_Y}{d_Y} \Delta d_Z \]

Take bottom of guy as origin of XYZ global axes then:

Initial X coord of top of guy \( X = 0 \text{ m} \)
Initial Y coord of top of guy \( Y = 5 \text{ m} \)
Initial Z coord of top of guy \( Z = 6 \text{ m} \)
Displ in X dirn at bottom of guy \( dX_b = 0 \text{ m} \)
Displ in Y dirn at bottom of guy \( dY_b = -0.0012757 \text{ m} \)
Displ in Z dirn at bottom of guy \( dZ_b = -0.0015309 \text{ m} \)
Defln in X dirn at top of guy \( dX_t = 0.0010757 \text{ m} \)
Defln in Y dirn at top of guy \( dY_t = -0.1014 \times 10^{-3} \text{ m} \)
Defln in Z dirn at top of guy \( dZ_t = 0 \text{ m} \)
Defln in X dirn at middle of guy \( dX_m = 0.068287 \text{ m} \)
Defln in Y dirn at middle of guy \( dY_m = -0.0049307 \text{ m} \)
Defln in Z dirn at middle of guy \( dZ_m = 0.002754 \text{ m} \)

Ignoring displacement component in local X direction as we are only interested in transverse displacements.

Defln of middle of guy in local y 
\[ \text{del} y = -\frac{C_x C_y \text{del} X}{H} + \frac{H \text{del} Y - C_y C_z \text{del} Z}{H} = -0.005512 \text{ m} \]

local z 
\[ \text{del} z = \frac{C_z \text{del} X + C_x \text{del} Z}{H} = -0.067749 \text{ m} \]

Elastic modulus for the guy \( E = 210 \times 10^6 \text{ kN/m}^2 \)
True diameter of the guy \( d = 0.0048402 \text{ m} \)
Guy diameter modelling factor \( s = 20 \)
Corrn transv loading in local Y 
\[ w_{fy} = 3.8 \times 10^6 \times (d/L)^4 s^2 \Delta \text{del} y = -0.25952 \times 10^{-3} \text{ kN/m} \]

Compensating end point loads 
\[ W_{fy} = -w_{fy} L/2 = 0.0010134 \text{ kN} \]
Corrn transv loading in local Z 
\[ w_{fz} = 3.8 \times 10^6 \times (d/L)^4 s^2 \Delta \text{del} z = -0.0031898 \text{ kN/m} \]

Compensating end point loads 
\[ W_{fz} = -w_{fz} L/2 = 0.012456 \text{ kN} \]
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

Basic snow load \( S_b = 0.5 \, \text{kN/m}^2 \)
Site altitude is less than 100 m above mean sea level.
Site snow load \( S_o = S_b = 0.5 \, \text{kN/m}^2 \)

Symmetrical pitched Roof

Symmetrical pitched roof where the angle of pitch is > 15°.

For symmetrical roofs with pitches > 15° two load types are to be considered.

CASE 1: Uniform load over whole roof.

CASE 2: Uniform load over one half of roof only.

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient \( u_1 \) is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient \( u_{1a} \) is considered to act on one half of the roof.

Angle of pitch of the roof \( \alpha \) \( \alpha = 16^\circ \)
Snow loading

Location:

NOTE: For roofs of more than two spans b3 is the largest stretch of roof away from the taller ridge.

Snow drift profile extends as a sloping line between the ridges. Height h is the depth of snow at the valley line.

Frame 1 half span length \( b_1 = 10 \text{ m} \)
Frame 2 half span length \( b_2 = 20 \text{ m} \)
Max ridge to outer eaves length \( b_3 = 50 \text{ m} \)
Frame 1 Valley to ridge height \( h_{o1} = 2.5 \text{ m} \)
Frame 2 Valley to ridge height \( h_{o2} = 3.2 \text{ m} \)
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

Basic snow load \( S_b = 0.8 \) kN/m\(^2\)
Site altitude is less than 100 m above mean sea level.
Site snow load \( S_o = S_b = 0.8 \) kN/m\(^2\)

Symmetrical pitched Roof

Symmetrical pitched roof where the angle of pitch is > 15°.

For symmetrical roofs with pitches > 15° two load types are to be considered.

CASE 1: Uniform load over whole roof.

CASE 2: Uniform load over one half of roof only.

The value of the snow load shape coefficient \( u_l \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient \( u_l \) is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient \( u_{la} \) is considered to act on one half of the roof.

Angle of pitch of the roof \( \alpha \) \( \alpha = 75^\circ \)
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

<table>
<thead>
<tr>
<th>Basic snow load</th>
<th>Sb=0.8 kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site altitude</td>
<td>less than 100 m above mean sea level.</td>
</tr>
<tr>
<td>Site snow load</td>
<td>So=Sb=0.8 kN/m²</td>
</tr>
</tbody>
</table>

Asymmetrical pitched Roof

The value of the snow load shape coefficient $u_1$ is dependent upon the angle of pitch of the roof measured from the horizontal.

Asymmetrical pitched roof.

Roof section

CASE 1: Uniform load - BS6399-3:1988 Clause 7.2.3.2

Left side of roof:
- Angle of left pitch of roof: $\alpha_L=75^\circ$
- Slope of roof is greater than 60° therefore the shape coefficient is zero and the snow loading is zero on this side of the roof.
- Snow load shape coefficient: $u_{1L}=0$
- Snow load on left side of roof: $S_{d1}=u_{1L}*S_0=0$ kN/m²

Right side of roof:
- Angle of right pitch of roof: $\alpha_R=25^\circ$
- Snow load shape coefficient: $u_{1R}=0.8$
- Snow load on right side of roof: $S_{d2}=u_{1R}*S_0=0.64$ kN/m²
Summary of loading

CASE 1: Uniform load

CASE 2: Asymmetric load (only the left side of roof loaded)

CASE 3: Asymmetric load (only the right side of roof loaded)
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

Basic snow load \( S_b = 0.5 \text{ kN/m}^2 \)

Site altitude is less than 100 m above mean sea level.

Site snow load \( S_o = S_b = 0.5 \text{ kN/m}^2 \)

Symmetrical pitched Roof

Symmetrical pitched roof where the angle of pitch is > 15°.

For symmetrical roofs with pitches > 15° two load types are to be considered.

CASE 1: Uniform load over whole roof.

CASE 2: Uniform load over one half of roof only.

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient \( u_1 \) is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient \( u_{1a} \) is considered to act on one half of the roof.

Angle of pitch of the roof \( \alpha \) = 16°
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Difference in eaves levels</td>
<td>$h_{o1}=5$ m</td>
</tr>
<tr>
<td>Length of lower building(s)</td>
<td>$b_1=30$ m</td>
</tr>
<tr>
<td>Width of higher building</td>
<td>$b_2=10$ m</td>
</tr>
<tr>
<td>Length of lower building</td>
<td>$b_3=10$ m</td>
</tr>
<tr>
<td>Low level roof slope</td>
<td>$\alpha_6=15^\circ$</td>
</tr>
</tbody>
</table>
**Location: Roof and snow loading**

**Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)**

Basic snow load \( S_b = 0.5 \text{ kN/m}^2 \)

Site altitude is less than 100 m above mean sea level.

Site snow load \( S_o = S_b = 0.5 \text{ kN/m}^2 \)

**Symmetrical pitched Roof**

Symmetrical pitched roof where the angle of pitch is > 15°.

For symmetrical roofs with pitches > 15° two load types are to be considered.

CASE 1: Uniform load over whole roof.

CASE 2: Uniform load over one half of roof only.

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient \( u_1 \) is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient \( u_{1a} \) is considered to act on one half of the roof.

Angle of pitch of the roof \( \alpha \) \( \alpha = 16^\circ \)
Location:

Difference in eaves levels  $h_{ol}=2.5 \text{ m}$
Length of lower building(s)  $b_1=10 \text{ m}$
Width of higher building  $b_2=20 \text{ m}$
Angle of pitch of low level roof  $\alpha_7=30^\circ$
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

Basic snow load \( S_b = 0.5 \text{ kN/m}^2 \)
Site altitude is less than 100 m above mean sea level.
Site snow load \( S_o = S_b = 0.5 \text{ kN/m}^2 \)

Symmetrical pitched Roof

Symmetrical pitched roof where the angle of pitch is > 15°.

For symmetrical roofs with pitches > 15° two load types are to be considered.

CASE 1: Uniform load over whole roof.

CASE 2: Uniform load over one half of roof only.

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient \( u_1 \) is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient \( u_{la} \) is considered to act on one half of the roof.

Angle of pitch of the roof \( \alpha \) \( \text{alpha}=16^\circ \)
Location:

Difference in eaves levels $h_{01}=2.5\ m$
Length of lower building(s) $b_1=10\ m$
Width of higher building $b_2=20\ m$
Angle of pitch of low level roof $\alpha_7=30^\circ$
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

- Basic snow load: \( S_b = 0.5 \ \text{kN/m}^2 \)
- Site altitude is less than 100 m above mean sea level.
- Site snow load: \( S_o = S_b = 0.5 \ \text{kN/m}^2 \)

Flat or Monopitch Roof

Flat or monopitch roof where the angle of pitch is \( \alpha \)°.

A single load case of a uniform layer of snow across the whole roof will be considered.

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient is considered to be constant across the whole roof.

Angle of pitch of the roof \( \alpha \)° \( \alpha = 16^\circ \)
Location:

Difference in eaves levels  $h_{o1}=2.5$ m  
Length of lower building(s)  $b_1=10$ m  
Width of higher building  $b_2=20$ m
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

Basic snow load $S_b = 0.5 \text{ kN/m}^2$
Site altitude is less than 100 m above mean sea level.
Site snow load $S_c = S_b = 0.5 \text{ kN/m}^2$

Flat or Monopitch Roof

Flat or monopitch roof where the angle of pitch is $\alpha^\circ$.

Roof section

A single load case of a uniform layer of snow across the whole roof will be considered.

The value of the snow load shape coefficient $u_1$ is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient is considered to be constant across the whole roof.

Angle of pitch of the roof $\alpha^\circ$ = 16°
Location:

Difference in eaves levels \( h_{ol} = 2.5 \text{ m} \)
Length of lower building(s) \( b_1 = 10 \text{ m} \)
Width of higher building \( b_2 = 20 \text{ m} \)
Location: Roof and Snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

Basic snow load \( S_b = 0.5 \) kN/m²
Site altitude is less than 100 m above mean sea level.
Site snow load \( S_o = S_b = 0.5 \) kN/m²

Flat or Monopitch Roof

Flat or monopitch roof where the angle of pitch is \( \alpha \)°.

Roof section

A single load case of a uniform layer of snow across the whole roof will be considered.

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal.
This coefficient is considered to be constant across the whole roof.
Angle of pitch of the roof \( \alpha \)°  \( \alpha = 16^\circ \)
Location:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parapet height</td>
<td>ho1=2.5 m</td>
</tr>
<tr>
<td>Length of building</td>
<td>b1=10 m</td>
</tr>
<tr>
<td>Low level roof slope</td>
<td>alpha6=12.5°</td>
</tr>
</tbody>
</table>
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

Basic snow load \( S_b = 0.5 \text{ kN/m}^2 \)

Site altitude is less than 100 m above mean sea level.

Site snow load \( S_c = S_b = 0.5 \text{ kN/m}^2 \)

**Flat or Monopitch Roof**

Flat or monopitch roof where the angle of pitch is \( \alpha \)°.

A single load case of a uniform layer of snow across the whole roof will be considered.

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient is considered to be constant across the whole roof.

Angle of pitch of the roof \( \alpha \)° \( \alpha = 16\)°
Location:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parapet height</td>
<td>h_{1}=2.5\ m</td>
</tr>
<tr>
<td>Length of building</td>
<td>b_{1}=10\ m</td>
</tr>
</tbody>
</table>
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

Basic snow load $S_b = 0.5 \text{ kN/m}^2$
Site altitude is less than 100 m above mean sea level.
Site snow load $S_o = S_b = 0.5 \text{ kN/m}^2$

Symmetrical pitched Roof

Symmetrical pitched roof where the angle of pitch is $> 15^\circ$.

For symmetrical roofs with pitches $> 15^\circ$ two load types are to be considered.

CASE 1: Uniform load over whole roof.

CASE 2: Uniform load over one half of roof only.

The value of the snow load shape coefficient $u_1$ is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient $u_1$ is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient $u_{1a}$ is considered to act on one half of the roof.

Angle of pitch of the roof $\alpha^\circ$  \( \alpha = 16^\circ \)
Location:

- Parapet height: $h_0=2.5$ m
- Parapet to ridge length: $b_1=10$ m
- Building width: $b_2=20$ m
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic snow load</td>
<td>$S_b = 0.5 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Site altitude is less than 100 m above mean sea level.</td>
<td></td>
</tr>
<tr>
<td>Site snow load</td>
<td>$S_o = S_b = 0.5 \text{ kN/m}^2$</td>
</tr>
</tbody>
</table>

Symmetrical pitched Roof

Symmetrical pitched roof where the angle of pitch is $> 15^\circ$.

For symmetrical roofs with pitches $> 15^\circ$ two load types are to be considered.

**CASE 1:** Uniform load over whole roof.

**CASE 2:** Uniform load over one half of roof only.

The value of the snow load shape coefficient $u_l$ is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient $u_l$ is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient $u_{la}$ is considered to act on one half of the roof.

Angle of pitch of the roof $\alpha^\circ$ alpha=$16^\circ$
Location:

Eaves to ridge height
- $h_{ol} = 2.5$ m
- $b_1 = 20$ m
- $b_2 = 10$ m

Slope of roof to building B
- $\alpha = 16^\circ$
- $h = \tan(\text{RAD}(\alpha)) \times \left( \frac{l_{s2}}{l_{s1_2}} \right) = 1.0753$ m
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

<table>
<thead>
<tr>
<th>Basic snow load</th>
<th>Sb=0.5 kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site altitude is less than 100 m above mean sea level.</td>
<td></td>
</tr>
<tr>
<td>Site snow load</td>
<td>So=Sb=0.5 kN/m²</td>
</tr>
</tbody>
</table>

Flat or Monopitch Roof

Flat or monopitch roof where the angle of pitch is \( \alpha \)°.

\[ u_1 \]

A single load case of a uniform layer of snow across the whole roof will be considered.

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient is considered to be constant across the whole roof.

Angle of pitch of the roof \( \alpha \)° \( \alpha = 16° \)
Location:

- Left side obstruction height: $h_{o1} = 2.5\, m$
- Roof Length to left side: $b_1 = 10\, m$
- Right side obstruction height: $h_{o2} = 2\, m$
- Roof Length to right side: $b_2 = 20\, m$
Location: Roof and snow loading

Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)

Basic snow load $S_b = 0.5 \text{ kN/m}^2$
Site altitude is less than 100 m above mean sea level.
Site snow load $S_o = S_b = 0.5 \text{ kN/m}^2$

Symmetrical pitched Roof

Symmetrical pitched roof where the angle of pitch is $> 15^\circ$.

For symmetrical roofs with pitches $> 15^\circ$ two load types are to be considered.

CASE 1: Uniform load over whole roof.

CASE 2: Uniform load over one half of roof only.

The value of the snow load shape coefficient $u_1$ is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient $u_1$ is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient $u_{1a}$ is considered to act on one half of the roof.

Angle of pitch of the roof $\alpha^\circ$  

$\alpha = 16^\circ$
Location:

Left side obstruction height  \( h_{o1} = 2.5 \text{ m} \)
Roof Length to left side  \( b_1 = 10 \text{ m} \)
Right side obstruction height  \( h_{o2} = 2 \text{ m} \)
Roof Length obstruct. to ridge  \( b_2 = 5 \text{ m} \)
Obstruction to right Wall  \( b_3 = 20 \text{ m} \)
**Location:** Roof and snow loading

**Snow loading on roof structures to BS6399-3:1988 (Inc amend't 3)**

<table>
<thead>
<tr>
<th>Snow load</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic snow load</td>
<td>$S_b = 0.5 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Site altitude less than 100 m</td>
<td></td>
</tr>
<tr>
<td>Site snow load</td>
<td>$S_o = S_b = 0.5 , \text{kN/m}^2$</td>
</tr>
</tbody>
</table>

**Flat or Monopitch Roof**

Flat or monopitch roof where the angle of pitch is $\alpha^\circ$.

Roof section

A single load case of a uniform layer of snow across the whole roof will be considered.

The value of the snow load shape coefficient $u_1$ is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient is considered to be constant across the whole roof.

Angle of pitch of the roof $\alpha^\circ = 16^\circ$
Location:

- Canopy to roof height: $h_{o1} = 2.5 \text{ m}$
- Span of canopy: $b_1 = 2 \text{ m}$
- Length of building: $b_2 = 10 \text{ m}$
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l  \( S_b = 0.5 \, \text{kN/m}^2 \)
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load  \( S_k = 0.5 \, \text{kN/m}^2 \)

For UK designs the following exposure and thermal coefficient values as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003 will be adopted for all topographies and all roofing materials:

Exposure coefficient  \( C_e = 1.0 \)
Thermal coefficient  \( C_t = 1.0 \)

Symmetrical pitched Roof

Two load types will be considered. The pitch angle is > 15°

CASE 1: Uniform load over whole roof

CASE 2: Uniform load over half the roof only

The value of the snow load shape coefficient  \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient  \( u_1 \) is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient  \( u_{1a} \) is considered to act over half the roof only.
Angle of roof pitch $\alpha^\circ$  
$\alpha = 16^\circ$

Location:

Frame 1 half span length  
$b_1 = 10$ m

Frame 2 half span length  
$b_2 = 20$ m

Max ridge to outer eaves length  
$b_3 = 50$ m

Frame 1 Valley to ridge height  
$h_{o1} = 2.5$ m

Frame 2 Valley to ridge height  
$h_{o2} = 3.2$ m
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l  $S_b=0.8 \text{ kN/m}^2$
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load  $S_k=0.8 \text{ kN/m}^2$
For UK designs the following exposure and thermal coefficient values
as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003
will be adopted for all topographies and all roofing materials:
Exposure coefficient  $C_e=1.0$
Thermal coefficient  $C_t=1.0$

Symmetrical pitched Roof

Two load types will be considered. The pitch angle is $>15^\circ$

CASE 1: Uniform load over whole roof

CASE 2: Uniform load over half the roof only

The value of the snow load shape coefficient $u_1$ is dependent upon
the angle of pitch of the roof measured from the horizontal. This
coefficient $u_1$ is considered to be constant across the whole roof
in load case 1 and in load case 2 the coefficient $u_{1a}$ is considered
to act over half the roof only.
Angle of roof pitch $\alpha^\circ$  $\alpha=75^\circ$
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l $S_b=0.8 \text{ kN/m}^2$
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load $S_k=0.8 \text{ kN/m}^2$
For UK designs the following exposure and thermal coefficient values as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003 will be adopted for all topographies and all roofing materials:
Exposure coefficient $C_e=1.0$
Thermal coefficient $C_t=1.0$

Asymmetrical pitched roof

The value of the snow load shape coefficient $u_1$ is dependent upon the angle of pitch of the roof measured from the horizontal.

Asymmetrical pitched roof

Roof section

Angle of left roof pitch $\alpha_l=75^\circ$
Angle of right roof pitch $\alpha_r=25^\circ$
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l  \( S_b = 0.5 \) kN/m²  
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load  \( S_k = 0.5 \) kN/m²

For UK designs the following exposure and thermal coefficient values as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003 will be adopted for all topographies and all roofing materials:

- Exposure coefficient  \( C_e = 1.0 \)
- Thermal coefficient  \( C_t = 1.0 \)

**Symmetrical pitched Roof**

Two load types will be considered. The pitch angle is > 15°

Roof section

CASE 1: Uniform load over whole roof

CASE 2: Uniform load over half the roof only

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient \( u_1 \) is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient \( u_{1a} \) is considered to act over half the roof only.
Angle of roof pitch $\alpha^\circ$  alpha=16°

Location:

Difference in eaves levels  ho1=5 m
Length of lower building(s)  b1=30 m
Width of higher building  b2=10 m
Length of lower building  b3=10 m
Low level roof slope  alpha6=15°
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l Sb=0.5 kN/m²
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load Sk=0.5 kN/m²

For UK designs the following exposure and thermal coefficient values as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003 will be adopted for all topographies and all roofing materials:
Exposure coefficient Ce=1.0
Thermal coefficient Ct=1.0

Symmetrical pitched Roof

Two load types will be considered. The pitch angle is > 15°

CASE 1: Uniform load over whole roof

CASE 2: Uniform load over half the roof only

The value of the snow load shape coefficient u1 is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient u1 is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient ula is considered to act over half the roof only.
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of roof pitch $\alpha^\circ$</td>
<td>$\alpha=16^\circ$</td>
</tr>
<tr>
<td>Difference in eaves levels $h_01$</td>
<td>2.5 m</td>
</tr>
<tr>
<td>Length of lower building(s) $b_1$</td>
<td>10 m</td>
</tr>
<tr>
<td>Width of higher building $b_2$</td>
<td>20 m</td>
</tr>
<tr>
<td>Angle of pitch of low level roof $\alpha_7$</td>
<td>30$^\circ$</td>
</tr>
</tbody>
</table>
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l  \( S_b = 0.5 \text{ kN/m}^2 \)
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load  \( S_k = 0.5 \text{ kN/m}^2 \)
For UK designs the following exposure and thermal coefficient values
as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003
will be adopted for all topographies and all roofing materials:
Exposure coefficient  \( C_e = 1.0 \)
Thermal coefficient  \( C_t = 1.0 \)

Symmetrical pitched Roof

Two load types will be considered. The pitch angle is > 15°

CASE 1: Uniform load over whole roof

CASE 2: Uniform load over half the roof only

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient \( u_1 \) is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient \( u_{1a} \) is considered to act over half the roof only.
Angle of roof pitch $\alpha^\circ$  
alpha=16°

Location:

Difference in eaves levels  $h_{o1}=2.5$ m
Length of lower building(s)  $b_{1}=10$ m
Width of higher building  $b_{2}=20$ m
Angle of pitch of low level roof  $\alpha_{7}=30^\circ$
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l  $S_b=0.5 \text{ kN/m}^2$
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load  $S_k=0.5 \text{ kN/m}^2$
For UK designs the following exposure and thermal coefficient values as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003 will be adopted for all topographies and all roofing materials:
Exposure coefficient  $C_e=1.0$
Thermal coefficient  $C_t=1.0$

Flat or monopitch roof - BS EN 1991-1-3:2003, Table 5.2

Flat or monopitch roof where the pitch angle is $\alpha^\circ$

For this type of roof it is necessary to consider a single load case of a uniform layer of snow across the whole roof.

The value of the snow load shape coefficient $u_1$ is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient is considered to be constant across the whole roof.

Angle of roof pitch $\alpha^\circ$  $\alpha=16^\circ$
Location:

Difference in eaves levels      \( h_{01} = 2.5 \text{ m} \)
Length of lower building(s)    \( b_1 = 10 \text{ m} \)
Width of higher building        \( b_2 = 20 \text{ m} \)
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l  \( S_b = 0.5 \text{ kN/m}^2 \)
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load  \( S_k = 0.5 \text{ kN/m}^2 \)
For UK designs the following exposure and thermal coefficient values as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003 will be adopted for all topographies and all roofing materials:
Exposure coefficient  \( C_e = 1.0 \)
Thermal coefficient  \( C_t = 1.0 \)

Flat or monopitch roof - BS EN 1991-1-3:2003, Table 5.2

Flat or monopitch roof where the pitch angle is \( \alpha \)

Roof section

For this type of roof it is necessary to consider a single load case of a uniform layer of snow across the whole roof

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient is considered to be constant across the whole roof.
Angle of roof pitch \( \alpha \)  \( \alpha = 16^\circ \)
Location:

Difference in eaves levels  \( h_{ol} = 2.5 \) m
Length of lower building(s)  \( b_1 = 10 \) m
Width of higher building  \( b_2 = 20 \) m
Location: Roof and Snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l  $S_b=0.5$ kN/m²
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load  $S_k=0.5$ kN/m²
For UK designs the following exposure and thermal coefficient values
as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003
will be adopted for all topographies and all roofing materials:
Exposure coefficient  $C_e=1.0$
Thermal coefficient  $C_t=1.0$

Flat or monopitch roof - BS EN 1991-1-3:2003, Table 5.2

Flat or monopitch roof where
the pitch angle is $\alpha^\circ$

Roof section

For this type of roof it is
necessary to consider a single
load case of a uniform layer
of snow across the whole roof

The value of the snow load shape coefficient $u_1$ is dependent upon
the angle of pitch of the roof measured from the horizontal.
This coefficient is considered to be constant across the whole roof.
Angle of roof pitch $\alpha^\circ$  $\alpha=16^\circ$
### Location:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parapet height</td>
<td>$h_{01}=2.5$ m</td>
</tr>
<tr>
<td>Length of building</td>
<td>$b_1=10$ m</td>
</tr>
<tr>
<td>Low level roof slope</td>
<td>$\alpha_6=12.5^\circ$</td>
</tr>
</tbody>
</table>
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l  \( S_b = 0.5 \) kN/m²
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load  \( S_k = 0.5 \) kN/m²

For UK designs the following exposure and thermal coefficient values as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003 will be adopted for all topographies and all roofing materials:
Exposure coefficient  \( C_e = 1.0 \)
Thermal coefficient  \( C_t = 1.0 \)

Flat or monopitch roof - BS EN 1991-1-3:2003, Table 5.2

Flat or monopitch roof where the pitch angle is \( \alpha \)°

Roof section

\[ u_1 \]

For this type of roof it is necessary to consider a single load case of a uniform layer of snow across the whole roof

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient is considered to be constant across the whole roof.
Angle of roof pitch \( \alpha \)°  \( \text{alpha} = 16^\circ \)
Location:

Parapet height \( h_{o1} = 2.5 \text{ m} \)
Length of building \( b_1 = 10 \text{ m} \)
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l $S_b=0.5 \text{ kN/m}^2$
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load $S_k=0.5 \text{ kN/m}^2$
For UK designs the following exposure and thermal coefficient values as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003 will be adopted for all topographies and all roofing materials:

- Exposure coefficient $C_e=1.0$
- Thermal coefficient $C_t=1.0$

Symmetrical pitched Roof

Two load types will be considered. The pitch angle is $>15^\circ$

CASE 1: Uniform load over whole roof

CASE 2: Uniform load over half the roof only

The value of the snow load shape coefficient $u_1$ is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient $u_1$ is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient $u_{1a}$ is considered to act over half the roof only.
Angle of roof pitch $\alpha^\circ$  
alpha=16°

Location:

Parapet height  $h_{o1}$=2.5 m
Parapet to ridge length  $b_1$=10 m
Building width  $b_2$=20 m
**Location:** Roof and snow loading

**Snow loading on roof structures to BS EN 1991-1-3:2003**

Ground snow load at 100m a.m.s.l  \( S_b = 0.5 \) kN/m\(^2\)

Site altitude is less than 100 m above mean sea level.

Characteristic ground snow load  \( S_k = 0.5 \) kN/m\(^2\)

For UK designs the following exposure and thermal coefficient values as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003 will be adopted for all topographies and all roofing materials:

- Exposure coefficient  \( C_e = 1.0 \)
- Thermal coefficient  \( C_t = 1.0 \)

**Symmetrical pitched Roof**

Two load types will be considered. The pitch angle is > 15°

CASE 1: Uniform load over whole roof

CASE 2: Uniform load over half the roof only

The value of the snow load shape coefficient \( u_1 \) is dependent upon the angle of pitch of the roof measured from the horizontal. This coefficient \( u_1 \) is considered to be constant across the whole roof in load case 1 and in load case 2 the coefficient \( u_{1a} \) is considered to act over half the roof only.
Angle of roof pitch $\alpha$  

$\alpha = 16^\circ$

Location:

Eaves to ridge height

$ho_1 = 2.5 \, \text{m}$

$b_1 = 20 \, \text{m}$

$b_2 = 10 \, \text{m}$

Slope of roof to building B

$\alpha = 16^\circ$

$h = \tan(\text{RAD}(\alpha)) \times \left(\frac{ls_12}{ls_1 \_2}\right)$

$= 1.0753 \, \text{m}$
**Location:** Roof and snow loading

**Snow loading on roof structures to BS EN 1991-1-3:2003**

Ground snow load at 100m a.m.s.l $S_b=0.5$ kN/m²
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load $S_k=0.5$ kN/m²
For UK designs the following exposure and thermal coefficient values as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003 will be adopted for all topographies and all roofing materials:

- Exposure coefficient $C_e=1.0$
- Thermal coefficient $C_t=1.0$

**Flat or monopitch roof - BS EN 1991-1-3:2003, Table 5.2**

Flat or monopitch roof where the pitch angle is $\alpha^\circ$

![Roof section diagram](image)

For this type of roof it is necessary to consider a single load case of a uniform layer of snow across the whole roof

The value of the snow load shape coefficient $u_1$ is dependent upon the angle of pitch of the roof measured from the horizontal.
This coefficient is considered to be constant across the whole roof.

Angle of roof pitch $\alpha^\circ = 16^\circ$
Location:

Left side obstruction height \( h_{o1} = 2.5 \text{ m} \)
Roof Length to left side \( b_1 = 10 \text{ m} \)
Right side obstruction height \( h_{o2} = 2 \text{ m} \)
Roof Length to right side \( b_2 = 20 \text{ m} \)
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l  $S_b=0.5$ kN/m$^2$
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load  $S_k=0.5$ kN/m$^2$
For UK designs the following exposure and thermal coefficient values
as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003
will be adopted for all topographies and all roofing materials:
Exposure coefficient  $C_e=1.0$
Thermal coefficient  $C_t=1.0$

Symmetrical pitched Roof

Two load types will be considered. The pitch angle is $>15^\circ$

Roof section

CASE 1: Uniform load over whole roof

CASE 2: Uniform load over half the roof only

The value of the snow load shape coefficient $u_l$ is dependent upon
the angle of pitch of the roof measured from the horizontal. This
coefficient $u_l$ is considered to be constant across the whole roof
in load case 1 and in load case 2 the coefficient $u_{la}$ is considered
to act over half the roof only.
Angle of roof pitch $\alpha^\circ$  
alpha=16°

Location:

Left side obstruction height  ho1=2.5 m  
Roof Length to left side  b1=10 m  
Right side obstruction height  ho2=2 m  
Roof Length obstruct. to ridge  b2=5 m  
Obstruction to right Wall  b3=20 m
Location: Roof and snow loading

Snow loading on roof structures to BS EN 1991-1-3:2003

Ground snow load at 100m a.m.s.l  $S_b=0.5$ kN/m$^2$
Site altitude is less than 100 m above mean sea level.
Characteristic ground snow load  $S_k=0.5$ kN/m$^2$
For UK designs the following exposure and thermal coefficient values
as per Clauses NA.2.15 and NA.2.16 in the NA to BS EN 1991-1-3:2003
will be adopted for all topographies and all roofing materials:
Exposure coefficient  $C_e=1.0$
Thermal coefficient  $C_t=1.0$

Flat or monopitch roof - BS EN 1991-1-3:2003, Table 5.2

![Flat or monopitch roof diagram]

Flat or monopitch roof where
the pitch angle is $\alpha^\circ$

For this type of roof it is
necessary to consider a single
load case of a uniform layer
of snow across the whole roof

The value of the snow load shape coefficient $u_1$ is dependent upon
the angle of pitch of the roof measured from the horizontal.
This coefficient is considered to be constant across the whole roof.
Angle of roof pitch $\alpha^\circ$  $\alpha=16^\circ$
Location:

Canopy to roof height       ho1=2.5 m
Span of canopy              b1=2 m
Length of building          b2=10 m
Location: Wind loads

Displacement heights

\[ \text{Ho} = \text{Mean height of other buildings} \]

Building reference height \( H_r = 13.77 \text{ m} \)
Upwind spacing of building \( X_o = 20 \text{ m} \)
Displacement height \( H_{dw0} = 3.8 \text{ m} \)
Displacement height \( H_{dw90} = 3.8 \text{ m} \)
Building type factor \( K_b = 2 \)
Building height \( H = 13.77 \text{ m} \)

Standard wind loads

Basic wind speed \( V_b = 21 \text{ m/s} \)
Site altitude above mean sea level \( \Delta S = 25 \text{ m} \)
Direction factor \( S_{dw0} = 1 \)
Direction factor \( S_{dw90} = 0.73 \)

<table>
<thead>
<tr>
<th>EFFECTIVE HEIGHT</th>
<th>9.97 m</th>
<th>9.97 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>LONG FACE</td>
<td></td>
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<tr>
<td>SHORT FACE</td>
<td></td>
<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>ALTITUDE FACTOR</th>
<th>1.025</th>
<th>1.025</th>
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<td></td>
</tr>
<tr>
<td>SHORT FACE</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>DESIGN SUMMARY</th>
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<th></th>
</tr>
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<tbody>
<tr>
<td>LONG FACE</td>
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<tr>
<td>SHORT FACE</td>
<td></td>
<td></td>
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<tr>
<td>DIRECTION FACTOR</td>
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<td></td>
</tr>
<tr>
<td>SEASONAL FACTOR</td>
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<tr>
<td>PROBABILITY FACTOR</td>
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<tr>
<td>DYNAMIC WIND PRESSURE</td>
<td>0.8461 kN/m²</td>
<td>0.4509 kN/m²</td>
</tr>
</tbody>
</table>

SCALE 5.48 Office 1007 Proforma 702
Dynamic pressure & pressure coeffs on walls of rectangular buildings

Greater horiz. dimens of building  L=60 m
Lesser horiz. dimens of building  W=20 m
Height of wall including parapet  Hr' = 13.77 m

Wind acting on long face of building:
Dynamic pressure based on height above ground 9.97 m.
Dynamic pressure @ eaves  qew0 = 0.8461 kN/m²

Wind acting on short face of building:
Dynamic pressure based on height above ground 9.97 m.
Dynamic pressure @ eaves  qew90 = 0.4509 kN/m²

Case 1: Wind on long face of building (i.e. angle Theta = 0°)

Key zones for wall pressure data.
Scaling width > D
H = 13.77 m  27.54 m > 20 m
Case 2: Wind on short face of building (i.e. angle $\Theta = 90^\circ$)

Windward (front) face
Leeward (rear) face
Isolated Zone A side face
Isolated Zone B side face

<table>
<thead>
<tr>
<th>Wall net pressure $C_{pi}=0.2$</th>
<th>Wind on long face of building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>$N_{1LW1} = PeLW1 - Pi1ww0 = 0.5181 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>$N_{1LW2} = PeLW2 - Pi1ww0 = -0.5923 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N_{1LW3} = PeLW3 - Pi1ww0 = -1.269 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N_{1LW4} = PeLW4 - Pi1ww0 = -0.8461 , \text{kN/m}^2$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wall net pressure $C_{pi}=-0.3$</th>
<th>Wind on short face of building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>$N_{1SW1} = PeSW1 - Pi1ww90 = 0.1804 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>$N_{1SW2} = PeSW2 - Pi1ww90 = -0.3156 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N_{1SW3} = PeSW3 - Pi1ww90 = -0.6764 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N_{1SW4} = PeSW4 - Pi1ww90 = -0.4509 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>$N_{1SW5} = PeSW5 - Pi1ww90 = -0.3156 , \text{kN/m}^2$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wall net pressure $C_{pi}=-0.3$</th>
<th>Wind on long face of building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>$N_{2LW1} = PeLW1 - Pi2ww0 = 0.9411 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>$N_{2LW2} = PeLW2 - Pi2ww0 = -0.1692 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N_{2LW3} = PeLW3 - Pi2ww0 = -0.8461 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N_{2LW4} = PeLW4 - Pi2ww0 = -0.4231 , \text{kN/m}^2$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wall net pressure $C_{pi}=-0.3$</th>
<th>Wind on short face of building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>$N_{2SW1} = PeSW1 - Pi2ww90 = 0.4058 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>$N_{2SW2} = PeSW2 - Pi2ww90 = -0.0902 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N_{2SW3} = PeSW3 - Pi2ww90 = -0.4509 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N_{2SW4} = PeSW4 - Pi2ww90 = -0.2255 , \text{kN/m}^2$</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>$N_{2SW5} = PeSW5 - Pi2ww90 = -0.0902 , \text{kN/m}^2$</td>
</tr>
</tbody>
</table>
External pressure coefficients $C_{pe}$ for duopitch roofs of rectangular clad buildings to Table 10 of BS6399-2:1997 (June 02)

Case 1: Wind angle Theta = 0°

Roof slope to horizontal $\alpha = 30°$

Case 2: Wind angle Theta = 90°

Roof net pressure $C_{pi}=0.2$  

| Roof zone A    | $N_{1RA0} = P_{RA0} - P_{iW1w0} = -0.5923$ kN/m² |
| Roof zone B    | $N_{1RB0} = P_{RB0} - P_{iW1w0} = -0.5923$ kN/m² |
| Roof zone C    | $N_{1RC0} = P_{RC0} - P_{iW1w0} = -0.3384$ kN/m² |
| Roof zone E    | $N_{1RE0} = P_{RE0} - P_{iW1w0} = -0.9307$ kN/m² |
| Roof zone F    | $N_{1RF0} = P_{RF0} - P_{iW1w0} = -0.5923$ kN/m² |
| Roof zone G    | $N_{1RG0} = P_{RG0} - P_{iW1w0} = -0.5923$ kN/m² |
| Roof zone A (+Ve Value) | $N_{1RAA0} = P_{RAA0} - P_{iW1w0} = 0.5077$ kN/m² |
| Roof zone B (+Ve Value) | $N_{1RAB0} = P_{RAB0} - P_{iW1w0} = 0.2538$ kN/m² |
| Roof zone C (+Ve Value) | $N_{1RAC0} = P_{RAC0} - P_{iW1w0} = 0.1692$ kN/m² |
| Roof zone E (+Ve Value) | $N_{1RAE0} = P_{RAE0} - P_{iW1w0} = 0.9307$ kN/m² |
| Roof zone F (+Ve Value) | $N_{1RAF0} = P_{RAF0} - P_{iW1w0} = 0.5923$ kN/m² |
| Roof zone G (+Ve Value) | $N_{1RAG0} = P_{RAG0} - P_{iW1w0} = 0.5923$ kN/m² |
### Roof net pressure Cpi=0.2

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Wind load calculation</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>N1RA9=PeRA9-PiW1w90</td>
<td>-0.6313 kN/m²</td>
</tr>
<tr>
<td>B</td>
<td>N1RB9=PeRB9-PiW1w90</td>
<td>-0.5862 kN/m²</td>
</tr>
<tr>
<td>C</td>
<td>N1RC9=PeRC9-PiW1w90</td>
<td>-0.3607 kN/m²</td>
</tr>
<tr>
<td>D</td>
<td>N1RD9=PeRD9-PiW1w90</td>
<td>-0.3156 kN/m²</td>
</tr>
</tbody>
</table>

### Roof net pressure Cpi=-0.3

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Wind load calculation</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>N2RA0=PeRA0-PiW2w0</td>
<td>-0.1692 kN/m²</td>
</tr>
<tr>
<td>B</td>
<td>N2RB0=PeRB0-PiW2w0</td>
<td>-0.1692 kN/m²</td>
</tr>
<tr>
<td>C</td>
<td>N2RC0=PeRC0-PiW2w0</td>
<td>0.0846 kN/m²</td>
</tr>
<tr>
<td>E</td>
<td>N2RE0=PeRE0-PiW2w0</td>
<td>-0.5077 kN/m²</td>
</tr>
<tr>
<td>F</td>
<td>N2RF0=PeRF0-PiW2w0</td>
<td>-0.1692 kN/m²</td>
</tr>
<tr>
<td>G</td>
<td>N2RG0=PeRG0-PiW2w0</td>
<td>-0.1692 kN/m²</td>
</tr>
<tr>
<td>A (+Ve Value)</td>
<td>N2RAA0=PRAA0-PiW2w0</td>
<td>0.9307 kN/m²</td>
</tr>
<tr>
<td>B (+Ve Value)</td>
<td>N2RAB0=PRAB0-PiW2w0</td>
<td>0.6769 kN/m²</td>
</tr>
<tr>
<td>C (+Ve Value)</td>
<td>N2RAC0=PRAC0-PiW2w0</td>
<td>0.5923 kN/m²</td>
</tr>
<tr>
<td>E (+Ve Value)</td>
<td>N2RAE0=PRAE0-PiW2w0</td>
<td>-0.5077 kN/m²</td>
</tr>
<tr>
<td>F (+Ve Value)</td>
<td>N2RAF0=PRAF0-PiW2w0</td>
<td>-0.1692 kN/m²</td>
</tr>
<tr>
<td>G (+Ve Value)</td>
<td>N2RAG0=PRAG0-PiW2w0</td>
<td>-0.1692 kN/m²</td>
</tr>
</tbody>
</table>

### Roof net pressure Cpi=-0.3

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Wind load calculation</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>N2RA9=PeRA9-PiW2w90</td>
<td>-0.4058 kN/m²</td>
</tr>
<tr>
<td>B</td>
<td>N2RB9=PeRB9-PiW2w90</td>
<td>-0.3607 kN/m²</td>
</tr>
<tr>
<td>C</td>
<td>N2RC9=PeRC9-PiW2w90</td>
<td>-0.1353 kN/m²</td>
</tr>
<tr>
<td>D</td>
<td>N2RD9=PeRD9-PiW2w90</td>
<td>-0.0902 kN/m²</td>
</tr>
</tbody>
</table>
Location: Default case 2 Flat roof; topography critical

Displacement heights

<table>
<thead>
<tr>
<th>Wind</th>
<th>6Ho</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Ho</td>
<td></td>
</tr>
<tr>
<td>0.8Ho</td>
<td></td>
</tr>
<tr>
<td>He</td>
<td></td>
</tr>
<tr>
<td>2Ho</td>
<td></td>
</tr>
<tr>
<td>Ho=Mean height of other buildings</td>
<td></td>
</tr>
<tr>
<td>Xo</td>
<td></td>
</tr>
</tbody>
</table>

Building reference height Hr=18 m
Displacement height Hdw0=0 m
Displacement height Hdw90=0 m
Building type factor Kb=2
Building height H=18 m

Standard wind loads

Basic wind speed Vb=22.5 m/s
Site altitude above mean sea level deltaS=35 m

Topography is significant - Wind on long face of building

Slope details and site location relative to crest

Site altitude is 35 m above mean sea level.
Xt is positive if site is located downwind of crest.
Xt is negative if site is located upwind of crest.
Direction factor           Sdw0=0.85
Direction factor           Sdw90=0.85
Seasonal factor            Ss=0.86
Probability factor         Sp=1.05

Effective height (long face) Hew0 18 m
Effective height (short face) Hew90 18 m
Altitude factor (long face) Saw0 1.113
Altitude factor (short face) Saw90 1.035
Direction factor (long face) Sdw0 0.85
Direction factor (short face) Sdw90 0.85
Seasonal factor            Ss 0.86
Probability factor         Sp 1.05
Dynamic wind pressure (long face) qsw0 0.7996 kN/m²
Dynamic wind pressure (short face) qsw90 0.6915 kN/m²

Dynamic pressure & pressure coeffs on walls of rectangular buildings

Greater horiz.dimens of building  L=150 m
Lesser horiz.dimens of building  W=40.5 m
Height of wall including parapet Hr'=18 m
Wind acting on long face of building:
Dynamic pressure based on height above ground 18 m.
Dynamic pressure @ eaves qew0=0.7996 kN/m²
Wind acting on short face of building:
Dynamic pressure based on height above ground 18 m.
Dynamic pressure @ eaves qew90=0.6915 kN/m²
Diagonal dimension of Ext long wall dima1=25 m
Diagonal dimension of Ext short wall dima2=35 m

Case 1: Wind on long face of building (i.e. angle Theta = 0°)

PLAN VIEW
Case 2: Wind on short face of building (i.e. angle Theta = 90°)

Inwind depth \( D = 150 \) m
Cross wind width \( B = 40.5 \) m

Internal volume of storey \( \text{SinVol}=40000 \) m³
External pressure coefficients \( C_{pe} \) for flat roofs of rectangular clad buildings to Table 8 of BS6399-2:1997 (June 02)

**PLAN VIEW**

Wind on long face \( (\theta=0^\circ) \)

- Inwind depth: \( D=W=40.5 \text{ m} \)
- Cross wind width: \( B=L=150 \text{ m} \)

**PLAN VIEW**

Wind on short face \( (\theta=90^\circ) \)

- Inwind depth: \( D=L=150 \text{ m} \)
- Cross wind width: \( B=W=40.5 \text{ m} \)

Diagonal dimension of roof at 0 degree \( \text{dima4}=5 \text{ m} \)
Diagonal dimension of roof at 90 degree \( \text{dima5}=5 \text{ m} \)
Location: Default case 3 mono roof; topography critical

Displacement heights

\[ \text{Profile of displacement height} \]

\[ \text{Wind} \]

\[ \text{6Ho} \]

\[ \text{2Ho} \]

\[ \text{Ho} \]

\[ \text{0.8Ho} \]

\[ \text{He} \]

\[ \text{Hd} \]

\[ \text{Hr} \]

\[ Ho = \text{Mean height of other buildings} \]

\[ Xo \]

Building reference height \( \text{Hr}=18.5 \text{ m} \)

Upwind of site \( \text{Ho}=6.5 \text{ m} \)

Upwind spacing of building \( \text{Xo}=20 \text{ m} \)

Displacement height \( \text{Hdw0}=6.5 \text{ m} \)

Displacement height \( \text{Hdw90}=6.5 \text{ m} \)

Building type factor \( \text{Kb}=8 \)

Building height \( \text{H}=18.5 \text{ m} \)

Standard wind loads

Basic wind speed \( \text{Vb}=24 \text{ m/s} \)

Site altitude above mean sea level \( \text{deltaS}=30 \text{ m} \)

Topography is significant - Wind on long face of building

Slope details and site location relative to crest

\[ \text{Xt}<0 \] \[ \text{Xt}>0 \]

\[ \text{Site} \]

\[ \text{crest} \]

\[ \text{Z}=5 \text{ m} \]

Site altitude is 30 m above mean sea level.

Xt is positive if site is located downwind of crest.

Xt is negative if site is located upwind of crest.
Loadings and foundations  
Made by: IFB  
Date: 02/12/19  
Ref No: SC702 BS

Direction factor  
Sdw0=0.85  
Sdw90=0.85  
Seasonal factor  
Ss=0.66  
Probability factor  
Sp=1.05

Effective height (long face)  
Hew0  12 m  
Effective height (short face)  
Hew90  12 m  
Altitude factor (long face)  
Saw0  1.09  
Altitude factor (short face)  
Saw90  1.03  
Direction factor (long face)  
Sdw0  0.85  
Direction factor (short face)  
Sdw90  0.85  
Seasonal factor  
Ss  0.66  
Probability factor  
Sp  1.05  
Dynamic wind pressure (long face)  
qsw0  0.4172 kN/m²  
Dynamic wind pressure (short face)  
qsw90  0.3725 kN/m²

Dynamic pressure & pressure coeffs on walls of rectangular buildings

Greater horiz.dimens of building  
L=40 m  
Lesser horiz.dimens of building  
W=20 m  
Height of wall including parapet  
Hr'=18.5 m  
Wind acting on long face of building:  
Dynamic pressure at height Hr'  
qew0=0.613*Veew0^2/1000=0.4806 kN/m²  
Wind acting on short face of building:  
Dynamic pressure at height Hr'  
qew90=0.613*Veew90^2/1000=0.4291 kN/m²

Case 1: Wind on long face of building (i.e. angle Theta = 0°)

![Plan View Diagram]

![Elevation on Side Face Diagram]
**Case 2: Wind on short face of building (i.e. angle $\Theta = 90^\circ$)**

**PLAN VIEW**

- Inwind depth $D=40$ m
- Cross wind width $B=20$ m

**ELEVATION ON SIDE FACE**

- $0.75$ times $Cpe$
- **Internal pressure coefficient 1** $Cpi(1)=-0.8$
- **Internal pressure coefficient 2** $Cpi(2)=0.3$

**Wall net pressure $Cpi=-0.8$**

- **Windward (front) face**
  - $N1LW1=PeLW1-Pi1ww0=0.7897$ kN/m
- **Leeward (rear) face**
  - $N1LW2=PeLW2-Pi1ww0=0.1442$ kN/m
- **Isolated Zone A side face**
  - $N1LW3=PeLW3-Pi1ww0=-0.2403$ kN/m
- **Isolated Zone B side face**
  - $N1LW4=PeLW4-Pi1ww0=0$ kN/m

**Wall net pressure $Cpi=0.3$**

- **Windward (front) face**
  - $N1SW1=PeSW1-Pi1ww90=0.6665$ kN/m
- **Leeward (rear) face**
  - $N1SW2=PeSW2-Pi1ww90=0.1287$ kN/m
- **Isolated Zone A side face**
  - $N1SW3=PeSW3-Pi1ww90=-0.2146$ kN/m
- **Isolated Zone B side face**
  - $N1SW4=PeSW4-Pi1ww90=0$ kN/m
- **Isolated Zone C side face**
  - $N1SW5=PeSW5-Pi1ww90=0.1287$ kN/m
Windward (front) face: N2SW1 = PeSW1 - Pi2ww90 = 0.1945 kN/m²
Leeward (rear) face: N2SW2 = PeSW2 - Pi2ww90 = -0.3433 kN/m²
Isolated Zone A side face: N2SW3 = PeSW3 - Pi2ww90 = -0.6866 kN/m²
Isolated Zone B side face: N2SW4 = PeSW4 - Pi2ww90 = -0.472 kN/m²
Isolated Zone C side face: N2SW5 = PeSW5 - Pi2ww90 = -0.3433 kN/m²

Coefficients Cpe for monopitch roofs of rectangular clad

read in conjunction with BS6399-2:1997 (June 2002)

Case 1: Wind angle Theta = 0°

Case 3: Wind angle Theta = 90°
### Roof net pressure $C_{pi}=-0.8$ Wind at 0° (long face)

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Loadings</th>
<th>Foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone A</td>
<td>$N_{1RA0} = P_{RA0} - P_{iW1w0} = 0.3338$ kN/m²</td>
<td>Made by: IFB</td>
</tr>
<tr>
<td>Zone B</td>
<td>$N_{1RB0} = P_{RB0} - P_{iW1w0} = 0.3338$ kN/m²</td>
<td>Date: 02/12/19</td>
</tr>
<tr>
<td>Zone C</td>
<td>$N_{1RC0} = P_{RC0} - P_{iW1w0} = 0.3338$ kN/m²</td>
<td>Ref No: SC702 BS</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Loadings</th>
<th>Foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone A (+Ve Value)</td>
<td>$N_{1RAA0} = P_{RAA0} - P_{iW1w0} = 0.6675$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone B (+Ve Value)</td>
<td>$N_{1RAB0} = P_{RAB0} - P_{iW1w0} = 0.6675$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone C (+Ve Value)</td>
<td>$N_{1RAC0} = P_{RAC0} - P_{iW1w0} = 0.6675$ kN/m²</td>
<td></td>
</tr>
</tbody>
</table>

### Roof net pressure $C_{pi}=-0.8$ Wind at 90° (short face)

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Loadings</th>
<th>Foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone Au</td>
<td>$N_{1Au9} = P_{RA9} - P_{iW1w90} = 0.149$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone Al</td>
<td>$N_{1Al9} = P_{RA9} - P_{iW1w90} = 0.149$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone B</td>
<td>$N_{1RB9} = P_{RB9} - P_{iW1w90} = 0.149$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone C</td>
<td>$N_{1RC9} = P_{RC9} - P_{iW1w90} = 0.149$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone D</td>
<td>$N_{1RD9} = P_{RD9} - P_{iW1w90} = 0.2235$ kN/m²</td>
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</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Loadings</th>
<th>Foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone Au (+ve value)</td>
<td>$N_{1Rup9} = P_{Ru9} - P_{iW1w90} = 0.5961$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone Al (+ve value)</td>
<td>$N_{1Rlp9} = P_{Ru9} - P_{iW1w90} = 0.5961$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone B (+ve value)</td>
<td>$N_{1RBp9} = P_{Ru9} - P_{iW1w90} = 0.5961$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone C (+ve value)</td>
<td>$N_{1RCp9} = P_{Ru9} - P_{iW1w90} = 0.5588$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone D (+ve value)</td>
<td>$N_{1RDp9} = P_{Ru9} - P_{iW1w90} = 0.5216$ kN/m²</td>
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</tr>
</tbody>
</table>

### Roof net pressure $C_{pi}=0.3$ Wind at 0° (long face)

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Loadings</th>
<th>Foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone A</td>
<td>$N_{2RA0} = P_{RA0} - P_{iW2w0} = 0.1252$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone B</td>
<td>$N_{2RB0} = P_{RB0} - P_{iW2w0} = 0.0417$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone C</td>
<td>$N_{2RC0} = P_{RC0} - P_{iW2w0} = 0.0417$ kN/m²</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Loadings</th>
<th>Foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone A (+Ve Value)</td>
<td>$N_{2RAA0} = P_{RAA0} - P_{iW2w0} = 0.2086$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone B (+Ve Value)</td>
<td>$N_{2RAB0} = P_{RAB0} - P_{iW2w0} = 0.2086$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone C (+Ve Value)</td>
<td>$N_{2RAC0} = P_{RAC0} - P_{iW2w0} = 0.2086$ kN/m²</td>
<td></td>
</tr>
</tbody>
</table>

### Roof net pressure $C_{pi}=0.3$ Wind at 90° (short face)

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Loadings</th>
<th>Foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone Au upper</td>
<td>$N_{2Au9} = Pu_{RA9} - P_{iW2w90} = -0.5588$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone A lower</td>
<td>$N_{2Al9} = Pu_{RA9} - P_{iW2w90} = -0.5588$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone B</td>
<td>$N_{2RB9} = Pu_{RB9} - P_{iW2w90} = -0.5588$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone C</td>
<td>$N_{2RC9} = Pu_{RC9} - P_{iW2w90} = -0.2608$ kN/m²</td>
<td></td>
</tr>
<tr>
<td>Zone D</td>
<td>$N_{2RD9} = Pu_{RD9} - P_{iW2w90} = -0.1863$ kN/m²</td>
<td></td>
</tr>
</tbody>
</table>
Roof net pressure $C_p=0.3$  Wind at 180°

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Expression</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$N_{2RA8} = P_{ERA8} - P_{IW2w0}$</td>
<td>$-0.5841$ kN/m²</td>
</tr>
<tr>
<td>B</td>
<td>$N_{2RB8} = P_{ERB8} - P_{IW2w0}$</td>
<td>$-0.4172$ kN/m²</td>
</tr>
<tr>
<td>C</td>
<td>$N_{2RC8} = P_{ERC8} - P_{IW2w0}$</td>
<td>$-0.4172$ kN/m²</td>
</tr>
</tbody>
</table>
Location: Default case 4 Hipped roof structure

Displacement heights

<table>
<thead>
<tr>
<th>Ho</th>
<th>Mean height of other buildings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Xo</td>
<td>Upwind of site</td>
</tr>
<tr>
<td>Ho</td>
<td>6.5 m</td>
</tr>
<tr>
<td>Xo</td>
<td>10 m</td>
</tr>
<tr>
<td>Hr</td>
<td>13.8 m</td>
</tr>
</tbody>
</table>

Building reference height
Upwind spacing of building
Displacement height
Displacement height
Building type factor
Building height

Standard wind loads

| Basic wind speed | 20 m/s |
| Upwind altitude above mean sea level | 25 m |
| Direction factor (long face) | Sd0 = 1 |
| Direction factor (short face) | Sd90 = 1 |

Effective height (long face)
Effective height (short face)
Altitude factor (long face)
Altitude factor (short face)
Direction factor (long face)
Direction factor (short face)
Seasonal factor
Probability factor
Dynamic wind pressure (long face)
Dynamic wind pressure (short face)

| Hew0 | 7.3 m |
| Hew90 | 7.3 m |
| Saw0 | 1.025 |
| Saw90 | 1.025 |
| Sd0 | 1 |
| Sd90 | 1 |
| Ss | 1 |
| Sp | 1 |
| qsw0 | 0.662 kN/m² |
| qsw90 | 0.662 kN/m² |
Dynamic pressure & pressure coeffs on walls of rectangular buildings

Greater horiz.dimens of building \( L = 60 \text{ m} \)
Lesser horiz.dimens of building \( W = 30 \text{ m} \)
Height of wall including parapet \( H_r' = 13.8 \text{ m} \)

Wind acting on long face of building:
Dynamic pressure based on height above ground 7.3 m.
Dynamic pressure @ eaves \( q_{ew0} = 0.662 \text{ kN/m}^2 \)

Wind acting on short face of building:
Dynamic pressure based on height above ground 7.3 m.
Dynamic pressure @ eaves \( q_{ew90} = 0.662 \text{ kN/m}^2 \)

Case 1: Wind on long face of building (i.e. angle Theta = 0°)

![Plan View](Image)

Wind depth \( D = 30 \text{ m} \)
Cross wind width \( B = 60 \text{ m} \)

Elevation on side face

![Elevation](Image)

Key zones for wall pressure data.

Case 2: Wind on short face of building (i.e. angle Theta = 90°)

![Plan View](Image)

Wind depth \( D = 60 \text{ m} \)
Cross wind width \( B = 30 \text{ m} \)
"0.75 times Cpe" "0.9*Cpe"

Internal pressure coefficient 1 \( C_{pi}(1) = -0.8 \)

### Wall net pressure \( C_{pi} = -0.8 \)

<table>
<thead>
<tr>
<th></th>
<th>Windward (front) face</th>
<th>Leeward (rear) face</th>
<th>Isolated Zone A side face</th>
<th>Isolated Zone B side face</th>
<th>Isolated Zone C side face</th>
<th>Funnelling Zone A side face</th>
<th>Funnelling Zone B side face</th>
<th>Funnelling Zone C side face</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( N_{1LW1} = P_{eLW1} - P_{i1ww0} = 1.028 \text{ kN/m}^2 )</td>
<td>( N_{1LW2} = P_{eLW2} - P_{i1ww0} = 0.1986 \text{ kN/m}^2 )</td>
<td>( N_{1LW3} = P_{eLW3} - P_{i1ww0} = -0.331 \text{ kN/m}^2 )</td>
<td>( N_{1LW4} = P_{eLW4} - P_{i1ww0} = 0 \text{ kN/m}^2 )</td>
<td>( N_{1LW5} = P_{eLW5} - P_{i1ww0} = 0.1986 \text{ kN/m}^2 )</td>
<td>( N_{1LW6} = P_{eLW6} - P_{i1ww0} = -0.5296 \text{ kN/m}^2 )</td>
<td>( N_{1LW7} = P_{eLW7} - P_{i1ww0} = -0.0662 \text{ kN/m}^2 )</td>
<td>( N_{1LW8} = P_{eLW8} - P_{i1ww0} = 0.0662 \text{ kN/m}^2 )</td>
</tr>
</tbody>
</table>

### Wall net pressure \( C_{pi} = -0.8 \)

<table>
<thead>
<tr>
<th></th>
<th>Windward (front) face</th>
<th>Leeward (rear) face</th>
<th>Isolated Zone A side face</th>
<th>Isolated Zone B side face</th>
<th>Isolated Zone C side face</th>
<th>Funnelling Zone A side face</th>
<th>Funnelling Zone B side face</th>
<th>Funnelling Zone C side face</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( N_{1SW1} = P_{eSW1} - P_{i1ww90} = 0.9268 \text{ kN/m}^2 )</td>
<td>( N_{1SW2} = P_{eSW2} - P_{i1ww90} = 0.1986 \text{ kN/m}^2 )</td>
<td>( N_{1SW3} = P_{eSW3} - P_{i1ww90} = -0.331 \text{ kN/m}^2 )</td>
<td>( N_{1SW4} = P_{eSW4} - P_{i1ww90} = 0 \text{ kN/m}^2 )</td>
<td>( N_{1SW5} = P_{eSW5} - P_{i1ww90} = 0.1986 \text{ kN/m}^2 )</td>
<td>( N_{1SW6} = P_{eSW6} - P_{i1ww90} = -0.5296 \text{ kN/m}^2 )</td>
<td>( N_{1SW7} = P_{eSW7} - P_{i1ww90} = -0.0662 \text{ kN/m}^2 )</td>
<td>( N_{1SW8} = P_{eSW8} - P_{i1ww90} = 0.0662 \text{ kN/m}^2 )</td>
</tr>
</tbody>
</table>
External pressure coefficients $C_{pe}$ for hipped roofs of rectangular clad buildings to Table 11 of BS6399-2:1997 (June 2002)

Case 1: Wind angle $\Theta = 0^\circ$

Figure 21b
Zones for wind direction $\theta=0$ deg & using $\alpha(0)$ as the angle of slope of the main roof

External pressure coefficients $C_{pe}$ for hipped roofs of rectangular clad buildings to Table 11 of BS6399-2:1997 (June 2002)

Case 1: Wind angle $\Theta = 0^\circ$

Figure 21b
Zones for wind direction $\theta=0$ deg & using $\alpha(0)$ as the angle of slope of the main roof

Main roof slope to horizontal $\alpha=22.5^\circ$

Case 2: Wind angle $\Theta=90^\circ$

Figure 21c
Zones for wind direction $\theta=90^\circ$ using $\alpha(90)$ as the angle of slope of the hipped end

Hipped end roof slope to horizontal $\alpha=30^\circ$

Roof net pressure $C_{pi}=-0.8$ Wind at $0^\circ$ (long face)

| Roof zone A | $N_{1HA0}=Pe_{HA0}-Pi_{W1w0}=-0.0662$ kN/m² |
| Roof zone B | $N_{1HB0}=Pe_{HB0}-Pi_{W1w0}=0.0993$ kN/m² |
| Roof zone C | $N_{1HC0}=Pe_{HC0}-Pi_{W1w0}=-0.2979$ kN/m² |
| Roof zone E | $N_{1HE0}=Pe_{HE0}-Pi_{W1w0}=-0.3641$ kN/m² |
| Roof zone F | $N_{1HF0}=Pe_{HF0}-Pi_{W1w0}=-0.1655$ kN/m² |
| Roof zone G | $N_{1HG0}=Pe_{HG0}-Pi_{W1w0}=0.1324$ kN/m² |
| Roof zone H | $N_{1HH0}=Pe_{HH0}-Pi_{W1w0}=-0.0993$ kN/m² |
| Roof zone I | $N_{1HI0}=Pe_{HI0}-Pi_{W1w0}=0.1324$ kN/m² |
| Roof zone A (+Ve Value) | $N_{1HAA0}=PHAA0-Pi_{W1w0}=0.8606$ kN/m² |
| Roof zone B (+Ve Value) | $N_{1HAB0}=PHAB0-Pi_{W1w0}=0.7613$ kN/m² |
### Roof net pressure $C_{pi} = -0.8$

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>$N_{1HAC9} = PH_{AC9} - \Pi_{W1w90}$</th>
<th>kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.1986 kN/m²</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>0.1986 kN/m²</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>0.3972 kN/m²</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>-0.331 kN/m²</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>0.1324 kN/m²</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>-0.1324 kN/m²</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>0.1324 kN/m²</td>
<td></td>
</tr>
<tr>
<td>J</td>
<td>0.1986 kN/m²</td>
<td></td>
</tr>
<tr>
<td>A (+Ve Value)</td>
<td>1.059 kN/m²</td>
<td></td>
</tr>
<tr>
<td>B (+Ve Value)</td>
<td>0.8606 kN/m²</td>
<td></td>
</tr>
<tr>
<td>C (+Ve Value)</td>
<td>0.7944 kN/m²</td>
<td></td>
</tr>
<tr>
<td>E (+Ve Value)</td>
<td>-0.331 kN/m²</td>
<td></td>
</tr>
<tr>
<td>G (+Ve Value)</td>
<td>0.1324 kN/m²</td>
<td></td>
</tr>
<tr>
<td>H (+Ve Value)</td>
<td>0.5296 kN/m²</td>
<td></td>
</tr>
<tr>
<td>I (+Ve Value)</td>
<td>0.5296 kN/m²</td>
<td></td>
</tr>
<tr>
<td>J (+Ve Value)</td>
<td>0.5296 kN/m²</td>
<td></td>
</tr>
</tbody>
</table>

**Wind at 90° (short face)**

---

Roof zone C (+Ve Value) $N_{1HAC0} = PH_{AC0} - \Pi_{W1w0} = 0.7282$ kN/m²

Roof zone E (+Ve Value) $N_{1HAE0} = PH_{AE0} - \Pi_{W1w0} = -0.3641$ kN/m²

Roof zone F (+Ve Value) $N_{1HAF0} = PH_{AF0} - \Pi_{W1w0} = -0.1655$ kN/m²

Roof zone G (+Ve Value) $N_{1HAG0} = PH_{AG0} - \Pi_{W1w0} = -0.3641$ kN/m²

Roof zone I (+Ve Value) $N_{1HAI0} = PH_{AI0} - \Pi_{W1w0} = 0.5296$ kN/m²

---

Roof zone A $N_{1HA9} = Pe_{HA9} - \Pi_{W1w90} = 0.1986$ kN/m²

Roof zone B $N_{1HB9} = Pe_{HB9} - \Pi_{W1w90} = 0.1986$ kN/m²

Roof zone C $N_{1HC9} = Pe_{HC9} - \Pi_{W1w90} = 0.3972$ kN/m²

Roof zone E $N_{1HE9} = Pe_{HE9} - \Pi_{W1w90} = -0.331$ kN/m²

Roof zone G $N_{1HG9} = Pe_{HG9} - \Pi_{W1w90} = 0.1324$ kN/m²

Roof zone H $N_{1HH9} = Pe_{HH9} - \Pi_{W1w90} = -0.1324$ kN/m²

Roof zone I $N_{1HI9} = Pe_{HI9} - \Pi_{W1w90} = 0.1324$ kN/m²

Roof zone J $N_{1HJ9} = Pe_{HJ9} - \Pi_{W1w90} = 0.1986$ kN/m²

Roof zone A (+Ve Value) $N_{1HAA9} = PH_{A9} - \Pi_{W1w90} = 1.059$ kN/m²

Roof zone B (+Ve Value) $N_{1HAB9} = PH_{AB9} - \Pi_{W1w90} = 0.8606$ kN/m²

Roof zone C (+Ve Value) $N_{1HAC9} = PH_{AC9} - \Pi_{W1w90} = 0.7944$ kN/m²

Roof zone E (+Ve Value) $N_{1HAE9} = PH_{AE9} - \Pi_{W1w90} = -0.331$ kN/m²

Roof zone G (+Ve Value) $N_{1HAG9} = PH_{AG9} - \Pi_{W1w90} = 0.1324$ kN/m²

Roof zone H (+Ve Value) $N_{1HAH9} = PH_{AH9} - \Pi_{W1w90} = 0.5296$ kN/m²

Roof zone I (+Ve Value) $N_{1HAI9} = PH_{AI9} - \Pi_{W1w90} = 0.5296$ kN/m²

Roof zone J (+Ve Value) $N_{1HAJ9} = PH_{AJ9} - \Pi_{W1w90} = 0.5296$ kN/m²
Location: Default case 5

Displacement heights

Wind loads to BS6399-2:1997

Building reference height
Upwind spacing of building
Displacement height
Building type factor
Building height

Standard wind loads

Basic wind speed
Site altitude above mean sea level
deltaS=25 m
Direction factor
Direction factor

Effective height (long face)
Effective height (short face)
Altitude factor (long face)
Altitude factor (short face)
Direction factor (long face)
Direction factor (short face)
Seasonal factor
Probability factor
Dynamic wind pressure (long face)
Dynamic wind pressure (short face)
Dynamic pressure & pressure coeffs on walls of rectangular buildings

Greater horiz. dimens of building  \( L = 60 \text{ m} \)
Lesser horiz. dimens of building  \( W = 30 \text{ m} \)
Height of wall including parapet  \( H_r' = 13.8 \text{ m} \)

Wind acting on long face of building:
Dynamic pressure based on height above ground 8.6 m.
Dynamic pressure @ eaves  \( q_{ew0} = 0.7124 \text{ kN/m}^2 \)

Wind acting on short face of building:
Dynamic pressure based on height above ground 8.6 m.
Dynamic pressure @ eaves  \( q_{ew90} = 0.7124 \text{ kN/m}^2 \)

Diagonal dimension of Ext long wall  \( d_{ia1} = 5 \text{ m} \)
Diagonal dimension of Ext short wall  \( d_{ia2} = 5 \text{ m} \)

Case 1: Wind on long face of building (i.e. angle \( \Theta = 0^\circ \))
Case 2: Wind on short face of building (i.e. angle Theta = 90°)

Wind depth \( D = 60 \) m
Cross wind width \( B = 30 \) m

Key zones for wall pressure data.

Internal pressure coefficient 1 \( C_{pi}(1) = -0.8 \)
Diagonal dimension (internal) \( d_{ma3} = 79.35 \) m
External pressure coefficients $C_{pe}$ for hipped roofs of rectangular clad buildings to Table 11 of BS6399-2:1997 (June 2002)

Case 1: Wind angle Theta = 0°

Figure 21b

Zones for wind direction theta=0 deg & using alpha(0) as the angle of slope of the main roof

Main roof slope to horizontal $\alpha=\alpha(0)=22.5°$

Case 2: Wind angle Theta=90°

Figure 21c

Zones for wind direction theta=90° using Wind alpha(90) as the angle of slope of the hipped end

Hipped end roof slope to horizontal $\alpha=\alpha(90)=30°$

Diagonal dimension of roof at 0 degree $d_{a4}=5$ m
Diagonal dimension of roof at 90 degree $d_{a5}=5$ m
Location: Default case 6

Displacement heights

\[
\text{Wind} \quad \text{Profile of displacement height} \quad \text{6Ho}
\]

\[
\begin{align*}
\text{Ho} & = \text{Mean height of other buildings} \\
\text{Building reference height} & = \text{Hr}=10 \text{ m} \\
\text{upwind of site} & = \text{Ho}=4 \text{ m} \\
\text{Upwind spacing of building} & = \text{Xo}=10 \text{ m} \\
\text{Displacement height} & = \text{Hdw0}=2.8 \text{ m} \\
\text{Displacement height} & = \text{Hdw90}=2.8 \text{ m} \\
\text{Building type factor} & = \text{Kb}=1 \\
\text{Building height} & = \text{H}=10 \text{ m}
\end{align*}
\]

Standard wind loads

\[
\begin{align*}
\text{Basic wind speed} & = \text{Vb}=21 \text{ m/s} \\
\text{Site altitude above mean sea level} & = \text{deltaS}=25 \text{ m}
\end{align*}
\]

Topography is significant - Wind on long face of building

Slope details and site location relative to crest

\[
\begin{align*}
\text{Site altitude is} & = 25 \text{ m above mean sea level.} \\
\text{Xt is positive if site is located downwind of crest.} \\
\text{Xt is negative if site is located upwind of crest.}
\end{align*}
\]

\[
\begin{align*}
\text{Height to crest} & = \text{Z}=5 \text{ m} \\
\text{Altitude of upwind base of topography} & = \text{deltaT}=20 \text{ m}
\end{align*}
\]
Direction factor
  Sdw0=1
  Sdw90=1

Effective height (long face)  Hew0  7.2 m
Effective height (short face) Hew90  7.2 m
Altitude factor (long face)  Saw0  1.061
Altitude factor (short face) Saw90  1.025

DESIGN
  Altitude factor (long face)  Saw0  1.061
  Altitude factor (short face) Saw90  1.025
  Direction factor (long face) Sdw0  1
  Direction factor (short face) Sdw90  1
  Seasonal factor  Ss  1
  Probability factor  Sp  1

SUMMARY
  Direction factor (long face) Sdw0  1
  Direction factor (short face) Sdw90  1
  Seasonal factor  Ss  1
  Probability factor  Sp  1
  Dynamic wind pressure (long face) qsw0  0.7636 kN/m²
  Dynamic wind pressure (short face) qsw90  0.7126 kN/m²

Dynamic pressure & pressure coeffs on walls of rectangular buildings

Greater horiz.dimens of building  L=50 m
Lesser  horiz.dimens of building  W=30 m
Height of wall including parapet  Hr'=10 m

Wind acting on long face of building:
Dynamic pressure at height Hr'  qew0=0.613*Veew0^2/1000=0.8951 kN/m²

Wind acting on short face of building:
Dynamic pressure at height Hr'  qew90=0.613*Veew90^2/1000=0.8354 kN/m²

Case 1: Wind on long face of building (i.e. angle Theta = 0°)
Case 2: Wind on short face of building (i.e. angle $\Theta = 90^\circ$)

Windward (front) face
- $N1LW1=PeLW1-Pi1ww0=0.4326$ kN/m^2
- $N1LW2=PeLW2-Pi1ww0=-0.6266$ kN/m^2
- $N1LW3=PeLW3-Pi1ww0=-1.343$ kN/m^2
- $N1LW4=PeLW4-Pi1ww0=-0.8951$ kN/m^2
- $N1LW5=PeLW5-Pi1ww0=-0.6266$ kN/m^2

Leeward (rear) face
- $N1LW2=PeLW2-Pi1ww0=-0.179$ kN/m^2
- $N1LW4=PeLW4-Pi1ww0=-0.4475$ kN/m^2
- $N1LW5=PeLW5-Pi1ww0=-0.179$ kN/m^2

Isolated Zone A side face
- $N1LW3=PeLW3-Pi1ww90=-1.253$ kN/m^2
- $N1LW4=PeLW4-Pi1ww90=-0.8354$ kN/m^2
- $N1LW5=PeLW5-Pi1ww90=-0.1671$ kN/m^2

Isolated Zone B side face
- $N1LW3=PeLW3-Pi1ww90=-1.253$ kN/m^2
- $N1LW4=PeLW4-Pi1ww90=-0.8354$ kN/m^2
- $N1LW5=PeLW5-Pi1ww90=-0.1671$ kN/m^2

Isolated Zone C side face
- $N1LW4=PeLW4-Pi1ww90=-0.8354$ kN/m^2
- $N1LW5=PeLW5-Pi1ww90=-0.1671$ kN/m^2

Wind on long face of building

Windward (front) face
- $N2LW1=PeLW1-Pi2ww0=0.8801$ kN/m^2
- $N2LW2=PeLW2-Pi2ww0=-0.179$ kN/m^2
- $N2LW3=PeLW3-Pi2ww0=-0.8951$ kN/m^2
- $N2LW4=PeLW4-Pi2ww0=-0.4475$ kN/m^2
- $N2LW5=PeLW5-Pi2ww0=-0.179$ kN/m^2

Leeward (rear) face
- $N2LW2=PeLW2-Pi2ww0=-0.179$ kN/m^2
- $N2LW4=PeLW4-Pi2ww0=-0.4475$ kN/m^2
- $N2LW5=PeLW5-Pi2ww0=-0.179$ kN/m^2

Isolated Zone A side face
- $N2LW3=PeLW3-Pi2ww90=-0.8354$ kN/m^2
- $N2LW4=PeLW4-Pi2ww90=-0.4475$ kN/m^2
- $N2LW5=PeLW5-Pi2ww90=-0.179$ kN/m^2

Isolated Zone B side face
- $N2LW4=PeLW4-Pi2ww90=-0.4475$ kN/m^2
- $N2LW5=PeLW5-Pi2ww90=-0.179$ kN/m^2

Isolated Zone C side face
- $N2LW5=PeLW5-Pi2ww90=-0.179$ kN/m^2
- $N2LW2=PeLW2-Pi2ww90=-0.1671$ kN/m^2

Wind on short face of building

Windward (front) face
- $N2SW1=PeSW1-Pi2ww90=0.7518$ kN/m^2
- $N2SW2=PeSW2-Pi2ww90=-0.1671$ kN/m^2
- $N2SW3=PeSW3-Pi2ww90=-0.8354$ kN/m^2
Isolated Zone B side face \[ N2SW4 = PeSW4 - Pi2ww90 = -0.4177 \text{ kN/m}^2 \]
Isolated Zone C side face \[ N2SW5 = PeSW5 - Pi2ww90 = -0.1671 \text{ kN/m}^2 \]

**External pressure coefficients Cpe for flat roofs of rectangular clad buildings to Table 8 of BS6399-2:1997 (June 02)**

**Wind at 0° (long face)**

**Roof net pressure Cpi=0.2**
- Roof zone A: \[ N1RA0 = PeRA0 - PiW1w0 = -1.68 \text{ kN/m}^2 \]
- Roof zone B: \[ N1RB0 = PeRB0 - PiW1w0 = -1.222 \text{ kN/m}^2 \]
- Roof zone C: \[ N1RC0 = PeRC0 - PiW1w0 = -0.6872 \text{ kN/m}^2 \]
- Roof zone D 1: \[ N1RD10 = PeRD10 - PiW1w0 = 0 \text{ kN/m}^2 \]
- Roof zone D 2: \[ N1RD20 = PeRD20 - PiW1w0 = -0.3054 \text{ kN/m}^2 \]

**Wind at 90° (short face)**

**Roof net pressure Cpi=0.2**
- Roof zone A: \[ N1RA9 = PeRA9 - PiW1w90 = -1.568 \text{ kN/m}^2 \]
- Roof zone B: \[ N1RB9 = PeRB9 - PiW1w90 = -1.14 \text{ kN/m}^2 \]
- Roof zone C: \[ N1RC9 = PeRC9 - PiW1w90 = -0.6414 \text{ kN/m}^2 \]
- Roof zone D 1: \[ N1RD19 = PeRD19 - PiW1w90 = 0 \text{ kN/m}^2 \]
- Roof zone D 2: \[ N1RD29 = PeRD29 - PiW1w90 = -0.285 \text{ kN/m}^2 \]
<table>
<thead>
<tr>
<th>Roof zone A</th>
<th>Wind at 0° (long face)</th>
<th>Roof zone B</th>
<th>Wind at 0° (long face)</th>
<th>Roof zone C</th>
<th>Wind at 0° (long face)</th>
<th>Roof zone D 1</th>
<th>Wind at 0° (long face)</th>
<th>Roof zone D 2</th>
<th>Wind at 0° (long face)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N2RA0</td>
<td>PeRA0 - PiW2w0 = -1.298 kN/m²</td>
<td>N2RB0= PeRB0 - PiW2w0 = -0.8399 kN/m²</td>
<td>N2RC0= PeRC0 - PiW2w0 = -0.3054 kN/m²</td>
<td>N2RD10= PeRD10 - PiW2w0 = 0.3818 kN/m²</td>
<td>N2RD20= PeRD20 - PiW2w0 = 0.0764 kN/m²</td>
<td>N2RA9= PeRA9 - PiW2w90 = -1.211 kN/m²</td>
<td>N2RB9= PeRB9 - PiW2w90 = -0.7839 kN/m²</td>
<td>N2RC9= PeRC9 - PiW2w90 = -0.285 kN/m²</td>
<td>N2RD19= PeRD19 - PiW2w90 = 0.3563 kN/m²</td>
</tr>
</tbody>
</table>

Roof zone Cpi=-0.3

Wind at 90° (short face)

Roof zone A

Roof zone B

Roof zone C

Roof zone D 1

Roof zone D 2
Location: Default case 7 (free-standing monopitch roof)

Displacement heights

Displacement heights diagram showing:
- Wind direction
- 6Ho
- 2Ho
- Ho
- 0.8Ho
- He
- Hd
- Hr

Ho = Mean height of other buildings
Xo = Upwind spacing of building

Building reference height
Hr = 13.77 m
Upwind of site
Ho = 6.5 m
Upwind spacing of building
Xo = 20 m
Displacement height
Hd = 3.8 m
Building type factor
Kb = 2
Building height
H = 13.77 m

Standard wind loads

Basic wind speed
Vb = 21 m/s
Site altitude above mean sea level
deltaS = 25 m
Direction factor
Sd = 1

Effective height
He = 9.97 m
Altitude factor
Sa = 1.025
Direction factor
Sd = 1
Seasonal factor
Ss = 1
Probability factor
Sp = 1
Dynamic wind pressure
qs = 0.8461 kN/m²
Net pressure coefficients $C_p$ for free-standing monopitch roofs in accordance with BS6399-2:1997 (June 2002)

Tables 13 and 15 of the code will be utilised which take account of the combined effect of the wind on both upper and lower surfaces of the canopy for all wind directions.

Pitch of roof (0° to 30°)  $\alpha_2=25°$

Net downward roof pressure (i.e. normal to the roof)

Local net pressure for zone A  $C_{locA} = qs \times C_{locAp} = 1.692 \text{ kN/m}^2$
Local net pressure for zone B  $C_{locB} = qs \times C_{locBp} = 2.623 \text{ kN/m}^2$
Local net pressure for zone C  $C_{locC} = qs \times C_{locCp} = 1.946 \text{ kN/m}^2$

Net upwind roof pressure (i.e. normal to the roof)

Local net pressure for zone A  $C_{locA} = qs \times C_{locAn} = -1.782 \text{ kN/m}^2$
Local net pressure for zone B  $C_{locB} = qs \times C_{locBn} = -2.193 \text{ kN/m}^2$
Local net pressure for zone C  $C_{locC} = qs \times C_{locCn} = -2.193 \text{ kN/m}^2$

Net pressure on fascias and gables

In addition to pressure normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia at the eaves or any gable ends. The pressure for the windward and leeward faces could be taken as follows:

Net pressure (winward face)  $C_{pwin} = qs \times 1.3 = 1.1 \text{ kN/m}^2$
Net pressure (leeward face)  $C_{plee} = qs \times 0.6 = 0.5077 \text{ kN/m}^2$
Location: Default case 8 (free-standing monopitch roof)

Displacement heights

---

Wind loads to BS6399-2:1997

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Location: Default case 8 (free-standing monopitch roof)

Displacement heights

\[ \text{Profile of displacement height} \]

\[ \text{Ho} = \text{Mean height of other buildings} \]

\[ X_0 \]

Building reference height \( H_r = 13.77 \text{ m} \)

Upwind of site \( H_o = 6.5 \text{ m} \)

Upwind spacing of building \( X_0 = 20 \text{ m} \)

Displacement height \( H_d = 3.8 \text{ m} \)

Building type factor \( K_b = 2 \)

Building height \( H = 13.77 \text{ m} \)

Standard wind loads

Basic wind speed \( V_b = 21 \text{ m/s} \)

Site altitude above mean sea level \( \Delta S = 25 \text{ m} \)

Direction factor \( S_d = 1 \)

---

Effective height \( H_e = 9.97 \text{ m} \)

Altitude factor \( S_a = 1.025 \)

Direction factor \( S_d = 1 \)

Seasonal factor \( S_s = 1 \)

Probability factor \( S_p = 1 \)

Dynamic wind pressure \( q_s = 0.8461 \text{ kN/m}^2 \)
Net pressure coefficients \( C_p \) for free-standing monopitch roofs in accordance with BS6399-2:1997 (June 2002)

![Diagram of roof sections A, B, C with W/10 and L/10 labels]

Tables 13 and 15 of the code will be utilised which take account of the combined effect of the wind on both upper and lower surfaces of the canopy for all wind directions.

Pitch of roof (0° to 30°) \( \alpha_2 = 30° \)

**Net downward roof pressure (i.e. normal to the roof)**

- Local net pressure for zone A \( C_{locA} = q_s \times C_{locAp} = 1.861 \text{ kN/m}^2 \)
- Local net pressure for zone B \( C_{locB} = q_s \times C_{locBp} = 2.708 \text{ kN/m}^2 \)
- Local net pressure for zone C \( C_{locC} = q_s \times C_{locCp} = 2.031 \text{ kN/m}^2 \)

**Net upwind roof pressure (i.e. normal to the roof)**

- Local net pressure for zone A \( C_{locA} = q_s \times C_{locAn} = -0.9595 \text{ kN/m}^2 \)
- Local net pressure for zone B \( C_{locB} = q_s \times C_{locBn} = -1.371 \text{ kN/m}^2 \)
- Local net pressure for zone C \( C_{locC} = q_s \times C_{locCn} = -1.576 \text{ kN/m}^2 \)

**Net pressure on fascias and gables**

In addition to pressure normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia at the eaves or any gable ends. The pressure for the windward and leeward faces could be taken as follows:

- Net pressure (winward face) \( C_{win} = q_s \times 1.3 = 1.1 \text{ kN/m}^2 \)
- Net pressure (leeward face) \( C_{lee} = q_s \times 0.6 = 0.5077 \text{ kN/m}^2 \)
**Location:** Default case 9 (free-standing duopitch roof)

### Displacement heights

- **Wind**
- **Profile of displacement height**
- **6Ho**
- **2Ho**
- **He**
- **Hr**
- **Ho**
- **0.8Ho**
- **Xo**
- **Hd**

\[ Ho = \text{Mean height of other buildings} \]

**Building reference height**

- **Hr** = 13.77 m

**Upwind spacing of building**

- **Xo** = 20 m

**Displacement height**

- **Hd** = 3.8 m

**Building type factor**

- **Kb** = 2

**Building height**

- **H** = 13.77 m

### Standard wind loads

- **Basic wind speed**
  - **Vb** = 21 m/s

- **Site altitude above mean sea level**
  - **deltaS** = 25 m

- **Direction factor**
  - **Sd** = 1

**Effective height**

- **He** = 9.97 m

**Altitude factor**

- **Sa** = 1.025

**Direction factor**

- **Sd** = 1

**Seasonal factor**

- **Ss** = 1

**Probability factor**

- **Sp** = 1

**Dynamic wind pressure**

- **qs** = 0.8461 kN/m²
Net pressure coefficients $C_p$ for free-standing duopitch roofs in accordance with BS6399-2:1997 (June 2002)

Tables 14 and 15 of the code will be utilised which take account of the combined effect of the wind on both upper and lower surfaces of the canopy for all wind directions.

Pitch of roof (-20° to 30°) $\alpha_3=5°$

**Net downward roof pressure (i.e. normal to the roof)**

<table>
<thead>
<tr>
<th>Zone</th>
<th>Local net pressure for zone A $C_{plocA}=qs*C_{locA}$</th>
<th>Local net pressure for zone B $C_{plocB}=qs*C_{locB}$</th>
<th>Local net pressure for zone C $C_{plocC}=qs*C_{locC}$</th>
<th>Local net pressure for zone D $C_{plocD}=qs*C_{locD}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.5077 kN/m²</td>
<td>1.523 kN/m²</td>
<td>1.1 kN/m²</td>
<td>0.3384 kN/m²</td>
</tr>
<tr>
<td>B</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Net upwind roof pressure (i.e. normal to the roof)**

<table>
<thead>
<tr>
<th>Zone</th>
<th>Local net pressure for zone A $C_{plocA}=qs*C_{locA}$</th>
<th>Local net pressure for zone B $C_{plocB}=qs*C_{locB}$</th>
<th>Local net pressure for zone C $C_{plocC}=qs*C_{locC}$</th>
<th>Local net pressure for zone D $C_{plocD}=qs*C_{locD}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-0.4112 kN/m²</td>
<td>-0.9595 kN/m²</td>
<td>-0.9595 kN/m²</td>
<td>-0.7539 kN/m²</td>
</tr>
<tr>
<td>B</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Net pressure on fascias and gables**

In addition to pressure normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia at the eaves or any gable ends. The pressure for the windward and leeward faces could be taken as follows:

<table>
<thead>
<tr>
<th>Net pressure (winward face)</th>
<th>Net pressure (leeward face)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{pwin}=qs*1.3=1.1$ kN/m²</td>
<td>$C_{plee}=qs*0.6=0.5077$ kN/m²</td>
</tr>
</tbody>
</table>
Location: Default case 10 (free-standing duopitch roof)

Displacement heights

\[ \text{Displacement heights} \]

\[ \text{Wind} \]

\[ \text{Profile of displacement height} \]

\[ 6Ho \]

\[ 2Ho \]

\[ Ho \]

\[ 0.8Ho \]

\[ He \]

\[ Hd \]

\[ Hr \]

Ho=Mean height of other buildings

Building reference height \( H_r = 13.77 \text{ m} \)

Upwind of site \( H_o = 6.5 \text{ m} \)

Upwind spacing of building \( X_o = 20 \text{ m} \)

Displacement height \( H_d = 3.8 \text{ m} \)

Building type factor \( K_b = 2 \)

Building height \( H = 13.77 \text{ m} \)

Standard wind loads

Basic wind speed \( V_b = 21 \text{ m/s} \)

Site altitude above mean sea level \( \Delta \text{S} = 25 \text{ m} \)

Direction factor \( S_d = 1 \)

\[ \text{Effective height} \quad He = 9.97 \text{ m} \]

\[ \text{Altitude factor} \quad S_a = 1.025 \]

\[ \text{Direction factor} \quad S_d = 1 \]

\[ \text{Seasonal factor} \quad S_s = 1 \]

\[ \text{Probability factor} \quad S_p = 1 \]

\[ \text{Dynamic wind pressure} \quad q_s = 0.8461 \text{ kN/m}^2 \]
Net pressure coefficients $C_p$ for free-standing duopitch roofs in accordance with BS6399-2:1997 (June 2002)

![Tables 14 and 15 of the code will be utilised which take account of the combined effect of the wind on both upper and lower surfaces of the canopy for all wind directions.]

Pitch of roof (-20° to 30°) $\alpha_{3}=30°$

**Net downward roof pressure (i.e. normal to the roof)**

Local net pressure for zone A $C_{plocA}=q_s*C_{locAp}=1.1\ kN/m^2$
Local net pressure for zone B $C_{plocB}=q_s*C_{locBp}=1.608\ kN/m^2$
Local net pressure for zone C $C_{plocC}=q_s*C_{locCp}=1.354\ kN/m^2$
Local net pressure for zone D $C_{plocD}=q_s*C_{locDp}=0.5923\ kN/m^2$

**Net upwind roof pressure (i.e. normal to the roof)**

Local net pressure for zone A $C_{plocA}=q_s*C_{locAn}=-0.8224\ kN/m^2$
Local net pressure for zone B $C_{plocB}=q_s*C_{locBn}=-0.891\ kN/m^2$
Local net pressure for zone C $C_{plocC}=q_s*C_{locCn}=-0.7539\ kN/m^2$
Local net pressure for zone D $C_{plocD}=q_s*C_{locDn}=-1.097\ kN/m^2$

**Net pressure on fascias and gables**

In addition to pressure normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia at the eaves or any gable ends. The pressure for the windward and leeward faces could be taken as follows:

Net pressure (winward face) $C_{pwin}=q_s*1.3=1.1\ kN/m^2$
Net pressure (leeward face) $C_{plee}=q_s*0.6=0.5077\ kN/m^2$
Location: Default case 11 (duopitch roof)

Displacement heights

Wind

Profile of displacement height

Ho = Mean height of other buildings

6Ho

2Ho

He

Hr

Hd

Hdw0

Hdw90

Ho = 6.5 m

Hr = 13.77 m

Xo = 20 m

H = 13.77 m

Building type factor Kb = 2

Altitude factor (long face) Saw0 = 1.025

Effective height (long face) Hew0 = 9.97 m

Effective height (short face) Hew90 = 9.97 m

Direction factor (long face) Sdw0 = 1

Direction factor (short face) Sdw90 = 1

Seasonal factor Ss = 1

Probability factor Sp = 1

Dynamic wind pressure (long face) qsw0 = 0.8461 kN/m²

Dynamic wind pressure (short face) qsw90 = 0.8461 kN/m²
Dynamic pressure & pressure coeffs on walls of rectangular buildings

Greater horiz. dimens of building  L=60 m
Lesser horiz. dimens of building  W=20 m
Height of wall including parapet  Hr'=13.77 m

Wind acting on long face of building:
Dynamic pressure based on height above ground 9.97 m.
Dynamic pressure @ eaves  qew0=0.8461 kN/m²

Wind acting on short face of building:
Dynamic pressure based on height above ground 9.97 m.
Dynamic pressure @ eaves  qew90=0.8461 kN/m²

Case 1: Wind on long face of building (i.e. angle Theta = 0°)
Case 2: Wind on short face of building (i.e. angle Theta = 90°)

**Plan View**

- Windward (front) face
- Leeward (rear) face
- Isolated Zone A side face
- Isolated Zone B side face

**Elevation on Side Face**

- Key zones for wall pressure data.

### Wall net pressure Cpi=0.2

- Windward (front) face
- Leeward (rear) face
- Isolated Zone A side face
- Isolated Zone B side face

### Wall net pressure Cpi=-0.3

- Windward (front) face
- Leeward (rear) face
- Isolated Zone A side face
- Isolated Zone B side face
- Isolated Zone C side face
External pressure coefficients Cpe for duopitch roofs of
rectangular clad buildings to Table 10 of BS6399-2:1997 (June 02)

Case 1: Wind angle Theta = 0°

Roof slope to horizontal

Case 2: Wind angle Theta = 90°

Roof net pressure Cpi=0.2

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Wind at 0° (long face)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof zone A</td>
<td>N1RA0=PeRA0-PiW1w0=-0.5923 kN/m²</td>
</tr>
<tr>
<td>Roof zone B</td>
<td>N1RB0=PeRB0-PiW1w0=-0.5923 kN/m²</td>
</tr>
<tr>
<td>Roof zone C</td>
<td>N1RC0=PeRC0-PiW1w0=-0.3384 kN/m²</td>
</tr>
<tr>
<td>Roof zone E</td>
<td>N1RE0=PeRE0-PiW1w0=-0.9307 kN/m²</td>
</tr>
<tr>
<td>Roof zone F</td>
<td>N1RF0=PeRF0-PiW1w0=-0.5923 kN/m²</td>
</tr>
<tr>
<td>Roof zone G</td>
<td>N1RG0=PeRG0-PiW1w0=-0.5923 kN/m²</td>
</tr>
<tr>
<td>Roof zone A (+Ve Value)</td>
<td>N1RAA0=PRAA0-PiW1w0=0.5077 kN/m²</td>
</tr>
<tr>
<td>Roof zone B (+Ve Value)</td>
<td>N1RAM0=PRAB0-PiW1w0=0.2538 kN/m²</td>
</tr>
<tr>
<td>Roof zone C (+Ve Value)</td>
<td>N1RAC0=PRAC0-PiW1w0=0.1692 kN/m²</td>
</tr>
<tr>
<td>Roof zone E (+Ve Value)</td>
<td>N1RAE0=PRAE0-PiW1w0=-0.9307 kN/m²</td>
</tr>
<tr>
<td>Roof zone F (+Ve Value)</td>
<td>N1RAF0=PRAF0-PiW1w0=-0.5923 kN/m²</td>
</tr>
<tr>
<td>Roof zone G (+Ve Value)</td>
<td>N1RAG0=PRAG0-PiW1w0=-0.5923 kN/m²</td>
</tr>
</tbody>
</table>
### Roof net pressure Cpi=0.2

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>N1RA9=PeRA9-PiW1w90=</th>
<th>( \text{kN/m}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-1.185</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>-1.1</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>-0.6769</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>-0.5923</td>
<td></td>
</tr>
</tbody>
</table>

### Roof net pressure Cpi=-0.3

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>N2RA9=PeRA9-PiW2w90=</th>
<th>( \text{kN/m}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-0.7615</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>-0.6769</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>-0.2538</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>-0.1692</td>
<td></td>
</tr>
</tbody>
</table>

### Roof net pressure Cpi=-0.3

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>N2RA9=PeRA9-PiW2w90=</th>
<th>( \text{kN/m}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-0.7615</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>-0.6769</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>-0.2538</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>-0.1692</td>
<td></td>
</tr>
</tbody>
</table>
**Location: Default case 12 (duopitch roof)**

**Displacement heights**

![Diagram of displacement heights]

- **Ho** = Mean height of other buildings
- **Xo** = Building reference height
- **Hr** = 4.3 m (building reference height)
- **Ho** = 0.5 m (upwind of site)
- **Xo** = 20 m (upwind spacing of building)
- **He** = Displacement height
- **Hd** = Displacement height
- **Kb** = Building type factor
- **H** = Building height
- **Vb** = Basic wind speed
- **deltaS** = Site altitude above mean sea level

**Standard wind loads**

- **Vb** = 24 m/s
- **deltaS** = 46 m

**Topography is significant - Wind on long face of building**

Slope details and site location relative to crest:

- **Xt** is positive if site is located downwind of crest.
- **Xt** is negative if site is located upwind of crest.
- **Z** = 45 m

---

SCALE 5.48 Office 1007 Proforma 702
Direction factor  
Direction factor  

Effective height (long face)  
Effective height (short face)  
Altitude factor (long face)  
Altitude factor (short face)  
\[
\begin{align*}
\text{Sdw0} &= 1 \\
\text{Hew0} &= 4.3 \text{ m} \\
\text{Sdw90} &= 1 \\
\text{Hew90} &= 4.3 \text{ m} \\
\text{Saw0} &= 1.18 \\
\text{Saw90} &= 1.046 \\
\end{align*}
\]

DESIGN  
SUMMARY  
Direction factor (long face)  
Direction factor (short face)  
Seasonal factor  
Probability factor  
Dynamic wind pressure (long face)  
Dynamic wind pressure (short face)  
\[
\begin{align*}
\text{Sdw0} &= 1 \\
\text{Sdw90} &= 1 \\
\text{Ss} &= 1 \\
\text{Sp} &= 1 \\
\text{qsw0} &= 1.206 \text{ kN/m}^2 \\
\text{qsw90} &= 0.9474 \text{ kN/m}^2 \\
\end{align*}
\]

Dynamic pressure & pressure coeffs on walls of rectangular buildings  

Greater horiz. dimens of building  \( L = 60 \text{ m} \)  
Lesser horiz. dimens of building  \( W = 20 \text{ m} \)  
Height of wall including parapet  \( H_{r'} = 4.3 \text{ m} \)  

Wind acting on long face of building:  
Dynamic pressure based on height above ground 4.3 m.  
Dynamic pressure \( @ \) eaves  \( q_{ew0} = 1.206 \text{ kN/m}^2 \)  

Wind acting on short face of building:  
Dynamic pressure based on height above ground 4.3 m.  
Dynamic pressure \( @ \) eaves  \( q_{ew90} = 0.9474 \text{ kN/m}^2 \)  

Case 1: Wind on long face of building (i.e. angle \( \Theta = 0^\circ \))
Case 2: Wind on short face of building (i.e. angle Theta = 90°)

Windward (front) face
Leeward (rear) face
Isolated Zone A side face
Isolated Zone B side face
Isolated Zone C side face

Wall net pressure Cpi=0.2

Wind on long face of building

Windward (front) face
Leeward (rear) face
Isolated Zone A side face
Isolated Zone B side face
Isolated Zone C side face

Wall net pressure Cpi=0.2

Wind on short face of building

Windward (front) face
Leeward (rear) face
Isolated Zone A side face
Isolated Zone B side face
Isolated Zone C side face

Wall net pressure Cpi=-0.3

Wind on long face of building

Windward (front) face
Leeward (rear) face
Isolated Zone A side face
Isolated Zone B side face
Isolated Zone C side face

Wall net pressure Cpi=-0.3

Wind on short face of building

Windward (front) face
Leeward (rear) face
Isolated Zone A side face

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Ref No: SC702 BS

Plan View

Elevation on side face

Key zones for wall pressure data.
External pressure coefficients $C_{pe}$ for duopitch roofs of rectangular clad buildings to Table 10 of BS6399-2:1997 (June 02)

Case 1: Wind angle $\theta = 0^\circ$

Roof slope to horizontal

Case 2: Wind angle $\theta = 90^\circ$

Roof net pressure $C_{pi}=0.2$

<table>
<thead>
<tr>
<th>Roof zone A</th>
<th>$N1RA0=PeRA0-PiW1w0=-0.844 \text{ kN/m}^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof zone B</td>
<td>$N1RB0=PeRB0-PiW1w0=-0.844 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Roof zone C</td>
<td>$N1RC0=PeRC0-PiW1w0=-0.4823 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Roof zone E</td>
<td>$N1RE0=PeRE0-PiW1w0=-1.326 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Roof zone F</td>
<td>$N1RF0=PeRF0-PiW1w0=-0.844 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Roof zone G</td>
<td>$N1RG0=PeRG0-PiW1w0=-0.844 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Roof zone A (+Ve Value)</td>
<td>$N1RAA0=PRAA0-PiW1w0=0.7234 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Roof zone B (+Ve Value)</td>
<td>$N1RAB0=PRAB0-PiW1w0=0.3617 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Roof zone C (+Ve Value)</td>
<td>$N1RAC0=PRAC0-PiW1w0=0.2411 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Roof zone E (+Ve Value)</td>
<td>$N1RAE0=PRAE0-PiW1w0=-1.326 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Roof zone F (+Ve Value)</td>
<td>$N1RAF0=PRAF0-PiW1w0=-0.844 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>Roof zone G (+Ve Value)</td>
<td>$N1RAG0=PRAG0-PiW1w0=-0.844 \text{ kN/m}^2$</td>
</tr>
</tbody>
</table>
Roof net pressure $C_{pi}=0.2$  \hspace{1cm} Wind at $90^\circ$ (short face)

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>$N_{i} = P_{i} - C_{pi} W_{i}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$-1.326 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>B</td>
<td>$-1.232 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>C</td>
<td>$-0.7579 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>D</td>
<td>$-0.6632 \text{ kN/m}^2$</td>
</tr>
</tbody>
</table>

Roof net pressure $C_{pi}=-0.3$  \hspace{1cm} Wind at $0^\circ$ (long face)

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>$N_{i} = P_{i} - C_{pi} W_{i}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$-0.2411 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>B</td>
<td>$-0.2411 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>C</td>
<td>$0.1206 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>E</td>
<td>$-0.7234 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>F</td>
<td>$-0.2411 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>G</td>
<td>$-0.2411 \text{ kN/m}^2$</td>
</tr>
</tbody>
</table>

Roof net pressure $C_{pi}=-0.3$  \hspace{1cm} Wind at $90^\circ$ (short face)

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>$N_{i} = P_{i} - C_{pi} W_{i}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$-0.8527 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>B</td>
<td>$-0.7579 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>C</td>
<td>$-0.2842 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>D</td>
<td>$-0.1895 \text{ kN/m}^2$</td>
</tr>
</tbody>
</table>
Location: Default case 13 (free-standing wall)

Free-standing wall details

Effective height of free-standing wall \( H_e = 5 \) m

\[
\begin{array}{cccc}
20 \text{ m} & \text{10 m} & \text{1.5 m} \\
\end{array}
\]

\[
\begin{array}{c|c|c|c|c}
\text{He} & \text{A} & \text{B} & \text{C} & \text{D} \\
\end{array}
\]

ELEVATION ON FREE-STANDING WALL

Standard wind loads

Basic wind speed \( V_b = 21 \) m/s
Site altitude above mean sea level \( \Delta S = 25 \) m

Slope details and site location relative to crest

\[\begin{align*}
\text{Wind} & \quad \text{Xt<0} \quad \text{Xt>0} \\
\text{Site} & \quad \text{Xt} \quad \text{Site altitude is 25 m} \\
& \quad \text{above mean sea level.} \\
\text{Xt} & \quad \text{Xt is positive if site is} \\
& \quad \text{located downwind of crest.} \\
\text{Xt} & \quad \text{Xt is negative if site is} \\
& \quad \text{located upwind of crest.} \\
\end{align*}\]

Height to crest \( Z = 10 \) m
Direction factor \( S_d = 1 \)

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective height</td>
<td>He 5 m</td>
</tr>
<tr>
<td>Altitude factor</td>
<td>Sa 1.227</td>
</tr>
<tr>
<td>Direction factor</td>
<td>Sd 1</td>
</tr>
<tr>
<td>Seasonal factor</td>
<td>Ss 1</td>
</tr>
<tr>
<td>Probability factor</td>
<td>Sp 1</td>
</tr>
<tr>
<td>Dynamic wind pressure</td>
<td>qs 0.9121 kN/m²</td>
</tr>
</tbody>
</table>
Size effect factors

Length of wall \( L_{wall} = 25 \text{ m} \)
Length of the return corner \( r_{cornL} = 5 \text{ m} \)

Net pressure on wall

Net pressure on wall zone A \( C_{pwin} = q_{s} * C_{pA} * C_{a} = 1.341 \text{ kN/m}^2 \)
Net pressure on wall zone B \( C_{pwin} = q_{s} * C_{pB} * C_{a} = 1.149 \text{ kN/m}^2 \)
Net pressure on wall zone C \( C_{pwin} = q_{s} * C_{pC} * C_{a} = 1.094 \text{ kN/m}^2 \)
Net pressure on wall zone D \( C_{pwin} = q_{s} * C_{pD} * C_{a} = 1.094 \text{ kN/m}^2 \)
Location: Default case 14 (signboard)

Signboard details

Effective height of signboard \( He = 5 \text{ m} \)
Depth of signboard \( h1 = 3 \text{ m} \)

Standard wind loads

Basic wind speed \( V_b = 21 \text{ m/s} \)
Site altitude above mean sea level \( \Delta S = 25 \text{ m} \)

Slope details and site location relative to crest

Effective height \( He = 5 \text{ m} \)
Direction factor \( S_d = 1 \)

Size effect factors

Clause 2.1.3.4 in BS6399 offers the user to input diagonal dimensions to derive a size effect factor. This program will use the simple method where the size effect factor is set to 1.0.
Size effect factor \( Ca = 1.0 \)
Net pressure coefficient \( C_{pwf} = 1.8 \)
Net pressure on signboard \( C_{pw} = q_s C_{pwf} Ca = 1.44 \text{ kN/m}^2 \)
Location: Default case 15 (signboard)

Signboard details

Effective height of signboard \( He = 4 \) m
Depth of signboard \( h_1 = 3.5 \) m

Standard wind loads

Basic wind speed \( V_b = 21 \) m/s
Site altitude above mean sea level \( \delta S = 25 \) m

Slope details and site location relative to crest

Height to crest \( Z = 10 \) m
Direction factor \( S_d = 1 \)

Effective height \( He = 4 \) m
Altitude factor \( S_a = 1.149 \)
Direction factor \( S_d = 1 \)
Seasonal factor \( S_s = 1 \)
Probability factor \( S_p = 1 \)
Dynamic wind pressure \( q_s = 0.6905 \) kN/m²

Size effect factors

Clause 2.1.3.4 in BS6399 offers the user to input diagonal dimensions to derive a size effect factor. This program will use the simple method where the size effect factor is set to 1.0.
Size effect factor  \( Ca = 1.0 \)

ELEVATION ON FREE-STANDING WALL

Length of wall  \( L_{\text{wall}} = 10 \text{ m} \)

**Net pressure on wall**

- Net pressure on signboard zone A  \( C_{\text{pwin}} = q_s C_A Ca = 1.409 \text{ kN/m}^2 \)
- Net pressure on signboard zone B  \( C_{\text{pwin}} = q_s C_B Ca = 0.8701 \text{ kN/m}^2 \)
- Net pressure on signboard zone C  \( C_{\text{pwin}} = q_s C_C Ca = 0.8287 \text{ kN/m}^2 \)
- Net pressure on signboard zone D  \( C_{\text{pwin}} = q_s C_D Ca = 0.8287 \text{ kN/m}^2 \)
Location: Default case 16 (free-standing parapet)

Free-standing parapet details

Effective height of free-standing parapet  $H_e=1$ m

<table>
<thead>
<tr>
<th>He</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

ELEVATION ON FREE-STANDING PARAPET

Standard wind loads

Basic wind speed  $V_b=24$ m/s
Site altitude above mean sea level $\delta_S=25$ m

Slope details and site location relative to crest

$Z=10$ m
$S_d=1$

<table>
<thead>
<tr>
<th>Effective height</th>
<th>$H_e$</th>
<th>1 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Altitude factor</td>
<td>$S_a$</td>
<td>1.149</td>
</tr>
<tr>
<td>Direction factor</td>
<td>$S_d$</td>
<td>1</td>
</tr>
<tr>
<td>Seasonal factor</td>
<td>$S_s$</td>
<td>1</td>
</tr>
<tr>
<td>Probability factor</td>
<td>$S_p$</td>
<td>1</td>
</tr>
<tr>
<td>Dynamic wind pressure</td>
<td>$qs$</td>
<td>$0.6469$ kN/m²</td>
</tr>
</tbody>
</table>
## Size effect factors

Length of wall \( L_{\text{wall}} = 10 \text{ m} \)

### Net pressure on parapet

<table>
<thead>
<tr>
<th>Zone</th>
<th>Net pressure formula</th>
<th>Net pressure [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone A</td>
<td>( q_s * C_{pA} * C_a )</td>
<td>1.979</td>
</tr>
<tr>
<td>Zone B</td>
<td>( q_s * C_{pB} * C_a )</td>
<td>1.223</td>
</tr>
<tr>
<td>Zone C</td>
<td>( q_s * C_{pC} * C_a )</td>
<td>0.9897</td>
</tr>
<tr>
<td>Zone D</td>
<td>( q_s * C_{pD} * C_a )</td>
<td>0.7762</td>
</tr>
</tbody>
</table>
Location: Default case 17

Displacement heights

Wind loads to BS6399-2:1997

Building reference height
Ho=Mean height of other buildings
Hr=9.64 m

upwind of site
Ho=5 m

Upwind spacing of building
Xo=20 m

Displacement height
Hdw0=0 m

Displacement height
Hdw90=0 m

Building type factor
Kb=2

Building height
H=9.64 m

Standard wind loads

Basic wind speed
Vb=23.5 m/s

Site altitude above mean sea level
deltaS=123.6 m

Topography is significant in both orthogonal directions

Slope details and site location relative to crest

Site altitude is 123.6 m above mean sea level.

Xt is positive if site is located downwind of crest.

Xt is negative if site is located upwind of crest.

Height to crest
Z=3.6 m
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective height (long face)</td>
<td>Hew0 9.64 m</td>
</tr>
<tr>
<td>Effective height (short face)</td>
<td>Hew90 9.64 m</td>
</tr>
<tr>
<td>Altitude factor (long face)</td>
<td>Saw0 1.166</td>
</tr>
<tr>
<td>Altitude factor (short face)</td>
<td>Saw90 1.166</td>
</tr>
<tr>
<td>Direction factor (long face)</td>
<td>Sdw0 0.995</td>
</tr>
<tr>
<td>Direction factor (short face)</td>
<td>Sdw90 1</td>
</tr>
<tr>
<td>Seasonal factor</td>
<td>Ss 1</td>
</tr>
<tr>
<td>Probability factor</td>
<td>Sp 1</td>
</tr>
<tr>
<td>Dynamic wind pressure (long face)</td>
<td>qsw0 1.178 kN/m²</td>
</tr>
<tr>
<td>Dynamic wind pressure (short face)</td>
<td>qsw90 1.19 kN/m²</td>
</tr>
</tbody>
</table>
Location: Default case 18

Displacement heights

```
Wind                  6Ho
Profile of            2Ho
displacement height   Ho
He                   0.8Ho
Hd

Ho=Mean height
of other buildings
```

Building reference height  Hr=9.64 m
upwind of site            Ho=5 m
Upwind spacing of building Xo=20 m
Displacement height       Hdw0=7.57 m
Displacement height       Hdw90=0 m
Building type factor      Kb=2
Building height           H=9.64 m

Standard wind loads

Basic wind speed         Vb=23.5 m/s
Site altitude above mean sea level deltaS=123.6 m

Topography is significant - Wind on short face of building

Slope details and site location relative to crest

```
Xt<0       Xt>0
Wind

Site altitude is 123.6 m above mean sea level.
Xt is positive if site is located downwind of crest.
Xt is negative if site is located upwind of crest.

Z=3.6 m
```
Direction factor  $S_{dw0}=0.83$
Direction factor  $S_{dw90}=0.8$

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective height (long face)</td>
<td>$H_{ew0}$ 3.856 m</td>
</tr>
<tr>
<td>Effective height (short face)</td>
<td>$H_{ew90}$ 9.64 m</td>
</tr>
<tr>
<td>Altitude factor (long face)</td>
<td>$S_{aw0}$ 1.124</td>
</tr>
<tr>
<td>Altitude factor (short face)</td>
<td>$S_{aw90}$ 1.166</td>
</tr>
<tr>
<td>Direction factor (long face)</td>
<td>$S_{dw0}$ 0.83</td>
</tr>
<tr>
<td>Direction factor (short face)</td>
<td>$S_{dw90}$ 0.8</td>
</tr>
<tr>
<td>Seasonal factor</td>
<td>$S_s$ 1</td>
</tr>
<tr>
<td>Probability factor</td>
<td>$S_p$ 1</td>
</tr>
<tr>
<td>Dynamic wind pressure (long face)</td>
<td>$q_{sw0}$ 0.5987 kN/m²</td>
</tr>
<tr>
<td>Dynamic wind pressure (short face)</td>
<td>$q_{sw90}$ 0.8949 kN/m²</td>
</tr>
</tbody>
</table>
Location: Default case 19 Flat roof (topography critical, H > 58m)

Displacement heights

Wind

Profile of
displacement height

\[ \text{Ho} = \text{Mean height of other buildings} \]

\[ \text{Xo} \]

Building reference height \( H_r = 58 \text{ m} \)
Displacement height \( H_{dw0} = 0 \text{ m} \)
Displacement height \( H_{dw90} = 0 \text{ m} \)
Building type factor \( K_b = 2 \)
Building height \( H = 58 \text{ m} \)

Standard wind loads

Basic wind speed \( V_b = 22.5 \text{ m/s} \)
Site altitude above mean sea level \( \Delta S = 35 \text{ m} \)

Topography is significant - Wind on long face of building

Slope details and site location relative to crest

\[ X_t < 0 \quad X_t > 0 \]

Site altitude is 35 m above mean sea level.

\( X_t \) is positive if site is located downwind of crest.

\( X_t \) is negative if site is located upwind of crest.

Height to crest \( Z = 10 \text{ m} \)
### Dynamic pressure & pressure coeffs on walls of rectangular buildings

- **Greater horiz. dimens of building**: L = 150 m
- **Lesser horiz. dimens of building**: W = 40.5 m
- **Height of wall including parapet**: Hr' = 58 m

Wind acting on long face of building:

- Dynamic pressure based on height above ground 58 m.
- Dynamic pressure @ eaves: qew0 = 0.8675 kN/m²

Wind acting on short face of building:

- Dynamic pressure based on height above ground 58 m.
- Dynamic pressure @ eaves: qew90 = 0.8255 kN/m²

- **Diagonal dimension of Ext long wall**: dima1 = 25 m
- **Diagonal dimension of Ext short wall**: dima2 = 35 m

### Case 1: Wind on long face of building (i.e. angle Theta = 0°)

![Plan View Diagram]

- **W = D**
- **L = B**
- **Inwind depth**: D = 40.5 m
- **Cross wind width**: B = 150 m
Case 2: Wind on short face of building (i.e. angle Theta = 90°)

Internal volume of storey  
\[ \text{SinVol} = 40000 \text{ m}^3 \]
External pressure coefficients $C_{pe}$ for flat roofs of rectangular clad buildings to Table 8 of BS6399-2:1997 (June 02)

**PLAN VIEW**

Wind on long face ($\theta=0^\circ$)

Inwind depth $D=W=40.5$ m
Cross wind width $B=L=150$ m

**PLAN VIEW**

Wind on short face ($\theta=90^\circ$)

Inwind depth $D=L=150$ m
Cross wind width $B=W=40.5$ m

Diagonal dimension of roof at 0 degree $d_{ma4}=5$ m
Diagonal dimension of roof at 90 degree $d_{ma5}=5$ m
Location: Wind loads

Obstruction height and upwind spacing

Wind direction

\[ \text{Wind direction} \]

\[ \text{6Hav} \]

\[ \text{2Hav} \]

\[ \text{Hr} \]

\[ \text{He} \]

\[ \text{Hd} \]

\[ \text{Hav} = \text{Obstruction height of other buildings} \]

Building reference height \( \text{Hr} = 13.77 \text{ m} \)
buildings upwind of site \( \text{Hav} = 6.5 \text{ m} \)
Upwind spacing of building \( \text{Xo} = 20 \text{ m} \)
Displacement height (wind \( \theta = 0^\circ \)) \( \text{Hd}(0) = 3.8 \text{ m} \)
Displacement height (wind \( \theta = 90^\circ \)) \( \text{Hd}(90) = 3.8 \text{ m} \)
Building height \( \text{H} = 13.77 \text{ m} \)

Standard wind loads

Basic wind velocity (Fig. NA.1) \( V_{b\text{map}} = 21.5 \text{ m/s} \)
Altitude above mean sea level \( A_{\text{site}} = 25 \text{ m} \)
Directional factor (wind \( \theta = 0^\circ \)) \( C_{\text{di}}(0) = 1 \)
Directional factor (wind \( \theta = 90^\circ \)) \( C_{\text{di}}(90) = 0.73 \)
Distance upwind to shoreline \( \text{Dis}(0) = 2.5 \text{ km} \)
Distance inside town terrain \( \text{Dit}(0) = 2.5 \text{ km} \)
Distance upwind to shoreline \( \text{Dis}(90) = 2.5 \text{ km} \)
Distance inside town terrain \( \text{Dit}(90) = 2.5 \text{ km} \)
Probability factor \( C_p = 1 \)

Effective height (long face) \( \text{He}(0) = 9.97 \text{ m} \)
Effective height (short face) \( \text{He}(90) = 9.97 \text{ m} \)
Altitude factor (long face) \( \text{Ca}(0) = 1.025 \)
Altitude factor (short face) \( \text{Ca}(90) = 1.025 \)
Direction factor (long face) \( C_{\text{di}}(0) = 1 \)
Direction factor (short face) \( C_{\text{di}}(90) = 0.73 \)
Seasonal factor \( C_{\text{season}} = 1 \)
Probability factor \( C_p = 1 \)
Long face peak velocity pressure \( q_p(0) = 0.6726 \text{ kN/m}^2 \)
Short face peak velocity pressure \( q_p(90) = 0.3584 \text{ kN/m}^2 \)
Peak velocity pressure & pressure coefficients

on walls of rectangular buildings

Height of wall including parapet $H'=13.77 \text{ m}$
Wind acting on long face of building (wind $\theta=0^\circ$):
Peak velocity pressure based on height above ground 9.97 m.
Peak velocity pressure @ eaves $q_{pe}(0)=0.6726 \text{ kN/m}^2$
Wind acting on short face of building (wind $\theta=90^\circ$):
Peak velocity pressure based on height above ground 9.97 m.
Peak velocity pressure @ eaves $q_{pe}(90)=0.3584 \text{ kN/m}^2$

Case 1: Wind on long face of building (i.e. angle $\theta=0^\circ$)

![Diagram of wind acting on long face of building]

Inwind depth $d=20 \text{ m}$
Crosswind width $b=60 \text{ m}$

Key zones for wall pressure data:
Scaling width $> d$
27.54 m > 20 m

ELEVATION ON SIDE FACE
Case 2: Wind on short face of building (i.e. angle $\theta=90^\circ$)

Windward (front) face
Leeward (rear) face
Isolated Zone A side face
Isolated Zone B side face

Wall net pressure $C_{pi}=0.2$

Wind on long face of building

Windward (front) face
Leeward (rear) face
Isolated Zone A side face
Isolated Zone B side face

Wall net pressure $C_{pi}=-0.3$

Wind on short face of building

Windward (front) face
Leeward (rear) face
Isolated Zone A side face
Isolated Zone B side face
Isolated Zone C side face
\[ N2SW5 = PeSW5 - P12ww90 = -0.0717 \text{ kN/m}^2 \]

**External pressure coefficients \( C_{pe} \) for duopitch roofs**

**Case 1: Wind angle \( \theta = 0^\circ \)**

- Roof slope to horizontal \( \alpha = 30^\circ \)

**Case 2: Wind angle \( \theta = 90^\circ \)**

**Roof net pressure \( C_{pi} = 0.2 \)**

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>( N_{1RF0} = PeRF0 - PiW1w0 = -0.4708 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>G</td>
<td>( N_{1RG0} = PeRG0 - PiW1w0 = -0.4708 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>H</td>
<td>( N_{1RH0} = PeRH0 - PiW1w0 = -0.269 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>J</td>
<td>( N_{1RJ0} = PeRJ0 - PiW1w0 = -0.7398 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>I</td>
<td>( N_{1RI0} = PeRI0 - PiW1w0 = -0.4708 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>F (+Ve Value)</td>
<td>( N_{1RAF0} = PRAF0 - PiW1w0 = 0.4035 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>G (+Ve Value)</td>
<td>( N_{1RAG0} = PRAG0 - PiW1w0 = 0.2018 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>H (+Ve Value)</td>
<td>( N_{1RAH0} = PRAH0 - PiW1w0 = 0.1345 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>J (+Ve Value)</td>
<td>( N_{1RAJ0} = PRAJ0 - PiW1w0 = -0.7398 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>I (+Ve Value)</td>
<td>( N_{1RAI0} = PRAI0 - PiW1w0 = -0.4708 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>Roof zone F (Ve Value)</td>
<td>N1RF9 = PeRF9 - PiW1w90 = 0.1075 kN/m²</td>
</tr>
<tr>
<td>------------------------</td>
<td>--------------------------------------</td>
</tr>
<tr>
<td>Roof zone G (Ve Value)</td>
<td>N1RG9 = PeRG9 - PiW1w90 = -0.0717 kN/m²</td>
</tr>
<tr>
<td>Roof zone H (Ve Value)</td>
<td>N1RH9 = PeRH9 - PiW1w90 = 0.0358 kN/m²</td>
</tr>
<tr>
<td>Roof zone I (Ve Value)</td>
<td>N1RI9 = PeRI9 - PiW1w90 = 0 kN/m²</td>
</tr>
</tbody>
</table>

### Wind θ=0° (long face)

<table>
<thead>
<tr>
<th>Roof zone F</th>
<th>N2RF0 = PeRF0 - PiW2w0 = -0.1345 kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof zone G</td>
<td>N2RG0 = PeRG0 - PiW2w0 = -0.1345 kN/m²</td>
</tr>
<tr>
<td>Roof zone H</td>
<td>N2RH0 = PeRH0 - PiW2w0 = 0.0673 kN/m²</td>
</tr>
<tr>
<td>Roof zone J</td>
<td>N2RJ0 = PeRJ0 - PiW2w0 = -0.4035 kN/m²</td>
</tr>
<tr>
<td>Roof zone I</td>
<td>N2RI0 = PeRI0 - PiW2w0 = -0.1345 kN/m²</td>
</tr>
<tr>
<td>Roof zone F (Ve Value)</td>
<td>N2RAF0 = PRAF0 - PiW2w0 = 0.7398 kN/m²</td>
</tr>
<tr>
<td>Roof zone G (Ve Value)</td>
<td>N2RAG0 = PRAG0 - PiW2w0 = 0.538 kN/m²</td>
</tr>
<tr>
<td>Roof zone H (Ve Value)</td>
<td>N2RAH0 = PRAH0 - PiW2w0 = 0.4708 kN/m²</td>
</tr>
<tr>
<td>Roof zone J (Ve Value)</td>
<td>N2RAJ0 = PRAJ0 - PiW2w0 = -0.4035 kN/m²</td>
</tr>
<tr>
<td>Roof zone I (Ve Value)</td>
<td>N2RAI0 = PRAI0 - PiW2w0 = -0.1345 kN/m²</td>
</tr>
</tbody>
</table>

### Wind θ=90° (short face)

<table>
<thead>
<tr>
<th>Roof zone F</th>
<th>N2RF9 = PeRF9 - PiW2w90 = -0.3226 kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof zone G</td>
<td>N2RG9 = PeRG9 - PiW2w90 = -0.2867 kN/m²</td>
</tr>
<tr>
<td>Roof zone H</td>
<td>N2RH9 = PeRH9 - PiW2w90 = -0.1075 kN/m²</td>
</tr>
<tr>
<td>Roof zone I</td>
<td>N2RI9 = PeRI9 - PiW2w90 = -0.0717 kN/m²</td>
</tr>
<tr>
<td>Roof zone F (Ve Value)</td>
<td>N2RAF9 = PRAF9 - PiW2w90 = 0.2867 kN/m²</td>
</tr>
<tr>
<td>Roof zone G (Ve Value)</td>
<td>N2RAG9 = PRAG9 - PiW2w90 = 0.2509 kN/m²</td>
</tr>
<tr>
<td>Roof zone H (Ve Value)</td>
<td>N2RAH9 = PRAH9 - PiW2w90 = 0.215 kN/m²</td>
</tr>
<tr>
<td>Roof zone I (Ve Value)</td>
<td>N2RAI9 = PRAI9 - PiW2w90 = 0.1792 kN/m²</td>
</tr>
</tbody>
</table>
Location: Default case 2 Flat roof; topography critical

Obstruction height and upwind spacing

Wind direction

\[ \text{Hav} = \text{Obstruction height of other buildings} \]

\[ \text{He} = \text{Displacement height} \]

\[ \text{Hr} = \text{Building height} \]

\[ \text{Vbmap} = 22.5 \, \text{m/s} \]

\[ \text{Asite} = 35 \, \text{m} \]

Standard wind loads

Orography is significant - Wind on long face of building

Slope details and site location relative to crest as per Figure NA.2 of NA to BS EN 1991-1-4:2005+A1:2010.
Loadings and foundations

Wind loads to Eurocode 1

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Directional factor (wind θ=0°)  Cdi(0)=0.85
Directional factor (wind θ=90°)  Cdi(90)=0.85
Seasonal factor  Cseason=0.86
Distance upwind to shoreline  Dis(0)=2.5 km
Distance upwind to shoreline  Dis(90)=2.5 km
Probability factor  Cp=1

Effective height (long face )  He(0)  18 m
Effective height (short face)  He(90)  18 m
Altitude factor (long face)  Ca(0)  1.035
Altitude factor (short face)  Ca(90)  1.035
Direction factor (long face)  Cdi(0)  0.85
Direction factor (short face)  Cdi(90)  0.85
Seasonal factor  Cseason  0.86
Probability factor  Cp  1
Long face peak velocity pressure  qpe(0)  0.6463 kN/m²
Short face peak velocity pressure  qpe(90)  0.5427 kN/m²

Peak velocity pressure & pressure coefficients

on walls of rectangular buildings

Height of wall including parapet  Hr'=18 m
Wind acting on long face of building (wind θ=0°):
Peak velocity pressure based on height above ground 18 m.
Peak velocity pressure @ eaves  qpe(0)=0.6463 kN/m²
Wind acting on short face of building (wind θ=90°):
Peak velocity pressure based on height above ground 18 m.
Peak velocity pressure @ eaves  qpe(90)=0.5427 kN/m²

Case 1: Wind on long face of building (i.e. angle θ=0°)

Inwind depth  d=40.5 m
Crosswind width  b=150 m

Key zones for wall pressure data.
Case 2: Wind on short face of building (i.e. angle $\theta=90^\circ$)

Inwind depth $d=150$ m
Crosswind width $b=40.5$ m

Key zones for wall pressure data.

External pressure coefficients $C_{pe}$ for flat roofs

Wind on long face ($\theta=0^\circ$)
Inwind depth $d=40.5$ m
Crosswind width $b=150$ m
PLAN VIEW

Wind on short face (θ=90°)

Inwind depth \( d = 150 \) m
Crosswind width \( b = 40.5 \) m
Location: Default case 3 mono roof; topography critical

Obstruction height and upwind spacing

Wind direction

\[ \text{Hav} = \text{Obstruction height of other buildings} \]

Building reference height \( \text{Hr} = 18.5 \text{ m} \)
buildings upwind of site \( \text{Hav} = 6.5 \text{ m} \)
Upwind spacing of building \( \text{Xo} = 20 \text{ m} \)
Displacement height (wind \( \theta = 0^\circ \)) \( \text{Hd}(0) = 6.5 \text{ m} \)
Displacement height (wind \( \theta = 90^\circ \)) \( \text{Hd}(90) = 6.5 \text{ m} \)
Building height \( \text{H} = 18.5 \text{ m} \)

Standard wind loads

Basic wind velocity (Fig. NA.1) \( \text{Vbmap} = 24 \text{ m/s} \)
Site altitude above mean sea level \( \text{Asite} = 30 \text{ m} \)

Orography is significant - Wind on long face of building

Slope details and site location relative to crest as per Figure NA.2 of NA to BS EN 1991-1-4:2005+A1:2010.

Effective height of the feature \( Z = 5 \text{ m} \)
### Directional factor (wind θ=0°)
- Cdi(0) = 0.85

### Directional factor (wind θ=90°)
- Cdi(90) = 0.85

### Seasonal factor
- Cseason = 0.66

### Distance upwind to shoreline
- D(0) = 50 km

### Distance inside town terrain
- Dit(0) = 10 km

### Distance upwind to shoreline
- D(90) = 50 km

### Distance inside town terrain
- Dit(90) = 10 km

### Probability factor
- Cp = 1

<table>
<thead>
<tr>
<th>Effective height (long face)</th>
<th>He(0)</th>
<th>12 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective height (short face)</td>
<td>He(90)</td>
<td>12 m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Altitude factor (long face)</th>
<th>Ca(0)</th>
<th>1.03</th>
</tr>
</thead>
<tbody>
<tr>
<td>Altitude factor (short face)</td>
<td>Ca(90)</td>
<td>1.03</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design</th>
<th>Cdi(0)</th>
<th>0.85</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design</td>
<td>Cdi(90)</td>
<td>0.85</td>
</tr>
</tbody>
</table>

| Seasonal factor | Cseason | 0.66 |

| Probability factor | Cp | 1 |

<table>
<thead>
<tr>
<th>Long face peak velocity pressure</th>
<th>q(0)</th>
<th>0.2726 kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short face peak velocity pressure</td>
<td>q(90)</td>
<td>0.239 kN/m²</td>
</tr>
</tbody>
</table>

### Peak velocity pressure & pressure coefficients

**on walls of rectangular buildings**

Height of wall including parapet  Hr' = 18.5 m

Wind acting on long face of building (wind θ=0°):

- Peak velocity pressure based on height above ground 12 m.
- Peak velocity pressure @ eaves  q(0) = 0.2726 kN/m²

Wind acting on short face of building (wind θ=90°):

- Peak velocity pressure based on height above ground 12 m.
- Peak velocity pressure @ eaves  q(90) = 0.239 kN/m²

### Case 1: Wind on long face of building (i.e. angle θ=0°)

```
PLAN VIEW
```

- Inwind depth  d = 20 m
- Crosswind width  b = 40 m
Case 2: Wind on short face of building (i.e. angle $\theta=90^\circ$)

Key zones for wall pressure data.
Scaling width > d
37 m > 20 m

"0.75 times Cpe"

Internal pressure coefficient 1 $C_{pi}(1)=-0.8$
Internal pressure coefficient 2 $C_{pi}(2)=0.3$
**Wall net pressure $C_{pi}=-0.8$**  
Wind on long face of building

<table>
<thead>
<tr>
<th>Face</th>
<th>Net Pressure ($kN/m^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>$N1LW1=PeLW1-Pi1ww0=0.4334$</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>$N1LW2=PeLW2-Pi1ww0=0.0872$</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N1LW3=PeLW3-Pi1ww0=-0.109$</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N1LW4=PeLW4-Pi1ww0=0$</td>
</tr>
</tbody>
</table>

**Wall net pressure $C_{pi}=-0.8$**  
Wind on short face of building

<table>
<thead>
<tr>
<th>Face</th>
<th>Net Pressure ($kN/m^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>$N1SW1=PeSW1-Pi1ww90=0.3653$</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>$N1SW2=PeSW2-Pi1ww90=0.1059$</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N1SW3=PeSW3-Pi1ww90=-0.0956$</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N1SW4=PeSW4-Pi1ww90=0$</td>
</tr>
</tbody>
</table>

**Wall net pressure $C_{pi}=0.3$**  
Wind on long face of building

<table>
<thead>
<tr>
<th>Face</th>
<th>Net Pressure ($kN/m^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>$N2LW1=PeLW1-Pi2ww0=0.1336$</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>$N2LW2=PeLW2-Pi2ww0=-0.2126$</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N2LW3=PeLW3-Pi2ww0=-0.4089$</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N2LW4=PeLW4-Pi2ww0=-0.2999$</td>
</tr>
</tbody>
</table>

**Wall net pressure $C_{pi}=0.3$**  
Wind on short face of building

<table>
<thead>
<tr>
<th>Face</th>
<th>Net Pressure ($kN/m^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>$N2SW1=PeSW1-Pi2ww90=0.1024$</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>$N2SW2=PeSW2-Pi2ww90=-0.157$</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N2SW3=PeSW3-Pi2ww90=-0.3585$</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N2SW4=PeSW4-Pi2ww90=-0.2629$</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>$N2SW5=PeSW5-Pi2ww90=-0.1912$</td>
</tr>
</tbody>
</table>

**Coefficients $C_{pe}$ for monopitch roofs**

**Case 1: Wind angle $\theta=0^\circ$**

![Diagram of monopitch roof and wall net pressure calculations](image)

**Pitch of roof (5° to 75°) $\alpha=75^\circ$**

---

 SCALE 5.48  
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Case 3: Wind angle $\theta=90^\circ$

Roof net pressure $C_{pi}=-0.8$ \hspace{1cm} Wind $\theta=0^\circ$ (long face)

- Roof zone F
  - $N_{1RF0} = P_{RF0} - P_{iW1w0} = 0.2181$ kN/m²
- Roof zone G
  - $N_{1RG0} = P_{RG0} - P_{iW1w0} = 0.2181$ kN/m²
- Roof zone H
  - $N_{1RH0} = P_{RH0} - P_{iW1w0} = 0.2181$ kN/m²
- Roof zone F (+Ve Value)
  - $N_{1RAF0} = P_{RAF0} - P_{iW1w0} = 0.4362$ kN/m²
- Roof zone G (+Ve Value)
  - $N_{1RAG0} = P_{RAG0} - P_{iW1w0} = 0.4362$ kN/m²
- Roof zone H (+Ve Value)
  - $N_{1RAH0} = P_{RAH0} - P_{iW1w0} = 0.4362$ kN/m²

Roof net pressure $C_{pi}=-0.8$ \hspace{1cm} Wind $\theta=90^\circ$ (short face)

- Roof zone Fu
  - $N_{1Fu9} = P_{uRF9} - P_{iW1w90} = -0.0956$ kN/m²
- Roof zone Fl
  - $N_{1Fl9} = P_{lRF9} - P_{iW1w90} = -0.0956$ kN/m²
- Roof zone G
  - $N_{1RG9} = P_{RG9} - P_{iW1w90} = -0.0956$ kN/m²
- Roof zone H
  - $N_{1RH9} = P_{RH9} - P_{iW1w90} = 0.0956$ kN/m²
- Roof zone I
  - $N_{1RI9} = P_{RI9} - P_{iW1w90} = 0.1434$ kN/m²
- Roof zone Fu (+ve value)
  - $N_{1Rup9} = P_{pRu9} - P_{iW1w90} = 0.3823$ kN/m²
- Roof zone Fl (+ve value)
  - $N_{1Rlp9} = P_{pRl9} - P_{iW1w90} = 0.3823$ kN/m²
- Roof zone G (+ve value)
  - $N_{1RGp9} = P_{pRG9} - P_{iW1w90} = 0.3823$ kN/m²
- Roof zone H (+ve value)
  - $N_{1RHp9} = P_{pRH9} - P_{iW1w90} = 0.3584$ kN/m²
- Roof zone I (+ve value)
  - $N_{1RIp9} = P_{pRI9} - P_{iW1w90} = 0.3345$ kN/m²

Roof net pressure $C_{pi}=-0.8$ \hspace{1cm} Wind $\theta=180^\circ$

- Roof zone F
  - $N_{1RF8} = P_{RF8} - P_{iW1w0} = -0.0818$ kN/m²
- Roof zone G
  - $N_{1RG8} = P_{RG8} - P_{iW1w0} = -0.0818$ kN/m²
- Roof zone H
  - $N_{1RH8} = P_{RH8} - P_{iW1w0} = -0.0818$ kN/m²

Roof net pressure $C_{pi}=0.3$ \hspace{1cm} Wind $\theta=0^\circ$ (long face)

- Roof zone F
  - $N_{2RF0} = P_{RF0} - P_{iW2w0} = -0.0818$ kN/m²
- Roof zone G
  - $N_{2RG0} = P_{RG0} - P_{iW2w0} = -0.0818$ kN/m²
- Roof zone H
  - $N_{2RH0} = P_{RH0} - P_{iW2w0} = -0.0818$ kN/m²
- Roof zone F (+Ve Value)
  - $N_{2RAF0} = P_{RAF0} - P_{iW2w0} = 0.1363$ kN/m²
- Roof zone G (+Ve Value)
  - $N_{2RAG0} = P_{RAG0} - P_{iW2w0} = 0.1363$ kN/m²
- Roof zone H (+Ve Value)
  - $N_{2RAH0} = P_{RAH0} - P_{iW2w0} = 0.1363$ kN/m²
### Roof net pressure Cpi=0.3 Wind θ=90° (short face)

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Equation</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>F upper</td>
<td>$N_{2F u 9} = P_{u RF 9} - P_{i W 2 w 90}$</td>
<td>$-0.3584$ kN/m²</td>
</tr>
<tr>
<td>F lower</td>
<td>$N_{2F l 9} = P_{l RF 9} - P_{i W 2 w 90}$</td>
<td>$-0.3584$ kN/m²</td>
</tr>
<tr>
<td>G</td>
<td>$N_{2R G 9} = P_{e R G 9} - P_{i W 2 w 90}$</td>
<td>$-0.1673$ kN/m²</td>
</tr>
<tr>
<td>H</td>
<td>$N_{2R H 9} = P_{e R H 9} - P_{i W 2 w 90}$</td>
<td>$-0.1195$ kN/m²</td>
</tr>
<tr>
<td>I</td>
<td>$N_{2R I 9} = P_{e R I 9} - P_{i W 2 w 90}$</td>
<td>$-0.1195$ kN/m²</td>
</tr>
<tr>
<td>Fu (+ve value)</td>
<td>$N_{2R u p 9} = P_{p u R u 9} - P_{i W 2 w 90}$</td>
<td>$0.1195$ kN/m²</td>
</tr>
<tr>
<td>Fl (+ve value)</td>
<td>$N_{2R l p 9} = P_{p l R l 9} - P_{i W 2 w 90}$</td>
<td>$0.1195$ kN/m²</td>
</tr>
<tr>
<td>G (+ve value)</td>
<td>$N_{2R G p 9} = P_{p G R G 9} - P_{i W 2 w 90}$</td>
<td>$0.0956$ kN/m²</td>
</tr>
<tr>
<td>H (+ve value)</td>
<td>$N_{2R H p 9} = P_{p H R H 9} - P_{i W 2 w 90}$</td>
<td>$0.0717$ kN/m²</td>
</tr>
</tbody>
</table>

### Roof net pressure Cpi=0.3 Wind θ=180°

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Equation</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>$N_{2R F 8} = P_{e R F 8} - P_{i W 2 w 0}$</td>
<td>$-0.3817$ kN/m²</td>
</tr>
<tr>
<td>G</td>
<td>$N_{2R G 8} = P_{e R G 8} - P_{i W 2 w 0}$</td>
<td>$-0.2726$ kN/m²</td>
</tr>
<tr>
<td>H</td>
<td>$N_{2R H 8} = P_{e R H 8} - P_{i W 2 w 0}$</td>
<td>$-0.2726$ kN/m²</td>
</tr>
</tbody>
</table>
Location: Default case 4 Hipped roof structure

Obstruction height and upwind spacing

Wind direction

6Hav

2Hav

z=0

Hav

Xo

Hr

Hav=Obstruction height of other buildings

Building reference height
Hr=13.8 m

Buildings upwind of site
Hav=6.5 m

Upwind spacing of building
Xo=10 m

Displacement height
Hd(0)=5.2 m

Displacement height
Hd(90)=5.2 m

Building height
H=13.8 m

Standard wind loads

Basic wind velocity (Fig. NA.1)
Vbmap=20 m/s

Altitude above mean sea level
Asite=25 m

Directional factor (wind θ=0°)
Cdi(0)=1

Directional factor (wind θ=90°)
Cdi(90)=1

Distance upwind to shoreline
Dis(0)=2.5 km

Distance inside town terrain
Dit(0)=2.5 km

Distance upwind to shoreline
Dis(90)=2.5 km

Distance inside town terrain
Dit(90)=2.5 km

Probability factor
Cp=1

Effective height (long face)
He(0) = 8.6 m

Effective height (short face)
He(90) = 8.6 m

Altitude factor (long face)
Ca(0) = 1.025

Altitude factor (short face)
Ca(90) = 1.025

Direction factor (long face)
Cdi(0) = 1

Direction factor (short face)
Cdi(90) = 1

Seasonal factor
Cseason = 1

Probability factor
Cp = 1

Long face peak velocity pressure
qp(0) = 0.548 kN/m²

Short face peak velocity pressure
qp(90) = 0.548 kN/m²
Peak velocity pressure & pressure coefficients

on walls of rectangular buildings

Height of wall including parapet  $H' = 13.8$ m
Wind acting on long face of building (wind $\theta = 0^\circ$):
Peak velocity pressure based on height above ground $8.6$ m.
Peak velocity pressure @ eaves  $q_{pe}(0) = 0.548$ kN/m$^2$
Wind acting on short face of building (wind $\theta = 90^\circ$):
Peak velocity pressure based on height above ground $8.6$ m.
Peak velocity pressure @ eaves  $q_{pe}(90) = 0.548$ kN/m$^2$

Case 1: Wind on long face of building (i.e. angle $\theta = 0^\circ$)

![Plan View](image1)

Inwind depth  $d = 30$ m
Crosswind width  $b = 60$ m

Key zones for wall pressure data.

Case 2: Wind on short face of building (i.e. angle $\theta = 90^\circ$)

![Plan View](image2)

Inwind depth  $d = 60$ m
Crosswind width  $b = 30$ m
"0.75 times Cpe" "0.9*Cpe"

Internal pressure coefficient 1 Cpi(1)=-0.8

### Wall net pressure Cpi=-0.8 Wind on long face of building

<table>
<thead>
<tr>
<th>Side Face</th>
<th>Windward (front) face</th>
<th>Leeward (rear) face</th>
<th>Isolated Zone A side face</th>
<th>Isolated Zone B side face</th>
<th>Isolated Zone C side face</th>
<th>Funnelling Zone A side face</th>
<th>Funnelling Zone B side face</th>
<th>Funnelling Zone C side face</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N1LW1=PeLW1-Pi1ww0=0.8373 kN/m²</td>
<td>N1LW2=PeLW2-Pi1ww0=0.2433 kN/m²</td>
<td>N1LW3=PeLW3-Pi1ww0=-0.2192 kN/m²</td>
<td>N1LW4=PeLW4-Pi1ww0=0 kN/m²</td>
<td>N1LW5=PeLW5-Pi1ww0=0.1644 kN/m²</td>
<td>N1LW6=PeLW6-Pi1ww0=-0.4384 kN/m²</td>
<td>N1LW7=PeLW7-Pi1ww0=-0.0548 kN/m²</td>
<td>N1LW8=PeLW8-Pi1ww0=-0.0548 kN/m²</td>
</tr>
</tbody>
</table>

### Wall net pressure Cpi=-0.8 Wind on short face of building

<table>
<thead>
<tr>
<th>Side Face</th>
<th>Windward (front) face</th>
<th>Leeward (rear) face</th>
<th>Isolated Zone A side face</th>
<th>Isolated Zone B side face</th>
<th>Isolated Zone C side face</th>
<th>Funnelling Zone A side face</th>
<th>Funnelling Zone B side face</th>
<th>Funnelling Zone C side face</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N1SW1=PeSW1-Pi1ww90=0.822 kN/m²</td>
<td>N1SW2=PeSW2-Pi1ww90=0.274 kN/m²</td>
<td>N1SW3=PeSW3-Pi1ww90=-0.2192 kN/m²</td>
<td>N1SW4=PeSW4-Pi1ww90=0 kN/m²</td>
<td>N1SW5=PeSW5-Pi1ww90=0.1644 kN/m²</td>
<td>N1SW6=PeSW6-Pi1ww90=-0.4384 kN/m²</td>
<td>N1SW7=PeSW7-Pi1ww90=-0.0548 kN/m²</td>
<td>N1SW8=PeSW8-Pi1ww90=-0.0548 kN/m²</td>
</tr>
</tbody>
</table>
External pressure coefficients $C_{pe}$ for hipped roofs

Case 1: Wind angle $\theta=0^\circ$

Figure 7.9

Wind zones shown are for wind angle $\theta=0^\circ$.

$b = \text{cross wind dimension}$

Main roof slope to horizontal $\alpha=22.5^\circ$

Case 2: Wind angle $\theta=90^\circ$

Figure 7.9

Wind zones shown are for wind angle $\theta=90^\circ$.

Hipped end roof slope to horizontal $\alpha=30^\circ$

Roof net pressure $C_{pi}=-0.8$  

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Formula</th>
<th>Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>$N_{HF0}=Pe_{HF0}-Pi_{W1}w_0$</td>
<td>$-0.0548 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>G</td>
<td>$N_{HG0}=Pe_{HG0}-Pi_{W1}w_0$</td>
<td>$0.0822 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>H</td>
<td>$N_{HH0}=Pe_{HH0}-Pi_{W1}w_0$</td>
<td>$0.2466 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>J</td>
<td>$N_{HJ0}=Pe_{HJ0}-Pi_{W1}w_0$</td>
<td>$0.3014 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>K</td>
<td>$N_{HK0}=Pe_{HK0}-Pi_{W1}w_0$</td>
<td>$0.137 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>I</td>
<td>$N_{HI0}=Pe_{HI0}-Pi_{W1}w_0$</td>
<td>$0.1096 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>L</td>
<td>$N_{HL0}=Pe_{HL0}-Pi_{W1}w_0$</td>
<td>$0.0822 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>M</td>
<td>$N_{HM0}=Pe_{HM0}-Pi_{W1}w_0$</td>
<td>$0.1096 \text{ kN/m}^2$</td>
</tr>
<tr>
<td>J</td>
<td>$N_{HAJ0}=Pe_{HAJ0}-Pi_{W1}w_0$</td>
<td>$0.3014 \text{ kN/m}^2$</td>
</tr>
</tbody>
</table>
**Roof net pressure $C_{pi}=-0.8$**

Wind $\theta=90^\circ$ (short face)

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Calculation</th>
<th>Force in kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>$N_{1HF9} = P_{HF9} - P_{iW1w90}$</td>
<td>0.1644</td>
</tr>
<tr>
<td>G</td>
<td>$N_{1HG9} = P_{HG9} - P_{iW1w90}$</td>
<td>0.1644</td>
</tr>
<tr>
<td>H</td>
<td>$N_{1HH9} = P_{HH9} - P_{iW1w90}$</td>
<td>0.3288</td>
</tr>
<tr>
<td>J</td>
<td>$N_{1HJ9} = P_{HJ9} - P_{iW1w90}$</td>
<td>-0.274</td>
</tr>
<tr>
<td>I</td>
<td>$N_{1HI9} = P_{HI9} - P_{iW1w90}$</td>
<td>0.1096</td>
</tr>
<tr>
<td>L</td>
<td>$N_{1HL9} = P_{HL9} - P_{iW1w90}$</td>
<td>-0.1096</td>
</tr>
<tr>
<td>M</td>
<td>$N_{1HM9} = P_{HM9} - P_{iW1w90}$</td>
<td>0.1096</td>
</tr>
<tr>
<td>N</td>
<td>$N_{1HN9} = P_{HN9} - P_{iW1w90}$</td>
<td>0.1644</td>
</tr>
<tr>
<td>F (+Ve Value)</td>
<td>$N_{1HAF9} = P_{HAF9} - P_{iW1w90}$</td>
<td>0.8768</td>
</tr>
<tr>
<td>G (+Ve Value)</td>
<td>$N_{1HAG9} = P_{HAG9} - P_{iW1w90}$</td>
<td>0.7124</td>
</tr>
<tr>
<td>H (+Ve Value)</td>
<td>$N_{1HAH9} = P_{HAH9} - P_{iW1w90}$</td>
<td>0.6576</td>
</tr>
<tr>
<td>J (+Ve Value)</td>
<td>$N_{1HAJ9} = P_{HAJ9} - P_{iW1w90}$</td>
<td>-0.274</td>
</tr>
<tr>
<td>I (+Ve Value)</td>
<td>$N_{1HAI9} = P_{HAI9} - P_{iW1w90}$</td>
<td>0.1096</td>
</tr>
<tr>
<td>L (+Ve Value)</td>
<td>$N_{1HAL9} = P_{HAL9} - P_{iW1w90}$</td>
<td>0.4384</td>
</tr>
<tr>
<td>M (+Ve Value)</td>
<td>$N_{1HAM9} = P_{HAM9} - P_{iW1w90}$</td>
<td>0.4384</td>
</tr>
<tr>
<td>N (+Ve Value)</td>
<td>$N_{1HAN9} = P_{HAN9} - P_{iW1w90}$</td>
<td>0.4384</td>
</tr>
</tbody>
</table>
Location: Default case 5

Obstruction height and upwind spacing

Wind direction

6Hav

2Hav

z=0

Hav

Hr

Hd

Hav=Obstruction height of other buildings

Xo

Building reference height: Hr=13.8 m
Buildings upwind of site: Hav=6.5 m
Upwind spacing of building: Xo=10 m
Displacement height: Hd(0)=5.2 m, Hd(90)=5.2 m
Building height: H=13.8 m

Standard wind loads

Basic wind velocity (Fig. NA.1): Vbmap=20 m/s
Altitude above mean sea level: Asite=25 m
Directional factor (wind θ=0°): Cdi(0)=1
Directional factor (wind θ=90°): Cdi(90)=1
Distance upwind to shoreline: Dis(0)=2.5 km, Dis(90)=2.5 km
Distance inside town terrain: Dit(0)=2.5 km, Dit(90)=2.5 km
Probability factor: Cp=1

Effective height (long face): He(0)=8.6 m
Effective height (short face): He(90)=8.6 m
Altitude factor (long face): Ca(0)=1.025
Altitude factor (short face): Ca(90)=1.025
Direction factor (long face): Cdi(0)=1
Direction factor (short face): Cdi(90)=1
Seasonal factor: Cseason=1
Probability factor: Cp=1
Long face peak velocity pressure: qp(0)=0.548 kN/m²
Short face peak velocity pressure: qp(90)=0.548 kN/m²
**Peak velocity pressure & pressure coefficients**

on walls of rectangular buildings

Height of wall including parapet \( H_r' = 13.8 \text{ m} \)

Wind acting on long face of building (wind \( \theta = 0^\circ \)):
- Peak velocity pressure based on height above ground 8.6 m.
- Peak velocity pressure @ eaves \( q_{pe}(0) = 0.548 \text{ kN/m}^2 \)

Wind acting on short face of building (wind \( \theta = 90^\circ \)):
- Peak velocity pressure based on height above ground 8.6 m.
- Peak velocity pressure @ eaves \( q_{pe}(90) = 0.548 \text{ kN/m}^2 \)

**Case 1: Wind on long face of building (i.e. angle \( \theta = 0^\circ \))**

![Diagram](image)

**ELEVATION ON SIDE FACE**

key zones for wall pressure data.

**Case 2: Wind on short face of building (i.e. angle \( \theta = 90^\circ \))**

![Diagram](image)

Inwind depth \( d = 60 \text{ m} \)
Crosswind width \( b = 30 \text{ m} \)
Internal pressure coefficient 1  $C_{pi(1)} = -0.8$

**External pressure coefficients $C_{pe}$ for hipped roofs**

Case 1: Wind angle $\theta = 0^\circ$

$\frac{e}{10}$  

$\frac{e}{10}$  

$\frac{e}{10}$  

$\frac{e}{10}$

$b = $ cross wind dimension

Main roof slope to horizontal  $\alpha = 22.5^\circ$
Case 2: Wind angle $\theta=90^\circ$

Wind zones shown are for wind angle $\theta=90^\circ$.

Hipped end roof slope to horizontal $\alpha=30^\circ$
Location: Default case 6

Obstruction height and upwind spacing

Wind direction

\[ \text{Hav=Obstruction height of other buildings} \]

Building reference height \( \text{Hr}=10 \text{ m} \)
buildings upwind of site \( \text{Hav}=4 \text{ m} \)
Upwind spacing of building \( \text{Xo}=10 \text{ m} \)
Displacement height (wind \( \theta=0^\circ \)) \( \text{Hd}(0)=2.8 \text{ m} \)
Displacement height (wind \( \theta=90^\circ \)) \( \text{Hd}(90)=2.8 \text{ m} \)
Building height \( \text{H}=10 \text{ m} \)

Standard wind loads

Basic wind velocity (Fig. NA.1) \( \text{Vbmap}=21 \text{ m/s} \)
Site altitude above mean sea level \( \text{Asite}=25 \text{ m} \)

Orography is significant – Wind on long face of building

Slope details and site location relative to crest as per Figure NA.2 of NA to BS EN 1991-1-4:2005+A1:2010.

\[ \text{Effective height of the feature base of orography} \]

\( \text{Z}=5 \text{ m} \)

\( \text{Abase}=20 \text{ m} \)
Directional factor (wind $\theta=0^\circ$) $\text{Cdi}(0)=1$

Directional factor (wind $\theta=90^\circ$) $\text{Cdi}(90)=1$

Distance upwind to shoreline $\text{Dis}(0)=5$ km

Distance inside town terrain $\text{Dit}(0)=5$ km

Distance upwind to shoreline $\text{Dis}(90)=5$ km

Distance inside town terrain $\text{Dit}(90)=5$ km

Probability factor $\text{Cp}=1$

Effective height (long face) $\text{He}(0)=7.2$ m

Effective height (short face) $\text{He}(90)=7.2$ m

Altitude factor (long face) $\text{Ca}(0)=1.025$

Altitude factor (short face) $\text{Ca}(90)=1.025$

Direction factor (long face) $\text{Cdi}(0)=1$

Direction factor (short face) $\text{Cdi}(90)=1$

Seasonal factor $\text{Cseason}=1$

Probability factor $\text{Cp}=1$

Long face peak velocity pressure $q_p(0)=0.5679$ kN/m²

Short face peak velocity pressure $q_p(90)=0.5226$ kN/m²

Peak velocity pressure & pressure coefficients on walls of rectangular buildings

Height of wall including parapet $H_r'=10$ m

Wind acting on long face of building (wind $\theta=0^\circ$):

Peak velocity pressure based on height above ground 7.2 m.

Peak velocity pressure at eaves $q_p(0)=0.5679$ kN/m²

Wind acting on short face of building (wind $\theta=90^\circ$):

Peak velocity pressure based on height above ground 7.2 m.

Peak velocity pressure at eaves $q_p(90)=0.5226$ kN/m²

Case 1: Wind on long face of building (i.e. angle $\theta=0^\circ$)

Inwind depth $d=30$ m

Crosswind width $b=50$ m

PLAN VIEW
Case 2: Wind on short face of building (i.e. angle \( \theta = 90^\circ \))

Key zones for wall pressure data.

Inwind depth \( d = 50 \) m
Crosswind width \( b = 30 \) m
### Wall net pressure Cpi=0.2

<table>
<thead>
<tr>
<th>Face Type</th>
<th>Wall Net Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>( N_{1LW1} = P_{eLW1} - P_{i1ww0} = 0.2903 ) kN/m²</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>( N_{1LW2} = P_{eLW2} - P_{i1ww0} = -0.2966 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>( N_{1LW3} = P_{eLW3} - P_{i1ww0} = -0.7951 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>( N_{1LW4} = P_{eLW4} - P_{i1ww0} = -0.5679 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>( N_{1LW5} = P_{eLW5} - P_{i1ww0} = -0.3975 ) kN/m²</td>
</tr>
</tbody>
</table>

### Wall net pressure Cpi=0.2

<table>
<thead>
<tr>
<th>Face Type</th>
<th>Wall Net Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>( N_{1SW1} = P_{eSW1} - P_{i1ww90} = 0.2613 ) kN/m²</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>( N_{1SW2} = P_{eSW2} - P_{i1ww90} = -0.2613 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>( N_{1SW3} = P_{eSW3} - P_{i1ww90} = -0.7316 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>( N_{1SW4} = P_{eSW4} - P_{i1ww90} = -0.5226 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>( N_{1SW5} = P_{eSW5} - P_{i1ww90} = -0.3658 ) kN/m²</td>
</tr>
</tbody>
</table>

### Wall net pressure Cpi=-0.3

<table>
<thead>
<tr>
<th>Face Type</th>
<th>Wall Net Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>( N_{2LW1} = P_{eLW1} - P_{i2ww0} = 0.5742 ) kN/m²</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>( N_{2LW2} = P_{eLW2} - P_{i2ww0} = -0.0126 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>( N_{2LW3} = P_{eLW3} - P_{i2ww0} = -0.5111 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>( N_{2LW4} = P_{eLW4} - P_{i2ww0} = -0.284 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>( N_{2LW5} = P_{eLW5} - P_{i2ww0} = -0.1136 ) kN/m²</td>
</tr>
</tbody>
</table>

### Wall net pressure Cpi=-0.3

<table>
<thead>
<tr>
<th>Face Type</th>
<th>Wall Net Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (front) face</td>
<td>( N_{2SW1} = P_{eSW1} - P_{i2ww90} = 0.5226 ) kN/m²</td>
</tr>
<tr>
<td>Leeward (rear) face</td>
<td>( N_{2SW2} = P_{eSW2} - P_{i2ww90} = 0 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>( N_{2SW3} = P_{eSW3} - P_{i2ww90} = -0.4703 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>( N_{2SW4} = P_{eSW4} - P_{i2ww90} = -0.2613 ) kN/m²</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>( N_{2SW5} = P_{eSW5} - P_{i2ww90} = -0.1045 ) kN/m²</td>
</tr>
</tbody>
</table>

### External pressure coefficients Cpe for flat roofs

![Diagram](attachment:image.png)

- Wind on long face (\( \theta = 0^\circ \))
- Inwind depth: \( d = 30 \) m
- Crosswind width: \( b = 50 \) m
Loadings and foundations

Wind loads to Eurocode 1

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áäää b áääá´
úäääääääää¿    â
³     I     ³    ³        PLAN VIEW
³      ³           ³    ³        Wind on short face (θ=90°)
³      ³     H     ³    ³d
e/2     ³           ³    ³        Inwind depth       d=50 m
³  â   âäââãåâéââé´    ³        Crosswind width    b=30 m
³  ³   ³ F³  G  ³F ³    ³
á  å   àäåàäàäààààààààààà á
e/10         ãäåå e/4
³
Wind

Roof net pressure Cpi=0.2 Wind θ=0° (long face)

Roof zone F  N1RF0=PeRF0-PiW1w0=-1.249 kN/m²
Roof zone G  N1RG0=PeRG0-PiW1w0=-0.9087 kN/m²
Roof zone H  N1RH0=PeRH0-PiW1w0=-0.5111 kN/m²
Roof zone I 1 N1RI10=PeRI10-PiW1w0=0 kN/m²
Roof zone I 2 N1RI20=PeRI20-PiW1w0=-0.2272 kN/m²

Roof net pressure Cpi=0.2 Wind θ=90° (short face)

Roof zone F  N1RF9=PeRF9-PiW1w90=-1.15 kN/m²
Roof zone G  N1RG9=PeRG9-PiW1w90=-0.8361 kN/m²
Roof zone H  N1RH9=PeRH9-PiW1w90=-0.4703 kN/m²
Roof zone I 1 N1RI19=PeRI19-PiW1w90=0 kN/m²
Roof zone I 2 N1RI29=PeRI29-PiW1w90=-0.209 kN/m²

Roof net pressure Cpi=-0.3 Wind θ=0° (long face)

Roof zone F  N2RF0=PeRF0-PiW2w0=-0.9655 kN/m²
Roof zone G  N2RG0=PeRG0-PiW2w0=-0.6247 kN/m²
Roof zone H  N2RH0=PeRH0-PiW2w0=-0.2272 kN²/m²
Roof zone I 1 N2RI10=PeRI10-PiW2w0=0.284 kN/m²
Roof zone I 2 N2RI20=PeRI20-PiW2w0=0.0568 kN/m²

Roof net pressure Cpi=-0.3 Wind θ=90° (short face)

Roof zone F  N2RF9=PeRF9-PiW2w90=-0.8884 kN/m²
Roof zone G  N2RG9=PeRG9-PiW2w90=-0.5748 kN/m²
Roof zone H  N2RH9=PeRH9-PiW2w90=-0.209 kN²/m²
Roof zone I 1 N2RI19=PeRI19-PiW2w90=0.2613 kN/m²
Roof zone I 2 N2RI29=PeRI29-PiW2w90=0.0523 kN/m²
Location: Default case 7 (free-standing monopitch roof)

Obstruction height and upwind spacing

Wind direction

---

Hav=Obstruction height of other buildings

Building reference height

Hr=13.77 m

Buildings upwind of site

Hav=6.5 m

Upwind spacing of building

Xo=20 m

Displacement height

Hd=3.8 m

Building height

H=13.77 m

Standard wind loads

Basic wind velocity (Fig. NA.1)

Vbmap=21 m/s

Altitude above mean sea level

Asite=25 m

Directional factor

Cdi=1

Distance upwind to shoreline

Dist=2.5 km

Distance inside town terrain

Dit=2 km

Probability factor

Cp=1

Effective height

He 9.97 m

Altitude factor

Ca 1.025

Direction factor

Cdi 1

Seasonal factor

Cseason 1

Probability factor

Cp 1

Peak velocity pressure

qp 0.6512 kN/m²
Net pressure coefficients $C_{p_{net}}$ for free-standing monopitch roofs

in accordance with BS EN 1991-1-4:2005+A1:2010 – Table 7.6

Table 7.6 takes account of the combined wind effect on both upper & lower surfaces of the canopy for all wind directions.

Pitch of roof ($0^\circ$ to $30^\circ$) $\alpha_2=25^\circ$

**Net downward roof pressure (i.e. normal to the roof)**

Local net pressure for zone A $C_{p_{locA}}=q_p*C_{locAp}=1.302$ kN/m$^2$
Local net pressure for zone B $C_{p_{locB}}=q_p*C_{locBp}=2.019$ kN/m$^2$
Local net pressure for zone C $C_{p_{locC}}=q_p*C_{locCp}=1.498$ kN/m$^2$

**Net upward roof pressure (i.e. normal to the roof)**

Local net pressure for zone A $C_{p_{locA}}=q_p*C_{locAn}=-1.354$ kN/m$^2$
Local net pressure for zone B $C_{p_{locB}}=q_p*C_{locBn}=-1.667$ kN/m$^2$
Local net pressure for zone C $C_{p_{locC}}=q_p*C_{locCn}=-1.667$ kN/m$^2$

**Net pressure on fascias and gables**

In addition to pressure normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia at the eaves or any gable ends. The pressure for the windward and leeward faces could be taken as follows:

Net pressure (windward face) $C_{p_{win}}=q_p*1.3=0.8466$ kN/m$^2$
Net pressure (leeward face) $C_{plee}=q_p*0.6=0.3907$ kN/m$^2$
Location: Default case 8 (free-standing monopitch roof)

Obstruction height and upwind spacing

Wind direction

Wind direction

Obstruction height and upwind spacing

Xo

Hav=Obstruction height of other buildings

Building reference height

Hr=13.77 m

Hav=6.5 m

Xo=20 m

Hd=3.8 m

Building height

H=13.77 m

Standard wind loads

Basic wind velocity (Fig. NA.1)

Vbmap=21 m/s

Altitude above mean sea level

Asite=25 m

Directional factor

Cdi=1

Distance upwind to shoreline

Dist=2.5 km

Distance inside town terrain

Dit=2 km

Probability factor

Cp=1

Effective height

He 9.97 m

Altitude factor

Ca 1.025

Direction factor

Cdi 1

Seasonal factor

Cseason 1

Probability factor

Cp 1

Peak velocity pressure

qp 0.6512 kN/m²
Net pressure coefficients $C_{p_{\text{net}}}$ for free-standing monopitch roofs

in accordance with BS EN 1991-1-4:2005+A1:2010 - Table 7.6

Table 7.6 takes account of the combined wind effect on both upper & lower surfaces of the canopy for all wind directions.

Pitch of roof (0° to 30°) $\alpha_2=30°$

Net downward roof pressure (i.e. normal to the roof)

Local net pressure for zone A $C_{p_{\text{locA}}}=q_p*C_{\text{locA}}=1.433$ kN/m²
Local net pressure for zone B $C_{p_{\text{locB}}}=q_p*C_{\text{locB}}=2.084$ kN/m²
Local net pressure for zone C $C_{p_{\text{locC}}}=q_p*C_{\text{locC}}=1.563$ kN/m²

Net upward roof pressure (i.e. normal to the roof)

Local net pressure for zone A $C_{p_{\text{locA}}}=q_p*C_{\text{locA}}=-0.7293$ kN/m²
Local net pressure for zone B $C_{p_{\text{locB}}}=q_p*C_{\text{locB}}=-1.042$ kN/m²
Local net pressure for zone C $C_{p_{\text{locC}}}=q_p*C_{\text{locC}}=-1.198$ kN/m²

Net pressure on fascias and gables

In addition to pressure normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia at the eaves or any gable ends. The pressure for the windward and leeward faces could be taken as follows:

Net pressure (windward face) $C_{\text{win}}=q_p*1.3=0.8466$ kN/m²
Net pressure (leeward face) $C_{\text{lee}}=q_p*0.6=0.3907$ kN/m²
Location: Default case 9 (free-standing duopitch roof)

Obstruction height and upwind spacing

Wind direction

\[ \text{Hav} = \text{Obstruction height of other buildings} \]

\[ \text{Hr} = 13.77 \text{ m} \]

\[ \text{Hav} = 6.5 \text{ m} \]

\[ \text{Xo} = 20 \text{ m} \]

\[ \text{Displacement height} \]

\[ \text{Hd} = 3.8 \text{ m} \]

\[ \text{Building height} \]

\[ \text{H} = 13.77 \text{ m} \]

Standard wind loads

Basic wind velocity (Fig. NA.1) \[ \text{Vbmap} = 21 \text{ m/s} \]

Altitude above mean sea level \[ \text{Asite} = 25 \text{ m} \]

Directional factor \[ \text{Cdi} = 1 \]

Distance upwind to shoreline \[ \text{Dist} = 2.5 \text{ km} \]

Distance inside town terrain \[ \text{Dit} = 2 \text{ km} \]

Probability factor \[ \text{Cp} = 1 \]

Effective height \[ \text{He} = 9.97 \text{ m} \]

Altitude factor \[ \text{Ca} = 1.025 \]

Direction factor \[ \text{Cdi} = 1 \]

Seasonal factor \[ \text{Cseason} = 1 \]

Probability factor \[ \text{Cp} = 1 \]

Peak velocity pressure \[ \text{qp} = 0.6512 \text{ kN/m}^2 \]
Net pressure coefficients $C_{p_{net}}$ for free-standing duopitch roofs

in accordance with BS EN 1991-1-4:2005+A1:2010 - Table 7.7

Table 7.7 takes account of the combined effect of the wind on both upper and lower surfaces of the canopy for all wind directions.

Pitch of roof (-20° to 30°) $\alpha_3=5^\circ$

**Net downward roof pressure (i.e. normal to the roof)**

Local net pressure for zone A $C_{p_{locA}}=q_p*C_{locAp}=0.3907$ kN/m²
Local net pressure for zone B $C_{p_{locB}}=q_p*C_{locBp}=1.172$ kN/m²
Local net pressure for zone C $C_{p_{locC}}=q_p*C_{locCp}=0.8466$ kN/m²
Local net pressure for zone D $C_{p_{locD}}=q_p*C_{locDp}=0.2605$ kN/m²

**Net upward roof pressure (i.e. normal to the roof)**

Local net pressure for zone A $C_{p_{locA}}=q_p*C_{locAn}=-0.3126$ kN/m²
Local net pressure for zone B $C_{p_{locB}}=q_p*C_{locBn}=-0.7293$ kN/m²
Local net pressure for zone C $C_{p_{locC}}=q_p*C_{locCn}=-0.7293$ kN/m²
Local net pressure for zone D $C_{p_{locD}}=q_p*C_{locDn}=-0.573$ kN/m²

**Net pressure on fascias and gables**

In addition to pressure normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia at the eaves or any gable ends. The pressure for the windward and leeward faces could be taken as follows:

Net pressure (windward face) $C_{p_{win}}=q_p*1.3=0.8466$ kN/m²
Net pressure (leeward face) $C_{p_{lee}}=q_p*0.6=0.3907$ kN/m²
Location: Default case 10 (free-standing duopitch roof)

Obstruction height and upwind spacing

Wind direction

Wind direction

Hav

z=0

2Hav

6Hav

2Hav

He

Hd

Hr

Hav=Obstruction height of other buildings

Xo

Building reference height

Hr=13.77 m

buildings upwind of site

Hav=6.5 m

Upwind spacing of building

Xo=20 m

Displacement height

Hd=3.8 m

Building height

H=13.77 m

Standard wind loads

Basic wind velocity (Fig. NA.1)

Vbmap=21 m/s

Altitude above mean sea level

Asite=25 m

Directional factor

Cdi=1

Distance upwind to shoreline

Dist=2.5 km

Distance inside town terrain

Dit=2.5 km

Probability factor

Cp=1

Effective height

He 9.97 m

Altitude factor

Ca 1.025

Direction factor

Cdi 1

Seasonal factor

Cseason 1

Probability factor

Cp 1

Peak velocity pressure

qp 0.6416 kN/m²
Net pressure coefficients \( C_{\text{net}} \) for free-standing duopitch roofs

in accordance with BS EN 1991-1-4:2005+A1:2010 - Table 7.7

<table>
<thead>
<tr>
<th>Alpha 3 (°)</th>
<th>CplocA</th>
<th>CplocB</th>
<th>CplocC</th>
<th>CplocD</th>
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<tbody>
<tr>
<td>0</td>
<td>0.8341</td>
<td>1.219</td>
<td>1.027</td>
<td>0.4492</td>
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<tr>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
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<td></td>
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<tr>
<td>45</td>
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<td>60</td>
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</tr>
<tr>
<td>75</td>
<td></td>
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</tr>
<tr>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Pitch of roof (-20° to 30°)  alpha3=30°

**Net downward roof pressure (i.e. normal to the roof)**

- Local net pressure for zone A \( C_{\text{locA}}=q_{\text{p}}*C_{\text{locAp}}=0.8341 \text{ kN/m}^2 \)
- Local net pressure for zone B \( C_{\text{locB}}=q_{\text{p}}*C_{\text{locBp}}=1.219 \text{ kN/m}^2 \)
- Local net pressure for zone C \( C_{\text{locC}}=q_{\text{p}}*C_{\text{locCp}}=1.027 \text{ kN/m}^2 \)
- Local net pressure for zone D \( C_{\text{locD}}=q_{\text{p}}*C_{\text{locDp}}=0.4492 \text{ kN/m}^2 \)

**Net upward roof pressure (i.e. normal to the roof)**

- Local net pressure for zone A \( C_{\text{locA}}=q_{\text{p}}*C_{\text{locAn}}=-0.7186 \text{ kN/m}^2 \)
- Local net pressure for zone B \( C_{\text{locB}}=q_{\text{p}}*C_{\text{locBn}}=-0.924 \text{ kN/m}^2 \)
- Local net pressure for zone C \( C_{\text{locC}}=q_{\text{p}}*C_{\text{locCn}}=-0.7186 \text{ kN/m}^2 \)
- Local net pressure for zone D \( C_{\text{locD}}=q_{\text{p}}*C_{\text{locDn}}=-1.027 \text{ kN/m}^2 \)

**Net pressure on fascias and gables**

In addition to pressure normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia at the eaves or any gable ends. The pressure for the windward and leeward faces could be taken as follows:

- Net pressure (winward face) \( C_{\text{win}}=q_{\text{p}}*1.3=0.8341 \text{ kN/m}^2 \)
- Net pressure (leeward face) \( C_{\text{lee}}=q_{\text{p}}*0.6=0.385 \text{ kN/m}^2 \)
Location: Default case 11 (duopitch roof)

Obstruction height and upwind spacing

Wind direction

<table>
<thead>
<tr>
<th>Wind direction</th>
<th>6Hav</th>
</tr>
</thead>
<tbody>
<tr>
<td>z=0</td>
<td></td>
</tr>
<tr>
<td>2Hav</td>
<td>He</td>
</tr>
<tr>
<td>3Hav</td>
<td></td>
</tr>
<tr>
<td>4Hav</td>
<td></td>
</tr>
<tr>
<td>5Hav</td>
<td></td>
</tr>
<tr>
<td>6Hav</td>
<td></td>
</tr>
</tbody>
</table>

Hav=Obstruction height of other buildings

Building reference height  
Hr=13.77 m

Building upwind of site  
Hav=6.5 m

Upwind spacing of building  
Xo=20 m

Displacement height (wind θ=0°)  
Hd(0)=3.8 m

Displacement height (wind θ=90°)  
Hd(90)=3.8 m

Building height  
H=13.77 m

Standard wind loads

Basic wind velocity (Fig. NA.1)  
Vbmap=21 m/s

Altitude above mean sea level  
Asite=25 m

Directional factor (wind θ=0°)  
Cdi(0)=1

Directional factor (wind θ=90°)  
Cdi(90)=1

Distance upwind to shoreline  
Dis(0)=2.5 km

Distance inside town terrain  
Dit(0)=2.5 km

Distance upwind to shoreline  
Dis(90)=2.5 km

Distance inside town terrain  
Dit(90)=2.5 km

Probability factor  
Cp=1

Effective height (long face)  
He(0)=9.97 m

Effective height (short face)  
He(90)=9.97 m

Altitude factor (long face)  
Ca(0)=1.025

Altitude factor (short face)  
Ca(90)=1.025

Direction factor (long face)  
Cdi(0)=1

Direction factor (short face)  
Cdi(90)=1

Seasonal factor  
Cseason=1

Probability factor  
Cp=1

Long face peak velocity pressure  
qp(0)=0.6416 kN/m²

Short face peak velocity pressure  
qp(90)=0.6416 kN/m²
Peak velocity pressure & pressure coefficients

on walls of rectangular buildings

Height of wall including parapet Hr' = 13.77 m
Wind acting on long face of building (wind θ=0°):
Peak velocity pressure based on height above ground 9.97 m.
Peak velocity pressure @ eaves qpe(0) = 0.6416 kN/m²
Wind acting on short face of building (wind θ=90°):
Peak velocity pressure based on height above ground 9.97 m.
Peak velocity pressure @ eaves qpe(90) = 0.6416 kN/m²

Case 1: Wind on long face of building (i.e. angle θ=0°)

Inwind depth d = 20 m
Crosswind width b = 60 m

Key zones for wall pressure data.
Scaling width > d
27.54 m > 20 m

Elevation on side face
Case 2: Wind on short face of building (i.e. angle θ=90°)

![Plan View Diagram]

Key zones for wall pressure data.

Plan View:
- Inwind depth: d = 60 m
- Crosswind width: b = 20 m

Elevation on Side Face:
- Height: H = 13.77 m

Wall net pressure Cpi=0.2

Windward (front) face: N1LW1=PeLW1-Pi1ww0=0.3583 kN/m²
Leeward (rear) face: N1LW2=PeLW2-Pi1ww0=-0.3958 kN/m²
Isolated Zone A side face: N1LW3=PeLW3-Pi1ww0=-0.8982 kN/m²
Isolated Zone B side face: N1LW4=PeLW4-Pi1ww0=-0.6416 kN/m²

Wall net pressure Cpi=-0.3

Windward (front) face: N2LW1=PeLW1-Pi2ww0=0.6791 kN/m²
Leeward (rear) face: N2LW2=PeLW2-Pi2ww0=-0.0750 kN/m²
Isolated Zone A side face: N2LW3=PeLW3-Pi2ww0=-0.5774 kN/m²
Isolated Zone B side face: N2LW4=PeLW4-Pi2ww0=-0.3208 kN/m²

Wind on long face of building

Windward (front) face: N1LW1=PeLW1-Pi1ww0=0.3583 kN/m²
Leeward (rear) face: N1LW2=PeLW2-Pi1ww0=-0.3958 kN/m²
Isolated Zone A side face: N1LW3=PeLW3-Pi1ww0=-0.8982 kN/m²
Isolated Zone B side face: N1LW4=PeLW4-Pi1ww0=-0.6416 kN/m²

Wind on short face of building

Windward (front) face: N1SW1=PeSW1-Pi1ww90=0.3208 kN/m²
Leeward (rear) face: N1SW2=PeSW2-Pi1ww90=-0.3208 kN/m²
Isolated Zone A side face: N1SW3=PeSW3-Pi1ww90=-0.8982 kN/m²
Isolated Zone B side face: N1SW4=PeSW4-Pi1ww90=-0.6416 kN/m²
Isolated Zone C side face: N1SW5=PeSW5-Pi1ww90=-0.4491 kN/m²

Wall net pressure Cpi=-0.3

Windward (front) face: N2SW1=PeSW1-Pi2ww90=0.6416 kN/m²
Leeward (rear) face: N2SW2=PeSW2-Pi2ww90=0.0 kN/m²
Isolated Zone A side face: N2SW3=PeSW3-Pi2ww90=-0.5774 kN/m²

Wind on long face of building

Windward (front) face: N2LW1=PeLW1-Pi2ww0=0.6791 kN/m²
Leeward (rear) face: N2LW2=PeLW2-Pi2ww0=-0.0750 kN/m²
Isolated Zone A side face: N2LW3=PeLW3-Pi2ww0=-0.5774 kN/m²
Isolated Zone B side face: N2LW4=PeLW4-Pi2ww0=-0.3208 kN/m²

Wall net pressure Cpi=-0.3

Windward (front) face: N2SW1=PeSW1-Pi2ww90=0.6416 kN/m²
Leeward (rear) face: N2SW2=PeSW2-Pi2ww90=0.0 kN/m²
Isolated Zone A side face: N2SW3=PeSW3-Pi2ww90=-0.5774 kN/m²
Sample output for SCALE Proforma 702. (ans=11)                              Page:  4
Loadings and foundations                                         Made by:  IFB
Wind loads to Eurocode 1                                                Date:  02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.                                Ref No:  SC702 EC

Isolated Zone B side face  N2SW4=PeSW4-Pi2ww90=-0.3208 kN/m²
Isolated Zone C side face  N2SW5=PeSW5-Pi2ww90=-0.1283 kN/m²

External pressure coefficients Cpe for duopitch roofs

Case 1: Wind angle θ=0°

![Diagram of a duopitch roof showing b = cross wind dimension and roof slope to horizontal]

Case 2: Wind angle θ=90°

![Diagram of a duopitch roof showing roof net pressure Cpi=0.2 and Wind θ=0° (long face)]

Roof net pressure Cpi=0.2  Wind θ=0° (long face)

<table>
<thead>
<tr>
<th>Roof zone F</th>
<th>N1RF0=PeRF0-PiW1w0=-0.4492 kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof zone G</td>
<td>N1RG0=PeRG0-PiW1w0=-0.4492 kN/m²</td>
</tr>
<tr>
<td>Roof zone H</td>
<td>N1RH0=PeRH0-PiW1w0=-0.2567 kN/m²</td>
</tr>
<tr>
<td>Roof zone J</td>
<td>N1RJ0=PeRJ0-PiW1w0=-0.7058 kN/m²</td>
</tr>
<tr>
<td>Roof zone I</td>
<td>N1RI0=PeRI0-PiW1w0=-0.4492 kN/m²</td>
</tr>
<tr>
<td>Roof zone F (+Ve Value)</td>
<td>N1RAFO=PRAFO-PiW1w0=0.385 kN/m²</td>
</tr>
<tr>
<td>Roof zone G (+Ve Value)</td>
<td>N1RAGO=PRAGO-PiW1w0=0.1925 kN/m²</td>
</tr>
<tr>
<td>Roof zone H (+Ve Value)</td>
<td>N1RAHO=PRAHO-PiW1w0=0.1283 kN/m²</td>
</tr>
<tr>
<td>Roof zone J (+Ve Value)</td>
<td>N1RAJ0=PRAJ0-PiW1w0=-0.7058 kN/m²</td>
</tr>
<tr>
<td>Roof zone I (+Ve Value)</td>
<td>N1RAI0=PRAI0-PiW1w0=-0.4492 kN/m²</td>
</tr>
</tbody>
</table>

SCALE 5.48                     Office 1007                 Proforma 702
### Roof net pressure $C_{pi}=0.2$

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Calculation</th>
<th>Result ($kN/m^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>$N1RF9=PeRF9-PiW1w90$</td>
<td>-0.8983</td>
</tr>
<tr>
<td>G</td>
<td>$N1RG9=PeRG9-PiW1w90$</td>
<td>-0.8341</td>
</tr>
<tr>
<td>H</td>
<td>$N1RH9=PeRH9-PiW1w90$</td>
<td>-0.5133</td>
</tr>
<tr>
<td>I</td>
<td>$N1RI9=PeRI9-PiW1w90$</td>
<td>-0.4492</td>
</tr>
<tr>
<td>F (+Ve Value)</td>
<td>$N1RAF9=PRAF9-PiW1w90$</td>
<td>0.1925</td>
</tr>
<tr>
<td>G (+Ve Value)</td>
<td>$N1RAG9=PRAG9-PiW1w90$</td>
<td>0.1283</td>
</tr>
<tr>
<td>H (+Ve Value)</td>
<td>$N1RAH9=PRAH9-PiW1w90$</td>
<td>0.0642</td>
</tr>
<tr>
<td>I (+Ve Value)</td>
<td>$N1RAI9=PRAI9-PiW1w90$</td>
<td>0</td>
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</tbody>
</table>

### Roof net pressure $C_{pi}=-0.3$

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Calculation</th>
<th>Result ($kN/m^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>$N2RF0=PeRF0-PiW2w0$</td>
<td>-0.1283</td>
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<tr>
<td>G</td>
<td>$N2RG0=PeRG0-PiW2w0$</td>
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<tr>
<td>H</td>
<td>$N2RH0=PeRH0-PiW2w0$</td>
<td>0.0642</td>
</tr>
<tr>
<td>J</td>
<td>$N2RJ0=PeRJ0-PiW2w0$</td>
<td>-0.385</td>
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<tr>
<td>I</td>
<td>$N2RI0=PeRI0-PiW2w0$</td>
<td>-0.1283</td>
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<tr>
<td>F (+Ve Value)</td>
<td>$N2RAF0=PRAF0-PiW2w0$</td>
<td>0.7058</td>
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<tr>
<td>G (+Ve Value)</td>
<td>$N2RAG0=PRAG0-PiW2w0$</td>
<td>0.5133</td>
</tr>
<tr>
<td>H (+Ve Value)</td>
<td>$N2RAH0=PRAH0-PiW2w0$</td>
<td>0.4492</td>
</tr>
<tr>
<td>J (+Ve Value)</td>
<td>$N2RAJ0=PRAJ0-PiW2w0$</td>
<td>-0.385</td>
</tr>
<tr>
<td>I (+Ve Value)</td>
<td>$N2RAI0=PRAI0-PiW2w0$</td>
<td>-0.1283</td>
</tr>
</tbody>
</table>

### Roof net pressure $C_{pi}=-0.3$

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Calculation</th>
<th>Result ($kN/m^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>$N2RF9=PeRF9-PiW2w90$</td>
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<tr>
<td>G</td>
<td>$N2RG9=PeRG9-PiW2w90$</td>
<td>-0.5133</td>
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<tr>
<td>H</td>
<td>$N2RH9=PeRH9-PiW2w90$</td>
<td>-0.1925</td>
</tr>
<tr>
<td>I</td>
<td>$N2RI9=PeRI9-PiW2w90$</td>
<td>-0.1283</td>
</tr>
<tr>
<td>F (+Ve Value)</td>
<td>$N2RAF9=PRAF9-PiW2w90$</td>
<td>0.5133</td>
</tr>
<tr>
<td>G (+Ve Value)</td>
<td>$N2RAG9=PRAG9-PiW2w90$</td>
<td>0.4492</td>
</tr>
<tr>
<td>H (+Ve Value)</td>
<td>$N2RAH9=PRAH9-PiW2w90$</td>
<td>0.385</td>
</tr>
<tr>
<td>I (+Ve Value)</td>
<td>$N2RAI9=PRAI9-PiW2w90$</td>
<td>0.3208</td>
</tr>
</tbody>
</table>
Location: Default case 12 (duopitch roof)

Obstruction height and upwind spacing

Hav=Obstruction height of other buildings

Building reference height \( H_r=4.3 \) m
Buildings upwind of site \( Hav=0.5 \) m
Upwind spacing of building \( X_o=20 \) m
Displacement height (wind \( \theta=0^\circ \)) \( H_d(0)=0 \) m
Displacement height (wind \( \theta=90^\circ \)) \( H_d(90)=0 \) m
Building height \( H=4.3 \) m

Standard wind loads

Basic wind velocity (Fig. NA.1) \( V_{b\text{map}}=24 \) m/s
Site altitude above mean sea level \( A_{\text{site}}=46 \) m

Orography is significant - Wind on long face of building

Slope details and site location relative to crest as per Figure NA.2 of NA to BS EN 1991-1-4:2005+A1:2010.

Effective height of the feature \( Z=45 \) m
Directional factor (wind θ=0°)  Cdi(0)=1
Directional factor (wind θ=90°)  Cdi(90)=1
Distance upwind to shoreline Dis(0)=2.5 km
Distance upwind to shoreline Dis(90)=2.5 km
Probability factor Cp=1

Effective height (long face) He(0) 4.3 m
Effective height (short face) He(90) 4.3 m
Altitude factor (long face) Ca(0) 1.046
Altitude factor (short face) Ca(90) 1.046

DESIGN
Altitude factor (short face) Ca(90) 1.046
Direction factor (long face) Cdi(0) 1
Direction factor (short face) Cdi(90) 1
Seasonal factor Cseason 1
Probability factor Cp 1
Long face peak velocity pressure qp(0) 1.114 kN/m²
Short face peak velocity pressure qp(90) 0.7919 kN/m²

Peak velocity pressure & pressure coefficients

on walls of rectangular buildings

Height of wall including parapet Hr'=4.3 m
Wind acting on long face of building (wind θ=0°):
Peak velocity pressure based on height above ground 4.3 m.
Peak velocity pressure @ eaves qpe(0)=1.114 kN/m²
Wind acting on short face of building (wind θ=90°):
Peak velocity pressure based on height above ground 4.3 m.
Peak velocity pressure @ eaves qpe(90)=0.7919 kN/m²

Case 1: Wind on long face of building (i.e. angle θ=0°)

Inwind depth d=20 m
Crosswind width b=60 m

Key zones for wall pressure data.

SCALE 5.48 Office 1007 Proforma 702
Case 2: Wind on short face of building (i.e. angle $\theta=90^\circ$)

![Plan View Diagram]

**Plan View**

- Inwind depth $d=60$ m
- Crosswind width $b=20$ m

**Key zones for wall pressure data.**

**Elevation on Side Face**

- A
- B
- C
- H=4.3 m

**Wall net pressure $C_{pi}=0.2$**

<table>
<thead>
<tr>
<th>Windward (front) face</th>
<th>$N_{1LW1}=Pe_{LW1}-P_{i1ww0}=0.557$ kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leeward (rear) face</td>
<td>$N_{1LW2}=Pe_{LW2}-P_{i1ww0}=-0.557$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N_{1LW3}=Pe_{LW3}-P_{i1ww0}=-1.56$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N_{1LW4}=Pe_{LW4}-P_{i1ww0}=-1.114$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>$N_{1LW5}=Pe_{LW5}-P_{i1ww0}=-0.7798$ kN/m²</td>
</tr>
</tbody>
</table>

**Wall net pressure $C_{pi}=-0.3$**

<table>
<thead>
<tr>
<th>Windward (front) face</th>
<th>$N_{2LW1}=Pe_{LW1}-P_{i2ww0}=1.114$ kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leeward (rear) face</td>
<td>$N_{2LW2}=Pe_{LW2}-P_{i2ww0}=0$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N_{2LW3}=Pe_{LW3}-P_{i2ww0}=-1.003$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N_{2LW4}=Pe_{LW4}-P_{i2ww0}=-0.557$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>$N_{2LW5}=Pe_{LW5}-P_{i2ww0}=-0.2228$ kN/m²</td>
</tr>
</tbody>
</table>

**Wind on long face of building**

<table>
<thead>
<tr>
<th>Windward (front) face</th>
<th>$N_{1LW1}=Pe_{LW1}-P_{i1ww0}=0.557$ kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leeward (rear) face</td>
<td>$N_{1LW2}=Pe_{LW2}-P_{i1ww0}=-0.557$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N_{1LW3}=Pe_{LW3}-P_{i1ww0}=-1.56$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N_{1LW4}=Pe_{LW4}-P_{i1ww0}=-1.114$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>$N_{1LW5}=Pe_{LW5}-P_{i1ww0}=-0.7798$ kN/m²</td>
</tr>
</tbody>
</table>

**Wind on short face of building**

<table>
<thead>
<tr>
<th>Windward (front) face</th>
<th>$N_{2LW1}=Pe_{LW1}-P_{i2ww0}=1.114$ kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leeward (rear) face</td>
<td>$N_{2LW2}=Pe_{LW2}-P_{i2ww0}=0$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N_{2LW3}=Pe_{LW3}-P_{i2ww0}=-1.003$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N_{2LW4}=Pe_{LW4}-P_{i2ww0}=-0.557$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>$N_{2LW5}=Pe_{LW5}-P_{i2ww0}=-0.2228$ kN/m²</td>
</tr>
</tbody>
</table>

**Wall net pressure $C_{pi}=-0.3$**

<table>
<thead>
<tr>
<th>Windward (front) face</th>
<th>$N_{2LW1}=Pe_{LW1}-P_{i2ww0}=1.114$ kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leeward (rear) face</td>
<td>$N_{2LW2}=Pe_{LW2}-P_{i2ww0}=0$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone A side face</td>
<td>$N_{2LW3}=Pe_{LW3}-P_{i2ww0}=-1.003$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone B side face</td>
<td>$N_{2LW4}=Pe_{LW4}-P_{i2ww0}=-0.557$ kN/m²</td>
</tr>
<tr>
<td>Isolated Zone C side face</td>
<td>$N_{2LW5}=Pe_{LW5}-P_{i2ww0}=-0.2228$ kN/m²</td>
</tr>
</tbody>
</table>
Leeward (rear) face  
Isolated Zone A side face  
Isolated Zone B side face  
Isolated Zone C side face  

External pressure coefficients Cpe for duopitch roofs

Case 1: Wind angle $\theta=0^\circ$

Case 2: Wind angle $\theta=90^\circ$

Roof net pressure $Cpi=0.2$  
Wind $\theta=0^\circ$ (long face)
### Roof net pressure Cpi=0.2

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Calculation</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>N1RF9=PeRF9-PiW1w90</td>
<td>-1.109 kN/m²</td>
</tr>
<tr>
<td>G</td>
<td>N1RG9=PeRG9-PiW1w90</td>
<td>-1.029 kN/m²</td>
</tr>
<tr>
<td>H</td>
<td>N1RH9=PeRH9-PiW1w90</td>
<td>-0.6335 kN/m²</td>
</tr>
<tr>
<td>I</td>
<td>N1RI9=PeRI9-PiW1w90</td>
<td>-0.5543 kN/m²</td>
</tr>
<tr>
<td>F (+Ve Value)</td>
<td>N1RAF9=PRAF9-PiW1w90</td>
<td>0.2376 kN/m²</td>
</tr>
<tr>
<td>G (+Ve Value)</td>
<td>N1RAG9=PRAG9-PiW1w90</td>
<td>0.1584 kN/m²</td>
</tr>
<tr>
<td>H (+Ve Value)</td>
<td>N1RAH9=PRAH9-PiW1w90</td>
<td>0.0792 kN/m²</td>
</tr>
<tr>
<td>I (+Ve Value)</td>
<td>N1RAI9=PRAI9-PiW1w90</td>
<td>0 kN/m²</td>
</tr>
</tbody>
</table>

### Roof net pressure Cpi=-0.3

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Calculation</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>N2RF0=PeRF0-PiW2w0</td>
<td>-0.2229 kN/m²</td>
</tr>
<tr>
<td>G</td>
<td>N2RG0=PeRG0-PiW2w0</td>
<td>-0.2229 kN/m²</td>
</tr>
<tr>
<td>H</td>
<td>N2RH0=PeRH0-PiW2w0</td>
<td>0.1114 kN/m²</td>
</tr>
<tr>
<td>J</td>
<td>N2RJ0=PeRJ0-PiW2w0</td>
<td>-0.6686 kN/m²</td>
</tr>
<tr>
<td>I</td>
<td>N2RI0=PeRI0-PiW2w0</td>
<td>-0.2229 kN/m²</td>
</tr>
<tr>
<td>F (+Ve Value)</td>
<td>N2RAF0=PRAF0-PiW2w0</td>
<td>1.226 kN/m²</td>
</tr>
<tr>
<td>G (+Ve Value)</td>
<td>N2RAG0=PRAG0-PiW2w0</td>
<td>0.8915 kN/m²</td>
</tr>
<tr>
<td>H (+Ve Value)</td>
<td>N2RAH0=PRAH0-PiW2w0</td>
<td>0.7801 kN/m²</td>
</tr>
<tr>
<td>J (+Ve Value)</td>
<td>N2RAJ0=PRAJ0-PiW2w0</td>
<td>-0.6686 kN/m²</td>
</tr>
<tr>
<td>I (+Ve Value)</td>
<td>N2RAI0=PRAI0-PiW2w0</td>
<td>-0.2229 kN/m²</td>
</tr>
</tbody>
</table>

### Roof net pressure Cpi=-0.3

<table>
<thead>
<tr>
<th>Roof zone</th>
<th>Calculation</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>N2RF9=PeRF9-PiW2w90</td>
<td>-0.7127 kN/m²</td>
</tr>
<tr>
<td>G</td>
<td>N2RG9=PeRG9-PiW2w90</td>
<td>-0.6335 kN/m²</td>
</tr>
<tr>
<td>H</td>
<td>N2RH9=PeRH9-PiW2w90</td>
<td>0.2376 kN/m²</td>
</tr>
<tr>
<td>I</td>
<td>N2RI9=PeRI9-PiW2w90</td>
<td>0.1584 kN/m²</td>
</tr>
<tr>
<td>F (+Ve Value)</td>
<td>N2RAF9=PRAF9-PiW2w90</td>
<td>0.6335 kN/m²</td>
</tr>
<tr>
<td>G (+Ve Value)</td>
<td>N2RAG9=PRAG9-PiW2w90</td>
<td>0.5543 kN/m²</td>
</tr>
<tr>
<td>H (+Ve Value)</td>
<td>N2RAH9=PRAH9-PiW2w90</td>
<td>0.4751 kN/m²</td>
</tr>
<tr>
<td>I (+Ve Value)</td>
<td>N2RAI9=PRAI9-PiW2w90</td>
<td>0.396 kN/m²</td>
</tr>
</tbody>
</table>
Location: Default case 13 (free-standing wall)

Free-standing wall details

Eff. height of free-standing wall  \( H_e = 5 \) m

Free-standing wall details

\[
\begin{array}{c}
20 \text{ m} \\
10 \text{ m} \\
1.5 \text{ m} \\
\end{array}
\]

\[
\begin{array}{cccc}
& A & B & C & D \\
He & \ & \ & \ & \\
\end{array}
\]

ELEVATION ON FREE-STANDING WALL

Standard wind loads

Basic wind velocity (Fig. NA.1)  \( V_{b\text{map}} = 21 \) m/s
Site altitude above mean sea level  \( A_{\text{site}} = 25 \) m

Slope details and site location relative to crest as per Fig. NA.2


Xt<0  Xt>0
\[
\begin{array}{c}
\text{Wind} \\
\downarrow \\
\text{Site} \\
\text{crest} \\
\text{Abase} \\
\text{Lu} \\
\end{array}
\]

Site altitude is 25 m above mean sea level.
Xt is positive if site is located downwind of crest.
Xt is negative if site is located upwind of crest.

Height to crest  \( Z = 10 \) m
Directional factor \( C_{di} = 1 \)
Distance upwind to shoreline \( D_{dist} = 2.5 \) km
Distance inside town terrain \( D_{dit} = 2 \) km
Probability factor \( C_{p} = 1 \)

Effective height \( H_e = 5 \) m

**Design**
- Altitude factor \( C_a = 1.025 \)
- Direction factor \( C_{di} = 1 \)
- Seasonal factor \( C_{season} = 1 \)
- Probability factor \( C_{p} = 1 \)

**Summary**
- Peak velocity pressure \( q_p = 0.7211 \) kN/m²

Length of wall \( L_{wall} = 25 \) m
Length of the return corner \( r_{cornL} = 5 \) m

**Net pressure on wall**
- Net pressure on wall zone A \( C_{pwin} = q_p \cdot C_{pA} = 1.514 \) kN/m²
- Net pressure on wall zone B \( C_{pwin} = q_p \cdot C_{pB} = 1.298 \) kN/m²
- Net pressure on wall zone C \( C_{pwin} = q_p \cdot C_{pC} = 1.009 \) kN/m²
- Net pressure on wall zone D \( C_{pwin} = q_p \cdot C_{pD} = 0.8653 \) kN/m²
Location: Default case 14 (signboard)

Signboard details

Effective height of signboard \( H_e = 5 \text{ m} \)
Depth of signboard \( h_1 = 3 \text{ m} \)
Width of signboard \( b = 4 \text{ m} \)

Standard wind loads

Basic wind velocity (Fig. NA.1) \( V_{b,\text{map}} = 21 \text{ m/s} \)
Site altitude above mean sea level \( A_{\text{site}} = 25 \text{ m} \)

Slope details and site location relative to crest as per Fig. NA.2 of NA to BS EN 1991-1-4:2005+A1:2010

\( L_u \) and \( L_u \) are the distances from the base to the crest and base to the site, respectively.

Wind loads to Eurocode 1

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SCALE 5.48          Office 1007          Proforma 702
<table>
<thead>
<tr>
<th>Category</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Directional factor</td>
<td>$C_{di}=1$</td>
</tr>
<tr>
<td>Distance upwind to shoreline</td>
<td>$D_{ist}=2.5$ km</td>
</tr>
<tr>
<td>Distance inside town terrain</td>
<td>$D_{it}=2$ km</td>
</tr>
<tr>
<td>Probability factor</td>
<td>$C_p=1$</td>
</tr>
<tr>
<td>Effective height</td>
<td>$H_e=5$ m</td>
</tr>
<tr>
<td>Altitude factor</td>
<td>$C_a=1.025$</td>
</tr>
<tr>
<td>Direction factor</td>
<td>$C_{di}=1$</td>
</tr>
<tr>
<td>Seasonal factor</td>
<td>$C_{season}=1$</td>
</tr>
<tr>
<td>Probability factor</td>
<td>$C_p=1$</td>
</tr>
<tr>
<td>Peak velocity pressure</td>
<td>$q_p=0.6283$ kN/m$^2$</td>
</tr>
<tr>
<td>Net pressure coefficient</td>
<td>$C_{pwf}=1.80$</td>
</tr>
<tr>
<td>Net pressure on signboard</td>
<td>$C_{pwin}=q_p*C_{pwf}=1.131$ kN/m$^2$</td>
</tr>
</tbody>
</table>
Location: Default case 15 (signboard)

Signboard details

- Effective height of signboard: $H_e = 4\,\text{m}$
- Depth of signboard: $h_1 = 3.5\,\text{m}$
- Width of signboard: $b = 5\,\text{m}$

Standard wind loads

- Basic wind velocity (Fig. NA.1): $V_{b\text{map}} = 21\,\text{m/s}$
- Site altitude above mean sea level: $A_{\text{site}} = 25\,\text{m}$

Slope details and site location relative to crest as per Fig. NA.2


- $X_t < 0$: Wind is downwind of crest.
- $X_t > 0$: Wind is upwind of crest.

Site altitude is 25 m above mean sea level.

$X_t$ is positive if site is located downwind of crest.

$X_t$ is negative if site is located upwind of crest.

Height to crest: $Z = 10\,\text{m}$
Directional factor Cdi = 1
Distance upwind to shoreline Dist = 2.5 km
Distance inside town terrain Dit = 2 km
Probability factor Cp = 1

Effective height He = 4 m
Altitude factor Ca = 1.025
Direction factor Cdi = 1
Seasonal factor Cseason = 1
Probability factor Cp = 1
Peak velocity pressure qp = 0.5649 kN/m²

16 m
8 m
1.2 m

He A B C D

ELEVATION ON FREE-STANDING WALL

Length of wall Lwall = 10 m

Net pressure on wall

Net pressure on signboard zone A Cpwin = qp * CpA = 1.299 kN/m²
Net pressure on signboard zone B Cpwin = qp * CpB = 0.7909 kN/m²
Net pressure on signboard zone C Cpwin = qp * CpC = 0.6779 kN/m²
Net pressure on signboard zone D Cpwin = qp * CpD = 0.6779 kN/m²
Location: Default case 16 (free-standing parapet)

Free-standing parapet details

free-standing parapet He=1 m

4 m

2 m

0.3 m

He

A

B

C

D

ELEVATION ON FREE-STANDING PARAPET

Standard wind loads

Basic wind velocity (Fig. NA.1) Vbmap=24 m/s
Site altitude above mean sea level Asite=25 m

Slope details and site location relative to crest as per Fig. NA.2


Wind

Xt<0

Xt>0

Site altitude is 25 m above mean sea level.

Xt is positive if site is located downwind of crest.

Xt is negative if site is located upwind of crest.

Height to crest Z=10 m
Directional factor: $C_{di}=1$
Distance upwind to shoreline: $\text{Dist}=2.5 \text{ km}$
Distance inside town terrain: $\text{Dit}=2 \text{ km}$
Probability factor: $C_p=1$

Effective height: $H_e = 1 \text{ m}$

Design
Altitude factor: $C_a = 1.025$
Direction factor: $C_{di} = 1$
Seasonal factor: $C_{season} = 1$
Probability factor: $C_p = 1$

Summary
Peak velocity pressure: $q_p = 0.5091 \text{ kN/m}^2$

Length of wall: $L_{wall}=10 \text{ m}$

Net pressure on parapet

- Net pressure on parapet zone A: $C_{pwin}=q_p*C_{pA}=1.731 \text{ kN/m}^2$
- Net pressure on parapet zone B: $C_{pwin}=q_p*C_{pB}=1.069 \text{ kN/m}^2$
- Net pressure on parapet zone C: $C_{pwin}=q_p*C_{pC}=0.8655 \text{ kN/m}^2$
- Net pressure on parapet zone D: $C_{pwin}=q_p*C_{pD}=0.6109 \text{ kN/m}^2$
Location: Default case 17

Obstruction height and upwind spacing

Wind direction

\[ \text{Hav} = \text{Obstruction height of other buildings} \]

\[ \text{Building reference height} \quad \text{Hr} = 9.64 \text{ m} \]

\[ \text{buildings upwind of site} \quad \text{Hav} = 5 \text{ m} \]

\[ \text{Upwind spacing of building} \quad \text{Xo} = 20 \text{ m} \]

\[ \text{Displacement height (wind } \theta = 0^\circ) \quad \text{Hd}(0) = 0 \text{ m} \]

\[ \text{Displacement height (wind } \theta = 90^\circ) \quad \text{Hd}(90) = 0 \text{ m} \]

\[ \text{Building height} \quad \text{H} = 9.64 \text{ m} \]

Standard wind loads

Basic wind velocity (Fig. NA.1) \[ \text{Vbmap} = 23.5 \text{ m/s} \]

Site altitude above mean sea level \[ \text{Asite} = 123.6 \text{ m} \]

Orography is significant in both orthogonal directions

Slope details and site location relative to crest as per Figure NA.2 of NA to BS EN 1991-1-4:2005+A1:2010.

Effective height of the feature \[ \text{Z} = 3.6 \text{ m} \]
<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Directional factor (wind θ=0°)</td>
<td>Cdi(0)=0.995</td>
</tr>
<tr>
<td>Directional factor (wind θ=90°)</td>
<td>Cdi(90)=1</td>
</tr>
<tr>
<td>Distance upwind to shoreline</td>
<td>Dis(0)=154 km</td>
</tr>
<tr>
<td>Distance upwind to shoreline</td>
<td>Dis(90)=141 km</td>
</tr>
<tr>
<td>Probability factor</td>
<td>Cp=1</td>
</tr>
<tr>
<td>Effective height (long face)</td>
<td>He(0)=9.64 m</td>
</tr>
<tr>
<td>Effective height (short face)</td>
<td>He(90)=9.64 m</td>
</tr>
<tr>
<td>Altitude factor (long face)</td>
<td>Ca(0)=1.124</td>
</tr>
<tr>
<td>Altitude factor (short face)</td>
<td>Ca(90)=1.124</td>
</tr>
<tr>
<td>Design summary</td>
<td></td>
</tr>
<tr>
<td>Altitude factor (short face)</td>
<td>Ca(90)=1.124</td>
</tr>
<tr>
<td>Direction factor (long face)</td>
<td>Cdi(0)=0.995</td>
</tr>
<tr>
<td>Direction factor (short face)</td>
<td>Cdi(90)=1</td>
</tr>
<tr>
<td>Seasonal factor</td>
<td>Cseason=1</td>
</tr>
<tr>
<td>Probability factor</td>
<td>Cp=1</td>
</tr>
<tr>
<td>Long face peak velocity pressure</td>
<td>qp(0)=1.072 kN/m²</td>
</tr>
<tr>
<td>Short face peak velocity pressure</td>
<td>qp(90)=1.082 kN/m²</td>
</tr>
</tbody>
</table>
Location: Default case 18

Obstruction height and upwind spacing

Wind direction

Hav = Obstruction height of other buildings

Building reference height
Hr = 9.64 m

Buildings upwind of site
Hav = 5 m

Upwind spacing of building
Xo = 20 m

Displacement height (wind θ=0°)
Hd(0) = 7.57 m

Displacement height (wind θ=90°)
Hd(90) = 0 m

Building height
H = 9.64 m

Standard wind loads

Basic wind velocity (Fig. NA.1)
Vbmap = 23.5 m/s

Site altitude above mean sea level
Asite = 123.6 m

Orography is significant - Wind on short face of building

Slope details and site location relative to crest as per Figure NA.2 of NA to BS EN 1991-1-4:2005+A1:2010.

Effective height of the feature
Z = 3.6 m
### Loadings and foundations

**Wind loads to Eurocode 1**

**Made by:** IFB  
**Date:** 02/12/19

**Ref No:** SC702 EC

---

| Directional factor (wind $\theta = 0^\circ$) | $C_{di}(0) = 0.83$ |
| Directional factor (wind $\theta = 90^\circ$) | $C_{di}(90) = 0.8$ |
| Distance upwind to shoreline | $D_{is}(0) = 60$ km |
| Distance upwind to shoreline | $D_{is}(90) = 6$ km |
| Probability factor | $C_p = 1$ |

---

**Effective height (long face)**: $H_{e}(0) = 2.07$ m  
**Effective height (short face)**: $H_{e}(90) = 9.64$ m  
**Altitude factor (long face)**: $C_{a}(0) = 1.124$  
**Altitude factor (short face)**: $C_{a}(90) = 1.124$  
**Direction factor (long face)**: $C_{di}(0) = 0.83$  
**Direction factor (short face)**: $C_{di}(90) = 0.8$  
**Seasonal factor** | $C_{season} = 1$  
**Probability factor** | $C_p = 1$  
**Long face peak velocity pressure** | $q_{p}(0) = 0.4248$ kN/m$^2$  
**Short face peak velocity pressure** | $q_{p}(90) = 0.7544$ kN/m$^2$
Location: Default case 19 Flat roof (topography critical, H > 58m)

Obstruction height and upwind spacing

\[ \text{Hav} = \text{Obstruction height of other buildings} \]
\[ \text{Xo} = \text{Building reference height} \]
\[ \text{Hr} = 58 \text{ m} \]
\[ \text{Hd}(0) = 0 \text{ m} \]
\[ \text{Hd}(90) = 0 \text{ m} \]

Building height \( H = 58 \text{ m} \)

Standard wind loads

Basic wind velocity (Fig. NA.1) \( V_{bmap} = 22.5 \text{ m/s} \)
Site altitude above mean sea level \( A_{site} = 35 \text{ m} \)

Orography is significant – Wind on long face of building

Slope details and site location relative to crest as per Figure NA.2 of NA to BS EN 1991-1-4:2005+A1:2010.

\[ \text{Site altitude is 35 m above mean sea level.} \]
\[ \text{Xt is positive if site is located downwind of crest.} \]
\[ \text{Xt is negative if site is located upwind of crest.} \]

Effective height of the feature \( Z = 10 \text{ m} \)
Directional factor (wind $\theta=0^\circ$) $C_{di}(0)=0.85$
Directional factor (wind $\theta=90^\circ$) $C_{di}(90)=0.85$
Seasonal factor $C_{season}=0.86$
Distance upwind to shoreline $D_{is}(0)=2.5$ km
Distance upwind to shoreline $D_{is}(90)=2.5$ km
Probability factor $C_p=1$

**Effective height** (long face) $H_e(0)=58$ m
**Effective height** (short face) $H_e(90)=58$ m
**Altitude factor** (long face) $C_a(0)=1.035$
**Altitude factor** (short face) $C_a(90)=1.035$
**Direction factor** (long face) $C_{di}(0)=0.85$
**Direction factor** (short face) $C_{di}(90)=0.85$
**Seasonal factor** $C_{season}=0.86$
**Probability factor** $C_p=1$
**Long face peak velocity pressure** $q_{p(0)}=0.7266$ kN/m$^2$
**Short face peak velocity pressure** $q_{p(90)}=0.6672$ kN/m$^2$

**Peak velocity pressure & pressure coefficients**

On walls of rectangular buildings

Height of wall including parapet $H_r'=58$ m
Wind acting on long face of building (wind $\theta=0^\circ$):
Peak velocity pressure based on height above ground 58 m.
Peak velocity pressure @ eaves $q_{pe(0)}=0.7266$ kN/m$^2$
Wind acting on short face of building (wind $\theta=90^\circ$):
Peak velocity pressure based on height above ground 58 m.
Peak velocity pressure @ eaves $q_{pe(90)}=0.6672$ kN/m$^2$

**Case 1: Wind on long face of building (i.e. angle $\theta=0^\circ$)**

---

**PLAN VIEW**

- Inwind depth $d=40.5$ m
- Crosswind width $b=150$ m

---

SCALE 5.48 Office 1007 Proforma 702
Case 2: Wind on short face of building (i.e. angle $\theta=90^\circ$)
External pressure coefficients $C_{pe}$ for flat roofs

**PLAN VIEW**

Wind on long face ($\theta=0^\circ$)

- Inwind depth $d=40.5$ m
- Crosswind width $b=150$ m

**PLAN VIEW**

Wind on short face ($\theta=90^\circ$)

- Inwind depth $d=150$ m
- Crosswind width $b=40.5$ m

Assumes a rigid wall and Poisson's ratio = 0.5

dc = distance to centre of load
hs(n) = depth to subdivision n

Line loading  \( ll = 100 \, \text{kN/m} \)
Distance to centre of load  \( dc = 2 \, \text{m} \)
Height of wall  \( hw = 4 \, \text{m} \)
Number of subdivisions  \( ns = 8 \)
Depth to subdivision 1  \( hs(1) = hw/ns = 0.5 \, \text{m} \)
Depth ratio  \( nr(1) = hs(n)/hw = 0.125 \)
\( ma = mr^2 \times nr(n) = 0.03125 \)
\( mb = (mr^2 + (nr(n))^2)^2 = 0.070557 \)

Pressure at depth 0.5  \( qh(1) = 4/\pi \times ll/\text{hw} \times ma/mb = 14.098 \)
Calculations repeated for each depth.

<table>
<thead>
<tr>
<th>Depth</th>
<th>nr (depth ratio)</th>
<th>Horiz pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.5</td>
<td>0.125</td>
<td>14.098</td>
</tr>
<tr>
<td>1</td>
<td>0.25</td>
<td>20.372</td>
</tr>
<tr>
<td>1.5</td>
<td>0.375</td>
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<td>2</td>
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<tr>
<td>2.5</td>
<td>0.625</td>
<td>12.119</td>
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<tr>
<td>3</td>
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<tr>
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<td>0.875</td>
<td>6.7504</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>5.093</td>
</tr>
</tbody>
</table>

Pressures relate to surcharge load only.
Location: Lateral earth pressure due to surcharge (point load)


Assumes a rigid wall and Poisson's ratio = 0.5

dc = distance to centre of load
hs(n) = depth to subdivision n

Point loading
pl = 500 kN

Distance to centre of load
dc = 2 m

Height of wall
hw = 4 m

Number of subdivisions
ns = 10

Depth to subdivision 1
hs(1) = hw/ns = 0.4 m

Depth ratio
nr(1) = hs(n)/hw = 0.1

ma = mr^2 * (nr(n))^2 = 0.0025
mb = (mr^2 + (nr(n))^2)^3 = 0.017576

Pressure 0.5m to side
qhd(1) = qh(n) * (COS(1.1*ad))^2 = 7.31

Calculations repeated for each depth.

<table>
<thead>
<tr>
<th>Depth</th>
<th>Distance away from centre of load</th>
<th>2.0m</th>
<th>1.5m</th>
<th>1.0m</th>
<th>0.5m</th>
<th>centre</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.4</td>
<td>3.3184</td>
<td>4.5415</td>
<td>5.9925</td>
<td>7.31</td>
<td>7.8676</td>
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<tr>
<td>0.8</td>
<td>9.5657</td>
<td>13.091</td>
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<td>21.072</td>
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<td>31.664</td>
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<td>24.451</td>
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</tr>
<tr>
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<td>4.0868</td>
<td>5.3926</td>
<td>6.5782</td>
<td>7.08</td>
<td></td>
</tr>
</tbody>
</table>

Pressures relate to surcharge load only.
Location: Lateral earth pressure due to surcharge (strip load)


\[
dc\quad \text{Assumes a rigid wall and} \\
\text{Poisson's ratio } = 0.5 \\
\text{dc} = \text{distance to centre of load} \\
hs(n) = \text{depth to subdivision } n
\]

Strip loading \( sl = 100 \text{ kN/m}^2 \)
Distance to centre of load \( dc = 2 \text{ m} \)
Width of load \( dw = 1 \text{ m} \)
Height of wall \( hw = 4 \text{ m} \)
Number of subdivisions \( ns = 10 \)
Pressure at depth 0.4 \( qh(1) = 2*sl/PI*(ar(n)-sr*cc) = 12.471 \)
Calculations repeated for each depth.

<table>
<thead>
<tr>
<th>Depth m</th>
<th>A degrees</th>
<th>B degrees</th>
<th>Horiz pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.4</td>
<td>78.69</td>
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<td>59.036</td>
<td>13.019</td>
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<td>1.6</td>
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<td>4</td>
<td>26.565</td>
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</tbody>
</table>

Pressures relate to surcharge load only.
Location: Pressure from retained cohesionless soil

Depth to point considered: \( h = 5 \text{ m} \)

Pressure acting on vertical surface:

- **Surface of soil**
  - Surface of soil at \( \theta = 0^\circ \)
  - Maximum positive surcharge
  - Level fill
  - Maximum -ve surcharge

With maximum positive surcharge:
- Coefficient: \( k_1 = (\cos(\theta))^2 = 0.75 \)
- Horizontal pressure at depth \( h \): \( q_1 = k_1 \cdot D \cdot h = 60 \text{ kN/m}^2 \)

With level fill:
- Coefficient: \( k_2 = \frac{1 - \sin(\theta)}{1 + \sin(\theta)} = 0.33333 \)
- Horizontal pressure at depth \( h \): \( q_2 = k_2 \cdot D \cdot h = 26.667 \text{ kN/m}^2 \)

With maximum negative surcharge:
- Coefficient: \( k_3 = \frac{\cos(\theta)}{1 + \sin(\theta) \cdot \sqrt{2}} \cdot (\cos(\theta))^2 = 0.25736 \)
- Horizontal pressure at depth \( h \): \( q_3 = k_3 \cdot D \cdot h = 20.589 \text{ kN/m}^2 \)
Location: Pressure from contained granular material

Depth to point considered \( h = 5 \text{ m} \)

Pressure acting on inclined surface:

\[
\begin{align*}
q_v &= \text{vertical pressure} \\
q_n &= \text{pressure normal to surface considered} \\
q_h &= \text{horizontal pressure} \\
g_s &= \text{weight of inclined slab per unit area}
\end{align*}
\]

Angle of inclination of surface \( \theta_1 = 45^\circ \)

Weight of inclined slab (if any) \( g_s = 10 \text{ kN/m}^2 \)

Intensities of pressure:

- Vertical \( q_v = D \times h = 80 \text{ kN/m}^2 \)
- Horizontal \( q_h = k_2 \times D \times h = 26.667 \text{ kN/m}^2 \)
- Normal to inclined surface \( q_n = k_4 \times D \times h + g_s \times \cos(\theta_1) = 60.404 \text{ kN/m}^2 \)
Location: Pressure from retained cohesive soil

Cohesion factor \( C = 20 \text{ kN/m}^2 \)
Depth to point considered \( h = 5 \text{ m} \)
Pressure acting on vertical surface:

```
      \[ \text{surface of soil} \]
       /
      /   \ h
     /     \ q^2
    /       \
   /         \
  /           \
 /              \
/                \
/                    \ 
/                        \ 
/                            \ 
/                                            \ 
/                                                \ 
/                                                    \ 
/                                                        \ 
/                                                             \ 
/                                                                     \ 
/                                                                 \ 
/                                                                 \ 
/                                                          Level fill
```

With level fill:
Coefficient \( k_2 = \frac{1 - \sin(\theta)}{1 + \sin(\theta)} = 0.3333 \)
Depth of tension crack \( z_0 = \frac{2C}{D\sqrt{k_2}} = 4.3301 \text{ m} \)
Assuming tension crack is dry:
Horizontal pressure at depth \( h \) \( q_2 = k_2Dh - 2C\sqrt{k_2} = 3.5727 \text{ kN/m}^2 \)
Assuming tension crack is filled with water:
Horizontal pressure at depth \( h \) \( q_2 = k_2Dh = 26.667 \text{ kN/m}^2 \)
Note that this is less than the horizontal pressure of 42.464 kN/m\(^2\) acting at the bottom of the tension crack.
Location: Pressure from submerged granular material

- Depth to surface of material: $h_o = 2$ m
- Extra depth to point considered: $h = 5$ m
- Proportion of voids ($\beta$): $\beta = 0.3$
- Density of material: $D_m = D = 16$ kN/m$^3$
- Density of water: $D_l = 9.81$ kN/m$^3$
- Factor: $\alpha = 1 + k^2(D_m/D_l - (1 - \beta)) = 1.3103$
- Horizontal pressure at depth $h$: $q_2 = D_l h_o + D_l \alpha h = 83.892$ kN/m$^2$
Location: Typical hopper

Bending moments and shearing forces in hopper bottoms

See Reinforced Concrete Designer's Handbook, 10th edition. All loads and pressures are unfactored.

\[
\begin{align*}
&h1, h2, h3, h4 \quad \text{(dimensions)} \\
&l1, b1, a1, a2 \quad \text{(dimensions)} \\
&\theta_1, \theta_2 \quad \text{(angles)} \\
&t1, t2 \quad \text{(thicknesses)} \\
&q, qn, qh \quad \text{(loads)}
\end{align*}
\]

where, \(X\) = location of mean span and centre of pressure on inclined slab. Since this differs between end and side slabs, dimensions \(l2, b2, h3\) etc. are suffixed 'e' and 's' as appropriate in the following:

Density of contained material \(D=9 \text{ kN/m}^3\)
Angle of int.friction of material \(\theta=25^\circ\)

Total depth of bunker \(ho=h1+h2=6\) m
External dim. of side slabs \(l1=a1+(l-a1)*h2/(ho-h4)=9.3462\) m
External dim. of end slabs \(b1=a2+(b-a2)*h2/(ho-h4)=5.6538\) m
Moments and forces

In sides:

\[ X_s = \text{position of centroid of pressure on side panel} \]

\[ X_s = \frac{q_s \cdot h_s}{E N D} \]

Pressures:

\[ q_s = D \cdot h_s = 38.25 \text{ kN/m}^2 \]

\[ q_{hs} = D \cdot h_s \cdot \frac{1 - \sin(\theta)}{1 + \sin(\theta)} = 15.524 \text{ kN/m}^2 \]

\[ q_{ns} = q_{hs} \cdot (\sin(\theta))^2 + q_s \cdot (\cos(\theta))^2 + 24t2 \cdot \cos(\theta) = 30.198 \text{ kN/m}^2 \]

Horizontal moments and forces:

Bending moments in hopper bottom (per metre length of inclined slab)

At midspan \[ M_{hsm} = q_{ns} \cdot S_{Side}^2 / 32 = 13.787 \text{ kNm} \]

At support \[ M_{hss} = -M_{hsm} = -13.787 \text{ kNm} \]

Direct tension in hopper bottom (per metre length of inclined slab)

\[ V_{hs} = q_{ns} \cdot l_2s \cdot \sin(\theta) / 2 = 72.018 \text{ kN} \]

Moments and direct forces along slope:

Bending moments in inclined hopper bottom (per metre length)

At mid-height \[ M_{vsm} = q_{ns} \cdot S_{Side}^2 / 32 = 13.787 \text{ kNm} \]

At top \[ M_{vss} = -M_{vsm} = -13.787 \text{ kNm} \]

Tension in direction of slope (per metre width of inclined slab)

At mid-height \[ V_{vsm} = F_{xs} / (2 \cdot \sin(\theta) \cdot (l_2s + b_2s)) = 91.157 \text{ kN} \]

At slope top \[ V_{vss} = F / (2 \cdot (l_1 + b_1)) = 119.5 \text{ kN} \]

Vertical hanging-up force \[ V_s = F / (2 \cdot (l_1 + b_1)) = 93.79 \text{ kN} \]

In ends:

\[ X_e = \text{position of centroid of pressure on end panel} \]

\[ X_e = \frac{q_e \cdot h_e}{E N D} \]

Pressures:

\[ q_e = D \cdot h_e = 34.765 \text{ kN/m}^2 \]

\[ q_{he} = D \cdot h_e \cdot \frac{1 - \sin(\theta)}{1 + \sin(\theta)} = 14.11 \text{ kN/m}^2 \]

\[ q_{ne} = q_{he} \cdot (\sin(\theta))^2 + q_e \cdot (\cos(\theta))^2 + 24t2 \cdot \cos(\theta) = 36.331 \text{ kN/m}^2 \]

Horizontal moments and forces:

Bending moments in hopper bottom (per metre length of inclined slab)

At midspan \[ M_{hem} = q_{ne} \cdot S_{End}^2 / 32 = 18.131 \text{ kNm} \]

At support \[ M_{hes} = -M_{hem} = -18.131 \text{ kNm} \]

Direct tension in hopper bottom (per metre length of inclined slab)

\[ V_{he} = q_{ne} \cdot l_2e \cdot \sin(\theta) / 2 = 71.728 \text{ kN} \]
Moments and direct forces along slope:

Bending moments in inclined hopper bottom (per metre length)

At mid-height \( M_{vem} = q_n e \times S_{End}^2 / 32 = 18.131 \text{ kNm} \)

At top \( M_{ves} = -M_{vem} = -18.131 \text{ kNm} \)

Tension in direction of slope (per metre width of inclined slab)

At mid-height \( V_{vem} = F_{xe} / (2 \times \sin(\theta_1) \times (l_{2e} + b_{2e})) = 142.77 \text{ kN} \)

At slope top \( V_{ves} = F / (2 \times \sin(\theta_1) \times (l_{1} + b_{1})) = 168.4 \text{ kN} \)

Vertical hanging-up force \( V_e = F / (2 \times (l_1 + b_1)) = 93.79 \text{ kN} \)
Location: Extremely large hopper

Bending moments and shearing forces in hopper bottoms

See Reinforced Concrete Designer’s Handbook, 10th edition. All loads and pressures are unfactored.

\[ \begin{align*}
\text{Density of contained material} & \quad D = 20 \text{ kN/m}^3 \\
\text{Angle of int. friction of material} & \quad \Theta = 10^\circ
\end{align*} \]

where, \( X \) = location of mean span and centre of pressure on inclined slab

Since this differs between end and side slabs, dimensions \( l_2, b_2, h_3 \) etc. are suffixed ‘e’ and ‘s’ as appropriate in the following:

Total depth of bunker \( ho = h_1 + h_2 = 20 \text{ m} \)

External dim. of side slabs \( l_1 = a_1 + (l - a_1) \times h_2 / (ho - h_4) = 20 \text{ m} \)

External dim. of end slabs \( b_1 = a_2 + (b - a_2) \times h_2 / (ho - h_4) = 20 \text{ m} \)
Moments and forces

In sides:

\[ X_s = \text{position of centroid of pressure on side panel} \]

\[ \frac{q_s}{q_{ns}} \]

\[ \frac{h_s}{\beta} \]

\[ \frac{\beta}{E N D} \]

Pressures:

\[ q_s = D \times h_s = 300 \text{ kN/m}^2 \]

\[ q_{hs} = D \times h_s \times (1 - \sin(\theta)) / (1 + \sin(\theta)) = 211.23 \text{ kN/m}^2 \]

\[ q_{ns} = q_{hs} \times (\sin(\theta))^2 + q_s \times (\cos(\theta))^2 + 24 \times t^2 \times \cos(\theta) = 243.82 \text{ kN/m}^2 \]

Horizontal moments and forces:

Bending moments in hopper bottom (per metre length of inclined slab)

- At midspan: \( M_{hsm} = q_{ns} \times S\text{Side}^2 / 32 = 997.1 \text{ kNm} \)
- At support: \( M_{hss} = -M_{hsm} = -997.1 \text{ kNm} \)

Direct tension in hopper bottom (per metre length of inclined slab)

\( V_{hs} = q_{ns} \times l_2 \times \sin(\theta) / 2 = 1598.5 \text{ kN} \)

Moments and direct forces along slope:

Bending moments in inclined hopper bottom (per metre length)

- At mid-height: \( M_{vsm} = q_{ns} \times S\text{Side}^2 / 32 = 997.1 \text{ kNm} \)
- At top: \( M_{vss} = -M_{vsm} = -997.1 \text{ kNm} \)

Tension in direction of slope (per metre width of inclined slab)

- At mid-height: \( V_{vsm} = F_{xs} / (2 \times \sin(\theta) \times (l_2 + b_2)) = 1741.3 \text{ kN} \)
- At slope top: \( V_{vss} = F / (2 \times (l_1 + b_1)) = 2079.2 \text{ kN} \)

Vertical hanging-up force

\( V_{s} = F / (2 \times (l_1 + b_1)) = 1817.5 \text{ kN} \)

In ends:

\[ X_e = \text{position of centroid of pressure on end panel} \]

\[ \frac{q_e}{q_{ne}} \]

\[ \frac{h_e}{\beta} \]

\[ \frac{\beta}{SIDE} \]

\[ \frac{\beta}{E L E V A T I O N} \]

Pressures:

\[ q_e = D \times h_e = 300 \text{ kN/m}^2 \]

\[ q_{he} = D \times h_e \times (1 - \sin(\theta)) / (1 + \sin(\theta)) = 211.23 \text{ kN/m}^2 \]

\[ q_{ne} = q_{he} \times (\sin(\theta))^2 + q_e \times (\cos(\theta))^2 + 24 \times t^2 \times \cos(\theta) = 243.82 \text{ kN/m}^2 \]

Horizontal moments and forces:

Bending moments in hopper bottom (per metre length of inclined slab)

- At midspan: \( M_{hem} = q_{ne} \times S\text{End}^2 / 32 = 997.1 \text{ kNm} \)
- At support: \( M_{hes} = -M_{hem} = -997.1 \text{ kNm} \)

Direct tension in hopper bottom (per metre length of inclined slab)

\( V_{he} = q_{ne} \times l_2 \times \sin(\theta) / 2 = 1598.5 \text{ kN} \)
Moments and direct forces along slope:

Bending moments in inclined hopper bottom (per metre length)

At mid-height \( M_{vem} = q_{ne} \times S_{End} \times 2/32 = 997.1 \, \text{kNm} \)
At top \( M_{ves} = -M_{vem} = -997.1 \, \text{kNm} \)

Tension in direction of slope (per metre width of inclined slab)

At mid-height \( V_{vem} = F_{xe} / (2 \times \sin(\theta_1) \times (l_{2e} + b_{2e})) = 1741.3 \, \text{kN} \)
At slope top \( V_{ves} = F / (2 \times \sin(\theta_1) \times (l_{1} + b_{1})) = 2079.2 \, \text{kN} \)

Vertical hanging-up force \( V_{e} = F / (2 \times (l_{1} + b_{1})) = 1817.5 \, \text{kN} \)
Location: Extremely small hopper

Bending moments and shearing forces in hopper bottoms

See Reinforced Concrete Designer's Handbook, 10th edition. All loads and pressures are unfactored.

\[ \text{SIDE ELEVATION} \quad \text{END ELEVATION} \]

where, \( X \) = location of mean span and centre of pressure on inclined slab
Since this differs between end and side slabs, dimensions \( l_2, b_2, h_3 \)
etc. are suffixed 'e' and 's' as appropriate in the following:
Density of contained material \( D = 4 \text{ kN/m}^3 \)
Angle of int.friction of material \( \Theta = 10^\circ \)

Total depth of bunker \( h_0 = h_1 + h_2 = 2 \text{ m} \)
External dim. of side slabs \( l_1 = a_1 + (1-a_1) \cdot h_2 / (h_0 - h_4) = 2 \text{ m} \)
External dim. of end slabs \( b_1 = a_2 + (b-a_2) \cdot h_2 / (h_0 - h_4) = 2 \text{ m} \)
Moments and forces

In sides:

\[ \text{Xs} = \text{position of centroid} \]
\[ \text{of pressure on side panel} \]
\[ \text{hs} \quad \text{E N D} \]
\[ \text{qhs} \]

Pressures:
\[ \text{qs} = D \times \text{hs} = 5.6 \, \text{kN/m}^2 \]
\[ \text{qhs} = D \times \text{hs} \times (1 - \sin(\theta)) / (1 + \sin(\theta)) = 3.9429 \, \text{kN/m}^2 \]
\[ \text{qns} = \text{qhs} \times (\sin(\theta_2))^2 + \text{qs} \times (\cos(\theta_2))^2 + 24 \times t_2 \times \cos(\theta_2) = 7.317 \, \text{kN/m}^2 \]

Horizontal moments and forces:
- Bending moments in hopper bottom (per metre length of inclined slab)
  - At midspan: \[ M_{hsm} = qns \times SS_{ide}^2 / 32 = 0.29268 \, \text{kNm} \]
  - At support: \[ M_{hss} = -M_{hsm} = -0.29268 \, \text{kNm} \]
- Direct tension in hopper bottom (per metre length of inclined slab)
  \[ V_{hs} = qns \times l_2s \times \sin(\theta_2) / 2 = 3.4148 \, \text{kN} \]

Moments and direct forces along slope:
- Bending moments in inclined hopper bottom (per metre length)
  - At mid-height: \[ M_{vsm} = qns \times SS_{ide}^2 / 32 = 0.29268 \, \text{kNm} \]
  - At top: \[ M_{vss} = -M_{vsm} = -0.29268 \, \text{kNm} \]
- Tension in direction of slope (per metre width of inclined slab)
  - At mid-height: \[ V_{vsm} = F_{xs} / (2 \times \sin(\theta_2) \times (l_2s + b_2s)) = 6.4361 \, \text{kN} \]
  - At slope top: \[ V_{vss} = F / (2 \times (l_1 + b_1)) = 8.8375 \, \text{kN} \]
- Vertical hanging-up force
  \[ V_{s} = F / (2 \times (l_1 + b_1)) = 6.2491 \, \text{kN} \]

In ends:

\[ \text{Xe} = \text{position of centroid} \]
\[ \text{of pressure on end panel} \]
\[ \text{he} \quad \text{S I D E} \]

Pressures:
\[ \text{qe} = D \times \text{he} = 5.6 \, \text{kN/m}^2 \]
\[ \text{qhe} = D \times \text{he} \times (1 - \sin(\theta)) / (1 + \sin(\theta)) = 3.9429 \, \text{kN/m}^2 \]
\[ \text{qne} = \text{qhe} \times (\sin(\theta_1))^2 + \text{qe} \times (\cos(\theta_1))^2 + 24 \times t_2 \times \cos(\theta_1) = 7.317 \, \text{kN/m}^2 \]

Horizontal moments and forces:
- Bending moments in hopper bottom (per metre length of inclined slab)
  - At midspan: \[ M_{hem} = qne \times SE_{nd}^2 / 32 = 0.29268 \, \text{kNm} \]
  - At support: \[ M_{hes} = -M_{hem} = -0.29268 \, \text{kNm} \]
- Direct tension in hopper bottom (per metre length of inclined slab)
  \[ V_{he} = qne \times l_2e \times \sin(\theta_1) / 2 = 3.4148 \, \text{kN} \]
Moments and direct forces along slope:

Bending moments in inclined hopper bottom (per metre length)

- At mid-height: \( M_{vem} = qne \cdot S_{End}^2 / 32 = 0.29268 \text{ kNm} \)
- At top: \( M_{ves} = -M_{vem} = -0.29268 \text{ kNm} \)

Tension in direction of slope (per metre width of inclined slab)

- At mid-height: \( V_{vem} = F_{xe} / \left( 2 \cdot \sin(\theta_1) \cdot (l_2 + b_2) \right) = 6.4361 \text{ kN} \)
- At slope top: \( V_{ves} = F / \left( 2 \cdot \sin(\theta_1) \cdot (l_1 + b_1) \right) = 8.8375 \text{ kN} \)
- Vertical hanging-up force: \( V_e = F / \left( 2 \cdot (l_1 + b_1) \right) = 6.2491 \text{ kN} \)
Location: Typical tank

Analysis of cylindrical containers with wall monolithic with base

Analyses based on coefficients by
(a) Dr Reissner published in
'Reinforced Concrete Designer's
Handbook' 10th Edition,
Table 184.
(b) Dr J.E.Gibson in 'Thin Shells
Computing and Theory'.

Elastic modulus of wall material \( E = 25 \times 10^6 \text{ kN/m}^2 \)
Diameter of tank \( d = 6 \text{ m} \)
Depth of tank and liquid \( h = 10 \text{ m} \)
Thickness of tank wall at base \( h_A = 0.25 \text{ m} \)
Density of contained liquid \( D_l = 9.807 \text{ kN/m}^3 \)
Poisson's ratio for concrete \( \nu = 0.2 \)
No. of vert. intervals considered \( n = 10 \)

Analysis using Reissner coefficients

Vert. bending moment at wall base \( M_A = K_1 D_l h^3 \frac{d}{2} = 20.23 \text{ kNm/m of circumference.} \)
Height of maximum circum.tension \( h_{max} = K_2 h = 1.739 \text{ m} \)
Maximum circumferential tension \( N_{max} = K_3 D_l h \frac{d}{2} = 251.57 \text{ kN/m of height.} \)
Analysis using Gibson formulae

Basic coefficient

\[
\beta = \left(\frac{3(1-\nu^2)}{4h^2A^2}\right)^{0.25} = 1.5042
\]

Values of deflection, force and moment are then found by solving the equations below at the required heights \(x\) above the base:

- **Radial deflection**:
  \[
  \text{Radial deflection} = -1000Dl(d/2)^2h*(1-x/h-A-B*(1-1/(\beta *h)))/(E*ha) \text{ mm}
  \]

- **Vertical shear**:
  \[
  \text{Vertical shear} = Dl*h*(-2*A+(A+B)/(\beta *h))/(2*\beta) \text{ kN/m}
  \]

- **Ring tension**:
  \[
  \text{Ring tension} = Dl*d/2*h*(1-x/h-A-B*(1-1/(\beta *h))) \text{ kN/m}
  \]

- **Vertical moment**:
  \[
  \text{Vertical moment} = Dl*h*(A*(1-1/(\beta *h))-B)/(2*\beta^2) \text{ kNm/m}
  \]

- **Horizontal moment**:
  \[
  \text{Horizontal moment} = Mx*nu \text{ kNm/m}
  \]

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<tr>
<th>Height (m)</th>
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NOTE: All values in table are unfactored.
Location: Shallow tank with very large diameter

Analysis of cylindrical containers with wall monolithic with base

Elastic modulus of wall material $E = 15E6$ kN/m²
Diameter of tank $d = 20$ m
Depth of tank and liquid $h = 2$ m
Thickness of tank wall at base $h_A = 0.3$ m
Density of contained liquid $D_l = 9.807$ kN/m³
Poisson's ratio for concrete $\nu = 0.3$
No. of vert. intervals considered $n = 15$

Analysis using Reissner coefficients

where, $N_{\text{max}}$ is the maximum circumferential tension and $h_{\text{max}}$ is the height of maximum circumferential tension.

Vert. bending moment at wall base $M_A = K_1D_lh^3$
  $= 5.8096$ kNm/m of circumference.
Height of maximum circum. tension $h_{\text{max}} = K_2h = 0.965$ m
Maximum circumferential tension $N_{\text{max}} = K_3D_lh^3d/2$
  $= 8.6798$ kN/m of height.
Analysis using Gibson formulae

Basic coefficient  \( \beta = \left( \frac{3(1-\nu^2)}{(d^2/4*hA^2)} \right)^{0.25} = 0.74213 \)

Values of deflection, force and moment are then found by solving the equations below at the required heights \( x \) above the base:

- **Radial deflection** = \(-1000*Dl*(d/2)^2*h/(1-x/h-A-B*(1-1/(\beta*h)))/(E*hA) \) mm
- **Vertical shear** = \(Dl*h*(-2*A+(A+B)/(\beta*h))/(2*\beta) \) kN/m
- **Ring tension** = \(Dl*d/2*h*(1-x/h-A-B*(1-1/(\beta*h))) \) kN/m
- **Vertical moment** = \(Dl*h*(A*(1-1/(\beta*h))-B)/(2*\beta^2) \) kNm/m
- **Horizontal moment** = \(Mx*nu \) kNm/m

<table>
<thead>
<tr>
<th>Height (m)</th>
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<th>Vertical shear (kN/m)</th>
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**NOTE:** All values in table are unfactored.
Location: Tall, small diameter tank containing light liquid

Analysis of cylindrical containers with wall monolithic with base

Analyses based on coefficients by
(a) Dr Reissner published in 'Reinforced Concrete Designer's Handbook' 10th Edition, Table 184.
(b) Dr J.E.Gibson in 'Thin Shells Computing and Theory'.

Elastic modulus of wall material $E=40E6$ kN/m$^2$
Diameter of tank $d=3$ m
Depth of tank and liquid $h=20$ m
Thickness of tank wall at base $h_A=1$ m
Density of contained liquid $D_l=8$ kN/m$^3$
Poisson's ratio for concrete $\nu=0.15$
No.of vert.intervals considered $n=20$

Analysis using Reissner coefficients

Vert.bending moment at wall base $MA=K_1*D_l*h^3$
=66.796 kNm/m of circumference.
Height of maximum circum.tension $h_{max}=K_2*h=2.562$ m
Maximum circumferential tension $N_{max}=K_3*D_l*h*d/2$
=217.72 kN/m of height.
Analysis using Gibson formulae

Basic coefficient

\[
\beta = \frac{3(1-\nu^2)}{(d^2/4hA^2)}^{.25} = 1.0685
\]

Values of deflection, force and moment are then found by solving the equations below at the required heights \( x \) above the base:

Radial deflection
\[
= -1000Dl*(d/2)^2*h*(1-x/h-A-B*(1-1/(\beta*h)))/(E*hA) \text{ mm}
\]

Vertical shear
\[
= Dl*h*(-2*A+(A+B)/(\beta*h))/(2*\beta) \text{ kN/m}
\]

Ring tension
\[
= Dl*d/2*h*(1-x/h-A-B*(1-1/(\beta*h))) \text{ kN/m}
\]

Vertical moment
\[
= Dl*h*(A*(1-1/(\beta*h))-B)/(2*\beta^2) \text{ kNm/m}
\]

Horizontal moment
\[
= Mx*\nu \text{ kNm/m}
\]

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<tr>
<th>Height (m)</th>
<th>Radial deflection (mm)</th>
<th>Vertical shear (kN/m)</th>
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NOTE: All values in table are unfactored.
Location: Footing for south elevation

Eaves level

Roof area carried by 1 m of wall  $a(3)=4 \text{ m}^2$
DL of pitched roof on plan  $F_f(3)=1.5 \text{ kN/m}^2$
Dead load attic floor  $DL(3)=0.7 \text{ kN/m}^2$
Edge load (upstand etc)  $E(3)=0.5 \text{ kN/m}$
LL excluded from reductions  $LE(3)=2 \text{ kN/m}^2$
Live load not so excluded on plan  $LL(3)=3 \text{ kN/m}^2$

Floor level 2

Floor area carried by 1 m of wall  $a(2)=4 \text{ m}^2$
Floor finishes  $F_f(2)=0.2 \text{ kN/m}^2$
Dead load of floor  $DL(2)=1 \text{ kN/m}^2$
Loading from external wall  $E(2)=10.8 \text{ kN/m}$
LL excluded from reductions  $LE(2)=2 \text{ kN/m}^2$
Live load not so excluded on plan  $LL(2)=5 \text{ kN/m}^2$

Floor level 1

Floor area carried by 1 m of wall  $a(1)=4 \text{ m}^2$
Floor finishes  $F_f(1)=0.2 \text{ kN/m}^2$
Dead load of floor  $DL(1)=1 \text{ kN/m}^2$
Loading from external wall  $E(1)=10.8 \text{ kN/m}$
LL excluded from reductions  $LE(1)=2 \text{ kN/m}^2$
Live load not so excluded on plan  $LL(1)=5 \text{ kN/m}^2$

Wall loading table

Loads from floors shown at floor levels, load at top of wall shown between floors. Reduced imposed loads contain reduction factors given in Table 2 of BS6399, but note that IStructE Green Book advises that these reduction factors should not be taken advantage of in the initial design except when assessing the load on the foundation.

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<td>Level 1</td>
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</table>
Location: Footing for south elevation

Eaves level

Roof area carried by 1 m of wall \( a(3) = 4 \text{ m}^2 \)
Perm load of pitched roof on plan \( Ff(3) = 1.5 \text{ kN/m}^2 \)
Permanent load attic floor \( DL(3) = 0.7 \text{ kN/m}^2 \)
Edge load (upstand etc.) \( E(3) = 0.5 \text{ kN/m} \)
IL excluded from reductions \( LE(3) = 2 \text{ kN/m}^2 \)
IL not so excluded on plan \( LL(3) = 3 \text{ kN/m}^2 \)

Floor level 2

Floor area carried by 1 m of wall \( a(2) = 4 \text{ m}^2 \)
Floor finishes \( Ff(2) = 0.2 \text{ kN/m}^2 \)
Permanent load of floor \( DL(2) = 1 \text{ kN/m}^2 \)
Loading from external wall \( E(2) = 10.8 \text{ kN/m} \)
IL excluded from reductions \( LE(2) = 2 \text{ kN/m}^2 \)
IL not so excluded on plan \( LL(2) = 5 \text{ kN/m}^2 \)

Floor level 1

Floor area carried by 1 m of wall \( a(1) = 4 \text{ m}^2 \)
Floor finishes \( Ff(1) = 0.2 \text{ kN/m}^2 \)
Permanent load of floor \( DL(1) = 1 \text{ kN/m}^2 \)
Loading from external wall \( E(1) = 10.8 \text{ kN/m} \)
IL excluded from reductions \( LE(1) = 2 \text{ kN/m}^2 \)
IL not so excluded on plan \( LL(1) = 5 \text{ kN/m}^2 \)

Wall loading table

Loads from floors shown at floor levels, load at top of wall shown between floors. Reduced imposed loads contain reduction factors given in EC1 Part 1-1, but note that IStructE advises that these reduction factors should not be taken advantage of in the initial design except when assessing the load on the foundation.

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SCALE 5.48 Office 1007 Proforma 710
Location: Grid reference A1 & all other internal columns

Roof level

Roof area carried by column  a(3)=24 m²
Roof finishes  Ff(3)=2.2 kN/m²
Ceiling & service load beneath  Cs(3)=0.5 kN/m²
Dead load of roof  DL(3)=7.2 kN/m²
Parapet load  E(3)=8.5 kN/m
Length of parapet carried  L(3)=6 m
Additional dead load incl column  DLa(3)=0 kN
LL excluded from reductions  LE(3)=2 kN/m²
Live load not so excluded  LL(3)=3 kN/m²

Floor level 2

Floor area carried by column  a(2)=24 m²
Floor finishes  Ff(2)=1.8 kN/m²
Ceiling & service load  Cs(2)=0.5 kN/m²
Dead load of floor  DL(2)=6.6 kN/m²
Loading from external envelope  E(2)=10.8 kN/m
Length of envelope carried  L(2)=6 m
Additional dead load incl column  DLa(2)=7.8 kN
LL excluded from reductions  LE(2)=2 kN/m²
Live load not so excluded  LL(2)=5 kN/m²

Floor level 1

Floor area carried by column  a(1)=24 m²
Floor finishes  Ff(1)=1.8 kN/m²
Ceiling & service load  Cs(1)=0.5 kN/m²
Dead load of floor  DL(1)=6.6 kN/m²
Loading from external envelope  E(1)=10.8 kN/m
Length of envelope carried  L(1)=6 m
Additional dead load incl column  DLa(1)=7.8 kN
LL excluded from reductions  LE(1)=2 kN/m²
Live load not so excluded  LL(1)=5 kN/m²
## Column loading table

Loads from floors shown at floor levels, column loads shown between. Reduced imposed loads contain reduction factors given in Table 2 of BS6399, but note that IStructE Green Book advises that these reduction factors should not be taken advantage of in the initial design except when assessing the load on the foundation.

<table>
<thead>
<tr>
<th>Characteristic loads</th>
<th>Factored loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dead (kN)</td>
</tr>
<tr>
<td>Roof level</td>
<td>288.6</td>
</tr>
<tr>
<td></td>
<td>288.6</td>
</tr>
<tr>
<td>Level 2</td>
<td>286.2</td>
</tr>
<tr>
<td></td>
<td>574.8</td>
</tr>
<tr>
<td>Level 1</td>
<td>286.2</td>
</tr>
<tr>
<td></td>
<td>861</td>
</tr>
</tbody>
</table>
Location: Grid reference A1 & all other internal columns

**Roof level**

- Roof area carried by column: $a(3)=24 \, \text{m}^2$
- Roof finishes: $Ff(3)=2.2 \, \text{kN/m}^2$
- Ceiling & service load beneath: $Cs(3)=0.5 \, \text{kN/m}^2$
- Permanent load of roof: $DL(3)=7.2 \, \text{kN/m}^2$
- Parapet load: $E(3)=8.5 \, \text{kN/m}$
- Length of parapet carried: $L(3)=6 \, \text{m}$
- Additional permanent ld incl col. DLa(3): $0 \, \text{kN}$
- IL excluded from reductions: $LE(3)=2 \, \text{kN/m}^2$
- Imposed load not so excluded: $LL(3)=3 \, \text{kN/m}^2$

**Floor level 2**

- Floor area carried by column: $a(2)=24 \, \text{m}^2$
- Floor finishes: $Ff(2)=1.8 \, \text{kN/m}^2$
- Ceiling & service load: $Cs(2)=0.5 \, \text{kN/m}^2$
- Permanent load of floor: $DL(2)=6.6 \, \text{kN/m}^2$
- Loading from external envelope: $E(2)=10.8 \, \text{kN/m}$
- Length of envelope carried: $L(2)=6 \, \text{m}$
- Additional permanent ld incl col. DLa(2): $7.8 \, \text{kN}$
- IL excluded from reductions: $LE(2)=2 \, \text{kN/m}^2$
- Imposed load not so excluded: $LL(2)=5 \, \text{kN/m}^2$

**Floor level 1**

- Floor area carried by column: $a(1)=24 \, \text{m}^2$
- Floor finishes: $Ff(1)=1.8 \, \text{kN/m}^2$
- Ceiling & service load: $Cs(1)=0.5 \, \text{kN/m}^2$
- Permanent load of floor: $DL(1)=6.6 \, \text{kN/m}^2$
- Loading from external envelope: $E(1)=10.8 \, \text{kN/m}$
- Length of envelope carried: $L(1)=6 \, \text{m}$
- Additional permanent ld incl col. DLa(1): $7.8 \, \text{kN}$
- IL excluded from reductions: $LE(1)=2 \, \text{kN/m}^2$
- Imposed load not so excluded: $LL(1)=5 \, \text{kN/m}^2$
Column loading table

Loads from floors shown at floor levels, column loads shown between. Reduced imposed loads contain reduction factors given in EC1 Part 1-1, but note that IStructE advises that these reduction factors should not be taken advantage of in the initial design except when assessing the load on the foundation.

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>Characteristic loads</th>
<th>Factored loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Permnt (kN)</td>
<td>Imposed (kN)</td>
</tr>
<tr>
<td>Roof level</td>
<td>288.6</td>
<td>120</td>
</tr>
<tr>
<td>Level 2</td>
<td>288.6</td>
<td>120</td>
</tr>
<tr>
<td>Level 1</td>
<td>286.2</td>
<td>168</td>
</tr>
<tr>
<td></td>
<td>574.8</td>
<td>288</td>
</tr>
<tr>
<td></td>
<td>286.2</td>
<td>168</td>
</tr>
<tr>
<td></td>
<td>861</td>
<td>456</td>
</tr>
</tbody>
</table>
Location: The House

Unit loads for elements of structure

Sloping roofs with angle of 30 degrees

Serviceability loads

Dead load on plan for sloping member = Load / Cos roof angle

Rafters 50 mm x 100 mm at 400 mm c/c

<table>
<thead>
<tr>
<th>Loads</th>
<th>On horizontal</th>
<th>On slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rafters</td>
<td>0.0694</td>
<td>0.0601</td>
</tr>
<tr>
<td>Felt load</td>
<td>0.017</td>
<td>0.0147</td>
</tr>
<tr>
<td>Batten load</td>
<td>0.0385</td>
<td>0.0333</td>
</tr>
<tr>
<td>Tile/slate load</td>
<td>0.7188</td>
<td>0.6225</td>
</tr>
<tr>
<td><strong>Sub total for dead load (Gk)</strong></td>
<td><strong>0.9488 kN/m²</strong></td>
<td><strong>0.6225 kN/m²</strong></td>
</tr>
<tr>
<td><strong>Live load on horizontal (Qk)</strong></td>
<td><strong>0.75 kN/m²</strong></td>
<td><strong>0.75 kN/m²</strong></td>
</tr>
<tr>
<td><strong>Total load (Gk+Qk)</strong></td>
<td><strong>1.699 kN/m²</strong></td>
<td><strong>1.699 kN/m²</strong></td>
</tr>
<tr>
<td><strong>Ultimate dead load (Factor 1.4)</strong></td>
<td><strong>1.328 kN/m²</strong></td>
<td><strong>1.328 kN/m²</strong></td>
</tr>
<tr>
<td><strong>Ultimate live load (Factor 1.6)</strong></td>
<td><strong>1.2 kN/m²</strong></td>
<td><strong>1.2 kN/m²</strong></td>
</tr>
</tbody>
</table>

Loading with plaster/plasterboard for roofs with angle of 30°

Serviceability loads

<table>
<thead>
<tr>
<th>Loads</th>
<th>On horizontal</th>
<th>On slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load from above</td>
<td>0.9488 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Plaster/board load</td>
<td>0.2491 kN/m²</td>
<td>0.2157 kN/m²</td>
</tr>
<tr>
<td>Allowance for insulation etc.</td>
<td>0.1155 kN/m²</td>
<td>0.1 kN/m²</td>
</tr>
<tr>
<td><strong>Sub total for dead load (Gk)</strong></td>
<td><strong>1.313 kN/m²</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Live load on horizontal (Qk)</strong></td>
<td><strong>0.75 kN/m²</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Total Gk+Qk with plaster/board</strong></td>
<td><strong>2.063 kN/m²</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Ultimate dead load (Factor 1.4)</strong></td>
<td><strong>1.839 kN/m²</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Ultimate live load (Factor 1.6)</strong></td>
<td><strong>1.2 kN/m²</strong></td>
<td></td>
</tr>
</tbody>
</table>
Location: The House

Unit loads for elements of structure

Sloping roofs with angle of 30 degrees

Serviceability loads

Permanent load on plan for sloping member = Load / Cos roof angle

Rafters 50 mm x 100 mm at 400 mm c/c

<table>
<thead>
<tr>
<th>Loads</th>
<th>On horizontal</th>
<th>On slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rafters</td>
<td>0.0694</td>
<td>0.0601</td>
</tr>
<tr>
<td>Boarding load</td>
<td>0.0951</td>
<td>0.0824</td>
</tr>
<tr>
<td>Felt load</td>
<td>0.017</td>
<td>0.0147</td>
</tr>
<tr>
<td>Counterbatten load</td>
<td>0.01</td>
<td>0.0087</td>
</tr>
<tr>
<td>Batten load</td>
<td>0.0385</td>
<td>0.0333</td>
</tr>
<tr>
<td>Tile/slate load</td>
<td>0.7188</td>
<td>0.6225</td>
</tr>
</tbody>
</table>

Subtotal for permant load (Gk) 0.9488 kN/m²

Total load (Gk+Qk) 1.699 kN/m²

Ult permanent load (Factor 1.35) 1.281 kN/m²
Ult imposed load (Factor 1.5) 1.125 kN/m²

Loading with plaster/plasterboard for roofs with angle of 30°

Serviceability loads

<table>
<thead>
<tr>
<th>Loads</th>
<th>On horizontal</th>
<th>On slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent load from above</td>
<td>0.9488 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Plaster/board load</td>
<td>0.2491 kN/m²</td>
<td>0.2157 kN/m²</td>
</tr>
<tr>
<td>Allowance for insulation etc.</td>
<td>0.1155 kN/m²</td>
<td>0.1 kN/m²</td>
</tr>
</tbody>
</table>

Subtotal for permanent load (Gk) 1.313 kN/m²

Imposed load on horizontal (Qk) 0.75 kN/m²

Total Gk+Qk with plaster/board 2.063 kN/m²

Ult permanent load (Factor 1.35) 1.773 kN/m²
Ult imposed load (Factor 1.5) 1.125 kN/m²
Location: Inner leaf lintel above front door

Domestic lintel

Calculations in accordance with BS5977-1:1981.

For selection of lintels using published load-span tables, it may be convenient to convert triangular loads and part span loads to equivalent uniformly distributed loads over the whole span. However this may not give correct shear forces and the calculated BM may occur at a point other than mid-span.

NOTE: Load triangle sides are at 45° to the horizontal and Interaction zone triangle sides are at 60° to the horizontal.

Clear span of lintel \( \text{Lc}=1.8 \text{ m} \)

Unfactored masonry load \( \text{Wm}=2.1 \text{ kN/m}^2 \)

Floor/roof loading \( \text{Wrf}=4.8 \text{ kN/m} \)

Height above lintel \( \text{Hrf}=0.65 \text{ m} \)

Partition loading \( \text{Wp}=3.6 \text{ kN} \)

**SUMMARY OF LOADINGS**

<table>
<thead>
<tr>
<th>Source</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry</td>
<td>1.3825 kN/m</td>
</tr>
<tr>
<td>Floor/roof</td>
<td>2.6408 kN/m</td>
</tr>
<tr>
<td>Partition</td>
<td>3.5009 kN/m</td>
</tr>
<tr>
<td>Self weight</td>
<td>0.36 kN/m</td>
</tr>
<tr>
<td><strong>Total (unfactored)</strong></td>
<td><strong>7.8842 kN/m</strong></td>
</tr>
</tbody>
</table>
Location: Inner leaf lintel above front door

Domestic lintel

Calculations are in accordance with the EC6 Manual by IStructE, PD 6697:2010 and BS5977-1:1981.

For selection of lintels using published load-span tables, it may be convenient to convert triangular loads and part span loads to equivalent uniformly distributed loads over the whole span. However this may not give correct shear forces and the calculated BM may occur at a point other than mid-span.

NOTE: Load triangle sides are at 45° to the horizontal and Interaction zone triangle sides are at 60° to the horizontal.

Clear span of lintel \( L_c = 1.8 \) m
Unfactored masonry load \( W_m = 2.1 \) kN/m²
Floor/roof loading \( W_{rf} = 4.8 \) kN/m
Height above lintel \( H_{rf} = 0.65 \) m
Partition loading \( W_p = 3.6 \) kN

**SUMMARY OF LOADINGS**

**Equivalent udl values**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry</td>
<td>1.3825 kN/m</td>
</tr>
<tr>
<td>Floor/roof</td>
<td>2.6408 kN/m</td>
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<tr>
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<tr>
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</tr>
<tr>
<td><strong>Total (unfactored)</strong></td>
<td><strong>7.8842 kN/m</strong></td>
</tr>
</tbody>
</table>
Location: New building for The National Lottery

Unit loads for roof elements of structure

Portal frame roof with angle of 10°

Serviceability loads

Load on horizontal = Load on slope / Cos(roof angle)

Frame details: 30 m span x 6 m centres

<table>
<thead>
<tr>
<th>Loads</th>
<th>On horizontal</th>
<th>On slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheetling and insulation</td>
<td>0.073</td>
<td>0.072</td>
</tr>
<tr>
<td>Allowance for services etc.</td>
<td>0.071</td>
<td>0.07</td>
</tr>
<tr>
<td>Subtotal for dead load (Gk)</td>
<td>0.489</td>
<td>0.482</td>
</tr>
<tr>
<td>Live load on horizontal (Qk)</td>
<td>11(1)=0.75 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Total factored horizontal load</td>
<td>1.89 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Total factored load on slope</td>
<td>1.86 kN/m²</td>
<td></td>
</tr>
</tbody>
</table>

Wind loading

Description of wind load: Windward side of roof

Wind Load

\[ w(1)=-0.5 \text{ kN/m}^2 \]

Wind load factored by (1.2)

\[ -0.6 \text{ kN/m}^2 \]

Total factored load on slope

0.865 kN/m²
Location: New building for The National Lottery

Unit loads for roof elements of structure

Portal frame roof with angle θ=10°

Serviceability loads

Load on plan (for sloping member) = Load on slope / Cos θ

Frame details: 30 m span x 6 m centres

<table>
<thead>
<tr>
<th>Loads</th>
<th>On plan</th>
<th>On slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet ing and insulation</td>
<td>0.073</td>
<td>0.072</td>
</tr>
<tr>
<td>Frame</td>
<td>0.294</td>
<td>0.29</td>
</tr>
<tr>
<td>Allowance for services etc.</td>
<td>0.071</td>
<td>0.07</td>
</tr>
<tr>
<td>Subtotal for permanent load (Gk)</td>
<td>0.489</td>
<td>0.482</td>
</tr>
<tr>
<td>Imposed load on plan (Qk)</td>
<td>1.1(1)=0.75 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Imposed load on slope (Qk)</td>
<td>1.1s(1)=0.739 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Total factored load (on plan)</td>
<td>1.79 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Total factored load (on slope)</td>
<td>1.76 kN/m²</td>
<td></td>
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</tbody>
</table>

Wind loading

Description of wind load: Windward side of roof
Wind Load                     w(1)=-0.5 kN/m²
Wind load (factored by 0.75)   -0.375 kN/m²

Total factored load (on slope)  1.38 kN/m²
Location: Beam ref B1

Steel beam with welded bottom flange plate

Find centroid of loads taking moments about front edge of flange plate. The eccentricity of total load relative to beam centre line will be calculated.

Width of beam section \( B = 166.8 \text{ mm} \)
Total section width \( X = 290 \text{ mm} \)
Flange load position \( L_p = 41.5 \text{ mm} \)
Beam load position \( L_b = 240 \text{ mm} \)

Loadings - unfactored

On flange plate:  
- Dead load \( W_3 = 8 \text{ kN} \)
- Imposed load \( W_4 = 3 \text{ kN} \)

On beam:  
- Dead load \( W_1 = 14.5 \text{ kN} \)
- Imposed load \( W_2 = 11 \text{ kN} \)

Partial load factors

Dead load factor \( \gamma_d = 1.4 \)
Imposed load factor \( \gamma_i = 1.6 \)

Load case - unfactored

Flange moment \( M_p = (W_3 + W_4) \times L_p / 10^3 = 0.4565 \text{ kNm} \)
Beam moment \( M_b = (W_1 + W_2) \times L_b / 10^3 = 6.12 \text{ kNm} \)
Factored loading \( W_f = \gamma_d \times (W_3 + W_1) + \gamma_i \times (W_4 + W_2) = 53.9 \text{ kN} \)
Total moment \( M = M_p + M_b = 6.5765 \text{ kNm} \)
SUMMARY OF DESIGN PARAMETERS FOR TORSION

Factored loading: 53.9 kN
Load Eccentricity: 26.422 mm
Partial safety factor: 1.4767
Location: Beam ref B1

Steel beam with welded bottom flange plate

Width of beam section \( b = 166.8 \text{ mm} \)
Total section width \( X = 290 \text{ mm} \)
Flange load position \( L_p = 41.5 \text{ mm} \)
Beam load position \( L_b = 240 \text{ mm} \)

Loadings - unfactored

On flange plate:
- Permnt load \( W_3 = 8 \text{ kN} \)
- Imposed load \( W_4 = 3 \text{ kN} \)

On beam:
- Permnt load \( W_1 = 14.5 \text{ kN} \)
- Imposed load \( W_2 = 11 \text{ kN} \)

Partial load factors

- Permanent load factor \( \gamma_G = 1.35 \)
- Imposed load factor \( \gamma_Q = 1.5 \)

Load case - unfactored

- Flange moment \( M_p = (W_3 + W_4) \times L_p / 10^3 = 0.4565 \text{ kNm} \)
- Beam moment \( M_b = (W_1 + W_2) \times L_b / 10^3 = 6.12 \text{ kNm} \)
- Factored loading \( W_f = \gamma_G \times (W_3 + W_1) + \gamma_Q \times (W_4 + W_2) = 51.375 \text{ kN} \)
- Total moment \( M_{Ed} = M_p + M_b = 6.5765 \text{ kNm} \)
<table>
<thead>
<tr>
<th>SUMMARY OF</th>
<th>Factored loading: 51.375 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESIGN PARAMETERS</td>
<td>Load Eccentricity: 26.422 mm</td>
</tr>
<tr>
<td>FOR TORSION</td>
<td>Partial safety factor: 1.4075</td>
</tr>
</tbody>
</table>
### Location: I section beam - Mark 12a

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>$d = 77$ kN/m$^3$</td>
</tr>
<tr>
<td>Part description</td>
<td>Flanges</td>
</tr>
<tr>
<td>Length</td>
<td>$l(1) = 10$ m</td>
</tr>
<tr>
<td>X-sectional area</td>
<td>$a(1) = 0.030781$ m$^2$</td>
</tr>
<tr>
<td>Volume of part</td>
<td>$v = l(i) * a(i) = 0.30781$ m$^3$</td>
</tr>
<tr>
<td>Weight of part</td>
<td>$w = v * d = 23.701$ kN</td>
</tr>
<tr>
<td>Part description</td>
<td>Web</td>
</tr>
<tr>
<td>Length</td>
<td>$l(2) = 10$ m</td>
</tr>
<tr>
<td>X-sectional area</td>
<td>$a(2) = 0.018217$ m$^2$</td>
</tr>
<tr>
<td>Volume of part</td>
<td>$v = l(i) * a(i) = 0.18217$ m$^3$</td>
</tr>
<tr>
<td>Weight of part</td>
<td>$w = v * d = 14.027$ kN</td>
</tr>
</tbody>
</table>

**Totals for item:**
- Volume of item: $0.48998$ m$^3$
- Weight of item: $37.728$ kN
Location: First floor beams

Loading on elements of structure

Element name - Mark 12B

Factored load combination chosen 1.4Gk + 1.6Qk

Uniformly distributed load

Description of dead load: self weight
Dead load D(1)=15.2 kN/m
Dead load factored by 1.4 21.28 kN/m

Description of live load: Load from above
Imposed load L(1)=25 kN/m
Live load factored by 1.6 40 kN/m

Total unfactored load 40.2 kN/m
Total factored load 61.28 kN/m

Element name - Mark 12B

Factored load combination chosen 0.9Gk + 1.6Qk

Point load

Description of dead load: Tank self weight
Dead load D(2)=15.2 kN
Dead load factored by 0.9 13.68 kN

Description of live load: Tank contents
Imposed load L(2)=25 kN
Live load factored by 1.6 40 kN

Total unfactored load 40.2 kN
Total factored load 53.68 kN
**Location: Example 1 Three span beam using EC0 expression 6.10**

**Partial safety factors ULS**

Permanent actions (unfavourable) \( \gamma_G = 1.35 \)
Permanent actions (favourable) \( \gamma_G = 1.0 \)
Variable actions factor \( \gamma_Q = 1.5 \)

**ULS load combinations using EC0 expression 6.10**

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Qk</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5Qk</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Qk</td>
</tr>
<tr>
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<td>1.35Gk</td>
<td>1.35Gk+1.5Qk</td>
</tr>
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<td>1.0Gk</td>
<td>1.0Gk+1.5Qk</td>
<td>1.0Gk</td>
</tr>
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<td>1.35Gk+1.5Qk</td>
<td>1.35Gk+1.5Qk</td>
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<td>1.35Gk</td>
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<td>1.35Gk+1.5Qk</td>
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<td>1.35Gk+1.5Qk</td>
<td>1.35Gk+1.5Qk</td>
</tr>
</tbody>
</table>

**SLS load combinations using EC0 expression 6.10**

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>1.0Gk</td>
<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>9</td>
<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
<td>1.0Gk+1.0Qk</td>
</tr>
</tbody>
</table>

**Characteristic loads on beam**

- Char permanent load on span AB \( Gk1 = 25 \) kN/m
- Char permanent load on span BC \( Gk2 = 25 \) kN/m
- Char permanent load on span CD \( Gk3 = 25 \) kN/m
- Char variable load on span AB \( Qk1 = 10 \) kN/m
- Char variable load on span BC \( Qk2 = 10 \) kN/m
- Char variable load on span CD \( Qk3 = 10 \) kN/m
### ULS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load on span AB (kN/m)</th>
<th>Load on span BC (kN/m)</th>
<th>Load on span CD (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>33.75</td>
<td>48.75</td>
<td>33.75</td>
</tr>
<tr>
<td>2</td>
<td>40</td>
<td>25</td>
<td>40</td>
</tr>
<tr>
<td>3</td>
<td>48.75</td>
<td>33.75</td>
<td>48.75</td>
</tr>
<tr>
<td>4</td>
<td>25</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td>5</td>
<td>48.75</td>
<td>48.75</td>
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### SLS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
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</tr>
<tr>
<td>9</td>
<td>35</td>
<td>25</td>
<td>35</td>
</tr>
</tbody>
</table>
Location: Ex 2 Three span beam using EC0 expressions 6.10a & 6.10b

Partial safety factors ULS

Permanent actions (unfavourable) $g_a G = 1.35$ (expression 6.10a)
Permanent actions (unfavourable) $g_a G = 1.25$ (expression 6.10b)
Permanent actions (favourable) $g_a G = 1.0$
Variable actions factor $g_Q = 1.5$
Factor $\psi_0$ for variable load $\psi_0 = 0.7$

ULS load combinations using EC0 expressions 6.10a and 6.10b

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5\psi_0Qk</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5\psi_0Qk</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5\psi_0Qk</td>
</tr>
<tr>
<td>3</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Qk</td>
<td>1.25Gk</td>
</tr>
<tr>
<td>4</td>
<td>1.0Gk+1.5Qk</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Qk</td>
</tr>
<tr>
<td>5</td>
<td>1.35Gk+1.5\psi_0Qk</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5\psi_0Qk</td>
</tr>
<tr>
<td>6</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5\psi_0Qk</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>7</td>
<td>1.25Gk+1.5Qk</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Qk</td>
</tr>
<tr>
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<td>1.0Gk</td>
<td>1.0Gk+1.5Qk</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>9</td>
<td>1.35Gk+1.5\psi_0Qk</td>
<td>1.35Gk+1.5\psi_0Qk</td>
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<td>1.35Gk</td>
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<tr>
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<td>1.25Gk+1.5Qk</td>
<td>1.25Gk+1.5Qk</td>
<td>1.25Gk</td>
</tr>
<tr>
<td>12</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Qk</td>
<td>1.25Gk+1.5Qk</td>
</tr>
<tr>
<td>13</td>
<td>1.35Gk+1.5\psi_0Qk</td>
<td>1.35Gk+1.5\psi_0Qk</td>
<td>1.35Gk+1.5\psi_0Qk</td>
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<tr>
<td>14</td>
<td>1.25Gk+1.5Qk</td>
<td>1.25Gk+1.5Qk</td>
<td>1.25Gk+1.5Qk</td>
</tr>
</tbody>
</table>

SLS load combinations using EC0 expressions 6.10a and 6.10b

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>1.0Gk</td>
<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
</tr>
</tbody>
</table>
Characteristics loads on beam

Char permanent load on span AB \( Gk1 = 25 \text{ kN/m} \)
Char permanent load on span BC \( Gk2 = 25 \text{ kN/m} \)
Char permanent load on span CD \( Gk3 = 25 \text{ kN/m} \)
Char variable load on span AB \( Qk1 = 10 \text{ kN/m} \)
Char variable load on span BC \( Qk2 = 10 \text{ kN/m} \)
Char variable load on span CD \( Qk3 = 10 \text{ kN/m} \)

ULS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load on span AB (kN/m)</th>
<th>Load on span BC (kN/m)</th>
<th>Load on span CD (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>33.75</td>
<td>44.25</td>
<td>33.75</td>
</tr>
<tr>
<td>2</td>
<td>35.5</td>
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<tr>
<td>3</td>
<td>31.25</td>
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<td>31.25</td>
</tr>
<tr>
<td>4</td>
<td>40</td>
<td>25</td>
<td>40</td>
</tr>
<tr>
<td>5</td>
<td>44.25</td>
<td>33.75</td>
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</tr>
<tr>
<td>6</td>
<td>25</td>
<td>35.5</td>
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<td>7</td>
<td>46.25</td>
<td>31.25</td>
<td>46.25</td>
</tr>
<tr>
<td>8</td>
<td>25</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td>9</td>
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<td>44.25</td>
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<tr>
<td>10</td>
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<td>14</td>
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</table>
### SLS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
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</tr>
<tr>
<td>16</td>
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<td>25</td>
<td>35</td>
</tr>
</tbody>
</table>

If the permanent actions are ≤ 4.5 times the variable actions then loading combinations 1, 2, 5, 6, 9, 10 & 13 could be ignored.
Location: Example 3 Two span beam using EC0 expression 6.10

Partial safety factors ULS

Permanent actions (unfavourable) \( \gamma_G = 1.35 \)
Permanent actions (favourable) \( \gamma_G = 1.0 \)
Variable actions factor \( \gamma_Q = 1.5 \)

ULS load combinations using EC0 expression 6.10

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th></th>
<th>Span BC</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35Gk</td>
<td></td>
<td>1.35Gk+1.5Qk</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5Qk</td>
<td></td>
<td>1.0Gk</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.35Gk+1.5Qk</td>
<td></td>
<td>1.35Gk</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1.0Gk</td>
<td></td>
<td>1.0Gk+1.5Qk</td>
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<td>1.35Gk+1.5Qk</td>
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<td>1.35Gk+1.5Qk</td>
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</tbody>
</table>

SLS load combinations using EC0 expression 6.10

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th></th>
<th>Span BC</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>1.0Gk</td>
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<td>1.0Gk+1.0Qk</td>
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<tr>
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<td>1.0Gk+1.0Qk</td>
<td></td>
<td>1.0Gk</td>
<td></td>
</tr>
</tbody>
</table>

Characteristic loads on beam

Char permanent load on span AB \( G_{k1} = 25 \text{ kN/m} \)
Char permanent load on span BC \( G_{k2} = 25 \text{ kN/m} \)
Char variable load on span AB \( Q_{k1} = 10 \text{ kN/m} \)
Char variable load on span BC \( Q_{k2} = 10 \text{ kN/m} \)
### ULS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load span AB (kN/m)</th>
<th>Load span BC (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>33.75</td>
<td>48.75</td>
</tr>
<tr>
<td>2</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td>3</td>
<td>48.75</td>
<td>33.75</td>
</tr>
<tr>
<td>4</td>
<td>25</td>
<td>40</td>
</tr>
<tr>
<td>5</td>
<td>48.75</td>
<td>48.75</td>
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</table>

### SLS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>25</td>
<td>35</td>
</tr>
<tr>
<td>7</td>
<td>35</td>
<td>25</td>
</tr>
</tbody>
</table>
Location: Ex 4 Two span beam using EC0 expressions 6.10a & 6.10b

Partial safety factors ULS

Permanent actions (unfavourable) \( \gamma_G = 1.35 \) (expression 6.10a)
Permanent actions (unfavourable) \( \gamma_G = 1.25 \) (expression 6.10b)
Permanent actions (favourable) \( \gamma_G = 1.0 \)
Variable actions factor \( \gamma_Q = 1.5 \)
Factor \( \psi_0 \) for variable load \( \psi_0 = 0.7 \)

ULS load combinations using EC0 expressions 6.10a and 6.10b

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5( \psi_0 )Qk</td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5( \psi_0 )Qk</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>3</td>
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<tr>
<td>4</td>
<td>1.0Gk+1.5Qk</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>5</td>
<td>1.35Gk+1.5( \psi_0 )Qk</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>6</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5( \psi_0 )Qk</td>
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<td>8</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Qk</td>
</tr>
<tr>
<td>9</td>
<td>1.35Gk+1.5( \psi_0 )Qk</td>
<td>1.35Gk+1.5( \psi_0 )Qk</td>
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</table>

SLS load combinations using EC0 expressions 6.10a and 6.10b

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
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<td>12</td>
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<td>1.0Gk</td>
</tr>
</tbody>
</table>
Characteristic loads on beam

Char permanent load on span AB  $G_{k1}=25$ kN/m
Char permanent load on span BC  $G_{k2}=25$ kN/m
Char variable load on span AB  $Q_{k1}=10$ kN/m
Char variable load on span BC  $Q_{k2}=10$ kN/m

ULS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load span AB (kN/m)</th>
<th>Load span BC (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>33.75</td>
<td>44.25</td>
</tr>
<tr>
<td>2</td>
<td>35.5</td>
<td>25</td>
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<tr>
<td>3</td>
<td>31.25</td>
<td>46.25</td>
</tr>
<tr>
<td>4</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td>5</td>
<td>44.25</td>
<td>33.75</td>
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<tr>
<td>6</td>
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<td>10</td>
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SLS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
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<td>35</td>
</tr>
<tr>
<td>12</td>
<td>35</td>
<td>25</td>
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</tbody>
</table>

If the permanent actions are ≤ 4.5 times the variable actions then loading combinations 1, 2, 5, 6 & 9 could be ignored.
**Location:** Example 5 Cantilever span beam using EC0 expression 6.10

**Partial safety factors ULS**

- Permanent actions (unfavourable) \( \gamma_G = 1.35 \)
- Permanent actions (unfavourable) \( \gamma_G = 1.1 \) (for EQU)
- Permanent actions (favourable) \( \gamma_G = 1.0 \)
- Permanent actions (favourable) \( \gamma_G = 0.9 \) (for EQU)
- Variable actions factor \( \gamma_Q = 1.5 \)

**ULS load combinations using EC0 expression 6.10**

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35Gk+1.5Qk</td>
<td>1.35Gk+1.5Qk</td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5Qk</td>
<td>1.0Gk+1.5Qk</td>
</tr>
<tr>
<td>3</td>
<td>1.35Gk+1.5Qk</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>4</td>
<td>1.0Gk+1.5Qk</td>
<td>1.0Gk</td>
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<tr>
<td>5</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Qk</td>
</tr>
<tr>
<td>6</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Qk</td>
</tr>
</tbody>
</table>

**SLS load combinations using EC0 expression 6.10**

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>8</td>
<td>1.0Gk</td>
<td>1.0Gk+1.0Qk</td>
</tr>
</tbody>
</table>

**EQU overturning check using EC0 expression 6.10**

<table>
<thead>
<tr>
<th>Comb</th>
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<th>Span BC</th>
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<tbody>
<tr>
<td>9</td>
<td>0.9Gk</td>
<td>1.1Gk+1.5Qk</td>
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</tbody>
</table>
Characteristic loads on beam

Char permanent load on span AB    Gk1=25 kN/m
Char permanent load on span BC    Gk2=25 kN/m
Char variable load on span AB     Qk1=10 kN/m
Char variable load on span BC     Qk2=10 kN/m

ULS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load on span AB (kN/m)</th>
<th>Load on span BC (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>48.75</td>
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</tr>
<tr>
<td>2</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>3</td>
<td>48.75</td>
<td>33.75</td>
</tr>
<tr>
<td>4</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td>5</td>
<td>33.75</td>
<td>48.75</td>
</tr>
<tr>
<td>6</td>
<td>25</td>
<td>40</td>
</tr>
</tbody>
</table>

SLS & EQU load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>35</td>
<td>25</td>
</tr>
<tr>
<td>8</td>
<td>25</td>
<td>35</td>
</tr>
<tr>
<td>9</td>
<td>22.5</td>
<td>42.5</td>
</tr>
</tbody>
</table>
Location: Example 6 Single span beam using EC0 expression 6.10

Partial safety factors ULS

Permanent actions (unfavourable) \( \gamma_G = 1.35 \)
Variable actions factor \( \gamma_Q = 1.5 \)

ULS and SLS load combinations using expression 6.10

\[
\begin{array}{c|c}
\text{Comb} & \text{Span AB} \\
1 & 1.35Gk + 1.5Qk \\
2 & 1.0Gk + 1.0Qk \\
\end{array}
\]

Characteristic loads on beam

Char permanent load \( G_k = 25 \text{ kN/m} \)
Char variable load \( Q_k = 10 \text{ kN/m} \)

ULS & SLS load combination values

\[
\begin{array}{c|c}
\text{Comb} & \text{Load on span AB (kN/m)} \\
1 & 48.75 \\
2 & 35 \\
\end{array}
\]
Location: Ex 7 Single span beam using EC0 expressions 6.10 & 6.10b

Partial safety factors ULS

Permanent actions (unfavourable) $\gamma_G = 1.35$ (expression 6.10a)
Permanent actions (unfavourable) $\gamma_G = 1.25$ (expression 6.10b)
Variable actions factor $\gamma_Q = 1.5$
Factor $\psi_0$ for variable load $\psi_0 = 0.7$

ULS & SLS load combinations using EC0 expressions 6.10a & 6.10b

Characteristic loads on beam
Char permanent load $G_k = 25$ kN/m
Char variable load $Q_k = 10$ kN/m

ULS & SLS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load on span AB (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>44.25</td>
</tr>
<tr>
<td>2</td>
<td>46.25</td>
</tr>
<tr>
<td>3</td>
<td>35</td>
</tr>
</tbody>
</table>
Location: Example 8 Three span beam using EC0 expression 6.10

Partial safety factors ULS

Permanent actions (unfavourable)  $\gamma_aG = 1.35$
Permanent actions (favourable)  $\gamma_aG = 1.0$
Variable actions factor  $\gamma_Q = 1.5$

ULS load combinations using EC0 expression 6.10

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>1a</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q2</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5Q1</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q1</td>
</tr>
<tr>
<td>2a</td>
<td>1.0Gk+1.5Q2</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q2</td>
</tr>
<tr>
<td>3</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q1</td>
</tr>
<tr>
<td>3a</td>
<td>1.35Gk+1.5Q2</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q2</td>
</tr>
<tr>
<td>4</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q1</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>4a</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q2</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>5</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>5a</td>
<td>1.35Gk+1.5Q2</td>
<td>1.35Gk+1.5Q2</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>6</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk+1.5Q1</td>
</tr>
<tr>
<td>6a</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q2</td>
<td>1.35Gk+1.5Q2</td>
</tr>
<tr>
<td>7</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk+1.5Q1</td>
</tr>
<tr>
<td>7a</td>
<td>1.35Gk+1.5Q2</td>
<td>1.35Gk+1.5Q2</td>
<td>1.35Gk+1.5Q2</td>
</tr>
</tbody>
</table>

SLS load combinations using EC0 expression 6.10

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>1.0Gk</td>
<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>9</td>
<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
<td>1.0Gk+1.0Qk</td>
</tr>
</tbody>
</table>
Note: Use loading combinations 1-7 if Qk1 is the leading variable load and loading combinations 1a-7a if Qk2 is the leading variable load. If this is not readily evident then all the above load combinations need to be considered.
Location: Ex 9 Three span beam using EC0 expressions 6.10a & 6.10b

Partial safety factors ULS

Permanent actions (unfavourable) $\gamma_G = 1.35$ (expression 6.10a)
Permanent actions (unfavourable) $\gamma_G = 1.25$ (expression 6.10b)
Permanent actions (favourable) $\gamma_G = 1.0$
Variable actions factor $\gamma_Q = 1.5$

ULS load combinations using expressions 6.10a and 6.10b

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5Q1</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q1</td>
</tr>
<tr>
<td>3</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Q2</td>
<td>1.25Gk</td>
</tr>
<tr>
<td>3a</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Q2</td>
<td>1.25Gk</td>
</tr>
<tr>
<td>4</td>
<td>1.0Gk+1.5Q2</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q2</td>
</tr>
<tr>
<td>4a</td>
<td>1.0Gk+1.5Q3</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q3</td>
</tr>
<tr>
<td>5</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q1</td>
</tr>
<tr>
<td>6</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q1</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>7</td>
<td>1.25Gk+1.5Q2</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Q2</td>
</tr>
<tr>
<td>7a</td>
<td>1.25Gk+1.5Q3</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Q3</td>
</tr>
<tr>
<td>8</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q2</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>8a</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q3</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>9</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>10</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk+1.5Q1</td>
</tr>
</tbody>
</table>
### Loadings and foundations

Made by: IFB
Date: 02/12/19

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Ref No: SC726 EC

#### Eurocode load combinations

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1.25Gk+1.5Q2</td>
<td>1.25Gk+1.5Q2</td>
<td>1.25Gk</td>
</tr>
<tr>
<td>11a</td>
<td>1.25Gk+1.5Q3</td>
<td>1.25Gk+1.5Q3</td>
<td>1.25Gk</td>
</tr>
<tr>
<td>12</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Q2</td>
<td>1.25Gk+1.5Q2</td>
</tr>
<tr>
<td>12a</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Q3</td>
<td>1.25Gk+1.5Q3</td>
</tr>
<tr>
<td>13</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk+1.5Q1</td>
</tr>
<tr>
<td>14</td>
<td>1.25Gk+1.5Q2</td>
<td>1.25Gk+1.5Q2</td>
<td>1.25Gk+1.5Q2</td>
</tr>
<tr>
<td>14a</td>
<td>1.25Gk+1.5Q3</td>
<td>1.25Gk+1.5Q3</td>
<td>1.25Gk+1.5Q3</td>
</tr>
</tbody>
</table>

#### SLS load combinations using EC0 expressions 6.10a and 6.10b

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
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<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
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<tr>
<td>16</td>
<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
<td>1.0Gk+1.0Qk</td>
</tr>
</tbody>
</table>

Note: Use loading combinations 1-14 if Qk1 is the leading variable load and loading combinations 1,2,3a,4a,5,6,7a,8a,9,10,11a,12a,13 and 14a if Qk2 is the leading variable load. If this is not readily evident then all the above load combinations need to be considered. If the permanent actions are ≥ 4.5 times the variable actions then loading combinations 1,2,5,6,9,10 and 13 could be ignored.
**Location:** Example 10 Two span beam using EC0 expression 6.10

**Partial safety factors ULS**

- Permanent actions (unfavourable): $\gamma_{G}=1.35$
- Permanent actions (favourable): $\gamma_{G}=1.0$
- Variable actions factor: $\gamma_{Q}=1.5$

**ULS load combinations using EC0 expression 6.10**

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q1</td>
</tr>
<tr>
<td>1a</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q2</td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5Q1</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>2a</td>
<td>1.0Gk+1.5Q2</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>3</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>3a</td>
<td>1.35Gk+1.5Q2</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>4</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q1</td>
</tr>
<tr>
<td>4a</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q2</td>
</tr>
<tr>
<td>5</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk+1.5Q1</td>
</tr>
<tr>
<td>5a</td>
<td>1.35Gk+1.5Q2</td>
<td>1.35Gk+1.5Q2</td>
</tr>
</tbody>
</table>

**SLS load combinations using EC0 expression 6.10**

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>1.0Gk</td>
<td>1.0Gk+1.0Qk</td>
</tr>
<tr>
<td>7</td>
<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
</tr>
</tbody>
</table>

Note: Use loading combinations 1-5 if Qk1 is the leading variable load and loading combinations 1a-5a if Qk2 is the leading variable load. If this is not readily evident then all the above load combinations need to be considered.
Location: Ex 11 Two span beam using EC0 expressions 6.10a & 6.10b

Partial safety factors ULS

Permanent actions (unfavourable) \( \gamma_G=1.35 \) (expression 6.10a)
Permanent actions (unfavourable) \( \gamma_G=1.25 \) (expression 6.10b)
Permanent actions (favourable) \( \gamma_G=1.0 \)
Variable actions factor \( \gamma_Q=1.5 \)

ULS load combinations using expressions 6.10a & 6.10b

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q1</td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5Q1</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>3</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Q2</td>
</tr>
<tr>
<td>3a</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Q3</td>
</tr>
<tr>
<td>4</td>
<td>1.0Gk+1.5Q2</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>4a</td>
<td>1.0Gk+1.5Q3</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>5</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk</td>
</tr>
<tr>
<td>6</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q1</td>
</tr>
<tr>
<td>7</td>
<td>1.25Gk+1.5Q2</td>
<td>1.25Gk</td>
</tr>
<tr>
<td>7a</td>
<td>1.25Gk+1.5Q3</td>
<td>1.25Gk</td>
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<tr>
<td>8</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q2</td>
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<tr>
<td>8a</td>
<td>1.0Gk</td>
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<tr>
<td>9</td>
<td>1.35Gk+1.5Q1</td>
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<td>1.25Gk+1.5Q2</td>
<td>1.25Gk+1.5Q2</td>
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<td>10a</td>
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</table>
### SLS load combinations using EC0 expressions 6.10a & 6.10b

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>1.0Gk</td>
<td>1.0Gk+1.0Qk</td>
</tr>
<tr>
<td>7</td>
<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
</tr>
</tbody>
</table>

Note: Use loading combinations 1-10 if Qk1 is the leading variable load and loading combinations 1,2,3a,4a,5,6,7a,8a,9 and 10a if Qk2 is the leading variable load. If this is not readily evident then all the above load combinations need to be considered. Furthermore, if the permanent actions are ≤ 4.5 times the variable actions then loading combinations 1,2,5,6 and 9 could be ignored.
Location: Example 12 Cantilever span beam using EC0 expression 6.10

Partial safety factors ULS

Permanent actions (unfavourable) \( \gamma_G = 1.35 \)
Permanent actions (favourable) \( \gamma_G = 1.0 \)
Variable actions factor \( \gamma_Q = 1.5 \)

ULS load combinations using EC0 expression 6.10

<table>
<thead>
<tr>
<th>Comb</th>
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<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk+1.5Q1</td>
</tr>
<tr>
<td>1a</td>
<td>1.35Gk+1.5Q2</td>
<td>1.35Gk+1.5Q2</td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5Q1</td>
<td>1.0Gk+1.5Q1</td>
</tr>
<tr>
<td>2a</td>
<td>1.0Gk+1.5Q2</td>
<td>1.0Gk+1.5Q2</td>
</tr>
<tr>
<td>3</td>
<td>1.35Gk+1.5Q1</td>
<td>1.35Gk</td>
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<tr>
<td>3a</td>
<td>1.35Gk+1.5Q2</td>
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<tr>
<td>4</td>
<td>1.0Gk+1.5Qk1</td>
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</tr>
<tr>
<td>4a</td>
<td>1.0Gk+1.5Qk2</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>5</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q1</td>
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<tr>
<td>5a</td>
<td>1.35Gk</td>
<td>1.35Gk+1.5Q2</td>
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<tr>
<td>6</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q1</td>
</tr>
<tr>
<td>6a</td>
<td>1.0Gk</td>
<td>1.0Gk+1.5Q2</td>
</tr>
</tbody>
</table>

SLS load combinations using EC0 expression 6.10

<table>
<thead>
<tr>
<th>Comb</th>
<th>Span AB</th>
<th>Span BC</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>1.0Gk+1.0Qk</td>
<td>1.0Gk</td>
</tr>
<tr>
<td>8</td>
<td>1.0Gk</td>
<td>1.0Gk+1.0Qk</td>
</tr>
</tbody>
</table>

Note: Use loading combinations 1-6 if Qk1 is the leading variable load and loading combinations 1a-6a if Qk2 is the leading variable load. If this is not readily evident then all the above load combinations need to be considered.
Location: Example 13 Single span beam using EC0 expression 6.10

Partial safety factors ULS

Permanent actions (unfavourable) \( \gamma_G = 1.35 \)
Variable actions factor \( \gamma_Q = 1.5 \)
Factor \( \psi_0 \) for variable load 1 \( \psi_0 v_1 = 0.7 \)
Factor \( \psi_0 \) for variable load 2 \( \psi_0 v_2 = 0.5 \)

ULS & SLS load combinations using expression 6.10

\[
\begin{align*}
\text{Variable load 2} & : Q_k2 \\
\text{Variable load 1} & : Q_k1 \\
\text{Comb} & \quad \text{Load on span AB} \\
1 & 1.35G_k + 1.5Q_k1 + 1.5\psi_0Q_k2 \\
2 & 1.35G_k + 1.5Q_k2 + 1.5\psi_0Q_k1 \\
3 & 1.0G_k + 1.0Q_k1 + 1.0\psi_0Q_k2 \\
4 & 1.0G_k + 1.0Q_k2 + 1.0\psi_0Q_k1
\end{align*}
\]

Characteristic loads on beam

Char permanent load 1 \( G_k = 25 \text{ kN/m} \)
Char variable load 1 \( Q_k1 = 10 \text{ kN/m} \)
Char variable load 2 \( Q_k2 = 5 \text{ kN/m} \)

ULS & SLS load combination values

\[
\begin{align*}
\text{Comb} & \quad \text{Load on span AB} \\
1 & 52.5 \\
3 & 37.5
\end{align*}
\]

Governing load combinations
Partial safety factors ULS

Permanent actions (unfavourable) \( \gamma_G = 1.35 \) (expression 6.10a)
Permanent actions (unfavourable) \( \gamma_G = 1.25 \) (expression 6.10b)
Variable actions factor \( \gamma_Q = 1.5 \)
Factor \( \psi_0 \) for variable load 1 \( \psi_{0v1} = 0.7 \)
Factor \( \psi_0 \) for variable load 2 \( \psi_{0v2} = 0.5 \)

ULS & SLS load combinations using expressions 6.10a & 6.10b

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load on span AB (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>48</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>37.5</td>
</tr>
</tbody>
</table>

Characteristic loads on beam

Char permanent load \( G_k = 25 \) kN/m
Char variable load 1 \( Q_{k1} = 10 \) kN/m
Char variable load 2 \( Q_{k2} = 5 \) kN/m

ULS & SLS load combination values

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load on span AB (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>48</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>37.5</td>
</tr>
</tbody>
</table>

Governing load combinations
**Location:** Ex15 - Steel braced frame (simple construction)

**Partial safety factors ULS**

Permanent actions (unfavourable) \( \gamma_G = 1.35 \) (Expr. 6.10a)
Permanent actions (unfavourable) \( \gamma_G = 1.25 \) (Expr. 6.10b, governs)
Permanent actions (favourable) \( \gamma_G = 1.0 \) (for STR)
Permanent actions (favourable) \( \gamma_G = 0.9 \) (for EQU)
Variable actions factor \( \gamma_Q = 1.5 \)

**ULS Load Combinations to EC0 Expressions 6.10a & 6.10b**

The frame is assumed to be braced (simple construction) and hence pattern loading need not be considered. Second order effects need not be considered provided the frame is sufficiently stiff (i.e. sway deformation under the design loading is relatively small) - this is deemed to be the case for elastic analysis when \( acr > 10 \). The load combinations below assume the dead load is not substantially greater than the imposed load. For storage areas replace the load factor of 1.05 used below with 1.5 as the load combination factor is \( \psi = 1.0 \) and not \( \psi = 0.7 \).

\[
\begin{array}{c}
Wk + EHF \\
\downarrow \\
Wk + EHF \\
\downarrow \\
Wk + EHF \\
\downarrow \\
Wk + EHF \\
\downarrow \\
\end{array}
\]

**Symbols:**
- \( I_k \) = Imposed floor loading (Load combination factor \( \psi = 0.7 \))
- \( I_{kr} \) = Imposed roof loading (Load combination factor \( \psi = 0.7 \))
- \( S_k \) = Snow load applied to roof (Load combination factor \( \psi = 0.5 \))
- \( W_k \) = Horizontal wind load (Load combination factor \( \psi = 0.5 \))
- \( W_{ku} \) = Wind uplift
- \( EHF \) = Frame imperfections & equivalent horizontal forces (i.e. Notional Horizontal Forces)
### Load Combinations Using Equation 6.10b

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load at each floor level</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25Gk+1.5(Ikr or Sk)</td>
</tr>
<tr>
<td>2</td>
<td>1.0Gk+1.5Wku</td>
</tr>
<tr>
<td>3</td>
<td>1.25Gk+1.5Sk+0.75Wk</td>
</tr>
<tr>
<td>4</td>
<td>1.25Gk+1.5Wk+0.75Sk</td>
</tr>
<tr>
<td>5</td>
<td>1.25Gk+1.5Ik</td>
</tr>
<tr>
<td>6</td>
<td>1.25Gk+1.5Ik+1.5(Ikr or Sk)</td>
</tr>
<tr>
<td>7</td>
<td>1.25Gk+1.5Ik+1.5Sk+0.75Wk</td>
</tr>
<tr>
<td>8</td>
<td>1.25Gk+1.5Wk+1.05Ik+0.75Sk</td>
</tr>
<tr>
<td>9</td>
<td>1.25Gk+1.5Ik+1.5(Ikr or Sk)+EHF</td>
</tr>
<tr>
<td>10</td>
<td>1.25Gk+1.5Ik+1.5Sk+0.75Wk+EHF</td>
</tr>
<tr>
<td>11</td>
<td>1.25Gk+1.5Wk+1.05Ik+0.75Sk+EHF</td>
</tr>
</tbody>
</table>

**NOTE:** Equation 6.10b is assumed to govern the design.

**NOTE:** For storage areas replace load factor 1.05 in table with 1.5.

### SLS Load Combinations to EC0 Expressions 6.10a & 6.10b

Based on the UK NA to EN 1993-1-1 (Clauses NA.2.23 and NA.2.24) the vertical and horiz deflections may be checked using the characteristic combination with variable loads only (i.e. permanent loads should not be included). The following combinations are based on EC0 expression 6.10b. EHF are not required in SLS load combinations.

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load at each floor level</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>1.0Ik+1.0(Ikr or Sk)</td>
</tr>
<tr>
<td>13</td>
<td>1.0Wk</td>
</tr>
</tbody>
</table>

### EQU Overturning Check to EC0 Expression 6.10

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load at each floor level</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>0.9Gk+1.5Wk+EHF</td>
</tr>
</tbody>
</table>
Location: Ex16 - Steel frame (continuous construction)

Partial safety factors ULS

Permanent actions (unfavourable) \( \gamma_G = 1.35 \) (Expr. 6.10a)
Permanent actions (unfavourable) \( \gamma_G = 1.25 \) (Expr. 6.10b, governs)
Permanent actions (favourable) \( \gamma_G = 1.0 \) (for STR)
Permanent actions (favourable) \( \gamma_G = 0.9 \) (for EQU)
Variable actions factor \( \gamma_Q = 1.5 \)

ULS Load Combinations to EC0 Expressions 6.10a & 6.10b

As the steel frame is of continuous construction pattern loading will need to be considered. Second order effects need not be considered provided the frame is sufficiently stiff (i.e. sway deformation under the design loading is relatively small) - this is deemed to be the case for elastic analysis when acr>10. The load combinations below assume the dead load is not substantially greater than the imposed load. For storage areas replace the load factor of 1.05 used below with 1.5 as the load combination factor is \( \psi = 1.0 \) and not \( \psi = 0.7 \).

Symbols:
- \( I_k \) = Imposed floor loading (Load combination factor \( \psi = 0.7 \))
- \( I_{kr} \) = Imposed roof loading (Load combination factor \( \psi = 0.7 \))
- \( S_k \) = Snow load applied to roof (Load combination factor \( \psi = 0.5 \))
- \( W_k \) = Horizontal wind load (Load combination factor \( \psi = 0.5 \))
- \( W_{ku} \) = Wind uplift
- \( EHF \) = Frame imperfections & equivalent horizontal forces
  (i.e. Notional Horizontal Forces)
### Loadings and foundations

Made by: IFB  
Date: 02/12/19

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Ref No: SC726 EC

---

<table>
<thead>
<tr>
<th>Combination</th>
<th>Span AB</th>
<th>Span BC</th>
<th>Span CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25Gk+1.5Ik+1.5(Ikr or Sk)+EHF</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1.00Gk+1.5Wku+EHF</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.25Gk+1.5Ik+1.5Sk+0.75Wk+EHF</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1.25Gk+1.5Wk+1.05Ik+0.75Sk+EHF</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1.25Gk+1.5Qk</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Qk</td>
</tr>
<tr>
<td>6</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Qk</td>
<td>1.25Gk</td>
</tr>
<tr>
<td>7</td>
<td>1.25Gk+1.5Qk</td>
<td>1.25Gk</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>1.25Gk</td>
<td>1.25Gk+1.5Qk</td>
<td></td>
</tr>
</tbody>
</table>

For the pattern loading combinations 5 - 8 it has been assumed that the permanent actions are ≤ 4.5 times the variable actions. Storeys other than the one under consideration may be assumed to be uniformly loaded (EN 1993-1-1, Clause 6.2.1).  
NOTE: For storage areas replace 1.05 with 1.5 in table above

---

### SLS Load Combinations to EC0 Expressions 6.10a & 6.10b

Based on the UK NA to EN 1993-1-1 (Clauses NA.2.23 and NA.2.24) the vertical and horiz deflections may be checked using the characteristic combination with variable loads only (i.e. permanent loads should not be included). The following combinations are based on EC0 expression 6.10b. EHF are not required in SLS load combinations.

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load at each floor level</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>1.0Ik+1.0Sk+0.5Wk</td>
</tr>
<tr>
<td>10</td>
<td>1.0Wk+0.7Ik+0.5Sk</td>
</tr>
<tr>
<td>11</td>
<td>1.0Ik+1.0(Ikr or Sk)</td>
</tr>
<tr>
<td>12</td>
<td>1.0Wk</td>
</tr>
</tbody>
</table>

---

### EQU Overturning Check to EC0 Expression 6.10

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load at each floor level</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>0.9Gk+1.5Wk+EHF</td>
</tr>
</tbody>
</table>
**Location: Ex17 - Single span steel portal frame**

**Partial safety factors ULS**

Permanent actions (unfavourable) \( \text{gamG}=1.35 \) (Expr. 6.10a)
Permanent actions (unfavourable) \( \text{gamG}=1.25 \) (Expr. 6.10b, governs)
Permanent actions (favourable) \( \text{gamG}=1.0 \) (for STR)
Permanent actions (favourable) \( \text{gamG}=0.9 \) (for EQU)
Variable actions factor \( \text{gamQ}=1.5 \)

**ULS Load Combinations to EC0 Expression 6.10b**

Portal frame designers will generally choose Equations 6.10a & 6.10b over the more onerous Equation 6.10. Furthermore assuming the ratio of permanent to imposed loading is not high which is particularly unlikely for this form of construction, equation 6.10b combinations are more onerous than 6.10a combinations and will be adopted here.

For the single span steel portal frame below the combined permanent and service loads will be treated as one permanent load.

<table>
<thead>
<tr>
<th>B</th>
<th>Rafter</th>
<th>C</th>
<th>Apex Haunch</th>
<th>Rafter</th>
</tr>
</thead>
<tbody>
<tr>
<td>/Eaves</td>
<td></td>
<td>Haunch</td>
<td></td>
<td>Eaves</td>
</tr>
<tr>
<td>Haunch</td>
<td></td>
<td></td>
<td></td>
<td>Haunch</td>
</tr>
<tr>
<td>A</td>
<td></td>
<td></td>
<td></td>
<td>E</td>
</tr>
</tbody>
</table>

Symbols:
- \( G_{ksup} \) = Permanent load + Service load
- \( G_{kinf} \) = Permanent load
- \( I_k \) = Impose load (Load combination factor \( \psi=0.7 \))
- \( S_k \) = Uniform snow load (Load combination factor \( \psi=0.5 \))
- \( W_k \) = Horizontal wind load (Load combination factor \( \psi=0.5 \))
- \( W_{kup} \) = Wind uplift
- \( A_d \) = Load from snow drift (accidental load condition)
- \( EHF \) = Frame imperfections & equivalent horizontal forces (i.e. Notional Horizontal Forces)
Combination | Combinations From Equation 6.10b
--- | ---
1 | 1.25Gksup+1.5(Ik or Sk)+EHF
2 | 1.25Gksup+1.5Sk+0.75Wk+EHF
3 | 1.25Gksup+1.5Wk+0.75Sk+EHF
4 | 1.0Gkinf+1.5Wkup+EHF

Accidental Load Combinations From Equation 6.11b
5 | 1.0Gksup+1.0Ad+EHF
6 | 1.0Gksup+1.0Ad+0.2Wk+EHF

**SLS Load Combinations to EC0 Expressions 6.14b**

Based on the UK NA to EN 1993-1-1 (Clauses NA.2.23 and NA.2.24) the vertical and horizontal deflections may be checked using the characteristic combination with variable loads only (i.e. permanent loads should not be included). The following combinations are based on EC0 expression 6.14b. EHF are not required in SLS load combinations.

<table>
<thead>
<tr>
<th>Comb</th>
<th>Combinations From Equation 6.14b</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>1.0(Ik or Sk)</td>
</tr>
<tr>
<td>8</td>
<td>1.0Sk+0.5Wk</td>
</tr>
<tr>
<td>9</td>
<td>0.5Sk+1.0Wk</td>
</tr>
<tr>
<td>10</td>
<td>1.0Wkup</td>
</tr>
</tbody>
</table>

**EQU Overturning Check to EC0 Expression 6.10**

<table>
<thead>
<tr>
<th>Comb</th>
<th>Load at each floor level</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>0.9Gksup+1.5Wk+EHF</td>
</tr>
</tbody>
</table>
Location: Ex1 - Typical pilecap

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$

Total ultimate load on column $N = 2800 \text{ kN}$
Column dimension $cx = 500 \text{ mm}$
Column dimension $cy = 500 \text{ mm}$
Nominal pile diameter $PileD = 450 \text{ mm}$
Spacing of pile centres $A = 1350 \text{ mm}$
Overhang: pile face to cap edge $olap = 150 \text{ mm}$
Length of pile cap $lx = 2100 \text{ mm}$
Width of pile cap $ly = 750 \text{ mm}$
Concrete grade $f_{cu} = 35 \text{ N/mm}^2$
Steel strength $f_y = 500 \text{ N/mm}^2$
Cover to underside of main bars $Cover = 150 \text{ mm}$
Side and top cover to main bars $SCover = 75 \text{ mm}$
Chosen depth of pilecap $h = 920 \text{ mm}$

Main steel diameter $dia = 20 \text{ mm}$
Main steel spacing $Space = 60 \text{ mm}$

Distribution steel

Distribution steel diameter $LSize = 20 \text{ mm}$
Spacing of bars $LSpac = 260 \text{ mm}$

**MAIN REINFORCEMENT SUMMARY**

- Characteristic strength: $500 \text{ N/mm}^2$
- Diameter of bars: $20 \text{ mm}$
- Number of bars: $10$
- Spacing: $60 \text{ mm}$
- Area required: $2898 \text{ mm}^2$
- Area provided: $3141.6 \text{ mm}^2$
- Volume of steel: $8608 \text{ cm}^3$
- Weight of steel: $67.573 \text{ kg}$
<table>
<thead>
<tr>
<th>Part of Structure</th>
<th>Characteristic Strength</th>
<th>Diameter of Bars</th>
<th>Number of Bars</th>
<th>Spacing of Bars</th>
<th>Area Required</th>
<th>Area Provided</th>
<th>Volume of Steel</th>
<th>Weight of Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distribution</td>
<td>500 N/mm²</td>
<td>20 mm</td>
<td>8</td>
<td>260 mm</td>
<td>1196 mm²/m</td>
<td>1208.3 mm²/m</td>
<td>8419.5 cm³</td>
<td>66.093 kg</td>
</tr>
<tr>
<td>Top Bars</td>
<td>500 N/mm²</td>
<td>20 mm</td>
<td>3</td>
<td>260 mm</td>
<td></td>
<td></td>
<td>1837.8 cm³</td>
<td>14.427 kg</td>
</tr>
<tr>
<td>Side Bars</td>
<td>500 N/mm²</td>
<td>12 mm</td>
<td>3</td>
<td>250 mm</td>
<td></td>
<td></td>
<td>1323.2 cm³</td>
<td>10.387 kg</td>
</tr>
<tr>
<td>Quantities</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.449 m³</td>
<td>158.48 kg</td>
</tr>
<tr>
<td></td>
<td>Total Volume of Concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>20189 cm³</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total Area of Formwork</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.244 m²</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total Volume of Steel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>158.48 kg</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Approx. Self-weight</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>48.686 kN</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Approx. Total Load</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1425 kN</td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex2 - Heavily-loaded column.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Pilecap for two piles

Designed by Truss Analogy.

Calculations are in accordance with BS8110.

For bond calculations, any high-yield bars are assumed Type 2 deformed.

Total ultimate load on column \( N = 10000 \) kN

Column dimension \( cx = 500 \) mm

Column dimension \( cy = 500 \) mm

Nominal pile diameter \( \text{PileD} = 500 \) mm

Spacing of pile centres \( A = 1500 \) mm

Overhang: pile face to cap edge \( o_{\text{lap}} = 300 \) mm

Length of pile cap \( l_x = 2600 \) mm

Width of pile cap \( l_y = 1100 \) mm

Concrete grade \( f_{\text{cu}} = 35 \) N/mm²

Steel strength \( f_y = 500 \) N/mm²

Cover to underside of main bars \( \text{Cover} = 150 \) mm

Side and top cover to main bars \( \text{SCover} = 75 \) mm

Chosen depth of pilecap \( h = 1660 \) mm

Design of main reinforcement

Main steel diameter \( \text{dia} = 25 \) mm

Main steel spacing \( \text{Space} = 80 \) mm

Distribution steel

Distribution steel diameter \( L_{\text{Size}} = 20 \) mm

Spacing of bars \( L_{\text{Spac}} = 145 \) mm

MAIN REINFORCEMENT SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>12</td>
</tr>
<tr>
<td>Spacing</td>
<td>80 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>5788.6 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>5890.5 mm²</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>25388 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>199.3 kg</td>
</tr>
<tr>
<td>DISTRIBUTION REINFORCEMENT SUMMARY</td>
<td>Characteristic strength</td>
</tr>
<tr>
<td>------------------------------------</td>
<td>-------------------------</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
</tr>
<tr>
<td></td>
<td>Area required</td>
</tr>
<tr>
<td></td>
<td>Area provided</td>
</tr>
<tr>
<td></td>
<td>Volume of steel</td>
</tr>
<tr>
<td></td>
<td>Weight of steel</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TOP BARS SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
<td>145 mm</td>
</tr>
<tr>
<td></td>
<td>Volume of steel</td>
<td>5387.8 cm³</td>
</tr>
<tr>
<td></td>
<td>Weight of steel</td>
<td>42.294 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SIDE BARS SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td></td>
<td>Number of bars/side</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
<tr>
<td></td>
<td>Volume of steel</td>
<td>3325.1 cm³</td>
</tr>
<tr>
<td></td>
<td>Weight of steel</td>
<td>26.102 kg</td>
</tr>
</tbody>
</table>

| QUANTITIES | Total volume of concrete needed | 4.7476 m³ |
|           | Total area of formwork       | 12.284 m² |
|           | Total volume of steel        | 63635 cm³ |
|           | Total weight of steel        | 499.53 kg |

|               | Approx. self-weight of pile cap | 159.52 kN |
|               | Approx. total load on each pile | 5080 kN   |
Location: Ex3 - Lightly-loaded column.

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Pilecap for two piles

Designed by Truss Analogy.

Calculations are in accordance with BS8110.

For bond calculations, any high-yield bars are assumed Type 2 deformed.

Total ultimate load on column $N=100 \text{kN}$
Column dimension $cx = 500 \text{ mm}$
Column dimension $cy = 500 \text{ mm}$
Nominal pile diameter $PileD=450 \text{ mm}$
Spacing of pile centres $A=1350 \text{ mm}$
Overhang: pile face to cap edge $olap=600 \text{ mm}$
Length of pile cap $lx=3000 \text{ mm}$
Width of pile cap $ly=1650 \text{ mm}$
Concrete grade $f_{cu}=35 \text{ N/mm}^2$
Steel strength $f_y=500 \text{ N/mm}^2$
Cover to underside of main bars $Cover=150 \text{ mm}$
Side and top cover to main bars $SCover=75 \text{ mm}$
Chosen depth of pilecap $h=300 \text{ mm}$

Design of main reinforcement

Main steel diameter $dia=12 \text{ mm}$
Main steel spacing $Space=275 \text{ mm}$

Distribution steel

Distribution steel diameter $LSize=12 \text{ mm}$
Spacing of bars $LSpac=285 \text{ mm}$

MAIN REINFORCEMENT SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Number of bars</th>
<th>Spacing</th>
<th>Area required</th>
<th>Area provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 $\text{N/mm}^2$</td>
<td>12 $\text{mm}$</td>
<td>6</td>
<td>275 $\text{mm}$</td>
<td>643.5 $\text{mm}^2$</td>
<td>678.58 $\text{mm}^2$</td>
</tr>
</tbody>
</table>

Volume of steel $1628.6 \text{ cm}^3$
Weight of steel $12.785 \text{ kg}$
### DISTRIBUTION REINFORCEMENT SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>11</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>285 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>390 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>396.83 mm²/m</td>
</tr>
</tbody>
</table>

- **Volume of steel:** 4486.1 cm³
- **Weight of steel:** 35.216 kg

### TOP BARS SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>6</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>285 mm</td>
</tr>
</tbody>
</table>

- **Volume of steel:** 1934 cm³
- **Weight of steel:** 15.182 kg

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Number of bars / side</td>
<td>1</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

- **Volume of steel:** 644.65 cm³
- **Weight of steel:** 5.0605 kg

### QUANTITIES

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total volume of concrete needed</td>
<td>1.485 m³</td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>2.79 m²</td>
</tr>
<tr>
<td>Total volume of steel</td>
<td>8693.3 cm³</td>
</tr>
<tr>
<td>Total weight of steel</td>
<td>68.243 kg</td>
</tr>
<tr>
<td>Approx. self-weight of pile cap</td>
<td>49.896 kN</td>
</tr>
<tr>
<td>Approx. total load on each pile</td>
<td>75 kN</td>
</tr>
</tbody>
</table>
Location: Ex1 - Typical pilecap

Total design load on column $N=1500$ kN
Column dimension $cx=500$ mm
Column dimension $cy=500$ mm
Nominal pile diameter $PileD=450$ mm
Partial safety factor for steel $gams=1.15$
Partial safety factor for conc. $gamc=1.5$
Spacing of pile centres $A=1350$ mm
Overhang: pile face to cap edge $olap=150$ mm
Length of pile cap $lx=2100$ mm
Width of pile cap $ly=750$ mm
Char yield strength of reinft. $fyk=500$ N/mm$^2$
Cover to underside of main bars $Cover=150$ mm
Side and top cover to main bars $SCover=75$ mm
Chosen depth of pilecap $h=1000$ mm

Design of main reinforcement

Main steel diameter $dia=16$ mm
Main steel spacing $Space=90$ mm
Distribution steel diameter $LSize=12$ mm
Spacing of bars $LSpac=75$ mm

Check shear resistance

Shear force due to ultimate load $VEd=N/2=750$ kN
Shear resistance $VRdc=\beta'CRdc*ks*Vdc*bw*d/1000 =830.04$ kN
Modified shear resistance $VRdc=VRdm=848.61$ kN

SHEAR SUMMARY

Shear force $750$ kN
Shear resistance $848.61$ kN

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face

Check punching shear at column face:
Eccentricity factor $\beta$ $=1$
Shear stress at column perimeter $vEd=\beta*VEd*1000/(uo*d)=0.89074$ N/mm$^2$
Maximum punching shear resistance $vRdmax=0.5*v*fcd=4.9728$ N/mm$^2$
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.
Check strength of struts and nodes - Clauses 6.5.2(2) and 6.5.4(4)

Strength of concrete strut:
- Strut compressive stress: $S_{str} = C_{str} \times 1000 / (c_y \times W_{str}) = 4.928 \text{ N/mm}^2$
- Design strength of conc. strut: $S_{Rdmax} = 0.6 \times (1 - f_{ck} / 250) \times f_{cd} = 8.4538 \text{ N/mm}^2$

The strut compressive stress is ≤ the strut design strength, hence OK.

Strength of upper strut node:
- Stress on horizontal plane: $S_{Ed1} = 0.5 \times N \times 1000 / (c_x / 2 \times c_y) = 6 \text{ N/mm}^2$
- Stress on vertical plane: $S_{Ed2} = C_{str} \times 1000 \times A / (2 \times L_{str} \times \text{comp}) = 3.856 \text{ N/mm}^2$

Strength of compression node: $S_{Rdmax} = (1 - f_{ck} / 250) \times f_{cd} = 14.09 \text{ N/mm}^2$
The stress in horizontal and vertical planes is satisfactory.

Strength of lower strut node:
- Stress in strut: $S_{Ed1} = S_{str} = 4.928 \text{ N/mm}^2$
- Stress in pile: $S_{Ed2} = 0.5 \times N \times 1000 / (P_{ileD}^2 \times \pi / 4) = 4.7157 \text{ N/mm}^2$

Strength of compression node: $S_{Rdmax} = 0.85 \times (1 - f_{ck} / 250) \times f_{cd} = 11.976 \text{ N/mm}^2$
The stress in strut and pile is satisfactory.

### MAIN REINFORCEMENT

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Number of bars</th>
<th>Spacing</th>
<th>Area required</th>
<th>Area provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 N/mm²</td>
<td>16 mm</td>
<td>7</td>
<td>90 mm</td>
<td>1402.9 mm²</td>
<td>1407.4 mm²</td>
</tr>
</tbody>
</table>

### DISTRIBUTION REINFORCEMENT

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Number of bars</th>
<th>Spacing of bars</th>
<th>Area required</th>
<th>Area provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 N/mm²</td>
<td>12 mm</td>
<td>27</td>
<td>75 mm</td>
<td>1500 mm²/m</td>
<td>1508 mm²/m</td>
</tr>
</tbody>
</table>

### TOP BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Number of bars</th>
<th>Spacing of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 N/mm²</td>
<td>12 mm</td>
<td>9</td>
<td>75 mm</td>
</tr>
</tbody>
</table>

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Number of bars</th>
<th>Spacing of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 N/mm²</td>
<td>12 mm</td>
<td>4</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

- Total concrete volume: 1.575 m³
- Total area of formwork: 5.7 m²
Location: Ex2 - Heavily-loaded column

Total design load on column \( N = 2500 \text{ kN} \)
Column dimension \( cx = 500 \text{ mm} \)
Column dimension \( cy = 500 \text{ mm} \)
Nominal pile diameter \( \text{PileD} = 500 \text{ mm} \)
Partial safety factor for steel \( g_{ams} = 1.15 \)
Partial safety factor for conc. \( g_{amc} = 1.5 \)
Spacing of pile centres \( A = 1500 \text{ mm} \)
Overhang: pile face to cap edge \( o_{lap} = 300 \text{ mm} \)
Length of pile cap \( lx = 2600 \text{ mm} \)
Width of pile cap \( ly = 1100 \text{ mm} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Cover to underside of main bars \( \text{Cover} = 150 \text{ mm} \)
Side and top cover to main bars \( \text{SCover} = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 1200 \text{ mm} \)

Design of main reinforcement

Main steel diameter \( \text{dia} = 12 \text{ mm} \)
Main steel spacing \( \text{Space} = 50 \text{ mm} \)
Distribution steel diameter \( \text{LSize} = 25 \text{ mm} \)
Spacing of bars \( \text{LSpac} = 270 \text{ mm} \)

Check shear resistance

Shear force due to ultimate load \( V_{Ed} = N/2 = 1250 \text{ kN} \)
Shear resistance \( V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b \times d / 1000 \)
\( = 1439.1 \text{ kN} \)
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 1567.8 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>1250 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>1567.8 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face:

Check punching shear at column face:
Eccentricity factor \( \beta \) \( = 1 \)
Shear stress at column perimeter \( v_{Ed} = \beta \times V_{Ed} \times 1000 / (u_o \times d) = 1.1973 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5 \times v \times f_{cd} = 5.5808 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.
<table>
<thead>
<tr>
<th>Section</th>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Number of bars</th>
<th>Spacing</th>
<th>Area required</th>
<th>Area provided</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>MAIN REINFORCEMENT</strong></td>
<td>500 N/mm²</td>
<td>12 mm</td>
<td>19</td>
<td>50 mm</td>
<td>2093.4 mm²</td>
<td>2148.8 mm²</td>
</tr>
<tr>
<td><strong>DISTRIBUTION REINFORCEMENT</strong></td>
<td>500 N/mm²</td>
<td>25 mm</td>
<td>10</td>
<td>270 mm</td>
<td>1800 mm²/m</td>
<td>1818.1 mm²/m</td>
</tr>
<tr>
<td><strong>TOP BARS SUMMARY</strong></td>
<td>500 N/mm²</td>
<td>25 mm</td>
<td>4</td>
<td>270 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>SIDE BARS SUMMARY</strong></td>
<td>500 N/mm²</td>
<td>12 mm</td>
<td>4</td>
<td>250 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>QUANTITIES</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total concrete volume</td>
<td>3.432 m³</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>8.88 m²</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Location: Ex3 - Lightly-loaded column

Pilecap for two piles


Ultimate load and moment values to be used.

Total design load on column \( N = 100 \text{ kN} \)
Column dimension \( cx = 500 \text{ mm} \)
Column dimension \( cy = 500 \text{ mm} \)
Nominal pile diameter \( \text{PileD}=450 \text{ mm} \)
Partial safety factor for steel \( \gamma_m=1.15 \)
Partial safety factor for conc. \( \gamma_c=1.5 \)
Spacing of pile centres \( A=1350 \text{ mm} \)
Overhang: pile face to cap edge \( olap=600 \text{ mm} \)
Length of pile cap \( lx=3000 \text{ mm} \)
Width of pile cap \( ly=1650 \text{ mm} \)
Char yield strength of reinft. \( f_{yk}=500 \text{ N/mm}^2 \)
Cover to underside of main bars \( Cover=150 \text{ mm} \)
Side and top cover to main bars \( SCover=75 \text{ mm} \)
Chosen depth of pilecap \( h=300 \text{ mm} \)

Design of main reinforcement

Main steel diameter \( \text{dia}=12 \text{ mm} \)
Main steel spacing \( \text{Space}=225 \text{ mm} \)
Distribution steel diameter \( \text{LSize}=12 \text{ mm} \)
Spacing of bars \( \text{LSpac}=250 \text{ mm} \)

Check shear resistance

Shear force due to ultimate load \( V_{Ed}=N/2=50 \text{ kN} \)
Shear resistance \( V_{Rdc}=\beta'*CR_{dc}*ks*v_{Rdc}*bw*d/1000 =120.05 \text{ kN} \)
Modified shear resistance \( V_{Rdc}=V_{Rdm}=124.46 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>50 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>124.46 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face:

Check punching shear at column face:
Eccentricity factor \( \beta \) \( \text{beta}=1 \)
Shear stress at column perimeter \( v_{Ed}=\beta'VEd*1000/(uo*d)=0.34722 \text{ N/mm}^2 \)
Maximum punching shear resistance \( V_{Rdmax}=0.5*v*f_{cd}=4.9728 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.
### MAIN REINFORCEMENT SUMMARY
- **Characteristic strength**: 500 N/mm²
- **Diameter of bars**: 12 mm
- **Number of bars**: 7
- **Spacing**: 225 mm
- **Area required**: 742.5 mm²
- **Area provided**: 791.68 mm²

### DISTRIBUTION REINFORCEMENT SUMMARY
- **Characteristic strength**: 500 N/mm²
- **Diameter of bars**: 12 mm
- **Number of bars**: 12
- **Spacing of bars**: 250 mm
- **Area required**: 450 mm²/m
- **Area provided**: 452.39 mm²/m

### TOP BARS SUMMARY
- **Characteristic strength**: 500 N/mm²
- **Diameter of bars**: 12 mm
- **Number of bars**: 7
- **Spacing of bars**: 250 mm

### SIDE BARS SUMMARY
- **Characteristic strength**: 500 N/mm²
- **Diameter of bars**: 12 mm
- **Number of bars/side**: 1
- **Spacing of bars**: 250 mm

### QUANTITIES
- **Total concrete volume**: 1.485 m³
- **Total area of formwork**: 2.79 m²
Location: Ex4 - Typical pilecap

Pilecap for two piles


Ultimate load and moment values to be used.

Total design load on column $N = 1800$ kN
Column dimension $cx = 500$ mm
Column dimension $cy = 500$ mm
Nominal pile diameter $PileD = 450$ mm
Partial safety factor for steel $gams = 1.15$
Partial safety factor for conc. $gamc = 1.5$
Spacing of pile centres $A = 1350$ mm
Overhang: pile face to cap edge $olap = 150$ mm
Length of pile cap $lx = 2100$ mm
Width of pile cap $ly = 750$ mm
Char yield strength of reinft. $fyk = 500$ N/mm²
Cover to underside of main bars $Cover = 150$ mm
Side and top cover to main bars $SCover = 75$ mm
Chosen depth of pilecap $h = 1000$ mm

Design of main reinforcement

Main steel diameter $dia = 16$ mm
Main steel spacing $Space = 65$ mm
Distribution steel diameter $LSize = 12$ mm
Spacing of bars $LSpac = 75$ mm

Check shear resistance

Shear force due to ultimate load $VEd = N/2 = 900$ kN
Shear resistance $VRdc = \beta' \cdot CRdc \cdot k_s \cdot vRdc \cdot bw \cdot d / 1000$

$$= 943.67 \text{ kN}$$

Shear force $900$ kN
Shear resistance $943.67$ kN

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face

Check punching shear at column face:
Eccentricity factor $\beta$ $\beta = 1$
Shear stress at column perimeter $vEd = \beta \cdot VEd \cdot 1000 / (uo \cdot d) = 1.0689$ N/mm²
Maximum punching shear resistance $vRdmax = 0.5 \cdot v \cdot fcd = 5.5808$ N/mm²
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.
### MAIN REINFORCEMENT

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>9</td>
</tr>
<tr>
<td>Spacing</td>
<td>65 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>1683.4 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>1809.6 mm²</td>
</tr>
</tbody>
</table>

### DISTRIBUTION REINFORCEMENT

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>27</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>75 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>1500 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>1508 mm²/m</td>
</tr>
</tbody>
</table>

### TOP BARS SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>9</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>75 mm</td>
</tr>
</tbody>
</table>

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Number of bars/side</td>
<td>4</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total concrete volume</td>
<td>1.575 m³</td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>5.7 m²</td>
</tr>
</tbody>
</table>
Location: Ex1 - Typical pilecap

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for four piles

Designed by Truss Analogy.
Calculations are in accordance with BS8110.

Pile cap assumed square (i.e. $lx=ly$) with main reinforcement arranged in bands parallel to principal axes and symmetrical about pile centres (i.e. chain lines in adjoining diagram).
For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column $N=5600$ kN
Column dimension $cx$ $cx=500$ mm
Column dimension $cy$ $cy=500$ mm
Nominal pile diameter $PileD=450$ mm
Spc.of pile ctrs in X-direction $AX=1350$ mm
Overhang: pile face to cap edge $olap=150$ mm
Length of pile cap $lx=2100$ mm
Spc.of pile ctrs in Y-direction $AY=1350$ mm
Breadth of pile cap $ly=2100$ mm
Concrete grade $fcu=35$ N/mm$^2$
Steel strength $fy=500$ N/mm$^2$
Cover to underside of main bars $Cover=150$ mm
Side and top cover to main bars $Scvr=75$ mm
Chosen depth of pilecap $h=820$ mm

Design of main steel in X-direction

Main steel diameter $diaX=16$ mm
Main steel spacing $SpcX=55$ mm

Design of main steel in Y-direction

Main steel diameter $diaY=16$ mm
Main steel spacing $SpcY=55$ mm

Distribution steel

Diameter of bar $LSize=12$ mm
Spacing of bars $LSpac=105$ mm
**MAIN REINFORCEMENT**  
**IN X-DIRECTION:**  
**SUMMARY**  
<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>55 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>36</td>
</tr>
<tr>
<td>Total area required</td>
<td>6687.7 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>7238.2 mm²</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>18986 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>149.04 kg</td>
</tr>
</tbody>
</table>

**MAIN REINFORCEMENT**  
**IN Y-DIRECTION:**  
**SUMMARY**  
<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>55 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>36</td>
</tr>
<tr>
<td>Total area required</td>
<td>6856.5 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>7238.2 mm²</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>18950 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>148.76 kg</td>
</tr>
</tbody>
</table>

**DISTRIBUTION REINFORCEMENT**  
**SUMMARY**  
<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
</tbody>
</table>

**IN X-DIRECTION:**  
In top:  
| Number of bars | 19 |
| Spacing of bars | 105 mm |
| Area required   | 1066 mm²/m |
| Area provided   | 1077.1 mm²/m |
| Volume of steel | 4190.3 cm³ |
| Weight of steel | 32.894 kg |

In bottom:  
No distribution bars required

**IN Y-DIRECTION:**  
In top:  
| Number of bars | 19 |
| Spacing of bars | 105 mm |
| Area required   | 1066 mm²/m |
| Area provided   | 1077.1 mm²/m |
| Volume of steel | 4190.3 cm³ |
| Weight of steel | 32.894 kg |

In bottom:  
No distribution bars required

**SIDE BARS SUMMARY**  
<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>6</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>2995.9 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>23.518 kg</td>
</tr>
</tbody>
</table>

**QUANTITIES**  
| Total volume of concrete needed | 3.6162 m³ |
| Total area of formwork         | 6.888 m²  |
| Total volume of steel         | 49312 cm³ |
| Total weight of steel         | 387.1 kg  |
| Approx. self-weight of pile cap | 121.5 kN  |
| Approx. total load on each pile | 1431 kN  |
Location: Ex2 - Large, heavily-loaded pilecap w. mild steel reinf.

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for four piles

Designed by Truss Analogy.
Calculations are in accordance with BS8110.

Pile cap assumed square (i.e. $lx=ly$) with main reinforcement arranged in bands parallel to principal axes and symmetrical about pile centres (i.e. chain lines in adjoining diagram).
For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column $N=10000$ kN
Column dimension $cx=cy=500$ mm
Nominal pile diameter $D=450$ mm
Spec. of pile ctrs in X-direction $AX=1350$ mm
Overhang: pile face to cap edge $olap=725$ mm
Length of pile cap $lx=3250$ mm
Spec. of pile ctrs in Y-direction $AY=1350$ mm
Breadth of pile cap $ly=3250$ mm
Concrete grade $fcu=25$ N/mm$^2$
Steel strength $fy=250$ N/mm$^2$
Cover to underside of main bars $Cover=150$ mm
Side and top cover to main bars $Scvr=75$ mm
Chosen depth of pilecap $h=1440$ mm

Design of main steel in X-direction

Main steel diameter $diaX=16$ mm
Main steel spacing $SpcX=50$ mm

Design of main steel in Y-direction

Main steel diameter $diaY=16$ mm
Main steel spacing $SpcY=50$ mm

Distribution steel

Diameter of bar $LSize=20$ mm
Spacing of bars $LSpac=90$ mm
### MAIN REINFORCEMENT

| IN X-DIRECTION: | Diameter of bars | 16 mm |
| Spacing          | 50 mm            |
| Total number of bars | 62               |
| Total area required | 12224 mm²        |
| Total area provided | 12466 mm²        |
| Volume of steel  | 49564 cm³        |
| Weight of steel  | 389.08 kg        |

### MAIN REINFORCEMENT

| IN Y-DIRECTION: | Diameter of bars | 16 mm |
| Spacing          | 50 mm            |
| Total number of bars | 62               |
| Total area required | 12380 mm²        |
| Total area provided | 12466 mm²        |
| Volume of steel  | 49564 cm³        |
| Weight of steel  | 389.08 kg        |

### DISTRIBUTION REINFORCEMENT

| SUMMARY | Characteristic strength | 250 N/mm² |
| Diameter of bars | 20 mm |

| IN X-DIRECTION: |
| Number of bars | 35 |
| Spacing of bars | 90 mm |
| Area required | 3456 mm²/m |
| Area provided | 3490.7 mm²/m |
| Volume of steel | 34086 cm³ |
| Weight of steel | 267.58 kg |

| In bottom: | No distribution bars required |

| IN Y-DIRECTION: |
| Number of bars | 35 |
| Spacing of bars | 90 mm |
| Area required | 3456 mm²/m |
| Area provided | 3490.7 mm²/m |
| Volume of steel | 34086 cm³ |
| Weight of steel | 267.58 kg |

| In bottom: | No distribution bars required |

### SIDE BARS SUMMARY

| Characteristic strength | 250 N/mm² |
| Diameter of bars | 12 mm |
| Total number of bars | 10    |
| Spacing of bars | 250 mm |
| Volume of steel | 7617.1 cm³ |
| Weight of steel | 59.794 kg |

### QUANTITIES

| Total volume of concrete needed | 15.21 m³ |
| Total area of formwork | 18.72 m² |
| Total volume of steel | 174918 cm³ |
| Total weight of steel | 1373.1 kg |

| Approx. self-weight of pile cap | 511.06 kN|
| Approx. total load on each pile | 2628 kN |
Location: Ex3 - Small cap with closely-spaced piles

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for four piles

Designed by Truss Analogy. Calculations are in accordance with BS8110.

Pile cap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to principal axes and symmetrical about pile centres (i.e. chain lines in adjoining diagram). For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column \( N = 5600 \) kN
Column dimension \( cx = 350 \) mm
Column dimension \( cy = 350 \) mm
Nominal pile diameter \( PileD = 250 \) mm
Spc.of pile ctrs in X-direction \( AX = 750 \) mm
Overhang: pile face to cap edge \( olap = 150 \) mm
Length of pile cap \( lx = 1300 \) mm
Spc.of pile ctrs in Y-direction \( AY = 750 \) mm
Breadth of pile cap \( ly = 1300 \) mm
Concrete grade \( fcu = 60 \) N/mm²
Steel strength \( fy = 500 \) N/mm²
Cover to underside of main bars \( Cover = 150 \) mm
Side and top cover to main bars \( Scvr = 75 \) mm
Chosen depth of pilecap \( h = 990 \) mm

Design of main steel in X-direction

Main steel diameter \( diaX = 25 \) mm
Main steel spacing \( SpcX = 275 \) mm

Design of main steel in Y-direction

Main steel diameter \( diaY = 20 \) mm
Main steel spacing \( SpcY = 135 \) mm

Distribution steel

Diameter of bar \( LSize = 16 \) mm
Spacing of bars \( LSpac = 155 \) mm
<table>
<thead>
<tr>
<th><strong>MAIN REINFORCEMENT</strong></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN X-DIRECTION:</td>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>275 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>2945.1 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>2945.2 mm²</td>
</tr>
<tr>
<td></td>
<td>Volume of steel</td>
<td>6538.4 cm³</td>
</tr>
<tr>
<td></td>
<td>Weight of steel</td>
<td>51.327 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>MAIN REINFORCEMENT</strong></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN Y-DIRECTION:</td>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>135 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>3037.7 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>3141.6 mm²</td>
</tr>
<tr>
<td></td>
<td>Volume of steel</td>
<td>7052.9 cm³</td>
</tr>
<tr>
<td></td>
<td>Weight of steel</td>
<td>55.365 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>DISTRIBUTION REINFORCEMENT</strong></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>IN X-DIRECTION:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>In top:</td>
<td>Number of bars</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
<td>155 mm</td>
</tr>
<tr>
<td></td>
<td>Area required</td>
<td>1287 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Area provided</td>
<td>1297.2 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Volume of steel</td>
<td>1849.8 cm³</td>
</tr>
<tr>
<td></td>
<td>Weight of steel</td>
<td>14.521 kg</td>
</tr>
<tr>
<td>In bottom:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>In top:</td>
<td>Number of bars</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
<td>155 mm</td>
</tr>
<tr>
<td></td>
<td>Area required</td>
<td>1287 mm²/m</td>
</tr>
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<td>Area provided</td>
<td>1297.2 mm²/m</td>
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<td></td>
<td>Weight of steel</td>
<td>14.521 kg</td>
</tr>
<tr>
<td>In bottom:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SIDE BARS SUMMARY</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Total number of bars</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
<td></td>
</tr>
<tr>
<td>Volume of steel</td>
<td>2474.6 cm³</td>
<td></td>
</tr>
<tr>
<td>Weight of steel</td>
<td>19.425 kg</td>
<td></td>
</tr>
</tbody>
</table>

| **QUANTITIES**              |                          |           |
| Total volume of concrete     | needed                   | 1.6731 m³ |
| Total area of formwork       |                          | 5.148 m²  |
| Total volume of steel        |                          | 19765 cm³ |
| Total weight of steel        |                          | 155.16 kg |
| Approx. self-weight of pile  | cap                      | 56.216 kN |
| Approx. total load on each   | pile                      | 1415 kN   |

---

*SCALE 5.48  Office 1007  Proforma 734*
Location: Ex1 - Typical pilecap

Bottom layer of main steel is assumed to run in the X-direction.

Pilecap for four piles

Designed by truss-analogy to EC2 Part 1-1.

Pile cap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres.

Ultimate load & moment values to be used.

Total design load on column N=2800 kN
Column dimension cx cx=500 mm
Column dimension cy cy=500 mm
Nominal pile diameter PileD=450 mm
Spc.of pile ctrs in X-direction AX=1350 mm
Overhang: pile face to cap edge olap=150 mm
Length of pile cap lx=2100 mm
Spc.of pile ctrs in Y-direction AY=1350 mm
Breadth of pile cap ly=2100 mm
Char yield strength of reinft. fyk=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=1000 mm

Design of main steel in X-direction

Main steel diameter diaX=12 mm
Main steel spacing SpcX=70 mm

Design of main steel in Y-direction

Main steel diameter diaY=12 mm
Main steel spacing SpcY=70 mm

Distribution steel

Distribution steel diameter LSize=12 mm
Spacing of bars LSpac=75 mm

Check shear resistance in X-direction

Shear force due to ultimate load VEd=N/2=1400 kN
Shear resistance:  
\[ VR_{dc} = \beta'*CR_{dc}*ks*v_{Rdc}*bw*d/1000 = 2261.8 \, kN \]

Modified shear resistance:  
\[ VR_{dc} = VR_{dm} = 2544.7 \, kN \]

**SHEAR SUMMARY**  
- **Shear force**: 1400 kN  
- **Shear resistance**: 2544.7 kN

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction:

Shear force due to ultimate load:  
\[ V_{Ed} = N/2 = 1400 \, kN \]

Shear resistance:  
\[ VR_{dc} = \beta'*CR_{dc}*ks*v_{Rdc}*bw*d/1000 = 2245.6 \, kN \]

Modified shear resistance:  
\[ VR_{dc} = VR_{dm} = 2517.4 \, kN \]

**SHEAR SUMMARY**  
- **Shear force**: 1400 kN  
- **Shear resistance**: 2517.4 kN

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear:

Check punching shear at column face:  
- Eccentricity factor $\beta = 1$  
- Shear stress at column perimeter:  
\[ v_{Ed} = \beta*V_{Ed}*1000/(u_0*d) = 1.6827 \, N/mm^2 \]

Maximum punching shear resistance:  
\[ v_{Rd_{max}} = 0.5*v*f_{cd} = 5.5808 \, N/mm^2 \]

Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

Check strength of struts and nodes - Clauses 6.5.2(2) and 6.5.4(4):

Case 1: X-Direction

Strength of concrete strut:  
- Strut compressive stress:  
\[ S_{str} = C_{str}*1000/(c_y*W_{str}) = 4.5907 \, N/mm^2 \]
- Design strength of conc. strut:  
\[ S_{Rd_{max}} = 0.6*(1-f_{ck}/250)*f_{cd} = 9.4874 \, N/mm^2 \]

The strut compressive stress is $\leq$ the strut design strength, hence OK.

Strength of upper strut node:  
- Stress on horizontal plane:  
\[ S_{Ed1} = 0.25*N*1000/(c_x*2*c_y) = 5.6 \, N/mm^2 \]
- Stress on vertical plane:  
\[ S_{Ed2} = C_{str}*1000*A_{X}/(2*L_{str}*comp) = 3.5819 \, N/mm^2 \]

Strength of compression node:  
\[ S_{Rd_{max}} = (1-f_{ck}/250)*f_{cd} = 15.812 \, N/mm^2 \]

The stress in horizontal and vertical planes is satisfactory.

Strength of lower strut node:  
- Stress in strut:  
\[ S_{Ed1} = S_{str} = 4.5907 \, N/mm^2 \]
- Stress in pile:  
\[ S_{Ed2} = 0.25*N*1000/((PileD)^2*\pi/4) = 4.4013 \, N/mm^2 \]
- Strength of compression node:  
\[ S_{Rd_{max}} = 0.85*(1-f_{ck}/250)*f_{cd} = 13.44 \, N/mm^2 \]

The stress in strut and pile is satisfactory.
### MAIN REINFORCEMENT

**IN X-DIRECTION:**
- **Summary**
  - Characteristic strength: 500 N/mm²
  - Diameter of bars: 12 mm
  - Spacing: 70 mm
  - Total number of bars: 28
  - Total area required: 3150 mm²
  - Total area provided: 3166.7 mm²

**IN Y-DIRECTION:**
- **Summary**
  - Characteristic strength: 500 N/mm²
  - Diameter of bars: 12 mm
  - Spacing: 70 mm
  - Total number of bars: 28
  - Total area required: 3150 mm²
  - Total area provided: 3166.7 mm²

### DISTRIBUTION REINFORCEMENT

**Summary**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm

**IN X-DIRECTION:**
- **In top:**
  - Number of bars: 26
  - Spacing of bars: 75 mm
  - Area required: 1500 mm²/m
  - Area provided: 1508 mm²/m
- **In bottom:**
  - No distribution bars required

**IN Y-DIRECTION:**
- **In top:**
  - Number of bars: 26
  - Spacing of bars: 75 mm
  - Area required: 1500 mm²/m
  - Area provided: 1508 mm²/m
- **In bottom:**
  - No distribution bars required

### SIDE BARS SUMMARY
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 8
- Spacing of bars: 250 mm

### QUANTITIES
- Total concrete volume: 4.41 m³
- Total area of formwork: 8.4 m²
Location: Ex2 - Large, heavily-loaded pilecap w. mild steel reinf.

Bottom layer of main steel is assumed to run in the X-direction.

**Pilecap for four piles**

Designed by truss-analogy to EC2 Part 1-1.

Pile cap assumed square (i.e. \( lx=ly \)) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres.

Ultimate load & moment values to be used.

Total design load on column \( N=9000 \text{ kN} \)
Column dimension \( cx=500 \text{ mm} \)
Column dimension \( cy=500 \text{ mm} \)
Nominal pile diameter \( \text{PileD}=450 \text{ mm} \)
Spc.of pile ctrs in X-direction \( AX=1350 \text{ mm} \)
Overhang: pile face to cap edge \( \text{olap}=725 \text{ mm} \)
Length of pile cap \( 1x=3250 \text{ mm} \)
Spc.of pile ctrs in Y-direction \( AY=1350 \text{ mm} \)
Breadth of pile cap \( 1y=3250 \text{ mm} \)
Char yield strength of reinft. \( f_{yk}=500 \text{ N/mm}^2 \)
Cover to underside of main bars \( \text{Cover}=150 \text{ mm} \)
Side and top cover to main bars \( \text{Scvr}=75 \text{ mm} \)
Chosen depth of pilecap \( h=1460 \text{ mm} \)

**Design of main steel in X-direction**

Main steel diameter \( \text{diaX}=16 \text{ mm} \)
Main steel spacing \( \text{SpcX}=90 \text{ mm} \)

**Design of main steel in Y-direction**

Main steel diameter \( \text{diaY}=16 \text{ mm} \)
Main steel spacing \( \text{SpcY}=90 \text{ mm} \)

**Distribution steel**

Distribution steel diameter \( \text{LSize}=25 \text{ mm} \)
Spacing of bars \( \text{LSpac}=220 \text{ mm} \)

**Check shear resistance in X-direction**

Shear force due to ultimate load \( V_{Ed}=N/2=4500 \text{ kN} \)
Shear resistance \[ VR_{dc} = \beta' \cdot CR_{dc} \cdot k_s \cdot v_{Rdc} \cdot b_w \cdot d/1000 \]

= 4588.9 kN

Modified shear resistance \[ VR_{dc} = VR_{dm} = 4864.3 \text{ kN} \]

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>4500 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>4864.3 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \[ V_{Ed} = N/2 = 4500 \text{ kN} \]

Shear resistance \[ VR_{dc} = \beta' \cdot CR_{dc} \cdot k_s \cdot v_{Rdc} \cdot b_w \cdot d/1000 \]

= 4559.2 kN

Modified shear resistance \[ VR_{dc} = VR_{dm} = 4817.1 \text{ kN} \]

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>4500 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>4817.1 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:

Eccentricity factor \( \beta \) \( \beta = 1 \)

Shear stress at column perimeter \[ v_{Ed} = \beta \cdot V_{Ed} \cdot 1000/(u_0 \cdot d) = 3.4992 \text{ N/mm}^2 \]

Maximum punching shear resistance \[ v_{Rdmax} = 0.5 \cdot v \cdot f_{cd} = 4.5 \text{ N/mm}^2 \]

Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>90 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>36</td>
</tr>
<tr>
<td>Total area required</td>
<td>7117.5 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>7238.2 mm²</td>
</tr>
</tbody>
</table>

**IN Y-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
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<td>Total area required</td>
<td>7117.5 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>7238.2 mm²</td>
</tr>
</tbody>
</table>

**DISTRIBUTION REINFORCEMENT**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
</tbody>
</table>

**IN X-DIRECTION:**

In top:

| Number of bars | 14 |
| Spacing of bars | 220 mm |
| Area required | 2190 mm²/m |
| Area provided | 2231.2 mm²/m |

In bottom:

No distribution bars required
### IN Y-DIRECTION:

<table>
<thead>
<tr>
<th>In top:</th>
<th>Number of bars</th>
<th>14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing of bars</td>
<td>220 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>2190 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>2231.2 mm²/m</td>
<td></td>
</tr>
</tbody>
</table>

| In bottom:       | No distribution bars required |             |

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>10</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

<table>
<thead>
<tr>
<th>Total concrete volume</th>
<th>15.421 m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total area of formwork</td>
<td>18.98 m²</td>
</tr>
</tbody>
</table>
Location: Ex3 - Small cap with closely-spaced piles

Bottom layer of main steel is assumed to run in the X-direction.

Total design load on column \( N = 3600 \) kN
Column dimension \( cx \) \( cx = 350 \) mm
Column dimension \( cy \) \( cy = 350 \) mm
Nominal pile diameter \( \text{PileD} = 250 \) mm
Spc.of pile ctrs in X-direction \( AX = 750 \) mm
Overhang: pile face to cap edge \( olap = 150 \) mm
Length of pile cap \( 1x = 1300 \) mm
Spc.of pile ctrs in Y-direction \( AY = 750 \) mm
Breadth of pile cap \( 1y = 1300 \) mm
Char yield strength of reinft. \( f_{yk} = 500 \) N/mm\(^2\)
Cover to underside of main bars \( \text{Cover} = 150 \) mm
Side and top cover to main bars \( \text{Scvr} = 75 \) mm
Chosen depth of pilecap \( h = 990 \) mm

Design of main steel in X-direction
- Main steel diameter \( \text{diaX} = 12 \) mm
- Main steel spacing \( \text{SpcX} = 60 \) mm

Design of main steel in Y-direction
- Main steel diameter \( \text{diaY} = 12 \) mm
- Main steel spacing \( \text{SpcY} = 60 \) mm

Distribution steel
- Distribution steel diameter \( \text{LSize} = 20 \) mm
- Spacing of bars \( \text{LSpac} = 180 \) mm

Check shear resistance in X-direction
Shear force due to ultimate load \( V_{Ed} = N/2 = 1800 \) kN
Shear resistance
VRdc=beta'*CRdc*ks*vRdc*bw*d/1000
=1693.9 kN

Modified shear resistance
VRdc=VRdm=1951.5 kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>1800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>1951.5 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

**Check shear resistance in Y-direction**

Shear force due to ultimate load
VEd=N/2=1800 kN

Shear resistance
VRdc=beta'*CRdc*ks*vRdc*bw*d/1000
=1681.7 kN

Modified shear resistance
VRdc=VRdm=1930.3 kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>1800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>1930.3 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

**Check punching shear**

Check punching shear at column face:

Eccentricity factor ß
beta=1

Shear stress at column perimeter
vEd=beta*VEd*1000/(uo*d)=3.1283 N/mm²

Maximum punching shear resistance
vRdmax=0.5*v*fcd=8 N/mm²

Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

**SUMMARY**

<table>
<thead>
<tr>
<th>Diameter of bars</th>
<th>12 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing</td>
<td>60 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>20</td>
</tr>
<tr>
<td>Total area required</td>
<td>2257 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>2261.9 mm²</td>
</tr>
</tbody>
</table>

**IN Y-DIRECTION:**

**SUMMARY**

<table>
<thead>
<tr>
<th>Diameter of bars</th>
<th>12 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing</td>
<td>60 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>20</td>
</tr>
<tr>
<td>Total area required</td>
<td>2224 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>2261.9 mm²</td>
</tr>
</tbody>
</table>

**DISTRIBUTION REINFORCEMENT**

**SUMMARY**

| Diameter of bars | 20 mm |

**IN X-DIRECTION:**

In top:

<table>
<thead>
<tr>
<th>Number of bars</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing of bars</td>
<td>180 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>1736.1 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>1745.3 mm²/m</td>
</tr>
</tbody>
</table>

In bottom:

No distribution bars required
## IN Y-DIRECTION:

<table>
<thead>
<tr>
<th>In top:</th>
<th>Number of bars</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Spacing of bars</td>
<td>180 mm</td>
</tr>
<tr>
<td></td>
<td>Area required</td>
<td>1736.1 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Area provided</td>
<td>1745.3 mm²/m</td>
</tr>
</tbody>
</table>

| In bottom: | No distribution bars required |

## SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>8</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

## QUANTITIES

<table>
<thead>
<tr>
<th>Total concrete volume</th>
<th>1.6731 m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total area of formwork</td>
<td>5.148 m²</td>
</tr>
</tbody>
</table>
Location: Ex1 - Typical pilecap

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15. Bottom layer of main steel is assumed to run in X-direction.

Pilecap for five piles

- Designed by Truss Analogy
- Calculations are in accordance with BS8110.
- Pile cap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to principal axes and symmetrical about pile centres (i.e. chain lines in diagram).
- For bond calculations, any high-yield bars are assumed Type 2:deformed.

Total ultimate load on column N=7000 kN
Column dimension cx cx=500 mm
Column dimension cy cy=500 mm
Nominal pile diameter PileD=450 mm
Spec.min.spacing of pile centres spc=1350 mm
Spec.of pile ctrs in X-direction AX=1910 mm
Overhang: pile face to cap edge olap=150 mm
Length of pile cap lx=2660 mm
Spec.of pile ctrs in Y-direction AY=1910 mm
Breadth of pile cap ly=2660 mm
Concrete grade fcu=35 N/mm²
Steel strength fy=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=1340 mm

Design of main steel in X-direction
- Main steel diameter diaX=16 mm
- Main steel spacing SpcX=70 mm

Design of main steel in Y-direction
- Main steel diameter diaY=16 mm
- Main steel spacing SpcY=70 mm

Distribution steel
- Diameter of bar LSize=20 mm
- Spacing of bars LSpac=180 mm
## MAIN REINFORCEMENT

### IN X-DIRECTION:

**SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm
- Spacing: 70 mm
- Total number of bars: 28
- Total area required: 5256.6 mm²
- Total area provided: 5629.7 mm²
- Volume of steel: 23774 cm³
- Weight of steel: 186.63 kg

### IN Y-DIRECTION:

**SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm
- Spacing: 70 mm
- Total number of bars: 28
- Total area required: 5329.5 mm²
- Total area provided: 5629.7 mm²
- Volume of steel: 23746 cm³
- Weight of steel: 186.41 kg

## DISTRIBUTION REINFORCEMENT

**SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 20 mm

### IN X-DIRECTION:

- In top:
  - Number of bars: 14
  - Spacing of bars: 180 mm
  - Area required: 1742 mm²/m
  - Area provided: 1745.3 mm²/m
  - Volume of steel: 11040 cm³
  - Weight of steel: 86.661 kg

- In bottom:
  - Number of bars: 3
  - Spacing of bars: 160 mm
  - Volume of steel: 2365.6 cm³
  - Weight of steel: 18.57 kg

### IN Y-DIRECTION:

- In top:
  - Number of bars: 14
  - Spacing of bars: 180 mm
  - Area required: 1742 mm²/m
  - Area provided: 1745.3 mm²/m
  - Volume of steel: 11040 cm³
  - Weight of steel: 86.661 kg

- In bottom:
  - Number of bars: 3
  - Spacing of bars: 160 mm
  - Volume of steel: 2365.6 cm³
  - Weight of steel: 18.57 kg

## SIDE BARS SUMMARY

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 10
- Spacing of bars: 250 mm
- Volume of steel: 6259.9 cm³
- Weight of steel: 49.141 kg
<table>
<thead>
<tr>
<th>QUANTITIES</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total volume of concrete needed</td>
<td>9.4813 m³</td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>14.258 m²</td>
</tr>
<tr>
<td>Total volume of steel</td>
<td>80591 cm³</td>
</tr>
<tr>
<td>Total weight of steel</td>
<td>632.64 kg</td>
</tr>
<tr>
<td>Approx. self-weight of pile cap</td>
<td>318.57 kN</td>
</tr>
<tr>
<td>Approx. total load on each pile</td>
<td>1464 kN</td>
</tr>
</tbody>
</table>
Location: Ex2 - Heavily-loaded pilecap reinforced with mild steel

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for five piles

- Designed by Truss Analogy
- Calculations are in accordance with BS8110.
- Pile cap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to principal axes and symmetrical about pile centres (i.e. chain lines in diagram).
- For bond calculations, any high-yield bars are assumed Type 2:deformed.

Total ultimate load on column \( N = 12500 \) kN
Column dimension \( cx = 500 \) mm
Column dimension \( cy = 500 \) mm
Nominal pile diameter \( \text{PileD} = 500 \) mm
Spec.min.spacing of pile centres \( \text{spc} = 1350 \) mm
Spec.of pile ctrs in X-direction \( AX = 1910 \) mm
Overhang: pile face to cap edge \( \text{olap} = 700 \) mm
Length of pile cap \( lx = 3810 \) mm
Spec.of pile ctrs in Y-direction \( AY = 1910 \) mm
Breadth of pile cap \( ly = 3810 \) mm
Concrete grade \( f_{cu} = 25 \) N/mm²
Steel strength \( f_y = 250 \) N/mm²
Cover to underside of main bars \( \text{Cover} = 150 \) mm
Side and top cover to main bars \( \text{Scvr} = 75 \) mm
Chosen depth of pilecap \( h = 1720 \) mm

Design of main steel in X-direction

- Main steel diameter \( \text{diaX} = 16 \) mm
- Main steel spacing \( \text{SpcX} = 40 \) mm

Design of main steel in Y-direction

- Main steel diameter \( \text{diaY} = 16 \) mm
- Main steel spacing \( \text{SpcY} = 40 \) mm

Distribution steel

- Diameter of bar \( \text{LSize} = 40 \) mm
- Spacing of bars \( \text{LSpac} = 300 \) mm
### MAIN REINFORCEMENT

**IN X-DIRECTION:**
- **Characteristic strength:** 250 N/mm$^2$
- **Diameter of bars:** 16 mm
- **Spacing:** 40 mm
- **Total number of bars:** 72
- **Total area required:** 14171 mm$^2$
- **Total area provided:** 14476 mm$^2$
- **Volume of steel:** 74206 cm$^3$
- **Weight of steel:** 582.52 kg

**SUMMARY**
- **Spacing:** 40 mm
- **Total number of bars:** 72
- **Total area required:** 14171 mm$^2$
- **Total area provided:** 14476 mm$^2$
- **Volume of steel:** 74206 cm$^3$
- **Weight of steel:** 582.52 kg

### MAIN REINFORCEMENT

**IN Y-DIRECTION:**
- **Characteristic strength:** 250 N/mm$^2$
- **Diameter of bars:** 16 mm
- **Spacing:** 40 mm
- **Total number of bars:** 72
- **Total area required:** 14319 mm$^2$
- **Total area provided:** 14476 mm$^2$
- **Volume of steel:** 74206 cm$^3$
- **Weight of steel:** 582.52 kg

**SUMMARY**
- **Spacing:** 40 mm
- **Total number of bars:** 72
- **Total area required:** 14319 mm$^2$
- **Total area provided:** 14476 mm$^2$
- **Volume of steel:** 74206 cm$^3$
- **Weight of steel:** 582.52 kg

### DISTRIBUTION REINFORCEMENT

**SUMMARY**
- **Characteristic strength:** 250 N/mm$^2$
- **Diameter of bars:** 40 mm

**IN X-DIRECTION:**
- **In top:**
  - Number of bars: 13
  - Spacing of bars: 300 mm
  - Area required: 4128 mm$^2$/m
  - Area provided: 4188.8 mm$^2$/m
  - Volume of steel: 59791 cm$^3$
  - Weight of steel: 469.36 kg

- **In bottom:**
  - Number of bars: 2
  - Spacing of bars: 275 mm
  - Volume of steel: 9198.6 cm$^3$
  - Weight of steel: 72.209 kg

**IN Y-DIRECTION:**
- **In top:**
  - Number of bars: 13
  - Spacing of bars: 300 mm
  - Area required: 4128 mm$^2$/m
  - Area provided: 4188.8 mm$^2$/m
  - Volume of steel: 59791 cm$^3$
  - Weight of steel: 469.36 kg

- **In bottom:**
  - Number of bars: 2
  - Spacing of bars: 275 mm
  - Volume of steel: 9198.6 cm$^3$
  - Weight of steel: 72.209 kg

### SIDE BARS SUMMARY
- **Characteristic strength:** 250 N/mm$^2$
- **Diameter of bars:** 12 mm
- **Total number of bars:** 12
- **Spacing of bars:** 250 mm
- **Volume of steel:** 10661 cm$^3$
- **Weight of steel:** 83.685 kg
<table>
<thead>
<tr>
<th>QUANTITIES</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total volume of concrete needed</td>
<td>24.968 m³</td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>26.213 m²</td>
</tr>
<tr>
<td>Total volume of steel</td>
<td>297052 cm³</td>
</tr>
<tr>
<td>Total weight of steel</td>
<td>2331.9 kg</td>
</tr>
<tr>
<td>Approx. self-weight of pile cap</td>
<td>838.91 kN</td>
</tr>
<tr>
<td>Approx. total load on each pile</td>
<td>2668 kN</td>
</tr>
</tbody>
</table>
Location: Ex3 - Small pilecap with high-grade concrete

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for five piles

- Designed by Truss Analogy
- Calculations are in accordance with BS8110.
- Pile cap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to principal axes and symmetrical about pile centres (i.e. chain lines in diagram).
- For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column N=7000 kN
Column dimension cx cx=350 mm
Column dimension cy cy=350 mm
Nominal pile diameter PileD=250 mm
Spec.min.spacing of pile centres spc=500 mm
Spec.of pile ctrs in X-direction AX=710 mm
Overhang: pile face to cap edge olap=150 mm
Length of pile cap lx=1260 mm
Spec.of pile ctrs in Y-direction AY=710 mm
Breadth of pile cap ly=1260 mm
Concrete grade fcu=60 N/mm²
Steel strength fy=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=990 mm

Design of main steel in X-direction

- Main steel diameter diaX=16 mm
- Main steel spacing SpcX=85 mm

Design of main steel in Y-direction

- Main steel diameter diaY=12 mm
- Main steel spacing SpcY=45 mm

Distribution steel

- Diameter of bar LSize=16 mm
- Spacing of bars LSpac=155 mm
### MAIN REINFORCEMENT

**IN X-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>85 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>14</td>
</tr>
<tr>
<td>Total area required</td>
<td>2788 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>2814.9 mm²</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>6341.9 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>49.784 kg</td>
</tr>
</tbody>
</table>

**SUMMARY**

- Spacing: 85 mm
- Total number of bars: 14
- Total area required: 2788 mm²
- Total area provided: 2814.9 mm²
- Volume of steel: 6341.9 cm³
- Weight of steel: 49.784 kg

### MAIN REINFORCEMENT

**IN Y-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>45 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>26</td>
</tr>
<tr>
<td>Total area required</td>
<td>2843.5 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>2940.5 mm²</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>6810.3 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>53.461 kg</td>
</tr>
</tbody>
</table>

**SUMMARY**

- Spacing: 45 mm
- Total number of bars: 26
- Total area required: 2843.5 mm²
- Total area provided: 2940.5 mm²
- Volume of steel: 6810.3 cm³
- Weight of steel: 53.461 kg

### DISTRIBUTION REINFORCEMENT

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
</tbody>
</table>

**IN X-DIRECTION:**

- In top:
  - Number of bars: 8
  - Spacing of bars: 155 mm
  - Area required: 1287 mm²/m
  - Area provided: 1297.2 mm²/m
  - Volume of steel: 1785.4 cm³
  - Weight of steel: 14.016 kg

- In bottom:
  - No distribution bars required

**IN Y-DIRECTION:**

- In top:
  - Number of bars: 8
  - Spacing of bars: 155 mm
  - Area required: 1287 mm²/m
  - Area provided: 1297.2 mm²/m
  - Volume of steel: 1785.4 cm³
  - Weight of steel: 14.016 kg

- In bottom:
  - No distribution bars required

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>8</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>2375 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>18.644 kg</td>
</tr>
</tbody>
</table>

### QUANTITIES

- Total volume of concrete needed: 1.5717 m³
- Total area of formwork: 4.9896 m²
- Total volume of steel: 19098 cm³
- Total weight of steel: 149.92 kg

- Approx. self-weight of pile cap: 52.81 kN
- Approx. total load on each pile: 1411 kN
Location: Ex1 - Typical pilecap

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for five piles

- Designed by truss-analogy to EC2 Part 1-1.

- Pile cap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres.

Ultimate load & moment values to be used.

Total design load on column  \( N=7000 \) kN
Column dimension \( cx \)  \( cx=500 \) mm
Column dimension \( cy \)  \( cy=500 \) mm
Nominal pile diameter  \( \text{PileD}=450 \) mm
Min spacing of pile centres  \( \text{spc}=1350 \) mm
Partial safety factor for steel  \( gams=1.15 \)
Partial safety factor for conc.  \( \text{gamc}=1.5 \)
Spc.of pile ctrs in X-direction  \( AX=1910 \) mm
Overhang: pile face to cap edge  \( olap=150 \) mm
Length of pile cap  \( lx=2660 \) mm
Spc.of pile ctrs in Y-direction  \( AY=1910 \) mm
Breadth of pile cap  \( ly=2660 \) mm
Char yield strength of reinft.  \( fyk=500 \) N/mm²
Cover to underside of main bars  \( \text{Cover}=150 \) mm
Side and top cover to main bars  \( \text{Scvr}=75 \) mm
Chosen depth of pilecap  \( h=1600 \) mm

Design of main steel in X-direction

Main steel diameter  \( \text{diaX}=16 \) mm
Main steel spacing  \( \text{SpcX}=75 \) mm

Design of main steel in Y-direction

Main steel diameter  \( \text{diaY}=16 \) mm
Main steel spacing  \( \text{SpcY}=75 \) mm
Distribution steel

- Distribution steel diameter: LSize = 20 mm
- Spacing of bars: LSpac = 130 mm

Check shear resistance in X-direction

- Shear force due to ultimate load: VEd = N/2.5 = 2800 kN
- Shear resistance: VRdc = beta'*CRdc*ks*vRdc*bw*d/1000
  = 3372.9 kN
- Modified shear resistance: VRdc = VRdm = 3606.8 kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>2800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>3606.8 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

- Shear force due to ultimate load: VEd = N/2.5 = 2800 kN
- Shear resistance: VRdc = beta'*CRdc*ks*vRdc*bw*d/1000
  = 3353 kN
- Modified shear resistance: VRdc = VRdm = 3574.9 kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>2800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>3574.9 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

- Check punching shear at column face:
  - Eccentricity factor β: beta = 1
  - Shear stress at column perimeter: vEd = beta*VEd*1000/(uo*d) = 1.9635 N/mm²
  - Maximum punching shear resistance: VRdmax = 0.5*v*fcd = 4.9728 N/mm²
  - Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

- Check punching shear at 2d:
  - Shear stress at control perimeter: vRdc = (0.18/gamc)*k*(100*p1 *fck)^0.333 = 0.27925 N/mm²
  - Enhancement factor to be used: senh = 5.0035
  - Enhanced punching shear resist.: VRdc = senh*VRdc = 1.3973 N/mm²
  - Design punching shear resistance: VRdc = senh*0.035*k^1.5*fck^0.5 = 1.4933 N/mm²

Check strength of struts and nodes - Clauses 6.5.2(2) and 6.5.4(4)

- Case 1: X-Direction
  - Strength of concrete strut:
    - Strut compressive stress: Sstr = Cstr*1000/(cy*Wstr) = 8.0565 N/mm²
    - Design strength of conc. strut: SRdmax = 0.6*(1-fck/250)*fcd = 8.4538 N/mm²
  - The strut compressive stress is ≤ the strut design strength, hence OK.
  - Strength of upper strut node:
Stress on horizontal plane: 
\[
\text{SEd1} = 0.20 \times N \times 1000 / (cx/2 \times cy) = 11.2 \text{ N/mm}^2
\]

Stress on vertical plane: 
\[
\text{SEd2} = C_{\text{str}} \times 1000 \times AX / (2 \times L_{\text{str}} \times \text{comp}) = 4.9124 \text{ N/mm}^2
\]

Strength of compression node: 
\[
\text{SRdmax} = (1 - f_{\text{ck}} / 250) \times f_{\text{cd}} = 14.09 \text{ N/mm}^2
\]

The stress in horizontal and vertical planes is satisfactory.

Strength of lower strut node:
\[
\text{SEd1} = S_{\text{str}} = 8.0565 \text{ N/mm}^2
\]

Stress in pile:
\[
\text{SEd2} = 0.20 \times N \times 1000 / (\text{PileD})^2 \times \pi / 4 = 8.8026 \text{ N/mm}^2
\]

Strength of compression node:
\[
\text{SRdmax} = 0.85 \times (1 - f_{\text{ck}} / 250) \times f_{\text{cd}} = 11.976 \text{ N/mm}^2
\]

The stress in strut and pile is satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>75 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>26</td>
</tr>
<tr>
<td>Total area required</td>
<td>5040 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>5227.6 mm²</td>
</tr>
</tbody>
</table>

**IN Y-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>75 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>26</td>
</tr>
<tr>
<td>Total area required</td>
<td>5040 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>5227.6 mm²</td>
</tr>
</tbody>
</table>

**DISTRIBUTION REINFORCEMENT**

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
</tbody>
</table>

**IN X-DIRECTION:**

In top:
| Number of bars | 20 |
| Spacing of bars | 130 mm |
| Area required   | 2400 mm²/m |
| Area provided   | 2416.6 mm²/m |

In bottom:
| Number of bars | 5 |
| Spacing of bars | 110 mm |

**IN Y-DIRECTION:**

In top:
| Number of bars | 20 |
| Spacing of bars | 130 mm |
| Area required   | 2400 mm²/m |
| Area provided   | 2416.6 mm²/m |

In bottom:
| Number of bars | 5 |
| Spacing of bars | 110 mm |

**SIDE BARS SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>12</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

**QUANTITIES**

| Total concrete volume   | 11.321 m³ |
| Total area of formwork  | 17.024 m² |
Location: Ex2 - Heavily-loaded pilecap reinforced with mild steel

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for five piles

- Designed by truss-analogy to EC2 Part 1-1.
- Pile cap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres.
- Ultimate load & moment values to be used.

Total design load on column \( N = 12500 \) kN
Column dimension \( c_x \) \( c_x = 500 \) mm
Column dimension \( c_y \) \( c_y = 500 \) mm
Nominal pile diameter \( \text{PileD} = 500 \) mm
Min spacing of pile centres \( \text{spc} = 1350 \) mm
Partial safety factor for steel \( \text{gams} = 1.15 \)
Partial safety factor for conc. \( \text{gamc} = 1.5 \)
Spec.of pile ctrs in X-direction \( \text{AX} = 1910 \) mm
Overhang: pile face to cap edge \( \text{olap} = 700 \) mm
Length of pile cap \( \text{lx} = 3810 \) mm
Spec.of pile ctrs in Y-direction \( \text{AY} = 1910 \) mm
Breadth of pile cap \( \text{ly} = 3810 \) mm
Char yield strength of reinft. \( f_{yk} = 500 \) N/mm²
Cover to underside of main bars \( \text{Cover} = 150 \) mm
Side and top cover to main bars \( \text{Scvr} = 75 \) mm
Chosen depth of pilecap \( h = 1720 \) mm

Design of main steel in X-direction

Main steel diameter \( \text{diaX} = 25 \) mm
Main steel spacing \( \text{SpcX} = 200 \) mm

Design of main steel in Y-direction

Main steel diameter \( \text{diaY} = 25 \) mm
Main steel spacing \( \text{SpcY} = 200 \) mm
Distribution steel

- Distribution steel diameter: \( \text{LSize}=25\text{ mm} \)
- Spacing of bars: \( \text{LSpac}=190\text{ mm} \)

Check shear resistance in X-direction

- Shear force due to ultimate load: \( \text{VEd}=\frac{N}{2.5}=5000\text{ kN} \)
- Shear resistance: \( \text{VRdc}=\beta'\cdot \text{CRdc} \cdot \text{ks} \cdot vRdc \cdot bw \cdot d / 1000 \)
  \[ = 4916.2\text{ kN} \]
- Modified shear resistance: \( \text{VRdc}=\text{VRdm}=5178\text{ kN} \)

**SHEAR SUMMARY**

- Shear force: 5000 kN
- Shear resistance: 5178 kN

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

- Shear force due to ultimate load: \( \text{VEd}=\frac{N}{2.5}=5000\text{ kN} \)
- Shear resistance: \( \text{VRdc}=\beta'\cdot \text{CRdc} \cdot \text{ks} \cdot vRdc \cdot bw \cdot d / 1000 \)
  \[ = 4873.9\text{ kN} \]
- Modified shear resistance: \( \text{VRdc}=\text{VRdm}=5111.3\text{ kN} \)

**SHEAR SUMMARY**

- Shear force: 5000 kN
- Shear resistance: 5111.3 kN

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

- Check punching shear at column face:
  - Eccentricity factor \( \beta \) = 1
  - Shear stress at column perimeter: \( v_{Ed}=\beta \cdot VE_{d} \cdot 1000/(u_{d} \cdot d) = 3.2626 \text{ N/mm}^2 \)
  - Maximum punching shear resistance: \( v_{Rd_{max}}=0.5 \cdot v \cdot f_{cd} = 4.5 \text{ N/mm}^2 \)
  - Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

- Check punching shear at 2d:
  - Shear stress at control perimeter: \( v_{Rd}=(0.18/\gamma_{cm}) \cdot k \cdot (100 \cdot p_{1} \cdot f_{ck})^{0.333}=0.26418 \text{ N/mm}^2 \)
  - Enhancement factor to be used: \( \gamma_{enh}=5.5225 \)
  - Enhanced punching shear resist.: \( v_{Rdc}=\gamma_{enh} \cdot v_{Rdc}=1.459 \text{ N/mm}^2 \)
  - Design punching shear resistance: \( v_{Rdc}=\gamma_{enh} \cdot 0.035 \cdot k \cdot 1.5 \cdot f_{ck}^{0.5} \cdot 1.5349 \text{ N/mm}^2 \)

**MAIN REINFORCEMENT IN X-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>200 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>16</td>
</tr>
<tr>
<td>Total area required</td>
<td>7740 mm^2</td>
</tr>
<tr>
<td>Total area provided</td>
<td>7854 mm^2</td>
</tr>
<tr>
<td><strong>MAIN REINFORCEMENT</strong></td>
<td>Characteristic strength</td>
</tr>
<tr>
<td>------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td><strong>IN Y-DIRECTION:</strong></td>
<td>Diameter of bars</td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td>Spacing</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>DISTRIBUTION REINFORCEMENT</strong></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SUMMARY</strong></td>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
</tbody>
</table>

| **IN X-DIRECTION:**            |                          |           |
| **In top:**                    | Number of bars           | 20        |
|                                | Spacing of bars          | 190 mm    |
|                                | Area required            | 2580 mm²/m |
|                                | Area provided            | 2583.5 mm²/m |
| **In bottom:**                 | Number of bars           | 4         |
|                                | Spacing of bars          | 160 mm    |

| **IN Y-DIRECTION:**            |                          |           |
| **In top:**                    | Number of bars           | 20        |
|                                | Spacing of bars          | 190 mm    |
|                                | Area required            | 2580 mm²/m |
|                                | Area provided            | 2583.5 mm²/m |
| **In bottom:**                 | Number of bars           | 4         |
|                                | Spacing of bars          | 160 mm    |

<table>
<thead>
<tr>
<th><strong>SIDE BARS SUMMARY</strong></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

| **QUANTITIES**                 |                          |           |
| Total concrete volume          | 24.968 m³               |
| Total area of formwork         | 26.213 m²               |
Location: Ex3 - Small pilecap with high-grade concrete

Bottom layer of main steel is assumed to run in X-direction.

![Diagram of pilecap for five piles]

- Designed by truss-analogy to EC2 Part 1-1.

Pile cap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres.

Ultimate load & moment values to be used.

Total design load on column \( N = 7000 \text{ kN} \)
Column dimension \( cx = 350 \text{ mm} \)
Column dimension \( cy = 350 \text{ mm} \)
Nominal pile diameter \( \text{PileD}=250 \text{ mm} \)
Min spacing of pile centres \( \text{spc}=500 \text{ mm} \)
Partial safety factor for steel \( gams=1.15 \)
Partial safety factor for conc. \( gamc=1.5 \)
Spc.of pile ctrs in X-direction \( AX=710 \text{ mm} \)
Overhang: pile face to cap edge \( olap=150 \text{ mm} \)
Length of pile cap \( lx=1260 \text{ mm} \)
Spc.of pile ctrs in Y-direction \( AY=710 \text{ mm} \)
Breadth of pile cap \( ly=1260 \text{ mm} \)
Char yield strength of reinft. \( fyk=500 \text{ N/mm}^2 \)
Cover to underside of main bars \( \text{Cover}=150 \text{ mm} \)
Side and top cover to main bars \( \text{Scvr}=75 \text{ mm} \)
Chosen depth of pilecap \( h=1600 \text{ mm} \)

Design of main steel in X-direction

- Main steel diameter \( \text{diaX}=25 \text{ mm} \)
- Main steel spacing \( \text{SpcX}=175 \text{ mm} \)

Design of main steel in Y-direction

- Main steel diameter \( \text{diaY}=20 \text{ mm} \)
- Main steel spacing \( \text{SpcY}=105 \text{ mm} \)
Distribution steel

Distribution steel diameter \( \text{LSize}=32 \text{ mm} \)
Spacing of bars \( \text{LSpac}=265 \text{ mm} \)

Check shear resistance in X-direction

Shear force due to ultimate load \( V_{E_d}=N/2.5=2800 \text{ kN} \)
Shear resistance \( V_{R_{dc}}=\beta'*C_{R_{dc}}*k_s*V_{R_{dc}}*b_w*d/1000 \)
\( =2641.9 \text{ kN} \)
Modified shear resistance \( V_{R_{dc}}=V_{R_{dm}}=2884.7 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>2800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>2884.7 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{E_d}=N/2.5=2800 \text{ kN} \)
Shear resistance \( V_{R_{dc}}=\beta'*C_{R_{dc}}*k_s*V_{R_{dc}}*b_w*d/1000 \)
\( =2584.4 \text{ kN} \)
Modified shear resistance \( V_{R_{dc}}=V_{R_{dm}}=2848.7 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>2800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>2848.7 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor \( \beta \)
\( \beta=1 \)
Shear stress at column perimeter \( v_{E_d}=\beta*V_{E_d}*1000/(u_o*d)=2.8269 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{R_{dmax}}=0.5*v*f_{cd}=8 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>175 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>8</td>
</tr>
<tr>
<td>Total area required</td>
<td>3814.9 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>3927 mm²</td>
</tr>
</tbody>
</table>

**MAIN REINFORCEMENT**

**IN Y-DIRECTION:**

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>105 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>12</td>
</tr>
<tr>
<td>Total area required</td>
<td>3748.2 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>3769.9 mm²</td>
</tr>
</tbody>
</table>

**DISTRIBUTION REINFORCEMENT**

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>32 mm</td>
</tr>
</tbody>
</table>
In top:  
Number of bars  5  
Spacing of bars  265 mm  
Area required  3027.7 mm²/m  
Area provided  3034.9 mm²/m  

In bottom:  
No distribution bars required  

IN Y-DIRECTION:  
In top:  
Number of bars  5  
Spacing of bars  265 mm  
Area required  3027.7 mm²/m  
Area provided  3034.9 mm²/m  

In bottom:  
No distribution bars required  

SIDE BARS SUMMARY  
Characteristic strength  500 N/mm²  
Diameter of bars  12 mm  
Total number of bars  12  
Spacing of bars  250 mm  

QUANTITIES  
Total concrete volume  2.5402 m³  
Total area of formwork  8.064 m²
Location: Ex1 - Typical pilecap

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$

Pilecap for two piles

Designed by Bending Theory.

Calculations are in accordance with BS8110.

For bond calculations, any high-yield bars are assumed Type 2 deformed.

Total ultimate load on column $N=2800$ kN
Column dimension $cx=500$ mm
Column dimension $cy=500$ mm
Nominal pile diameter $PileD=450$ mm
Spacing of pile centres $A=1350$ mm
Overhang: pile face to cap edge $olap=150$ mm
Length of pile cap $lx=2100$ mm
Width of pile cap $ly=750$ mm
Concrete grade $fcu=35$ N/mm$^2$
Steel strength $fy=500$ N/mm$^2$
Cover to underside of main bars $Cover=150$ mm
Side and top cover to main bars $SCover=75$ mm
Chosen depth of pilecap $h=1020$ mm

Design of main reinforcement

Main steel diameter $dia=16$ mm
Main steel spacing $Space=65$ mm

Distribution steel

Diameter of bar $LSize=12$ mm
Spacing of bars $LSpac=85$ mm

MAIN REINFORCEMENT SUMMARY

Characteristic strength $500$ N/mm$^2$
Diameter of bars $16$ mm
Number of bars $9$
Spacing $65$ mm
Area required $1694.7$ mm$^2$
Area provided $1809.6$ mm$^2$
Volume of steel $4565.5$ cm$^3$
Weight of steel $35.839$ kg

DISTRIBUTION REINFORCEMENT SUMMARY

Characteristic strength $500$ N/mm$^2$
Diameter of bars $12$ mm
Number of bars $23$
Spacing of bars $85$ mm
Area required $1326$ mm$^2$
Area provided $1330.6$ mm$^2$
Volume of steel $8443.6$ cm$^3$
Weight of steel $66.282$ kg
<table>
<thead>
<tr>
<th>TOP BARS SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>85 mm</td>
<td></td>
</tr>
<tr>
<td>Volume of steel</td>
<td>1764.3 cm³</td>
<td></td>
</tr>
<tr>
<td>Weight of steel</td>
<td>13.85 kg</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SIDE BARS SUMMARY</td>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Number of bars/side</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
<td></td>
</tr>
<tr>
<td>Volume of steel</td>
<td>1764.3 cm³</td>
<td></td>
</tr>
<tr>
<td>Weight of steel</td>
<td>13.85 kg</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>QUANTITIES</td>
<td>Total volume of concrete needed</td>
<td>1.6065 m³</td>
</tr>
<tr>
<td></td>
<td>Total area of formwork</td>
<td>5.814 m²</td>
</tr>
<tr>
<td></td>
<td>Total volume of steel</td>
<td>16538 cm³</td>
</tr>
<tr>
<td></td>
<td>Total weight of steel</td>
<td>129.82 kg</td>
</tr>
<tr>
<td></td>
<td>Approx. self-weight of pile cap</td>
<td>53.978 kN</td>
</tr>
<tr>
<td></td>
<td>Approx. total load on each pile</td>
<td>1427 kN</td>
</tr>
</tbody>
</table>
Location: Ex2 - Heavily-loaded column

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

Pilecap for two piles

Designed by Bending Theory.

Calculations are in accordance with BS8110.

For bond calculations, any high-yield bars are assumed Type 2 deformed.

Total ultimate load on column \( N = 10000 \text{ kN} \)
Column dimension \( cx = 500 \text{ mm} \)
Column dimension \( cy = 500 \text{ mm} \)
Nominal pile diameter \( \text{PileD} = 500 \text{ mm} \)
Spacing of pile centres \( A = 1500 \text{ mm} \)
Overhang: pile face to cap edge \( olap = 300 \text{ mm} \)
Length of pile cap \( lx = 2600 \text{ mm} \)
Width of pile cap \( ly = 1100 \text{ mm} \)
Concrete grade \( f_{cu} = 35 \text{ N/mm}^2 \)
Steel strength \( f_y = 500 \text{ N/mm}^2 \)
Cover to underside of main bars \( \text{Cover} = 150 \text{ mm} \)
Side and top cover to main bars \( \text{SCover} = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 1830 \text{ mm} \)

Design of main reinforcement

Main steel diameter \( \text{dia} = 20 \text{ mm} \)
Main steel spacing \( \text{Space} = 80 \text{ mm} \)

Distribution steel

Diameter of bar \( \text{LSize} = 25 \text{ mm} \)
Spacing of bars \( \text{LSpac} = 205 \text{ mm} \)

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Spacing</td>
<td>80 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>3646.2 mm²</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>3769.9 mm²</td>
<td></td>
</tr>
<tr>
<td>Volume of steel</td>
<td>15871 cm³</td>
<td></td>
</tr>
<tr>
<td>Weight of steel</td>
<td>124.59 kg</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DISTRIBUTION REINFORCEMENT SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>205 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>2379 mm²</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>2394.5 mm²</td>
<td></td>
</tr>
<tr>
<td>Volume of steel</td>
<td>35696 cm³</td>
<td></td>
</tr>
<tr>
<td>Weight of steel</td>
<td>280.22 kg</td>
<td></td>
</tr>
<tr>
<td><strong>TOP BARS SUMMARY</strong></td>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>----------------------</td>
<td>-------------------------</td>
<td>-----------</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>205 mm</td>
<td></td>
</tr>
<tr>
<td>Volume of steel</td>
<td>6013.2 cm³</td>
<td></td>
</tr>
<tr>
<td>Weight of steel</td>
<td>47.204 kg</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>SIDE BARS SUMMARY</strong></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Number of bars/side</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
<td></td>
</tr>
<tr>
<td>Volume of steel</td>
<td>3879.2 cm³</td>
<td></td>
</tr>
<tr>
<td>Weight of steel</td>
<td>30.452 kg</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>QUANTITIES</strong></th>
<th>Total volume of concrete needed</th>
<th>5.2338 m³</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total area of formwork</td>
<td>13.542 m²</td>
</tr>
<tr>
<td></td>
<td>Total volume of steel</td>
<td>61460 cm³</td>
</tr>
<tr>
<td></td>
<td>Total weight of steel</td>
<td>482.46 kg</td>
</tr>
</tbody>
</table>

Approx. self-weight of pile cap | 175.86 kN  
Approx. total load on each pile | 5088 kN
Location: Ex3 - Lightly-loaded column

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$

Pilecap for two piles

 Designed by Bending Theory.

Calculations are in accordance with BS8110.

For bond calculations, any high-yield bars are assumed Type 2 deformed.

Total ultimate load on column $N = 100 \text{ kN}$
Column dimension $c_x = 500 \text{ mm}$
Column dimension $c_y = 500 \text{ mm}$
Nominal pile diameter $PileD = 450 \text{ mm}$
Spacing of pile centres $A = 1350 \text{ mm}$
Overhang: pile face to cap edge $olap = 600 \text{ mm}$
Length of pile cap $l_x = 3000 \text{ mm}$
Width of pile cap $l_y = 1650 \text{ mm}$
Concrete grade $f_{cu} = 35 \text{ N/mm}^2$
Steel strength $f_y = 500 \text{ N/mm}^2$
Cover to underside of main bars $Cover = 150 \text{ mm}$
Side and top cover to main bars $SCover = 75 \text{ mm}$
Chosen depth of pilecap $h = 300 \text{ mm}$

Design of main reinforcement

Main steel diameter $dia = 12 \text{ mm}$
Main steel spacing $Space = 275 \text{ mm}$

Distribution steel

Diameter of bar $LSize = 12 \text{ mm}$
Spacing of bars $LSpac = 285 \text{ mm}$

**MAIN REINFORCEMENT SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Number of bars: 6
- Spacing: 275 mm
- Area required: 643.5 mm²
- Area provided: 678.58 mm²
- Volume of steel: 1302.9 cm³
- Weight of steel: 10.228 kg

**DISTRIBUTION REINFORCEMENT SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Number of bars: 11
- Spacing of bars: 285 mm
- Area required: 390 mm²
- Area provided: 396.83 mm²
- Volume of steel: 4486.1 cm³
- Weight of steel: 35.216 kg
### TOP BARS SUMMARY
- **Characteristic strength**: 500 N/mm²
- **Diameter of bars**: 12 mm
- **Number of bars**: 6
- **Spacing of bars**: 285 mm
- **Volume of steel**: 1934 cm³
- **Weight of steel**: 15.182 kg

### SIDE BARS SUMMARY
- **Characteristic strength**: 500 N/mm²
- **Diameter of bars**: 12 mm
- **Number of bars/side**: 1
- **Spacing of bars**: 250 mm
- **Volume of steel**: 644.65 cm³
- **Weight of steel**: 5.0605 kg

### QUANTITIES
- **Total volume of concrete needed**: 1.485 m³
- **Total area of formwork**: 2.79 m²
- **Total volume of steel**: 8367.6 cm³
- **Total weight of steel**: 65.686 kg
- **Approx. self-weight of pile cap**: 49.896 kN
- **Approx. total load on each pile**: 75 kN
Location: Ex1 - Typical pilecap

Pilecap for two piles

Designed using bending theory to EC2 Part 1-1.

Use ultimate values for load and moment.

Total design load on column \( N = 1500 \text{ kN} \)
Column dimension \( c_x = 500 \text{ mm} \)
Column dimension \( c_y = 500 \text{ mm} \)
Nominal pile diameter \( \text{PileD}=450 \text{ mm} \)
Partial safety factor for steel \( g_{ms}=1.15 \)
Partial safety factor for conc. \( g_{mc}=1.5 \)
Spacing of pile centres \( A=1350 \text{ mm} \)
Overhang: pile face to cap edge \( o_{lap}=150 \text{ mm} \)
Length of pile cap \( l_x=2100 \text{ mm} \)
Width of pile cap \( l_y=750 \text{ mm} \)
Char yield strength of reinft. \( f_{yk}=500 \text{ N/mm}^2 \)
Cover to underside of main bars \( \text{Cover}=150 \text{ mm} \)
Side and top cover to main bars \( \text{SCover}=75 \text{ mm} \)
Chosen depth of pilecap \( h=1000 \text{ mm} \)

Design of main reinforcement

Main steel diameter \( \text{dia}=12 \text{ mm} \)
Main steel spacing \( \text{Space}=60 \text{ mm} \)

Distribution steel

Distribution steel diameter \( \text{LSize}=12 \text{ mm} \)
Spacing of bars \( \text{LSpac}=75 \text{ mm} \)
Area of dist.steel provided \( \text{A}^2=\pi L_{\text{Size}}^2 \times 250/L_{\text{Spac}}=1508 \text{ mm}^2/\text{m} \)

Check shear resistance

Shear force due to ultimate load \( V_{Ed}=N/2+M_{x}^\prime \times 1000/A=750 \text{ kN} \)
Shear resistance \( V_{Rdc}=\beta_{1}^\prime C_{Rdc} \times \text{ks} \times v_{Rdc} \times b \times d/1000 \approx 772.6 \text{ kN} \)
Modified shear resistance \( V_{Rdc}=V_{Rdm}=850.1 \text{ kN} \)

**Shear Summary**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>750 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>850.1 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.
Check punching shear at column face

Check punching shear at column face:
Eccentricity factor \( \beta \) \( \beta = 1 \)
Shear stress at column perimeter \( \nu_{Ed} = \beta \nu_{Ed} * 1000 / (u_0*d) \) = 0.8886 N/mm²
Maximum punching shear resistance \( \nu_{Rdmax} = 0.5 \nu * f_{cd} = 4.973 \) N/mm²
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

<table>
<thead>
<tr>
<th><strong>MAIN REINFORCEMENT</strong></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Spacing</td>
<td>60 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>1125 mm²</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>1131 mm²</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>DISTRIBUTION REINFORCEMENT</strong></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>27</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>75 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>1500 mm²</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>1508 mm²</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>TOP BARS SUMMARY</strong></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>75 mm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>SIDE BARS SUMMARY</strong></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Number of bars/side</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>QUANTITIES</strong></th>
<th>Total concrete volume</th>
<th>1.575 m³</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total area of formwork</td>
<td>5.7 m²</td>
</tr>
</tbody>
</table>
**Location: Ex2 - Heavily-loaded column**

---

**Pilecap for two piles**

Designed using bending theory to EC2 Part 1-1.

Use ultimate values for load and moment.

---

**Total design load on column**  
N=5000 kN

**Column dimension cx**  
Cx=500 mm

**Column dimension cy**  
Cy=500 mm

**Nominal pile diameter**  
PileD=500 mm

**Partial safety factor for steel**  
gams=1.15

**Partial safety factor for conc.**  
gamc=1.5

**Spacing of pile centres**  
A=1500 mm

**Overhang: pile face to cap edge**  
Olap=300 mm

**Length of pile cap**  
1x=2600 mm

**Width of pile cap**  
Ly=1100 mm

**Char yield strength of reinft.**  
Fyk=500 N/mm²

**Cover to underside of main bars**  
Cover=150 mm

**Side and top cover to main bars**  
SCover=75 mm

**Chosen depth of pilecap**  
h=2200 mm

---

**Design of main reinforcement**

- **Main steel diameter**  
  Dia=20 mm

---

**Distribution steel**

- **Distribution steel diameter**  
  LSize=20 mm

---

**Check shear resistance**

- **Shear force due to ultimate load**  
  VEd=N/2+Mx'*1000/A=2500 kN

- **Shear resistance**  
  VRdc=beta'*CRdc*ks*vRdc*bw*d/1000  
  =2370 kN

- **Modified shear resistance**  
  VRdc=VRdm=2501 kN

**SHEAR SUMMARY**

- **Shear force**  
  2500 kN

---

**Hence section is satisfactory, no shear reinforcement is required.**
Check punching shear at column face

Check punching shear at column face:
Eccentricity factor $\beta$  $\beta = 1$
Shear stress at column perimeter $\sigma_{Ed} = \beta \cdot V_{Ed} \cdot 1000/(u_0 \cdot d) = 1.225 \text{ N/mm}^2$
Maximum punching shear resistance $V_{Rd,max} = 0.5 \cdot v \cdot f_{cd} = 4.973 \text{ N/mm}^2$
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 \text{ N/mm}^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>12</td>
</tr>
<tr>
<td>Spacing</td>
<td>80 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>3630 mm²</td>
</tr>
<tr>
<td>Area provided</td>
<td>3770 mm²</td>
</tr>
</tbody>
</table>

**SUMMARY**

| Diameter of bars       | 20 mm               |
| Number of bars         | 26                  |
| Spacing of bars        | 95 mm               |
| Area required          | 3300 mm²            |
| Area provided          | 3307 mm²            |

**TOP BARS SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 \text{ N/mm}^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Number of bars</td>
<td>11</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>95 mm</td>
</tr>
</tbody>
</table>

**SIDE BARS SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 \text{ N/mm}^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Number of bars/side</td>
<td>8</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

**QUANTITIES**

| Total concrete volume  | 6.292 m³            |
| Total area of formwork | 16.28 m²            |
Location: Ex3 - Lightly-loaded column

Pilecap for two piles

- Designed using bending theory to EC2 Part 1-1.
- Use ultimate values for load and moment.

Total design load on column \( N = 100 \text{ kN} \)
Column dimension \( cx = 500 \text{ mm} \)
Column dimension \( cy = 500 \text{ mm} \)
Nominal pile diameter \( \text{PileD} = 450 \text{ mm} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Spacing of pile centres \( A = 1350 \text{ mm} \)
Overhang: pile face to cap edge \( \text{olap} = 600 \text{ mm} \)
Length of pile cap \( lx = 3000 \text{ mm} \)
Width of pile cap \( ly = 1650 \text{ mm} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Cover to underside of main bars \( \text{Cover} = 150 \text{ mm} \)
Side and top cover to main bars \( \text{SCover} = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 300 \text{ mm} \)

Design of main reinforcement

- Main steel diameter \( \text{dia} = 12 \text{ mm} \)
- Main steel spacing \( \text{Space} = 225 \text{ mm} \)

Distribution steel

- Distribution steel diameter \( \text{LSize} = 12 \text{ mm} \)
- Spacing of bars \( \text{LSpac} = 250 \text{ mm} \)
- Area of dist.steel provided \( A2 = \pi \text{LSize}^2 \cdot 250 \cdot \text{LSpac} = 452.4 \text{ mm}^2/m \)

Check shear resistance

- Shear force due to ultimate load \( V_{Ed} = \frac{N}{2} + Mx' \cdot 1000/A = 50 \text{ kN} \)
- Shear resistance \( V_{Rdc} = \beta \cdot C_{Rdc} \cdot k_s \cdot v_{Rdc} \cdot b_w \cdot d/1000 = 120 \text{ kN} \)
- Modified shear resistance \( V_{Rdc} = V_{Rdm} = 124.5 \text{ kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>50 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear resistance</td>
<td>124.5 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.
Check punching shear at column face

Check punching shear at column face:
Eccentricity factor $\beta$ $\beta=1$
Shear stress at column perimeter $v_E=\beta V_E d/(u_0 d)=0.3472$ N/mm²
Maximum punching shear resistance $v_{Rd_{max}}=0.5 v f_{cd}=4.973$ N/mm²
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

### MAIN REINFORCEMENT

**SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Number of bars: 7
- Spacing: 225 mm
- Area required: 742.5 mm²
- Area provided: 791.7 mm²

### DISTRIBUTION REINFORCEMENT

**SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Number of bars: 12
- Spacing of bars: 250 mm
- Area required: 450 mm²
- Area provided: 452.4 mm²

### TOP BARS SUMMARY

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Number of bars: 7
- Spacing of bars: 250 mm

### SIDE BARS SUMMARY

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Number of bars/side: 1
- Spacing of bars: 250 mm

### QUANTITIES

- Total concrete volume: 1.485 m³
- Total area of formwork: 2.79 m²
Location: Pilecap at G2

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$

Pilecap for 3 piles

Calculations are in accordance with BS8110: 1985 (amended 1989).
Pile cap shaped as shown with main reinforcement parallel to principal axes (alternative system with reinforcing bars running between heads of piles will be subject of a future proforma).
For bond calculations any high-yield bars assumed to be Type 2: deformed.

Total ultimate load on column $N=4200$ kN
Column dimension $cx=cx=500$ mm
Column dimension $cy=cy=500$ mm
Nominal pile diameter $PileD=450$ mm
Spacing of pile ctrs in X-direction $AX=1350$ mm
Overlap from pile face to cap edge $olap=150$ mm
Spacing of pile ctrs in Y-direction $AY=1169$ mm
Distance $AY'$ to centre of column $AY'=779$ mm
Maximum cap length in X-direction $lx=2100$ mm
Minimum cap dimension in X-direction $BX=750$ mm
Maximum cap breadth in Y-direction $ly=1919$ mm
Minimum cap dimension in Y-direction $BY=700$ mm
Concrete grade $fcu=35$ N/mm$^2$
Steel strength $fy=500$ N/mm$^2$
Cover to underside of main bars $Cover=150$ mm
Side and top cover to main bars $Scvr=75$ mm
Chosen depth of pilecap $h=1020$ mm

Design of main steel in X-direction

Main steel diameter $diaX=16$ mm
Main steel spacing $SpcX=145$ mm

Design of main steel in Y-direction

Main steel diameter $diaY=16$ mm
Main steel spacing $SpcY=145$ mm

Distribution steel

Diameter of bar $LSize=12$ mm
Spacing of bars $LSpac=85$ mm
MAIN REINFORCEMENT

IN X-DIRECTION:

SUMMARY

Characteristic strength: 500 N/mm²
Diameter of bars: 16 mm
Spacing: 145 mm
Total number of bars: 13
Total area required: 2544.6 mm²
Total area provided: 2613.8 mm²
Volume of steel: 5483.8 cm³
Weight of steel: 43.048 kg

MAIN REINFORCEMENT

IN Y-DIRECTION:

SUMMARY

Characteristic strength: 500 N/mm²
Diameter of bars: 16 mm
Spacing: 145 mm
Total number of bars: 14
Area reqd.@ critical section: 2784.6 mm²
Area prov.@ critical section: 2814.9 mm²
Volume of steel: 5636.2 cm³
Weight of steel: 44.244 kg

DISTRIBUTION

REINFORCEMENT

SUMMARY

IN X-DIRECTION:

In top:
Number of bars: 21
Spacing of bars: 85 mm
Area required: 1326 mm²/m
Area provided: 1330.6 mm²/m
Volume of steel: 3651.9 cm³
Weight of steel: 28.668 kg

IN Y-DIRECTION:

In top:
Number of bars: 24
Spacing of bars: 85 mm
Area required: 1326 mm²/m
Area provided: 1330.6 mm²/m
Volume of steel: 3796.7 cm³
Weight of steel: 29.804 kg

SIDE BARS SUMMARY

Characteristic strength: 500 N/mm²
Diameter of bars: 12 mm
Total number of bars: 12
Spacing of bars: 250 mm

QUANTITIES

Total volume of concrete needed: 3.2712 m³
Total area of formwork: 7.1776 m²
Total volume of steel: 22160 cm³
Total weight of steel: 173.96 kg
Approx.self-weight of pile cap: 109.91 kN
Approx.total load on single pile: 1437 kN
Approx.total load on twin pile: 1437 kN
**Location**: Cap for heavily-loaded column using mild steel reinf.

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$

**Pilecap for 3 piles**

Calculations are in accordance with BS8110: 1985 (amended 1989). Pile cap shaped as shown with main reinforcement parallel to principal axes (alternative system with reinforcing bars running between heads of piles will be subject of a future proforma). For bond calculations any high-yield bars assumed to be Type 2: deformed.

### Design of main steel in X-direction

- **Main steel diameter**: diaX = 20 mm
- **Main steel spacing**: SpcX = 100 mm

### Design of main steel in Y-direction

- **Main steel diameter**: diaY = 20 mm
- **Main steel spacing**: SpcY = 75 mm

### Distribution steel

- **Diameter of bar**: LSize = 20 mm
- **Spacing of bars**: LSpac = 120 mm

**Total ultimate load on column** $N = 8000$ kN

**Column dimension cx** $cx = 500$ mm

**Column dimension cy** $cy = 500$ mm

**Nominal pile diameter** PileD = 500 mm

**Spac.of pile ctrs in X-direction** AX = 1500 mm

**Overlap from pile face to cap edge** olap = 300 mm

**Spac.of pile ctrs in Y-direction** AY = 1299 mm

**Distance AY' to centre of column** AY' = 866 mm

**Maximum cap length in X-direction** lx = 2600 mm

**Min.cap dimension in X-direction** BX = 1100 mm

**Max.cap breadth in Y-direction** ly = 2400 mm

**Min.cap dimension in Y-direction** BY = 1050 mm

**Concrete grade** fcu = 35 N/mm²

**Steel strength** fy = 250 N/mm²

**Cover to underside of main bars** Cover = 150 mm

**Side and top cover to main bars** Scvr = 75 mm

**Chosen depth of pilecap** h = 1080 mm
<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
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<tbody>
<tr>
<td>IN X-DIRECTION:</td>
<td>Diameter of bars</td>
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</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>100 mm</td>
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<td></td>
<td>Total number of bars</td>
<td>23</td>
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<td></td>
<td>Total area required</td>
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<td>Volume of steel</td>
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<table>
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<tbody>
<tr>
<td>IN Y-DIRECTION:</td>
<td>Diameter of bars</td>
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<td>SUMMARY</td>
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<td>Diameter of bars</td>
<td>20 mm</td>
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<tr>
<td>SUMMARY</td>
<td>In top:</td>
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<td>Area required</td>
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<table>
<thead>
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<th>SIDE BARS SUMMARY</th>
<th>Characteristic strength</th>
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<tbody>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>12 mm</td>
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<tr>
<td></td>
<td>Total number of bars</td>
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</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

| QUANTITIES      | Total volume of concrete needed | 5.6457 m³ |
|                 | Total area of formwork        | 9.5998 m² |
|                 | Total volume of steel         | 69561 cm³ |
|                 | Total weight of steel         | 546.06 kg |
| Approx.self-weight of pile cap | 189.7 kN |
| Approx.total load on single pile | 2730 kN |
| Approx.total load on twin pile  | 2730 kN |
Location: Cap for lightly-loaded column

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15

Total ultimate load on column N=150 kN
Column dimension cx cx=400 mm
Column dimension cy cy=400 mm
Nominal pile diameter PileD=250 mm
Spc.of pile ctrs in X-direction AX=750 mm
Overlap from pile face to cap edge olap=150 mm
Spc.of pile ctrs in Y-direction AY=650 mm
Distance AY' to centre of column AY'=433 mm
Maximum cap length in X-direction 1x=1300 mm
Min.cap dimension in X-direction BX=550 mm
Max.cap breadth in Y-direction ly=1200 mm
Min.cap dimension in Y-direction BY=500 mm
Concrete grade fcu=35 N/mm²
Steel strength fy=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=300 mm

Design of main steel in X-direction
Main steel diameter diaX=12 mm
Main steel spacing SpcX=250 mm

Design of main steel in Y-direction
Main steel diameter diaY=16 mm
Main steel spacing SpcY=300 mm

Distribution steel
Diameter of bar LSize=12 mm
Spacing of bars LSpac=285 mm
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<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Spacing</th>
<th>Total number of bars</th>
<th>Total area required</th>
<th>Total area provided</th>
<th>Volume of steel</th>
<th>Weight of steel</th>
</tr>
</thead>
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<tr>
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</tr>
<tr>
<td>IN X-DIRECTION</td>
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<td><strong>MAIN REINFORCEMENT</strong></td>
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</tr>
<tr>
<td>IN Y-DIRECTION</td>
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<td>DISTRIBUTION</td>
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<tr>
<td><strong>SIDE BARS SUMMARY</strong></td>
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</tr>
<tr>
<td>QUANTITIES</td>
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<tr>
<td>Total volume of concrete needed</td>
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<tr>
<td>Total area of formwork</td>
<td>1.3315 m²</td>
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<tr>
<td>Total volume of steel</td>
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<tr>
<td>Total weight of steel</td>
<td>22.082 kg</td>
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<tr>
<td>Approx.self-weight of pile cap</td>
<td>13.079 kN</td>
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</tr>
<tr>
<td>Approx.total load on single pile</td>
<td>55 kN</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Approx.total load on twin pile</td>
<td>55 kN</td>
<td></td>
<td></td>
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<td></td>
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<td></td>
</tr>
</tbody>
</table>
Location: Pilecap at G2

Pilecap for 3 piles

Design to EC2 Part 1-1.

Pilecap shaped as shown with main reinforcement parallel to principal axes.

Use ultimate values for load and moment.

Applied moment in the Y-direction only.

Total design load on column \( N = 3000 \text{ kN} \)
Column dimension \( cx = 500 \text{ mm} \)
Column dimension \( cy = 500 \text{ mm} \)
Design BM at bottom of column \( My' = 0 \text{ kNm} \)
Partial safety factor for steel \( gams = 1.15 \)
Partial safety factor for conc. \( gamc = 1.5 \)
Nominal pile diameter \( \text{PileD} = 450 \text{ mm} \)
Spacing of pile ctrs in X-dir. \( AX = 1350 \text{ mm} \)
Dist. from pile face to cap edge \( olap = 150 \text{ mm} \)
Spacing of pile ctrs in Y-dir. \( AY = 1169 \text{ mm} \)
Distance \( AY' \) to centre of column \( AY' = 779 \text{ mm} \)
Max cap length in X-direction \( lx = 2100 \text{ mm} \)
Min cap dimension in X-direction \( BX = 750 \text{ mm} \)
Max cap breadth in Y-direction \( ly = 1919 \text{ mm} \)
Min cap dimension in Y-direction \( BY = 700 \text{ mm} \)
Char yield strength of reinft. \( fyk = 500 \text{ N/mm}^2 \)
Cover to underside of main bars \( Cover = 150 \text{ mm} \)
Side and top cover to main bars \( Scvr = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 1020 \text{ mm} \)

Design of main steel in X-direction

Main steel diameter \( \text{diaX} = 12 \text{ mm} \)
Main steel spacing \( \text{SpcX} = 70 \text{ mm} \)

Design of main steel in Y-direction

Main steel diameter \( \text{diaY} = 16 \text{ mm} \)
Main steel spacing \( \text{SpcY} = 125 \text{ mm} \)
Distribution steel

Distribution steel diameter

Distribution steel diameter (LS): 20 mm
Spacing of bars (LSp): 205 mm
Area of dist steel provided

A2=PI*LS^2*2*250/LSp=1532 mm²/m

Check shear resistance in X-direction

Shear force due to ultimate load

VEd=N2=1000 kN
Shear resistance

VRdc=beta'*CRdc*ks*vRdc*bw*d/1000 =1797 kN
Modified shear resistance

VRdc=VRdm=1870 kN

SHEAR SUMMARY

Shear force 1000 kN
Shear resistance 1870 kN

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Case 1: Check plane passing through single pile on X-X axis
Shear force due to ultimate load

VEd=N1+My'*1000/Ay=1000 kN
Shear resistance

VRdc=beta'*CRdc*ks*vRdc*bw*d/1000 =1597 kN

SHEAR SUMMARY

Shear force 1000 kN
Shear resistance 1597 kN

Hence section is satisfactory, no shear reinforcement is required.

Case 2: Check plane passing through twin-piles
Shear force due to ultimate load

VEd=2*N2+My'*1000/Ay=2000 kN
Shear resistance

VRdc=beta'*CRdc*ks*vRdc*bw*d/1000 =2182 kN
Modified shear resistance

VRdc=VRdm=2393 kN

SHEAR SUMMARY

Shear force 2000 kN
Shear resistance 2393 kN

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor β

beta=1
Shear stress at column perimeter

vEd=beta*VEd*1000/(uo*d)=1.765 N/mm²
Maximum punching shear resistance

vRdmax=0.5*v*fcd=4.973 N/mm²
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.
MAIN REINFORCEMENT
**IN X-DIRECTION:**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Spacing: 70 mm
- Total number of bars: 26
- Total area required: 2936 mm²
- Total area provided: 2941 mm²

**SUMMARY**
- Spacing: 70 mm
- Total number of bars: 26
- Total area required: 2936 mm²
- Total area provided: 2941 mm²

MAIN REINFORCEMENT
**IN Y-DIRECTION:**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm
- Spacing: 125 mm
- Total number of bars: 16
- Area reqd.@ critical section: 3213 mm²
- Area prov.@ critical section: 3217 mm²

**SUMMARY**
- Spacing: 125 mm
- Total number of bars: 16
- Area reqd.@ critical section: 3213 mm²
- Area prov.@ critical section: 3217 mm²

DISTRIBUTION
**REINFORCEMENT**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 20 mm

**IN X-DIRECTION:**
- In top:
  - Number of bars: 9
  - Spacing of bars: 205 mm
  - Area required: 1530 mm²/m
  - Area provided: 1532 mm²/m

**IN Y-DIRECTION:**
- In top:
  - Number of bars: 10
  - Spacing of bars: 205 mm
  - Area required: 1530 mm²/m
  - Area provided: 1532 mm²/m

SIDE BARS SUMMARY
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 12
- Spacing of bars: 250 mm

QUANTITIES
- Total volume of concrete: 3.271 m³
- Total area of formwork: 7.178 m²
Location: Cap for heavily-loaded column

Pilecap for 3 piles

Design to EC2 Part 1-1.

Pilecap shaped as shown with main reinforcement parallel to principal axes.

Use ultimate values for load and moment.

Applied moment in the Y-direction only.

Total design load on column $N=4000$ kN
Column dimension $cx=500$ mm
Column dimension $cy=500$ mm
Design BM at bottom of column $My'=0$ kNm
Partial safety factor for steel $\gamma_{ams}=1.15$
Partial safety factor for conc. $\gamma_{amc}=1.5$
Nominal pile diameter $PileD=500$ mm
Spacing of pile ctrs in X-dir. $AX=1500$ mm
Dist. from pile face to cap edge $olap=300$ mm
Spacing of pile ctrs in Y-dir. $AY=1299$ mm
Distance $AY'$ to centre of column $AY'=866$ mm
Max cap length in X-direction $lx=2600$ mm
Min cap dimension in X-direction $BX=1100$ mm
Max cap breadth in Y-direction $ly=2400$ mm
Min cap dimension in Y-direction $BY=1050$ mm
Char yield strength of reinft. $f_{yk}=500$ N/mm$^2$
Cover to underside of main bars $Cover=150$ mm
Side and top cover to main bars $Scvr=75$ mm
Chosen depth of pilecap $h=1080$ mm

Design of main steel in X-direction

Main steel diameter $diaX=25$ mm
Main steel spacing $SpcX=300$ mm

Design of main steel in Y-direction

Main steel diameter $diaY=12$ mm
Main steel spacing $SpcY=65$ mm
Distribution steel

- Distribution steel diameter: LSize=25 mm
- Spacing of bars: LSpac=300 mm
- Area of dist steel provided: A2=PI*LSize^2*250/LSpac=1636 mm²/m

Check shear resistance in X-direction

Shear force due to ultimate load: VEd=N2=1333 kN
Shear resistance: VRdc=beta'*CRdc*ks*vRdc*bw*d/1000 =2701 kN
Modified shear resistance: VRdc=VRdm=2982 kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>1333 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>2982 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Case 1: Check plane passing through single pile on X-X axis
Shear force due to ultimate load: VEd=N1+My'*1000/AY=1333 kN
Shear resistance: VRdc=beta'*CRdc*ks*vRdc*bw*d/1000 =2315 kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>1333 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>2315 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Case 2: Check plane passing through twin-piles
Shear force due to ultimate load: VEd=2*N2+My'*1000/AY=2667 kN
Shear resistance: VRdc=beta'*CRdc*ks*vRdc*bw*d/1000 =2981 kN
Modified shear resistance: VRdc=VRdm=3305 kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>2667 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>3305 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
- Eccentricity factor β: beta=1
- Shear stress at column perimeter: vEd=beta*VEd*1000/(uo*d)=2.225 N/mm²
- Maximum punching shear resistance: vRdmax=0.5*v*fcd=5.581 N/mm²

Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.
**MAIN REINFORCEMENT**

**IN X-DIRECTION:**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 25 mm
- Spacing: 300 mm
- Total number of bars: 8
- Total area required: 3888 mm²
- Total area provided: 3927 mm²

**SUMMARY**

**MAIN REINFORCEMENT**

**IN Y-DIRECTION:**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Spacing: 65 mm
- Total number of bars: 38
- Area reqd.@ critical section: 4212 mm²
- Area prov.@ critical section: 4298 mm²

**SUMMARY**

**DISTRIBUTION REINFORCEMENT**

**IN X-DIRECTION:**
- Number of bars: 8
- Spacing of bars: 300 mm
- Area required: 1620 mm²/m
- Area provided: 1636 mm²/m

**IN Y-DIRECTION:**
- Number of bars: 10
- Spacing of bars: 300 mm
- Area required: 1620 mm²/m
- Area provided: 1636 mm²/m

**SIDE BARS SUMMARY**

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 12
- Spacing of bars: 250 mm

**QUANTITIES**

- Total volume of concrete: 5.646 m³
- Total area of formwork: 9.6 m²
Location: Cap for lightly-loaded column

Pilecap for 3 piles

Design to EC2 Part 1-1.

Pilecap shaped as shown with main reinforcement parallel to principal axes.

Use ultimate values for load and moment.

Applied moment in the Y-direction only.

Total design load on column \( N = 150 \text{ kN} \)
Column dimension \( cx = 400 \text{ mm} \)
Column dimension \( cy = 400 \text{ mm} \)
Design BM at bottom of column \( My' = 0 \text{ kNm} \)
Partial safety factor for steel \( \gamma_{Ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{Mc} = 1.5 \)
Nominal pile diameter \( \text{PileD} = 250 \text{ mm} \)
Spacing of pile ctrs in X-dir. \( AX = 750 \text{ mm} \)
Dist. from pile face to cap edge \( olap = 150 \text{ mm} \)
Spacing of pile ctrs in Y-dir. \( AY = 650 \text{ mm} \)
Distance \( AY' \) to centre of column \( AY' = 433 \text{ mm} \)
Max cap length in X-direction \( lx = 1300 \text{ mm} \)
Min cap dimension in X-direction \( BX = 550 \text{ mm} \)
Max cap breadth in Y-direction \( ly = 1200 \text{ mm} \)
Min cap dimension in Y-direction \( BY = 500 \text{ mm} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Cover to underside of main bars \( \text{Cover} = 150 \text{ mm} \)
Side and top cover to main bars \( \text{Scvr} = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 300 \text{ mm} \)

Design of main steel in X-direction

Main steel diameter \( \text{diaX} = 12 \text{ mm} \)
Main steel spacing \( \text{SpcX} = 250 \text{ mm} \)

Design of main steel in Y-direction

Main steel diameter \( \text{diaY} = 12 \text{ mm} \)
Main steel spacing \( \text{SpcY} = 185 \text{ mm} \)
Distribution steel

- Distribution steel diameter: LSize = 12 mm
- Spacing of bars: LSpac = 250 mm
- Area of dist steel provided: A2 = PI*LSize^2*250/LSpac = 452.4 mm²/m

Check shear resistance in X-direction

- Shear force due to ultimate load: VEd = 50 kN
- Shear resistance: VRdc = beta' * CRdc * ks * vRdc * bw * d / 1000
- Modified shear resistance: VRdc = VRdm = 267.9 kN

SHEAR SUMMARY

| Shear force | 50 kN |
| Shear resistance | 267.9 kN |

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Case 1: Check plane passing through single pile on X-X axis
- Shear force due to ultimate load: VEd = 50 kN
- Shear resistance: VRdc = 111.1 kN

SHEAR SUMMARY

| Shear force | 50 kN |
| Shear resistance | 111.1 kN |

Hence section is satisfactory, no shear reinforcement is required.

Case 2: Check plane passing through twin-piles
- Shear force due to ultimate load: VEd = 100 kN
- Shear enhancement: beta' = 4
- Shear resistance: VRdc = 384.4 kN

SHEAR SUMMARY

| Shear force | 100 kN |
| Shear resistance | 384.4 kN |

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
- Eccentricity factor β: beta = 1
- Shear stress at column perimeter: vEd = beta * VEd / (uo * d) = 0.7102 N/mm²
- Maximum punching shear resistance: vRdmax = 0.5 * v * fcd = 5.581 N/mm²
- Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.
Check punching shear at 2d:
Shear stress at control perimeter $v_{Ed}=\beta*V_{Ed}*1000/(u_1*d)=0.4383$ N/mm²
Design punching shear resistance $v_{Rdc}=(0.18/gamc)*k*(100*p_1 *f_{ck})^{0.333}=0.5701$ N/mm²
Enhanced punching shear resist. $v_{Rdc}=\text{senh} \times v_{Rdc}=0.9526$ N/mm²

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT IN X-DIRECTION:</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
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<tbody>
<tr>
<td>SUMMARY</td>
<td>Diameter of bars</td>
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<td>Spacing</td>
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<td>Total number of bars</td>
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<td>Total area required</td>
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<tr>
<td>Total area provided</td>
<td>565.5 mm²</td>
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<table>
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<tr>
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<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>185 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Area reqd. @ critical section</td>
<td>492.4 mm²</td>
<td></td>
</tr>
<tr>
<td>Area prov. @ critical section</td>
<td>678.6 mm²</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DISTRIBUTION REINFORCEMENT IN X-DIRECTION:</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>In top:</td>
<td>Number of bars</td>
<td>5</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>450 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>452.4 mm²/m</td>
<td></td>
</tr>
</tbody>
</table>

| IN Y-DIRECTION:                           | Number of bars          | 6         |
| Spacing of bars                           | 250 mm                  |
| Area required                              | 450 mm²/m               |
| Area provided                              | 452.4 mm²/m             |

<table>
<thead>
<tr>
<th>SIDE BARS SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Total number of bars</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
<td></td>
</tr>
</tbody>
</table>

| QUANTITIES                                 | Total volume of concrete | 0.3893 m³ |
|                                           | Total area of formwork   | 1.331 m²  |
Location: Typical pilecap

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for four piles

Designed using bending theory in accordance with BS8110.

Pile cap assumed square (i.e. $l_x = l_y$) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres.

Total ultimate load on column $N = 5600$ kN
Nominal pile diameter $Pile_D = 450$ mm
Column dimension $cx = 500$ mm
Column dimension $cy = 500$ mm
Spc.of pile ctrs in X-direction $AX = 1350$ mm
Overhang: pile face to cap edge $olap = 150$ mm
Length of pile cap $l_x = 2100$ mm
Spc.of pile ctrs in Y-direction $AY = 1350$ mm
Breadth of pile cap $l_y = 2100$ mm
Concrete grade $fcu = 35$ N/mm$^2$
Steel strength $fy = 500$ N/mm$^2$
Cover to underside of main bars $Cover = 150$ mm
Side and top cover to main bars $Scvr = 75$ mm
Chosen depth of pilecap $h = 900$ mm

Design of main steel in X-direction

Main steel diameter $dia_X = 16$ mm
Main steel spacing $Spc_X = 100$ mm

Design of main steel in Y-direction

Main steel diameter $dia_Y = 12$ mm
Main steel spacing $Spc_Y = 55$ mm

Distribution steel

Diameter of bar $LSize = 16$ mm
Spacing of bars $LSpac = 170$ mm
### MAIN REINFORCEMENT

#### IN X-DIRECTION:

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Spacing</th>
<th>Total number of bars</th>
<th>Total area required</th>
<th>Total area provided</th>
<th>Volume of steel</th>
<th>Weight of steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 N/mm²</td>
<td>16 mm</td>
<td>100 mm</td>
<td>20</td>
<td>3946.6 mm²</td>
<td>4021.2 mm²</td>
<td>9160.4 cm³</td>
<td>71.909 kg</td>
</tr>
</tbody>
</table>

#### SUMMARY

- **Spacing**: 100 mm
- **Total number of bars**: 20
- **Total area required**: 3946.6 mm²
- **Total area provided**: 4021.2 mm²
- **Volume of steel**: 9160.4 cm³
- **Weight of steel**: 71.909 kg

### MAIN REINFORCEMENT

#### IN Y-DIRECTION:

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Spacing</th>
<th>Total number of bars</th>
<th>Total area required</th>
<th>Total area provided</th>
<th>Volume of steel</th>
<th>Weight of steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 N/mm²</td>
<td>12 mm</td>
<td>55 mm</td>
<td>36</td>
<td>4035.1 mm²</td>
<td>4071.5 mm²</td>
<td>9779.8 cm³</td>
<td>76.771 kg</td>
</tr>
</tbody>
</table>

#### SUMMARY

- **Spacing**: 55 mm
- **Total number of bars**: 36
- **Total area required**: 4035.1 mm²
- **Total area provided**: 4071.5 mm²
- **Volume of steel**: 9779.8 cm³
- **Weight of steel**: 76.771 kg

### DISTRIBUTION REINFORCEMENT

#### SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Spacing</th>
<th>Number of bars</th>
<th>Area required</th>
<th>Area provided</th>
<th>Volume of steel</th>
<th>Weight of steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 N/mm²</td>
<td>16 mm</td>
<td></td>
<td>12</td>
<td>1170 mm²/m</td>
<td>1182.7 mm²/m</td>
<td>4704.8 cm³</td>
<td>36.933 kg</td>
</tr>
</tbody>
</table>

#### IN X-DIRECTION:

- **In top**: Number of bars 12, Spacing of bars 170 mm, Area required 1170 mm²/m, Area provided 1182.7 mm²/m, Volume of steel 4704.8 cm³, Weight of steel 36.933 kg

#### IN Y-DIRECTION:

- **In top**: Number of bars 12, Spacing of bars 170 mm, Area required 1170 mm²/m, Area provided 1182.7 mm²/m, Volume of steel 4704.8 cm³, Weight of steel 36.933 kg

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Total number of bars</th>
<th>Spacing of bars</th>
<th>Volume of steel</th>
<th>Weight of steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>500 N/mm²</td>
<td>12 mm</td>
<td>6</td>
<td>250 mm</td>
<td>2989.2 cm³</td>
<td>23.465 kg</td>
</tr>
</tbody>
</table>

### QUANTITIES

- **Total concrete volume**: 3.969 m³
- **Total area of formwork**: 7.56 m²
- **Total volume of steel**: 31339 cm³
- **Total weight of steel**: 246.01 kg
- **Approx. self-weight of pile cap**: 133.36 kN
- **Approx. total load on each pile**: 1434 kN
Location: Large, heavily-loaded pilecap with mild steel reinf.

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for four piles

Designed using bending theory in accordance with BS8110.

Pile cap assumed square (i.e. \( lx=ly \)) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres.

---

Total ultimate load on column \( N=10000 \) kN
Nominal pile diameter \( PileD=450 \) mm
Column dimension \( cx=500 \) mm
Column dimension \( cy=500 \) mm
Spc.of pile ctrs in X-direction \( AX=2000 \) mm
Overhang: pile face to cap edge \( olap=400 \) mm
Length of pile cap \( lx=3250 \) mm
Spc.of pile ctrs in Y-direction \( AY=2000 \) mm
Breadth of pile cap \( ly=3250 \) mm
Concrete grade \( fcu=25 \) N/mm²
Steel strength \( fy=250 \) N/mm²
Cover to underside of main bars \( Cover=150 \) mm
Side and top cover to main bars \( Scvr=75 \) mm
Chosen depth of pilecap \( h=1910 \) mm

Design of main steel in X-direction

Main steel diameter \( diaX=20 \) mm
Main steel spacing \( SpcX=65 \) mm

Design of main steel in Y-direction

Main steel diameter \( diaY=20 \) mm
Main steel spacing \( SpcY=65 \) mm

Distribution steel

Diameter of bar \( LSize=32 \) mm
Spacing of bars \( LSpac=175 \) mm
### Main Reinforcement

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>65 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>48</td>
</tr>
<tr>
<td>Total area required</td>
<td>14898 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>15080 mm²</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>74946 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>588.32 kg</td>
</tr>
</tbody>
</table>

### Main Reinforcement

**IN X-DIRECTION:**

**SUMMARY**

<table>
<thead>
<tr>
<th>Diameter of bars</th>
<th>20 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing</td>
<td>65 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>48</td>
</tr>
<tr>
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</tr>
<tr>
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<tr>
<td>Volume of steel</td>
<td>74946 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>588.32 kg</td>
</tr>
</tbody>
</table>

### Main Reinforcement

**IN Y-DIRECTION:**

**SUMMARY**

<table>
<thead>
<tr>
<th>Diameter of bars</th>
<th>20 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing</td>
<td>65 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>48</td>
</tr>
<tr>
<td>Total area required</td>
<td>14898 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>15080 mm²</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>74946 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>588.32 kg</td>
</tr>
</tbody>
</table>

### Distribution Reinforcement

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>32 mm</td>
</tr>
</tbody>
</table>

**IN X-DIRECTION:**

- **In top:**
  - Number of bars: 18
  - Spacing of bars: 175 mm
  - Area required: 4584 mm²/m
  - Area provided: 4595.7 mm²/m
  - Volume of steel: 44877 cm³
  - Weight of steel: 352.28 kg

**IN Y-DIRECTION:**

- **In top:**
  - Number of bars: 18
  - Spacing of bars: 175 mm
  - Area required: 4584 mm²/m
  - Area provided: 4595.7 mm²/m
  - Volume of steel: 44877 cm³
  - Weight of steel: 352.28 kg

### Side Bars Summary

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>14</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>10696 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>83.961 kg</td>
</tr>
</tbody>
</table>

### Quantities

- Total concrete volume: 20.174 m³
- Total area of formwork: 24.83 m²
- Total volume of steel: 250341 cm³
- Total weight of steel: 1965.2 kg

- Approx. self-weight of pile cap: 677.86 kN
- Approx. total load on each pile: 2670 kN
Location: Small cap with closely-spaced piles.

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \)
Bottom layer of main steel is assumed to run in X-direction.

**Pilecap for four piles**

Designed using bending theory in accordance with BS8110.

Pile cap assumed square (i.e. \( l_x = l_y \)) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres.

---

Total ultimate load on column \( N = 5600 \) kN
Nominal pile diameter \( PileD = 250 \) mm
Column dimension \( cx \) \( cx = 350 \) mm
Column dimension \( cy \) \( cy = 350 \) mm
Spc. of pile ctrs in X-direction \( AX = 500 \) mm
Overhang: pile face to cap edge \( olap = 150 \) mm
Length of pile cap \( lx = 1050 \) mm
Spc. of pile ctrs in Y-direction \( AY = 500 \) mm
Breadth of pile cap \( ly = 1050 \) mm
Concrete grade \( f_{cu} = 60 \) N/mm\(^2\)
Steel strength \( f_y = 500 \) N/mm\(^2\)
Cover to underside of main bars \( Cover = 150 \) mm
Side and top cover to main bars \( Scvr = 75 \) mm
Chosen depth of pilecap \( h = 990 \) mm

**Design of main steel in X-direction**

Main steel diameter \( diaX = 12 \) mm
Main steel spacing \( SpcX = 80 \) mm

**Design of main steel in Y-direction**

Main steel diameter \( diaY = 12 \) mm
Main steel spacing \( SpcY = 80 \) mm

**Distribution steel**

Diameter of bar \( LSize = 16 \) mm
Spacing of bars \( LSpac = 155 \) mm
### MAIN REINFORCEMENT

**IN X-DIRECTION:**
- **Summary:**
  - Characteristic strength: 500 N/mm²
  - Diameter of bars: 12 mm
  - Spacing: 80 mm
  - Total number of bars: 12
  - Total area required: 1351.4 mm²
  - Total area provided: 1357.2 mm²
  - Volume of steel: 2513.5 cm³
  - Weight of steel: 19.731 kg

**IN Y-DIRECTION:**
- **Summary:**
  - Characteristic strength: 500 N/mm²
  - Diameter of bars: 12 mm
  - Spacing: 80 mm
  - Total number of bars: 12
  - Total area required: 1351.4 mm²
  - Total area provided: 1357.2 mm²
  - Volume of steel: 2513.5 cm³
  - Weight of steel: 19.731 kg

### DISTRIBUTION REINFORCEMENT

**SUMMARY:**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm

**IN X-DIRECTION:**
- **In top:**
  - Number of bars: 6
  - Spacing of bars: 155 mm
  - Area required: 1287 mm²/m
  - Area provided: 1297.2 mm²/m
  - Volume of steel: 1085.7 cm³
  - Weight of steel: 8.523 kg

**IN Y-DIRECTION:**
- **In top:**
  - Number of bars: 6
  - Spacing of bars: 155 mm
  - Area required: 1287 mm²/m
  - Area provided: 1297.2 mm²/m
  - Volume of steel: 1085.7 cm³
  - Weight of steel: 8.523 kg

### SIDE BARS SUMMARY

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 8
- Spacing of bars: 250 mm
- Volume of steel: 1986 cm³
- Weight of steel: 15.59 kg

### QUANTITIES

- Total concrete volume: 1.0915 m³
- Total area of formwork: 4.158 m²
- Total volume of steel: 9184.4 cm³
- Total weight of steel: 72.098 kg

- Approx. self-weight of pile cap: 36.674 kN
- Approx. total load on each pile: 1410 kN
Location: Ex1 - Typical pilecap

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for four piles

- Designed using bending theory to EC2 Part 1-1.
- Pilecap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about the pile centres.

Use ultimate values for load and moment.

Total design load on column \( N = 3600 \text{ kN} \)
Column dimension \( cx \) \( cx = 500 \text{ mm} \)
Column dimension \( cy \) \( cy = 500 \text{ mm} \)
Nominal pile diameter \( \text{PileD} = 450 \text{ mm} \)
Partial safety factor for steel \( gams = 1.15 \)
Partial safety factor for conc. \( gamc = 1.5 \)
Spacing of pile ctrs in X-dir. \( AX = 1350 \text{ mm} \)
Overhang: pile face to cap edge \( olap = 150 \text{ mm} \)
Length of pile cap \( lx = 2100 \text{ mm} \)
Spacing of pile ctrs in Y-dir. \( AY = 1350 \text{ mm} \)
Breadth of pile cap \( ly = 2100 \text{ mm} \)
Char yield strength of reinf. \( fyk = 500 \text{ N/mm}^2 \)
Cover to underside of main bars \( \text{Cover} = 150 \text{ mm} \)
Side and top cover to main bars \( \text{Scvr} = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 1000 \text{ mm} \)

Design of main steel in X-direction

- Main steel diameter \( \text{diaX} = 12 \text{ mm} \)
- Main steel spacing \( \text{SpcX} = 70 \text{ mm} \)

Design of main steel in Y-direction

- Main steel diameter \( \text{diaY} = 12 \text{ mm} \)
- Main steel spacing \( \text{SpcY} = 70 \text{ mm} \)

Distribution steel

- Distribution steel diameter \( \text{LSize} = 12 \text{ mm} \)
- Spacing of bars \( \text{LSpac} = 75 \text{ mm} \)
- Area of dist.steel provided \( A_2 = \pi \times \text{LSize}^2 \times 250 / \text{LSpac} = 1508 \text{ mm}^2 / \text{m} \)
Check shear resistance in X-direction

Shear force due to ultimate load \( V_{Ed} = \frac{N}{2} + M_x' \times 1000/AX = 1800 \text{ kN} \)
Shear resistance \( V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d / 1000 \)
\[ = 2262 \text{ kN} \]
Modified shear resistance \( V_{Rd} = V_{Rdm} = 2545 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>1800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>2545 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = \frac{N}{2} + M_y' \times 1000/AY = 1800 \text{ kN} \)
Shear resistance \( V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d / 1000 \)
\[ = 2246 \text{ kN} \]
Modified shear resistance \( V_{Rd} = V_{Rdm} = 2517 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>1800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>2517 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor \( \beta \)
\( \beta = 1 \)
Shear stress at column perimeter \( v_{Ed} = \beta \times V_{Ed} \times 1000/(u_o \times d) = 2.163 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5 \times v \times f_{cd} = 5.581 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT IN X-DIRECTION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>70 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>28</td>
</tr>
<tr>
<td>Total area required</td>
<td>3150 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>3167 mm²</td>
</tr>
</tbody>
</table>

**MAIN REINFORCEMENT IN Y-DIRECTION**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>70 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>28</td>
</tr>
<tr>
<td>Total area required</td>
<td>3150 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>3167 mm²</td>
</tr>
</tbody>
</table>
## DISTRIBUTION REINFORCEMENT

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
</tbody>
</table>

### IN X-DIRECTION:

<table>
<thead>
<tr>
<th>In top:</th>
<th>Number of bars</th>
<th>26</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing of bars</td>
<td>75 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>1500 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>1508 mm²/m</td>
<td></td>
</tr>
</tbody>
</table>

### IN Y-DIRECTION:

<table>
<thead>
<tr>
<th>In top:</th>
<th>Number of bars</th>
<th>26</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing of bars</td>
<td>75 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>1500 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>1508 mm²/m</td>
<td></td>
</tr>
</tbody>
</table>

## SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>8</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

## QUANTITIES

| Total concrete volume   | 4.41 m³  |
| Total area of formwork  | 8.4 m²   |
Location: Ex2 - Large, heavily-loaded pilecap with mild steel reinf.

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for four piles

- Designed using bending theory to EC2 Part 1-1.
- Pilecap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about the pile centres.

Use ultimate values for load and moment.

Total design load on column  \( N=5000 \text{ kN} \)
Column dimension \( cx=500 \text{ mm} \)
Column dimension \( cy=500 \text{ mm} \)
Nominal pile diameter \( \text{PileD}=450 \text{ mm} \)
Partial safety factor for steel \( gams=1.15 \)
Partial safety factor for conc. \( gamc=1.5 \)
Spacing of pile ctrs in X-dir. \( AX=2000 \text{ mm} \)
Overhang: pile face to cap edge \( olap=400 \text{ mm} \)
Length of pile cap \( lx=3250 \text{ mm} \)
Spacing of pile ctrs in Y-dir. \( AY=2000 \text{ mm} \)
Breadth of pile cap \( ly=3250 \text{ mm} \)
Char yield strength of reinft. \( fyk=500 \text{ N/mm}^2 \)
Cover to underside of main bars \( \text{Cover}=150 \text{ mm} \)
Side and top cover to main bars \( \text{Scvr}=75 \text{ mm} \)
Chosen depth of pilecap \( h=1440 \text{ mm} \)

Design of main steel in X-direction

<table>
<thead>
<tr>
<th>Main steel diameter</th>
<th>diaX=16 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main steel spacing</td>
<td>SpcX=90 mm</td>
</tr>
</tbody>
</table>

Design of main steel in Y-direction

<table>
<thead>
<tr>
<th>Main steel diameter</th>
<th>diaY=16 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main steel spacing</td>
<td>SpcY=90 mm</td>
</tr>
</tbody>
</table>

Distribution steel

<table>
<thead>
<tr>
<th>Distribution steel diameter</th>
<th>LSize=20 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing of bars</td>
<td>LSpac=145 mm</td>
</tr>
<tr>
<td>Area of dist.steel provided</td>
<td>( A2=\pi\times LSize^2\times250/LSpac=2167 \text{ mm}^2/\text{m} )</td>
</tr>
</tbody>
</table>
Check shear resistance in X-direction

Shear force due to ultimate load \( V_{E,d} = \frac{N}{2} + M_x' \cdot \frac{1000}{A_x} = 2500 \text{ kN} \)

Shear resistance \( V_{R,d,c} = \beta' \cdot C_{R,d,c} \cdot k_s \cdot v_{R,d,c} \cdot b_w \cdot d \cdot 1000 = 4219 \text{ kN} \)

Modified shear resistance \( V_{R,d} = V_{R,d,m} = 4349 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force (kN)</th>
<th>Shear resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2500</td>
<td>4349</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{E,d} = \frac{N}{2} + M_y' \cdot \frac{1000}{A_y} = 2500 \text{ kN} \)

Shear resistance \( V_{R,d,c} = \beta' \cdot C_{R,d,c} \cdot k_s \cdot v_{R,d,c} \cdot b_w \cdot d \cdot 1000 = 4191 \text{ kN} \)

Modified shear resistance \( V_{R,d} = V_{R,d,m} = 4306 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force (kN)</th>
<th>Shear resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2500</td>
<td>4306</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:

Eccentricity factor \( \beta \) \( \beta = 1 \)

Shear stress at column perimeter \( \sigma_{Ed} = \beta \cdot V_{E,d} \cdot \frac{1000}{u_o \cdot d} = 1.975 \text{ N/mm}^2 \)

Maximum punching shear resistance \( \sigma_{R,d,max} = 0.5 \cdot \sigma_{f,c} = 5.581 \text{ N/mm}^2 \)

Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

Check punching shear at 2d:

Shear stress at control perimeter \( \sigma_{Ed} = \beta \cdot V_{E,d} \cdot \frac{1000}{u_1 \cdot d} = 0.6735 \text{ N/mm}^2 \)

Design punching shear resistance \( \sigma_{R,d,c} = (0.18 / \gamma_{mc}) \cdot k \cdot (100 \cdot p_1 \cdot f_{ck})^{0.333} = 0.3163 \text{ N/mm}^2 \)

Enhancement factor to be used \( \gamma_{enh} = 4.117 \)

Enhanced punching shear resist. \( \sigma_{R,d} = \gamma_{enh} \cdot \sigma_{R,d,c} = 1.302 \text{ N/mm}^2 \)

Design punching shear resistance \( \sigma_{R,d} = \gamma_{enh} \cdot 0.035 \cdot k^1.5 \cdot f_{ck}^0.5 \)

MAIN REINFORCEMENT

<table>
<thead>
<tr>
<th>Characteristic strength (N/mm²)</th>
<th>Diameter of bars (mm)</th>
<th>Spacing (mm)</th>
<th>Total number of bars</th>
<th>Total area required (mm²)</th>
<th>Total area provided (mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN X-DIRECTION:</td>
<td>500</td>
<td>16</td>
<td>90</td>
<td>35</td>
<td>7020</td>
</tr>
<tr>
<td>SUMMARY</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>IN Y-DIRECTION:</td>
<td>500</td>
<td>16</td>
<td>90</td>
<td>35</td>
<td>7020</td>
</tr>
</tbody>
</table>
### DISTRIBUTION REINFORCEMENT SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
</tbody>
</table>

#### IN X-DIRECTION:

**In top:**
- Number of bars: 22
- Spacing of bars: 145 mm
- Area required: 2160 mm²/m
- Area provided: 2167 mm²/m

#### IN Y-DIRECTION:

**In top:**
- Number of bars: 22
- Spacing of bars: 145 mm
- Area required: 2160 mm²/m
- Area provided: 2167 mm²/m

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>10</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

- Total concrete volume: 15.21 m³
- Total area of formwork: 18.72 m²
Location: Ex3 - Small cap with closely-spaced piles.

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for four piles

- Designed using bending theory to EC2 Part 1-1.
- Pilecap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about the pile centres.

Use ultimate values for load and moment.

Total design load on column N=3000 kN
Column dimension cx cx=350 mm
Column dimension cy cy=350 mm
Nominal pile diameter PileD=250 mm
Partial safety factor for steel gams=1.15
Partial safety factor for conc. gamc=1.5
Spacing of pile ctrs in X-dir. AX=500 mm
Overhang: pile face to cap edge olap=150 mm
Length of pile cap lx=1050 mm
Spacing of pile ctrs in Y-dir. AY=500 mm
Breadth of pile cap ly=1050 mm
Char yield strength of reinft. fyk=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=990 mm

Design of main steel in X-direction

Main steel diameter diaX=20 mm
Main steel spacing SpcX=175 mm

Design of main steel in Y-direction

Main steel diameter diaY=16 mm
Main steel spacing SpcY=110 mm

Distribution steel

Distribution steel diameter LSize=20 mm
Spacing of bars LSpac=180 mm
Area of dist.steel provided A2=\(\pi \times \text{LSize}^2 \times \frac{250}{\text{LSpac}} = 1745 \text{ mm}^2/\text{m}\)
Check shear resistance in X-direction

Shear force due to ultimate load \( V_{Ed} = \frac{N}{2} + M_x' \times \frac{1000}{A_X} = 1500 \text{ kN} \)

Shear enhancement \( \beta' = 4 \)

Shear resistance
\[
V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d / 1000 \\
= 1379 \text{ kN}
\]

Modified shear resistance
\( V_{Rdc} = V_{Rdm} = 1571 \text{ kN} \)

**SHEAR SUMMARY**

- Shear force: 1500 kN
- Shear resistance: 1571 kN

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = \frac{N}{2} + M_y' \times \frac{1000}{A_Y} = 1500 \text{ kN} \)

Shear enhancement \( \beta' = 4 \)

Shear resistance
\[
V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d / 1000 \\
= 1346 \text{ kN}
\]

Modified shear resistance
\( V_{Rdc} = V_{Rdm} = 1545 \text{ kN} \)

**SHEAR SUMMARY**

- Shear force: 1500 kN
- Shear resistance: 1545 kN

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:

- Eccentricity factor \( \beta \) \( \beta = 1 \)

Shear stress at column perimeter
\[
v_{Ed} = \beta \times V_{Ed} \times 1000 / (u_0 \times d) = 2.639 \text{ N/mm}^2
\]

Maximum punching shear resistance
\( v_{Rdmax} = 0.5 \times v \times f_{cd} = 8 \text{ N/mm}^2 \)

Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

- Characteristic strength: 500 N/mm²
- Diameter of bars: 20 mm
- Spacing: 175 mm
- Total number of bars: 6
- Total area required: 1823 mm²
- Total area provided: 1885 mm²

**IN Y-DIRECTION:**

- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm
- Spacing: 110 mm
- Total number of bars: 9
- Total area required: 1778 mm²
- Total area provided: 1810 mm²
### DISTRIBUTION REINFORCEMENT

**SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 20 mm

**IN X-DIRECTION:**
- **In top:**
  - Number of bars: 5
  - Spacing of bars: 180 mm
  - Area required: 1736 mm²/m
  - Area provided: 1745 mm²/m

**IN Y-DIRECTION:**
- **In top:**
  - Number of bars: 5
  - Spacing of bars: 180 mm
  - Area required: 1736 mm²/m
  - Area provided: 1745 mm²/m

### SIDE BARS SUMMARY

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 8
- Spacing of bars: 250 mm

### QUANTITIES

- Total concrete volume: 1.091 m³
- Total area of formwork: 4.158 m²
Location: Typical pilecap

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$
Bottom layer of main steel is assumed to run in X-direction.

Total ultimate load on column $N=7000$ kN
Column dimension $cx=500$ mm
Column dimension $cy=500$ mm
Nominal pile diameter $PileD=450$ mm
Spec.min.spacing of pile centres $spc=1350$ mm
Spec.of pile ctrs in X-direction $AX=1910$ mm
Overhang: pile face to cap edge $olap=150$ mm
Length of pile cap $lx=2660$ mm
Spec.of pile ctrs in Y-direction $AY=1910$ mm
Breadth of pile cap $ly=2660$ mm
Concrete grade $fcu=35$ N/mm$^2$
Steel strength $fy=500$ N/mm$^2$
Cover to underside of main bars $Cover=150$ mm
Side and top cover to main bars $Scvr=75$ mm
Chosen depth of pilecap $h=1420$ mm

Design of main steel in X-direction
Main steel diameter $diaX=16$ mm
Main steel spacing $SpcX=100$ mm

Design of main steel in Y-direction
Main steel diameter $diaY=16$ mm
Main steel spacing $SpcY=100$ mm

Distribution steel
Diameter of bar $LSize=20$ mm
Spacing of bars $LSpac=170$ mm
<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Spacing</th>
<th>Total number of bars</th>
<th>Total area required</th>
<th>Total area provided</th>
<th>Volume of steel</th>
<th>Weight of steel</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>X-DIRECTION</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>500 N/mm²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td></td>
<td>16 mm</td>
<td>100 mm</td>
<td>25</td>
<td>4910.4 mm²</td>
<td>5026.5 mm²</td>
<td>19493 cm³</td>
<td>153.02 kg</td>
</tr>
<tr>
<td><strong>Y-DIRECTION</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>500 N/mm²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td></td>
<td>16 mm</td>
<td>100 mm</td>
<td>25</td>
<td>4910.4 mm²</td>
<td>5026.5 mm²</td>
<td>19493 cm³</td>
<td>153.02 kg</td>
</tr>
<tr>
<td><strong>SIDE BARS</strong></td>
<td></td>
<td>12 mm</td>
<td>250 mm</td>
<td>10</td>
<td></td>
<td></td>
<td>6259.9 cm³</td>
<td>49.141 kg</td>
</tr>
<tr>
<td><strong>DISTRIBUTION</strong></td>
<td></td>
<td>500 N/mm²</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td></td>
<td>20 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>IN X-DIRECTION</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>In top:</td>
<td></td>
<td>15</td>
<td>170 mm</td>
<td></td>
<td>1846 mm²/m</td>
<td>1848 mm²/m</td>
<td>11828 cm³</td>
<td>92.851 kg</td>
</tr>
<tr>
<td><strong>IN Y-DIRECTION</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>In top:</td>
<td></td>
<td>15</td>
<td>170 mm</td>
<td></td>
<td>1846 mm²/m</td>
<td>1848 mm²/m</td>
<td>11828 cm³</td>
<td>92.851 kg</td>
</tr>
<tr>
<td><strong>QUANTITIES</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total concrete volume</td>
<td></td>
<td>10.047 m³</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total area of formwork</td>
<td></td>
<td>15.109 m²</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total volume of steel</td>
<td></td>
<td>68902 cm³</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total weight of steel</td>
<td></td>
<td>540.88 kg</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Approx. self-weight of pile cap: 337.59 kN
Approx. total load on each pile: 1468 kN
Location: Heavily-loaded pilecap reinforced with mild steel

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15. Bottom layer of main steel is assumed to run in X-direction.

Pilecap for five piles

Designed using bending theory in accordance with BS8110.

Pile cap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres (i.e. chain lines in adjoining diagram).

Total ultimate load on column  \(N=12500 \text{ kN}\)
Column dimension cx  \(cx=500 \text{ mm}\)
Column dimension cy  \(cy=500 \text{ mm}\)
Nominal pile diameter  \(\text{PileD}=450 \text{ mm}\)
Spec.min.spacing of pile centres  \(\text{spc}=1350 \text{ mm}\)
Spec.of pile ctrs in X-direction  \(AX=1910 \text{ mm}\)
Overhang: pile face to cap edge  \(\text{olap}=725 \text{ mm}\)
Length of pile cap  \(lx=3810 \text{ mm}\)
Spec.of pile ctrs in Y-direction  \(AY=1910 \text{ mm}\)
Breadth of pile cap  \(ly=3810 \text{ mm}\)
Concrete grade  \(fcu=35 \text{ N/mm}^2\)
Steel strength  \(fy=250 \text{ N/mm}^2\)
Cover to underside of main bars  \(\text{Cover}=150 \text{ mm}\)
Side and top cover to main bars  \(\text{Scvr}=75 \text{ mm}\)
Chosen depth of pilecap  \(h=1740 \text{ mm}\)

**Design of main steel in X-direction**

Main steel diameter  \(\text{diaX}=32 \text{ mm}\)
Main steel spacing  \(\text{SpcX}=190 \text{ mm}\)

**Design of main steel in Y-direction**

Main steel diameter  \(\text{diaY}=32 \text{ mm}\)
Main steel spacing  \(\text{SpcY}=190 \text{ mm}\)

**Distribution steel**

Diameter of bar  \(\text{LSize}=40 \text{ mm}\)
Spacing of bars  \(\text{LSpac}=300 \text{ mm}\)
### MAIN REINFORCEMENT

**IN X-DIRECTION:**

**SUMMARY**

- Characteristic strength: 250 N/mm²
- Diameter of bars: 32 mm
- Spacing: 190 mm
- Total number of bars: 20
- Total area required: 15911 mm²
- Total area provided: 16085 mm²
- Volume of steel: 70806 cm³
- Weight of steel: 555.83 kg

### MAIN REINFORCEMENT

**IN Y-DIRECTION:**

**SUMMARY**

- Characteristic strength: 250 N/mm²
- Diameter of bars: 32 mm
- Spacing: 190 mm
- Total number of bars: 20
- Total area required: 15911 mm²
- Total area provided: 16085 mm²
- Volume of steel: 70806 cm³
- Weight of steel: 555.83 kg

### DISTRIBUTION REINFORCEMENT

**SUMMARY**

- Characteristic strength: 250 N/mm²
- Diameter of bars: 40 mm

**IN X-DIRECTION:**

- In top:
  - Number of bars: 13
  - Spacing of bars: 300 mm
  - Area required: 4176 mm²/m
  - Area provided: 4188.8 mm²/m
  - Volume of steel: 59791 cm³
  - Weight of steel: 469.36 kg

**IN Y-DIRECTION:**

- In top:
  - Number of bars: 13
  - Spacing of bars: 300 mm
  - Area required: 4176 mm²/m
  - Area provided: 4188.8 mm²/m
  - Volume of steel: 59791 cm³
  - Weight of steel: 469.36 kg

### SIDE BARS SUMMARY

- Characteristic strength: 250 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 14
- Spacing of bars: 250 mm
- Volume of steel: 12421 cm³
- Weight of steel: 97.509 kg

### QUANTITIES

- Total concrete volume: 25.258 m³
- Total area of formwork: 26.518 m²
- Total volume of steel: 273615 cm³
- Total weight of steel: 2147.9 kg

- Approx. self-weight of pile cap: 848.67 kN
- Approx. total load on each pile: 2670 kN
Location: Small pilecap with high-grade concrete

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \). Bottom layer of main steel is assumed to run in X-direction.

![Diagram of pilecap for five piles]

**Pilecap for five piles**

Designed using bending theory in accordance with BS8110.

Pile cap assumed square (i.e. \( lx=ly \)) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres (i.e. chain lines in adjoining diagram).

**Total ultimate load on column**  \( N=7000 \text{ kN} \)

**Column dimension** \( cx=350 \text{ mm} \)

**Column dimension** \( cy=350 \text{ mm} \)

**Nominal pile diameter**  \( \text{PileD}=250 \text{ mm} \)

**Spec.min.spacing of pile centres**  \( \text{spc}=500 \text{ mm} \)

**Spec.of pile ctrs in X-direction**  \( AX=710 \text{ mm} \)

**Overhang: pile face to cap edge**  \( \text{olap}=170 \text{ mm} \)

**Length of pile cap**  \( lx=1300 \text{ mm} \)

**Spec.of pile ctrs in Y-direction**  \( AY=710 \text{ mm} \)

**Breadth of pile cap**  \( ly=1300 \text{ mm} \)

**Concrete grade**  \( f_{cu}=60 \text{ N/mm}^2 \)

**Steel strength**  \( f_y=500 \text{ N/mm}^2 \)

**Cover to underside of main bars**  \( \text{Cover}=150 \text{ mm} \)

**Side and top cover to main bars**  \( \text{Scvr}=75 \text{ mm} \)

**Chosen depth of pilecap**  \( h=990 \text{ mm} \)

**Design of main steel in X-direction**

**Main steel diameter**  \( \text{diaX}=12 \text{ mm} \)

**Main steel spacing**  \( \text{SpcX}=80 \text{ mm} \)

**Design of main steel in Y-direction**

**Main steel diameter**  \( \text{diaY}=12 \text{ mm} \)

**Main steel spacing**  \( \text{SpcY}=80 \text{ mm} \)

**Distribution steel**

**Diameter of bar**  \( \text{LSize}=16 \text{ mm} \)

**Spacing of bars**  \( \text{LSpac}=155 \text{ mm} \)
<table>
<thead>
<tr>
<th><strong>MAIN REINFORCEMENT</strong></th>
<th><strong>IN X-DIRECTION:</strong></th>
<th><strong>SUMMARY</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>80 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>1673.1 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>1696.5 mm²</td>
</tr>
<tr>
<td></td>
<td>Volume of steel</td>
<td>3498.1 cm³</td>
</tr>
<tr>
<td></td>
<td>Weight of steel</td>
<td>27.46 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>MAIN REINFORCEMENT</strong></th>
<th><strong>IN Y-DIRECTION:</strong></th>
<th><strong>SUMMARY</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>80 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>1673.1 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>1696.5 mm²</td>
</tr>
<tr>
<td></td>
<td>Volume of steel</td>
<td>3498.1 cm³</td>
</tr>
<tr>
<td></td>
<td>Weight of steel</td>
<td>27.46 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>DISTRIBUTION REINFORCEMENT</strong></th>
<th><strong>SUMMARY</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Characteristic strength</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>IN X-DIRECTION:</strong></th>
<th><strong>In top:</strong></th>
<th><strong>SUMMARY</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>155 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>1287 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>1297.2 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Volume of steel</td>
<td>1849.8 cm³</td>
<td></td>
</tr>
<tr>
<td>Weight of steel</td>
<td>14.521 kg</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>IN Y-DIRECTION:</strong></th>
<th><strong>In top:</strong></th>
<th><strong>SUMMARY</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>155 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>1287 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>1297.2 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Volume of steel</td>
<td>1849.8 cm³</td>
<td></td>
</tr>
<tr>
<td>Weight of steel</td>
<td>14.521 kg</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>SIDE BARS SUMMARY</strong></th>
<th><strong>SUMMARY</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>8</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>2438.4 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>19.141 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>QUANTITIES</strong></th>
<th><strong>SUMMARY</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Total concrete volume</td>
<td>1.6731 m³</td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>5.148 m²</td>
</tr>
<tr>
<td>Total volume of steel</td>
<td>13134 cm³</td>
</tr>
<tr>
<td>Total weight of steel</td>
<td>103.1 kg</td>
</tr>
<tr>
<td>Approx. self-weight of pile cap</td>
<td>56.216 kN</td>
</tr>
<tr>
<td>Approx. total load on each pile</td>
<td>1412 kN</td>
</tr>
</tbody>
</table>
Location: Ex1 - Typical pilecap

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for five piles

- Designed using bending theory to EC2 Part 1-1.
- Pilecap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres.
- Use ultimate values for load and moment.

Total design load on column: N=7000 kN
Column dimension cx: cx=500 mm
Column dimension cy: cy=500 mm
Partial safety factor for steel: gams=1.15
Partial safety factor for conc.: gamc=1.5
Nominal pile diameter: PileD=450 mm
Spec.min spacing of pile centres: spc=1350 mm
Spacing of pile ctrs in X-dir: AX=1910 mm
Dist. from pile face to cap edge: olap=150 mm
Length of pile cap: lx=2660 mm
Spacing of pile ctrs in Y-dir: AY=1910 mm
Breadth of pile cap: ly=2660 mm
Char yield strength of reinft.: fyk=500 N/mm²
Cover to underside of main bars: Cover=150 mm
Side and top cover to main bars: Scvr=75 mm
Chosen depth of pilecap: h=1420 mm

Design of main steel in X-direction

Main steel diameter: diaX=25 mm
Main steel spacing: SpcX=225 mm

Design of main steel in Y-direction

Main steel diameter: diaY=25 mm
Main steel spacing: SpcY=225 mm

Distribution steel

Distribution steel diameter: LSize=25 mm
Spacing of bars: LSpac=230 mm
Area of dist. steel provided: A2=Π*LSize²*250/LSpac=2134 mm²/m
Check shear resistance in X-direction

Shear force due to ultimate load \( V_{Ed} = N/2.5 + Mx'*1000/AX = 2800 \; \text{kN} \)
Shear resistance \( V_{Rdc} = \beta'*C_{Rdc}*k_s*v_{Rdc}*b*w*d/1000 = 3265 \; \text{kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>2800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear resistance</td>
<td>3265 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = N/2.5 + My'*1000/AY = 2800 \; \text{kN} \)
Shear resistance \( V_{Rdc} = \beta'*C_{Rdc}*k_s*v_{Rdc}*b*w*d/1000 = 3231 \; \text{kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>2800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear resistance</td>
<td>3231 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor \( \beta \) \( \beta = 1 \)
Shear stress at column perimeter \( v_{Ed} = \beta*V_{Ed}*1000/(u_o*d) = 2.272 \; \text{N/mm}^2 \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5*v*f_{cd} = 4.973 \; \text{N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

Check punching shear at 2d:
Shear stress at control perimeter \( v_{Ed} = \beta*V_{Ed}*1000/(u_1*d) = 0.8141 \; \text{N/mm}^2 \)
Design punching shear resistance \( v_{Rdc} = (0.18/gamc)*k*(100*p_1*f_{ck})^{0.333} = 0.3084 \; \text{N/mm}^2 \)
Enhancement factor to be used \( senh = 4.325 \)
Enhanced punching shear resist. \( v_{Rdc} = senh*v_{Rdc} = 1.334 \; \text{N/mm}^2 \)

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN X-DIRECTION:</td>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>225 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>5666 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>5890 mm²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN Y-DIRECTION:</td>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>225 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>5666 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>5890 mm²</td>
</tr>
</tbody>
</table>
## DISTRIBUTION REINFORCEMENT

### SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
</tr>
</tbody>
</table>

### IN X-DIRECTION:

<table>
<thead>
<tr>
<th>In top:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>11</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>230 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>2130 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>2134 mm²/m</td>
</tr>
</tbody>
</table>

### IN Y-DIRECTION:

<table>
<thead>
<tr>
<th>In top:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>11</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>230 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>2130 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>2134 mm²/m</td>
</tr>
</tbody>
</table>

## SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>10</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

## QUANTITIES

| Total concrete volume     | 10.05 m³ |
| Total area of formwork    | 15.11 m² |
Location: Ex2 - Heavily-loaded pilecap reinforced with mild steel

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for five piles
- Designed using bending theory to EC2 Part 1-1.
- Pilecap assumed square (i.e. lx=ly) with main reinforcement arranged in bands parallel to the principal axes and symmetrical about pile centres.
- Use ultimate values for load and moment.

Total design load on column $N=12500$ kN
Column dimension $cx = 500$ mm
Column dimension $cy = 500$ mm
Partial safety factor for steel $\gamma_{ms} = 1.15$
Partial safety factor for conc. $\gamma_{mc} = 1.5$
Nominal pile diameter $PileD = 450$ mm
Spec.min spacing of pile centres $spc = 1350$ mm
Spacing of pile ctrs in X-dir $AX = 1910$ mm
Dist. from pile face to cap edge $olap = 725$ mm
Length of pile cap $lx = 3810$ mm
Spacing of pile ctrs in Y-dir $AY = 1910$ mm
Breadth of pile cap $ly = 3810$ mm
Char yield strength of reinft. $f_yk = 500$ N/mm$^2$
Cover to underside of main bars $Cover = 150$ mm
Side and top cover to main bars $Scvr = 75$ mm
Chosen depth of pilecap $h = 1740$ mm

Design of main steel in X-direction
- Main steel diameter $diaX = 20$ mm
- Main steel spacing $SpcX = 115$ mm

Design of main steel in Y-direction
- Main steel diameter $diaY = 20$ mm
- Main steel spacing $SpcY = 115$ mm

Distribution steel
- Distribution steel diameter $LSize = 20$ mm
- Spacing of bars $LSpac = 120$ mm
- Area of dist. steel provided $A2 = \pi \times LSize^2 \times 250 / LSpac = 2618$ mm$^2$/m
Check shear resistance in X-direction

Shear force due to ultimate load \( V_{Ed} = \frac{N}{2.5} + \frac{M_x}{1000AX} = 5000 \text{ kN} \)

Shear resistance \( V_{Rdc} = \beta' * CRdc * ks * v_{Rdc} * bw * d / 1000 = 5444 \text{ kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>5000 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear resistance</td>
<td>5444 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = \frac{N}{2.5} + \frac{M_y}{1000AY} = 5000 \text{ kN} \)

Shear resistance \( V_{Rdc} = \beta' * CRdc * ks * v_{Rdc} * bw * d / 1000 = 5407 \text{ kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>5000 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear resistance</td>
<td>5407 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor \( \beta \) \( \beta = 1 \)
Shear stress at column perimeter \( v_{Ed} = \beta * V_{Ed} * 1000 / (u_0 * d) = 3.205 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5 * v_fcd = 5.581 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

Check punching shear at 2d:
Shear stress at control perimeter \( v_{Ed} = \beta * V_{Ed} * 1000 / (u_1 * d) = 1.149 \text{ N/mm}^2 \)
Design punching shear resistance \( v_{Rdc} = (0.18 / \gamma_{mc}) * k * (100 * p1 * fck)^{0.333} = 0.3213 \text{ N/mm}^2 \)
Enhancement factor to be used \( senh = 5.474 \)
Enhanced punching shear resist. \( v_{Rdc} = senh * v_{Rdc} = 1.759 \text{ N/mm}^2 \)

MAIN REINFORCEMENT
IN X-DIRECTION:

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>115 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>32</td>
</tr>
<tr>
<td>Total area required</td>
<td>9944 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>10053 mm²</td>
</tr>
</tbody>
</table>

MAIN REINFORCEMENT
IN Y-DIRECTION:

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>115 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>32</td>
</tr>
<tr>
<td>Total area required</td>
<td>9944 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>10053 mm²</td>
</tr>
<tr>
<td>DISTRIBUTION REINFORCEMENT SUMMARY</td>
<td>Characteristic strength</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
</tr>
</tbody>
</table>

**IN X-DIRECTION:**

| In top:                           | Number of bars           | 31        |
|                                  | Spacing of bars          | 120 mm    |
|                                  | Area required             | 2610 mm²/m |
|                                  | Area provided             | 2618 mm²/m |

**IN Y-DIRECTION:**

| In top:                           | Number of bars           | 31        |
|                                  | Spacing of bars          | 120 mm    |
|                                  | Area required             | 2610 mm²/m |
|                                  | Area provided             | 2618 mm²/m |

**SIDE BARS SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>14</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

**QUANTITIES**

| Total concrete volume    | 25.26 m³  |
| Total area of formwork   | 26.52 m²  |
Location: Ex3 - Small pilecap with high-grade concrete

Bottom layer of main steel is assumed to run in X-direction.

Total design load on column \( N = 5000 \text{ kN} \)
Column dimension \( cx = 350 \text{ mm} \)
Column dimension \( cy = 350 \text{ mm} \)
Partial safety factor for steel \( g_{ams} = 1.15 \)
Partial safety factor for conc. \( g_{amc} = 1.5 \)
Nominal pile diameter \( PileD = 250 \text{ mm} \)
Spec.min spacing of pile centres \( spc = 500 \text{ mm} \)
Spacing of pile ctrs in X-dir \( AX = 710 \text{ mm} \)
Dist. from pile face to cap edge \( olap = 170 \text{ mm} \)
Length of pile cap \( lx = 1300 \text{ mm} \)
Spacing of pile ctrs in Y-dir \( AY = 710 \text{ mm} \)
Breadth of pile cap \( ly = 1300 \text{ mm} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Cover to underside of main bars \( Cover = 150 \text{ mm} \)
Side and top cover to main bars \( Scvr = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 1200 \text{ mm} \)

**Design of main steel in X-direction**

Main steel diameter \( diaX = 12 \text{ mm} \)
Main steel spacing \( SpcX = 45 \text{ mm} \)

**Design of main steel in Y-direction**

Main steel diameter \( diaY = 16 \text{ mm} \)
Main steel spacing \( SpcY = 85 \text{ mm} \)

**Distribution steel**

Distribution steel diameter \( LSize = 25 \text{ mm} \)
Spacing of bars \( LSpac = 225 \text{ mm} \)
Area of dist. steel provided \( A2 = \pi \times LSize^2 \times 250/LSpac = 2182 \text{ mm}^2/m \)
Check shear resistance in X-direction

Shear force due to ultimate load \( V_{Ed} = N/2.5 + M_x' \times 1000/A_X = 2000 \text{ kN} \)

Shear resistance \( V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d/1000 \)

Modified shear resistance \( V_{Rdc} = V_{Rdm} = 2316 \text{ kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>2000 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear resistance</td>
<td>2316 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = N/2.5 + M_y' \times 1000/A_Y = 2000 \text{ kN} \)

Shear resistance \( V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d/1000 \)

Modified shear resistance \( V_{Rdc} = V_{Rdm} = 2292 \text{ kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>2000 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear resistance</td>
<td>2292 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:

Eccentricity factor \( \beta \)

Shear stress at column perimeter \( v_{Ed} = \beta \times V_{Ed} \times 1000/(u_o \times d) = 2.774 \text{ N/mm}^2 \)

Maximum punching shear resistance \( v_{Rd_{max}} = 0.5 \times v \times f_{cd} = 8 \text{ N/mm}^2 \)

Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

MAIN REINFORCEMENT

IN X-DIRECTION:

| SUMMARY |
|-----------------|----------------|
| Characteristic strength | 500 N/mm² |
| Diameter of bars | 12 mm |
| Spacing | 45 mm |
| Total number of bars | 26 |
| Total area required | 2835 mm² |
| Total area provided | 2941 mm² |

MAIN REINFORCEMENT

IN Y-DIRECTION:

| SUMMARY |
|-----------------|----------------|
| Characteristic strength | 500 N/mm² |
| Diameter of bars | 16 mm |
| Spacing | 85 mm |
| Total number of bars | 14 |
| Total area required | 2802 mm² |
| Total area provided | 2815 mm² |

DISTRIBUTION REINFORCEMENT

| SUMMARY |
|-----------------|----------------|
| Characteristic strength | 500 N/mm² |
| Diameter of bars | 25 mm |
### IN Y-DIRECTION:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>In top:</td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>6</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>225 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>2181 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>2182 mm²/m</td>
</tr>
</tbody>
</table>

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>8</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total concrete volume</td>
<td>2.028 m³</td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>6.24 m²</td>
</tr>
</tbody>
</table>
Location: Pilecap at G3

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for six piles

Designed by Bending Theory
Calculations are in accordance with BS8110.
Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.
For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column N=8400 kN
Column dimension cx cx=500 mm
Column dimension cy cy=500 mm
Nominal pile diameter PileD=450 mm
Spc.of pile ctrs in X-direction AX=1350 mm
Overhang: pile face to cap edge olap=150 mm
Length of pile cap lx=3450 mm
Spc.of pile ctrs in Y-direction AY=1350 mm
Breadth of pile cap ly=2100 mm
Concrete grade fcu=35 N/mm²
Steel strength fy=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=1730 mm

Design of main steel in X-direction
Main steel diameter diaX=32 mm
Main steel spacing SpcX=375 mm

Design of main steel in Y-direction
Main steel diameter diaY=16 mm
Main steel spacing SpcY=85 mm

Distribution steel
Diameter of bar LSize=25 mm
Spacing of bars LSpac=215 mm
### MAIN REINFORCEMENT

**IN X-DIRECTION:**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 32 mm
- Spacing: 375 mm
- Total number of bars: 6
- Total area required: 4780 mm²
- Total area provided: 4825.5 mm²
- Volume of steel: 23891 cm³
- Weight of steel: 187.54 kg

**IN Y-DIRECTION:**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm
- Spacing: 85 mm
- Total number of bars: 39
- Total area required: 7759.1 mm²
- Total area provided: 7841.4 mm²
- Volume of steel: 31350 cm³
- Weight of steel: 246.1 kg

### DISTRIBUTION REINFORCEMENT

**SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 25 mm

**IN X-DIRECTION:**
- In top:
  - Number of bars: 9
  - Spacing of bars: 215 mm
  - Area required: 2249 mm²/m
  - Area provided: 2283.1 mm²/m
  - Volume of steel: 14579 cm³
  - Weight of steel: 114.44 kg

**IN Y-DIRECTION:**
- In top:
  - Number of bars: 16
  - Spacing of bars: 215 mm
  - Area required: 2249 mm²/m
  - Area provided: 2283.1 mm²/m
  - Volume of steel: 15315 cm³
  - Weight of steel: 120.22 kg

### SIDE BARS SUMMARY

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 28
- Spacing of bars: 250 mm
- Volume of steel: 9880.2 cm³
- Weight of steel: 77.559 kg

### QUANTITIES

- Total concrete volume: 12.534 m³
- Total area of formwork: 19.203 m²
- Total volume of steel: 95015 cm³
- Total weight of steel: 745.87 kg

Approx. self-weight of pile cap: 421.14 kN
Approx. total load on each pile: 1471 kN
Location: Cap for heavily-loaded column using mild steel reinf.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for six piles

Designed by Bending Theory
Calculations are in accordance with BS8110.

Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.
For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column \( N=25000 \) kN
Column dimension \( cx=500 \) mm
Column dimension \( cy=500 \) mm
Nominal pile diameter \( PileD=500 \) mm
Spc.of pile ctrs in X-direction \( AX=1500 \) mm
Overhang: pile face to cap edge \( olap=300 \) mm
Length of pile cap \( lx=4100 \) mm
Spc.of pile ctrs in Y-direction \( AY=1500 \) mm
Breadth of pile cap \( ly=4100 \) mm
Concrete grade \( fcu=35 \) N/mm²
Steel strength \( fy=250 \) N/mm²
Cover to underside of main bars \( Cover=150 \) mm
Side and top cover to main bars \( Scvr=75 \) mm
Chosen depth of pilecap \( h=2840 \) mm

Design of main steel in X-direction

Main steel diameter \( diaX=25 \) mm
Main steel spacing \( SpcX=70 \) mm

Design of main steel in Y-direction

Main steel diameter \( diaY=25 \) mm
Main steel spacing \( SpcY=70 \) mm

Distribution steel

Diameter of bar \( LSize=50 \) mm
Spacing of bars \( LSpac=285 \) mm
MAIN REINFORCEMENT

**IN X-DIRECTION:**
- Characteristic strength: 250 N/mm²
- Diameter of bars: 25 mm
- Spacing: 70 mm
- Total number of bars: 57
- Total area required: 27946 mm²
- Total area provided: 27980 mm²
- Volume of steel: 216564 cm³
- Weight of steel: 1700 kg

**SUMMARY**
- Spacing: 70 mm
- Total number of bars: 57
- Total area required: 27946 mm²
- Total area provided: 27980 mm²
- Volume of steel: 216564 cm³
- Weight of steel: 1700 kg

MAIN REINFORCEMENT

**IN Y-DIRECTION:**
- Characteristic strength: 250 N/mm²
- Diameter of bars: 25 mm
- Spacing: 70 mm
- Total number of bars: 57
- Total area required: 27946 mm²
- Total area provided: 27980 mm²
- Volume of steel: 175993 cm³
- Weight of steel: 1381.5 kg

**SUMMARY**
- Spacing: 70 mm
- Total number of bars: 57
- Total area required: 27946 mm²
- Total area provided: 27980 mm²
- Volume of steel: 175993 cm³
- Weight of steel: 1381.5 kg

DISTRIBUTION REINFORCEMENT

**SUMMARY**
- Characteristic strength: 250 N/mm²
- Diameter of bars: 50 mm

**IN X-DIRECTION:**
- In top: Number of bars: 14
- Spacing of bars: 285 mm
- Area required: 6816 mm²/m
- Area provided: 6889.5 mm²/m
- Volume of steel: 108581 cm³
- Weight of steel: 852.36 kg

**IN Y-DIRECTION:**
- In top: Number of bars: 14
- Spacing of bars: 285 mm
- Area required: 6816 mm²/m
- Area provided: 6889.5 mm²/m
- Volume of steel: 108581 cm³
- Weight of steel: 852.36 kg

SIDE BARS SUMMARY

- Characteristic strength: 250 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 44
- Spacing of bars: 250 mm
- Volume of steel: 21896 cm³
- Weight of steel: 171.88 kg

QUANTITIES

- Total concrete volume: 47.74 m³
- Total area of formwork: 46.576 m²
- Total volume of steel: 631615 cm³
- Total weight of steel: 4958.2 kg
- Approx. self-weight of pile cap: 1604.1 kN
- Approx. total load on each pile: 4435 kN
Location: Cap for lightly-loaded column

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for six piles

Designed by Bending Theory
Calculations are in accordance with BS8110.

Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.
For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column $N=500$ kN
Column dimension $cx=300$ mm
Column dimension $cy=300$ mm
Nominal pile diameter $PileD=250$ mm
Spc.of pile ctrs in X-direction $AX=750$ mm
Overhang: pile face to cap edge $olap=300$ mm
Length of pile cap $lx=2350$ mm
Spc.of pile ctrs in Y-direction $AY=750$ mm
Breadth of pile cap $ly=2350$ mm
Concrete grade $fcu=35$ N/mm$^2$
Steel strength $fy=500$ N/mm$^2$
Cover to underside of main bars $Cover=150$ mm
Side and top cover to main bars $Scvr=75$ mm
Chosen depth of pilecap $h=310$ mm

Design of main steel in X-direction

Main steel diameter $diaX=12$ mm
Main steel spacing $SpcX=145$ mm

Design of main steel in Y-direction

Main steel diameter $diaY=12$ mm
Main steel spacing $SpcY=225$ mm

Distribution steel

Diameter of bar $LSIZE=12$ mm
Spacing of bars $LSpac=280$ mm
### MAIN REINFORCEMENT

#### IN X-DIRECTION:
- **Characteristic strength**: 500 N/mm²
- **Diameter of bars**: 12 mm
- **Spacing**: 145 mm
- **Total number of bars**: 16
- **Total area required**: 1775 mm²
- **Total area provided**: 1809.6 mm²
- **Volume of steel**: 4165.6 cm³
- **Weight of steel**: 32.7 kg

#### SUMMARY
- **Spacing**: 145 mm
- **Total number of bars**: 16
- **Total area required**: 1775 mm²
- **Total area provided**: 1809.6 mm²
- **Volume of steel**: 4165.6 cm³
- **Weight of steel**: 32.7 kg

### MAIN REINFORCEMENT

#### IN Y-DIRECTION:
- **Characteristic strength**: 500 N/mm²
- **Diameter of bars**: 12 mm
- **Spacing**: 225 mm
- **Total number of bars**: 10
- **Total area required**: 1063.9 mm²
- **Total area provided**: 1131 mm²
- **Volume of steel**: 1617.3 cm³
- **Weight of steel**: 12.696 kg

#### SUMMARY
- **Spacing**: 225 mm
- **Total number of bars**: 10
- **Total area required**: 1063.9 mm²
- **Total area provided**: 1131 mm²
- **Volume of steel**: 1617.3 cm³
- **Weight of steel**: 12.696 kg

### DISTRIBUTION REINFORCEMENT

#### SUMMARY
- **Characteristic strength**: 500 N/mm²
- **Diameter of bars**: 12 mm

#### IN X-DIRECTION:
- **In top:**
  - **Number of bars**: 8
  - **Spacing of bars**: 280 mm
  - **Area required**: 403 mm²/m
  - **Area provided**: 403.92 mm²/m
  - **Volume of steel**: 1990.5 cm³
  - **Weight of steel**: 15.626 kg

#### IN Y-DIRECTION:
- **In top:**
  - **Number of bars**: 8
  - **Spacing of bars**: 280 mm
  - **Area required**: 403 mm²/m
  - **Area provided**: 403.92 mm²/m
  - **Volume of steel**: 1990.5 cm³
  - **Weight of steel**: 15.626 kg

### SIDE BARS SUMMARY
- **Characteristic strength**: 500 N/mm²
- **Diameter of bars**: 12 mm
- **Total number of bars**: 4
- **Spacing of bars**: 250 mm
- **Volume of steel**: 1207.9 cm³
- **Weight of steel**: 9.4819 kg

### QUANTITIES
- **Total concrete volume**: 1.712 m³
- **Total area of formwork**: 2.914 m²
- **Total volume of steel**: 10972 cm³
- **Total weight of steel**: 86.129 kg

- **Approx. self-weight of pile cap**: 57.522 kN
- **Approx. total load on each pile**: 93 kN
Location: Ex1 - Pilecap at G3

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for six piles

Designed using bending theory to EC2 Part 1-1.

Main reinforcement is arranged in bands // to the principal axes and symmetrical about pile centres.

Use ultimate values for load and moment.

Total design load on column N=8400 kN
Column dimension cx cx=500 mm
Column dimension cy cy=500 mm
Nominal pile diameter PileD=450 mm
Partial safety factor for steel gams=1.15
Partial safety factor for conc. gamc=1.5
Spacing of pile ctrs in X-dir AX=1350 mm
Overhang: pile face to cap edge olap=150 mm
Length of pile cap lx=3450 mm
Spacing of pile ctrs in Y-dir AY=1350 mm
Breadth of pile cap ly=2100 mm
Char yield strength of reinf. fyk=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=1730 mm

Design of main steel in X-direction

Main steel diameter diaX=12 mm
Main steel spacing SpcX=40 mm

Design of main steel in Y-direction

Main steel diameter diaY=20 mm
Main steel spacing SpcY=115 mm

Distribution steel

Distribution steel diameter LSize=20 mm
Spacing of bars LSpac=120 mm
Area of dist. steel provided A2=π*LSize^2*250/LSpac=2618 mm²/m

Check shear resistance in X-direction

Shear force due to ultimate load VEd=N/3+Mx'*1000/AX=2800 kN
Shear resistance: \[ VR_{dc} = \beta \cdot CR_{dc} \cdot ks \cdot vR_{dc} \cdot bw \cdot d / 1000 \]
\[ = 2939 \text{ kN} \]
Modified shear resistance: \[ VR_{dc} = VR_{dm} = 3155 \text{ kN} \]

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear force</td>
<td>2800 kN</td>
</tr>
<tr>
<td>Shear resistance</td>
<td>3155 kN</td>
</tr>
</tbody>
</table>

**Hence section is satisfactory, no shear reinforcement is required.**

Check shear resistance in Y-direction:

Shear force due to ultimate load: \[ V_{Ed} = \frac{N}{2} + \frac{My^*1000}{AY} = 4200 \text{ kN} \]
Shear resistance: \[ VR_{dc} = \beta \cdot CR_{dc} \cdot ks \cdot vR_{dc} \cdot bw \cdot d / 1000 \]
\[ = 5889 \text{ kN} \]
Modified shear resistance: \[ VR_{dc} = VR_{dm} = 6303 \text{ kN} \]

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear force</td>
<td>4200 kN</td>
</tr>
<tr>
<td>Shear resistance</td>
<td>6303 kN</td>
</tr>
</tbody>
</table>

**Hence section is satisfactory, no shear reinforcement is required.**

Check punching shear at column face:

Eccentricity factor: \[ \beta = 1 \]
Shear stress at column perimeter: \[ v_{Ed} = \beta \cdot V_{Ed} \cdot 1000 / (uo \cdot d) = 2.696 \text{ N/mm}^2 \]
Maximum punching shear resistance: \[ v_{Rdmax} = 0.5 \cdot v \cdot f_{cd} = 4.973 \text{ N/mm}^2 \]
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>40 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>49</td>
</tr>
<tr>
<td>Total area required</td>
<td>5450 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>5542 mm²</td>
</tr>
</tbody>
</table>

**IN Y-DIRECTION:**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>115 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>29</td>
</tr>
<tr>
<td>Total area required</td>
<td>8953 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>9111 mm²</td>
</tr>
</tbody>
</table>

**DISTRIBUTION REINFORCEMENT**

**IN X-DIRECTION:**

In top:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>17</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>120 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>2595 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>2618 mm²/m</td>
</tr>
</tbody>
</table>

In Y-direction:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>28</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>120 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>2595 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>2618 mm²/m</td>
</tr>
</tbody>
</table>
### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>28</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

| Total concrete volume   | 12.53 m³  |
| Total area of formwork  | 19.2 m²    |
Location: Ex2 - Cap for heavily-loaded column w. mild steel reinf.

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for six piles

Designed using bending theory to EC2 Part 1-1.

Main reinforcement is arranged in bands // to the principal axes and symmetrical about pile centres.

Use ultimate values for load and moment.

Total design load on column \( N = 20000 \) kN
Column dimension \( cx = 500 \) mm
Column dimension \( cy = 500 \) mm
Nominal pile diameter \( \text{PileD} = 500 \) mm
Partial safety factor for steel \( g_{\text{ams}} = 1.15 \)
Partial safety factor for conc. \( g_{\text{amc}} = 1.5 \)
Spacing of pile ctrs in X-dir \( AX = 1500 \) mm
Overhang: pile face to cap edge \( \text{olap} = 300 \) mm
Length of pile cap \( 1x = 4100 \) mm
Spacing of pile ctrs in Y-dir \( AY = 1500 \) mm
Breadth of pile cap \( 1y = 4100 \) mm
Char yield strength of reinf. \( f_{yk} = 500 \) N/mm²
Cover to underside of main bars \( \text{Cover} = 150 \) mm
Side and top cover to main bars \( \text{Scvr} = 75 \) mm
Chosen depth of pilecap \( h = 2840 \) mm

Design of main steel in X-direction

Main steel diameter \( \text{diaX} = 40 \) mm
Main steel spacing \( \text{SpcX} = 300 \) mm

Design of main steel in Y-direction

Main steel diameter \( \text{diaY} = 40 \) mm
Main steel spacing \( \text{SpcY} = 300 \) mm

Distribution steel

Distribution steel diameter \( \text{LSize} = 25 \) mm
Spacing of bars \( \text{LSpac} = 115 \) mm
Area of dist. steel provided \( A2 = \pi \times \text{LSize}^2 \times 250 / \text{LSpac} = 4268 \) mm²/m

Check shear resistance in X-direction

Shear force due to ultimate load \( V_{Ed} = N / 3 + Mx' \times 1000 / AX = 6667 \) kN
Shear resistance  \[ VRdc = \beta' \cdot CRdc \cdot ks \cdot Vrdc \cdot bw \cdot d / 1000 \]
= 11049 kN

Modified shear resistance  \[ VRdc = VRdm = 11657 \text{ kN} \]

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>6667 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>11657 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load  \[ VEd = N/2 + My'*1000/AY = 10000 \text{ kN} \]
Shear resistance  \[ VRdc = \beta' \cdot CRdc \cdot ks \cdot Vrdc \cdot bw \cdot d / 1000 \]
= 10956 kN

Modified shear resistance  \[ VRdc = VRdm = 11511 \text{ kN} \]

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>10000 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>11511 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face

Check punching shear at column face:
Eccentricity factor \( \beta \) \[ \beta = 1 \]
Shear stress at column perimeter  \[ \nu Ed = \beta \cdot VEd \cdot 1000 / (uo \cdot d) = 3.802 \text{ N/mm}^2 \]
Maximum punching shear resistance  \[ vRdmax = 0.5 \cdot v \cdot fcd = 4.973 \text{ N/mm}^2 \]
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>40 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>300 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>14</td>
</tr>
<tr>
<td>Total area required</td>
<td>17466 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>17593 mm²</td>
</tr>
</tbody>
</table>

**IN Y-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>40 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>300 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>14</td>
</tr>
<tr>
<td>Total area required</td>
<td>17466 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>17593 mm²</td>
</tr>
</tbody>
</table>

**DISTRIBUTION REINFORCEMENT**

**IN X-DIRECTION:**

| Number of bars | 35 |
| Spacing of bars | 115 mm |
| Area required | 4260 mm²/m |
| Area provided | 4268 mm²/m |

**IN Y-DIRECTION:**

| Number of bars | 35 |
| Spacing of bars | 115 mm |
| Area required | 4260 mm²/m |
| Area provided | 4268 mm²/m |
### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>44</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

| Total concrete volume   | 47.74 m³  |
| Total area of formwork  | 46.58 m²  |
Location: Ex3 - Cap for lightly-loaded column

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for six piles

Designed using bending theory to EC2 Part 1-1.

Main reinforcement is arranged in bands // to the principal axes and symmetrical about pile centres.

Use ultimate values for load and moment.

Total design load on column \( N = 500 \text{ kN} \)
Column dimension \( c_x = 300 \text{ mm} \)
Column dimension \( c_y = 300 \text{ mm} \)
Nominal pile diameter \( PileD = 250 \text{ mm} \)
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Spacing of pile ctrs in X-dir \( AX = 750 \text{ mm} \)
Overhang: pile face to cap edge \( olap = 300 \text{ mm} \)
Length of pile cap \( lx = 2350 \text{ mm} \)
Spacing of pile ctrs in Y-dir \( AY = 750 \text{ mm} \)
Breadth of pile cap \( ly = 2350 \text{ mm} \)
Char yield strength of reinft. \( f_{yk} = 500 \text{ N/mm}^2 \)
Cover to underside of main bars \( Cover = 150 \text{ mm} \)
Side and top cover to main bars \( Scvr = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 400 \text{ mm} \)

Design of main steel in X-direction

Main steel diameter \( diaX = 12 \text{ mm} \)
Main steel spacing \( SpcX = 180 \text{ mm} \)

Design of main steel in Y-direction

Main steel diameter \( diaY = 12 \text{ mm} \)
Main steel spacing \( SpcY = 180 \text{ mm} \)

Distribution steel

Distribution steel diameter \( LSize = 12 \text{ mm} \)
Spacing of bars \( LSpac = 185 \text{ mm} \)
Area of dist. steel provided \( A2 = \pi \times LSize^2 \times 250 / LSpac = 611.3 \text{ mm}^2 / \text{m} \)

Check shear resistance in X-direction

Shear force due to ultimate load \( V_{Ed} = N / 3 + Mx' \times 1000 / AX = 166.7 \text{ kN} \)
Shear resistance \[ VR_{dc} = \beta' \cdot C_{Rdc} \cdot k_s \cdot v_{Rdc} \cdot b_w \cdot d / 1000 \] = 264.4 kN

Modified shear resistance \[ VR_{dc} = VR_{dm} = 298.6 \text{ kN} \]

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Shear force</th>
<th>Shear resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>166.7 kN</td>
<td>298.6 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \[ V_{Ed} = N/2 + M_y ' \cdot 1000 / A_Y = 250 \text{ kN} \]

Shear resistance \[ VR_{dc} = \beta' \cdot C_{Rdc} \cdot k_s \cdot v_{Rdc} \cdot b_w \cdot d / 1000 \] = 800.5 kN

Modified shear resistance \[ VR_{dc} = VR_{dm} = 894.2 \text{ kN} \]

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Shear force</th>
<th>Shear resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>250 kN</td>
<td>894.2 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face

Check punching shear at column face:

- Eccentricity factor \( \beta \): \( \beta = 1 \)
- Shear stress at column perimeter \( v_{Ed} = \beta \cdot V_{Ed} \cdot 1000 / (u_o \cdot d) = 1.796 \text{ N/mm}^2 \)
- Maximum punching shear resistance \( v_{Rd_{max}} = 0.5 \times v \times f_{cd} = 5.581 \text{ N/mm}^2 \)

Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Spacing: 180 mm
- Total number of bars: 13
- Total area required: 1410 mm²
- Total area provided: 1470 mm²

**IN Y-DIRECTION:**

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Spacing: 180 mm
- Total number of bars: 13
- Total area required: 1410 mm²
- Total area provided: 1470 mm²

**DISTRIBUTION REINFORCEMENT**

**SUMMARY**

**IN X-DIRECTION:**

In top:
- Number of bars: 12
- Spacing of bars: 185 mm
- Area required: 600 mm²/m
- Area provided: 611.3 mm²/m

**IN Y-DIRECTION:**

In top:
- Number of bars: 12
- Spacing of bars: 185 mm
- Area required: 600 mm²/m
- Area provided: 611.3 mm²/m
<table>
<thead>
<tr>
<th>SIDE BARS SUMMARY</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Total number of bars</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>QUANTITIES</th>
<th>Total concrete volume</th>
<th>2.209 m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total area of formwork</td>
<td>3.76 m²</td>
<td></td>
</tr>
</tbody>
</table>
Location: Pilecap at G3

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for seven piles

Designed by Bending Theory.
7th pile located directly beneath column.
Calculations are in accordance with BS8110.
Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.
For bond calculations, any high-yield bars are assumed Type2:deformed.

Total ultimate load on column N=9800 kN
Column dimension cx cx=500 mm
Column dimension cy cy=500 mm
Nominal pile diameter PileD=450 mm
Spc.of pile ctrs in X-direction AX=1350 mm
Overhang: pile face to cap edge olap=150 mm
Spc.of pile ctrs in Y-direction AY=1169 mm
Breadth of pile cap ly=3088 mm
Maximum length of pile cap lx=3566 mm
Minimum length of pile cap lx'=1783 mm
Concrete grade fcu=35 N/mm²
Steel strength fy=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=1500 mm

Design of main steel in X-direction

Main steel diameter diaX=16 mm
Main steel spacing SpcX=95 mm

Design of main steel in Y-direction

Main steel diameter diaY=16 mm
Main steel spacing SpcY=95 mm

Distribution steel

Diameter of bar LSize=20 mm
Spacing of bars LSpac=160 mm
MAIN REINFORCEMENT

IN X-DIRECTION:

Summary

- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm
- Spacing: 95 mm
- Total number of bars: 30
- Total area required: 6021.6 mm²
- Total area provided: 6031.9 mm²
- Volume of steel: 24359 cm³
- Weight of steel: 191.22 kg

IN Y-DIRECTION:

Summary

- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm
- Spacing: 95 mm
- Total number of bars: 32
- Total area required: 6390.7 mm²
- Total area provided: 6434 mm²
- Volume of steel: 26023 cm³
- Weight of steel: 204.28 kg

DISTRIBUTION REINFORCEMENT

Summary

- Characteristic strength: 500 N/mm²
- Diameter of bars: 20 mm

IN X-DIRECTION:

In top:

- Number of bars: 18
- Spacing of bars: 160 mm
- Area required: 1950 mm²/m
- Area provided: 1963.5 mm²/m
- Volume of steel: 14599 cm³
- Weight of steel: 114.6 kg

IN Y-DIRECTION:

In top:

- Number of bars: 22
- Spacing of bars: 160 mm
- Area required: 1950 mm²/m
- Area provided: 1963.5 mm²/m
- Volume of steel: 15190 cm³
- Weight of steel: 119.24 kg

SIDE BARS SUMMARY

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 36
- Spacing of bars: 250 mm
- Volume of steel: 9099.8 cm³
- Weight of steel: 71.434 kg

QUANTITIES

- Total concrete volume: 12.388 m³
- Total area of formwork: 16.046 m²
- Total volume of steel: 89270 cm³
- Total weight of steel: 700.77 kg

Approx. self-weight of pile cap: 416.25 kN
Approx. total load on each pile: 1460 kN
Location: Cap for heavily-loaded column w. mild steel reinf.

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for seven piles

Designed by Bending Theory. 7th pile located directly beneath column. Calculations are in accordance with BS8110. Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres. For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column $N = 40000$ kN
Column dimension $cx = 500$ mm
Column dimension $cy = 500$ mm
Nominal pile diameter $PileD = 450$ mm
Spc.of pile ctrs in X-direction $AX = 1350$ mm
Overhang: pile face to cap edge $olap = 150$ mm
Spc.of pile ctrs in Y-direction $AY = 1169$ mm
Breadth of pile cap $ly = 3088$ mm
Maximum length of pile cap $lx = 3566$ mm
Minimum length of pile cap $lx' = 1783$ mm
Concrete grade $fcu = 35$ N/mm$^2$
Steel strength $fy = 250$ N/mm$^2$
Cover to underside of main bars $Cover = 150$ mm
Side and top cover to main bars $Scvr = 75$ mm
Chosen depth of pilecap $h = 4420$ mm

Design of main steel in X-direction

Main steel diameter $diaX = 32$ mm
Main steel spacing $SpcX = 70$ mm

Design of main steel in Y-direction

Main steel diameter $diaY = 40$ mm
Main steel spacing $SpcY = 110$ mm

Distribution steel

Diameter of bar $LSize = 50$ mm
Spacing of bars $LSpac = 185$ mm
<table>
<thead>
<tr>
<th>Section</th>
<th>Characteristic strength</th>
<th>Diameter of bars</th>
<th>Spacing</th>
<th>Total number of bars</th>
<th>Total area required</th>
<th>Total area provided</th>
<th>Volume of steel</th>
<th>Weight of steel</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>MAIN REINFORCEMENT</strong></td>
<td>250 N/mm²</td>
<td>32 mm</td>
<td>70 mm</td>
<td>42</td>
<td>32758 mm²</td>
<td>33778 mm²</td>
<td>322601 cm³</td>
<td>2532.4 kg</td>
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<tr>
<td><strong>IN X-DIRECTION:</strong></td>
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<tr>
<td><strong>MAIN REINFORCEMENT</strong></td>
<td>250 N/mm²</td>
<td>40 mm</td>
<td>110 mm</td>
<td>28</td>
<td>34766 mm²</td>
<td>35186 mm²</td>
<td>336289 cm³</td>
<td>2639.9 kg</td>
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<td><strong>IN Y-DIRECTION:</strong></td>
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<td><strong>DISTRIBUTION REINFORCEMENT</strong></td>
<td>250 N/mm²</td>
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<td>Number of bars</td>
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<td>Spacing of bars</td>
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<tr>
<td>Area required</td>
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<tr>
<td>Volume of steel</td>
<td>80386 cm³</td>
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<tr>
<td>Weight of steel</td>
<td>631.03 kg</td>
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<td></td>
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</tr>
<tr>
<td><strong>IN Y-DIRECTION:</strong></td>
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</tr>
<tr>
<td>Number of bars</td>
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<td></td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>185 mm</td>
<td></td>
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<tr>
<td>Area required</td>
<td>10608 mm²/m</td>
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<tr>
<td>Area provided</td>
<td>10613 mm²/m</td>
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<tr>
<td>Volume of steel</td>
<td>81780 cm³</td>
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<tr>
<td>Weight of steel</td>
<td>641.97 kg</td>
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</tr>
<tr>
<td><strong>SIDE BARS SUMMARY</strong></td>
<td>250 N/mm²</td>
<td>12 mm</td>
<td></td>
<td>102</td>
<td>25206 cm³</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>QUANTITIES</strong></td>
<td>Total concrete volume</td>
<td>36.504 m³</td>
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<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Total area of formwork</td>
<td>47.283 m²</td>
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<tr>
<td></td>
<td>Total volume of steel</td>
<td>846261 cm³</td>
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<tr>
<td></td>
<td>Total weight of steel</td>
<td>6643.2 kg</td>
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<td></td>
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</tr>
<tr>
<td></td>
<td>Approx.self-weight of pile cap</td>
<td>1226.5 kN</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Approx.total load on each pile</td>
<td>5890 kN</td>
<td></td>
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</tr>
</tbody>
</table>
Location: Cap for lightly-loaded column

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$
Bottom layer of main steel is assumed to run in X-direction.

**Pilecap for seven piles**

Designed by Bending Theory.
7th pile located directly beneath column.
Calculations are in accordance with BS8110.
Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.
For bond calculations, any high-yield bars are assumed Type2: deformed.

Total ultimate load on column $N = 500$ kN
Column dimension $cx$ $cy = 300$ mm
Nominal pile diameter $PileD = 250$ mm
Spc.of pile ctrs in X-direction $AX = 750$ mm
Overhang: pile face to cap edge $olap = 150$ mm
Spc.of pile ctrs in Y-direction $AY = 650$ mm
Breadth of pile cap $ly = 1850$ mm
Maximum length of pile cap $lx = 2135$ mm
Minimum length of pile cap $lx' = 1067.6$ mm
Concrete grade $fcu = 35$ N/mm²
Steel strength $fy = 500$ N/mm²
Cover to underside of main bars $Cover = 150$ mm
Side and top cover to main bars $Scvr = 75$ mm
Chosen depth of pilecap $h = 300$ mm

**Design of main steel in X-direction**

Main steel diameter $diaX = 12$ mm
Main steel spacing $SpcX = 125$ mm

**Design of main steel in Y-direction**

**WARNING:**
No bar arrangement is satisfactory. Increase the cap thickness, or possibly, the cap width, or the concrete and/or steel strengths.
Location: Ex1 - Pilecap at G3

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for seven piles

Designed using bending theory to EC2 Part 1-1. Seventh pile is located directly beneath column. Main reinforcement is arranged in bands // to principal axes and is symmetrical about pile centres. Ultimate values for load & moment to be used.

Total design load on column N=9800 kN
Column dimension cx cx=500 mm
Column dimension cy cy=500 mm
Nominal pile diameter PileD=450 mm
Partial safety factor for steel gam=1.15
Partial safety factor for conc. gamc=1.5
Spacing of pile ctrs in X-dir AX=1350 mm
Overhang: pile face to cap edge olap=150 mm
Spacing of pile ctrs in Y-dir AY=1169 mm
Breadth of pile cap ly=3088 mm
Maximum length of pile cap lx=3566 mm
Minimum length of pile cap lx'=1783 mm
Char yield strength of reinf. fyk=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=1500 mm

Design of main steel in X-direction

Main steel diameter diaX=16 mm
Main steel spacing SpcX=80 mm

Design of main steel in Y-direction

Main steel diameter diaY=20 mm
Main steel spacing SpcY=125 mm

Distribution steel

Distribution steel diameter LSize=25 mm
Spacing of bars LSpac=215 mm
Area of dist steel provided A2=π*LSize²*2*250/LSpac=2283 mm²/m
Check shear resistance in X-direction

Shear force due to ultimate load \( V_{Ed} = 3N/7 + M_x'*1000/AX = 4200 \text{ kN} \)
Shear resistance \( V_{Rdc} = \beta'C_Rdc*k_s*V_{Rdc}*bw*d/1000 = 4680 \text{ kN} \)
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 5010 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Shear force</th>
<th>Shear resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>X-direction</strong></td>
<td>4200 kN</td>
<td>5010 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = 2N/7 + M_y'*1000/AY = 2800 \text{ kN} \)
Shear resistance \( V_{Rdc} = \beta'C_Rdc*k_s*V_{Rdc}*bw*d/1000 = 3336 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Shear force</th>
<th>Shear resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Y-direction</strong></td>
<td>2800 kN</td>
<td>3336 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor \( \beta \) \( \beta = 1 \)
Shear stress at column perimeter \( \nu_{Ed} = \beta*V_{Ed}*1000/(uo*d) = 3.701 \text{ N/mm}^2 \)
Maximum punching shear resistance \( V_{Rdmax} = 0.5*v*f_{cd} = 4.973 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>80 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>36</td>
</tr>
<tr>
<td>Total area required</td>
<td>6948 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>7238 mm²</td>
</tr>
</tbody>
</table>

**IN Y-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>125 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>24</td>
</tr>
<tr>
<td>Total area required</td>
<td>7374 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>7540 mm²</td>
</tr>
</tbody>
</table>
### DISTRIBUTION REINFORCEMENT

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>25 mm</td>
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</table>

**IN X-DIRECTION:**

<table>
<thead>
<tr>
<th>In top:</th>
<th>Number of bars</th>
<th>14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing of bars</td>
<td>215 mm</td>
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</tr>
<tr>
<td>Area required</td>
<td>2250 mm²/m</td>
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</tr>
<tr>
<td>Area provided</td>
<td>2283 mm²/m</td>
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</tbody>
</table>

**IN Y-DIRECTION:**

<table>
<thead>
<tr>
<th>In top:</th>
<th>Number of bars</th>
<th>16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing of bars</td>
<td>215 mm</td>
<td></td>
</tr>
<tr>
<td>Area required</td>
<td>2250 mm²/m</td>
<td></td>
</tr>
<tr>
<td>Area provided</td>
<td>2283 mm²/m</td>
<td></td>
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</tbody>
</table>

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>36</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

| Total concrete volume | 12.39 m³ |
| Total area of formwork | 16.05 m² |
Location: Ex2 - Cap for heavily-loaded column w. mild steel reinf.

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for seven piles

Designed using bending theory to EC2 Part 1-1. Seventh pile is located directly beneath column. Main reinforcement is arranged in bands to principal axes and is symmetrical about pile centres. Ultimate values for load & moment to be used.

Total design load on column \( N = 30000 \) kN
Column dimension \( cx = 500 \) mm
Column dimension \( cy = 500 \) mm
Nominal pile diameter \( PileD = 450 \) mm
Partial safety factor for steel \( gams = 1.15 \)
Partial safety factor for conc. \( gamc = 1.5 \)
Spacing of pile ctrs in X-dir \( AX = 1350 \) mm
Overhang: pile face to cap edge \( olap = 150 \) mm
Spacing of pile ctrs in Y-dir \( AY = 1169 \) mm
Breadth of pile cap \( ly = 3088 \) mm
Maximum length of pile cap \( lx = 3566 \) mm
Minimum length of pile cap \( lx' = 1783 \) mm
Char yield strength of reinft. \( fyk = 500 \) N/mm²
Cover to underside of main bars \( Cover = 150 \) mm
Side and top cover to main bars \( Scvr = 75 \) mm
Chosen depth of pilecap \( h = 4420 \) mm

Design of main steel in X-direction

Main steel diameter \( diaX = 32 \) mm
Main steel spacing \( SpcX = 110 \) mm

Design of main steel in Y-direction

Main steel diameter \( diaY = 32 \) mm
Main steel spacing \( SpcY = 110 \) mm

Distribution steel

Distribution steel diameter \( LSize = 50 \) mm
Spacing of bars \( LSpac = 295 \) mm
Area of dist steel provided \( A2 = \pi \times LSize^2 \times 250 / LSpac = 6656 \) mm²/m
Check shear resistance in X-direction

Shear force due to ultimate load \( V_{Ed} = 3\times N/7 + M_x' \times 1000/AX = 12857 \text{ kN} \)
Shear resistance \( V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d/1000 = 12627 \text{ kN} \)
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 13062 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>12857 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>13062 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Shear force due to ultimate load \( V_{Ed} = N/7 + M_x' \times 1000/AX = 4286 \text{ kN} \)
Shear resistance \( V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d/1000 = 12627 \text{ kN} \)
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 13062 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>4286 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>13062 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor \( \beta \) \( = \beta' = 1 \)
Shear stress at column perimeter \( v_{Ed} = \beta \times v_{Ed} \times 1000/(u_0 \times d) = 3.553 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5 \times v_{fcd} = 4.973 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>32 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>110 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>26</td>
</tr>
<tr>
<td>Total area required</td>
<td>20473 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>20910 mm²</td>
</tr>
</tbody>
</table>

**IN Y-DIRECTION:**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>32 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>110 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>28</td>
</tr>
<tr>
<td>Total area required</td>
<td>21729 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>22519 mm²</td>
</tr>
</tbody>
</table>
### DISTRIBUTION REINFORCEMENT

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>50 mm</td>
</tr>
</tbody>
</table>

**IN X-DIRECTION:**

- **In top:**
  - Number of bars: 10
  - Spacing of bars: 295 mm
  - Area required: 6630 mm²/m
  - Area provided: 6656 mm²/m

**IN Y-DIRECTION:**

- **In top:**
  - Number of bars: 12
  - Spacing of bars: 295 mm
  - Area required: 6630 mm²/m
  - Area provided: 6656 mm²/m

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>102</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

- Total concrete volume: 36.5 m³
- Total area of formwork: 47.28 m²
Location: Ex3 - Cap for lightly-loaded column

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for seven piles

Designed using bending theory to EC2 Part 1-1. Seventh pile is located directly beneath column. Main reinforcement is arranged in bands // to principal axes and is symmetrical about pile centres. Ultimate values for load & moment to be used.

Total design load on column N=500 kN
Column dimension cx cx=300 mm
Column dimension cy cy=300 mm
Nominal pile diameter PileD=250 mm
Partial safety factor for steel gams=1.15
Partial safety factor for conc. gamc=1.5
Spacing of pile ctrs in X-dir AX=750 mm
Overhang: pile face to cap edge olap=150 mm
Spacing of pile ctrs in Y-dir AY=650 mm
Breadth of pile cap ly=1850 mm
Maximum length of pile cap lx=2135 mm
Minimum length of pile cap lx'=1067.6 mm
Char yield strength of reinft. fyk=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=400 mm

Design of main steel in X-direction

Main steel diameter diaX=12 mm
Main steel spacing SpcX=165 mm

Design of main steel in Y-direction

Main steel diameter diaY=12 mm
Main steel spacing SpcY=160 mm

Distribution steel

Distribution steel diameter LSize=12 mm
Spacing of bars LSpac=185 mm
Area of dist steel provided A2=PI*LSize^2*250/LSpac=611.3 mm²/m
Check shear resistance in X-direction

Shear force due to ultimate load  \( V_{Ed} = 3 \times N/7 + M_{x}' \times 1000/AX = 214.3 \text{ kN} \)

Shear resistance  \( VR_{dc} = \beta' \times CR_{dc} \times ks \times v_{Rdc} \times bw \times d/1000 = 642.8 \text{ kN} \)

Modified shear resistance  \( VR_{dc} = VR_{dm} = 715.3 \text{ kN} \)

SHEAR SUMMARY

<table>
<thead>
<tr>
<th>shear force</th>
<th>214.3 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>shear resistance</td>
<td>715.3 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load  \( V_{Ed} = N/7 + M_{y}' \times 1000/AY = 142.9 \text{ kN} \)

Shear resistance  \( VR_{dc} = \beta' \times CR_{dc} \times ks \times v_{Rdc} \times bw \times d/1000 = 192.6 \text{ kN} \)

SHEAR SUMMARY

<table>
<thead>
<tr>
<th>shear force</th>
<th>142.9 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>shear resistance</td>
<td>192.6 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor \( \beta \)  \( \beta = 1 \)
Shear stress at column perimeter  \( v_{Ed} = \beta \times V_{Ed} \times 1000/(uo \times d) = 1.796 \text{ N/mm}^2 \)

Maximum punching shear resistance  \( v_{Rd}_{max} = 0.5 \times v \times f_{cd} = 4.973 \text{ N/mm}^2 \)

Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

MAIN REINFORCEMENT

IN X-DIRECTION:
SUMMARY

| Characteristic strength | 500 N/mm² |
| Diameter of bars | 12 mm |
| Spacing | 165 mm |
| Total number of bars | 10 |
| Total area required | 1110 mm² |
| Total area provided | 1131 mm² |

MAIN REINFORCEMENT

IN Y-DIRECTION:
SUMMARY

| Characteristic strength | 500 N/mm² |
| Diameter of bars | 12 mm |
| Spacing | 160 mm |
| Total number of bars | 12 |
| Total area required | 1177 mm² |
| Total area provided | 1357 mm² |
## DISTRIBUTION REINFORCEMENT SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
</tbody>
</table>

### IN X-DIRECTION:

- **In top:**
  - Number of bars: 10
  - Spacing of bars: 185 mm
  - Area required: 600 mm²/m
  - Area provided: 611.3 mm²/m

### IN Y-DIRECTION:

- **In top:**
  - Number of bars: 10
  - Spacing of bars: 185 mm
  - Area required: 600 mm²/m
  - Area provided: 611.3 mm²/m

## SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
</tbody>
</table>

- Total number of bars: 6
- Spacing of bars: 250 mm

## QUANTITIES

- Total concrete volume: 1.185 m³
- Total area of formwork: 2.563 m²
Location: Pilecap at G3

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$.
Bottom layer of main steel is assumed to run in X-direction.

### Pilecap for eight piles

Designed by Bending Theory. Calculations are in accordance with BS8110.
Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.

For bond calculations, any high-yield bars are assumed Type 2: deformed.

---

**Total ultimate load on column** $N=11200$ kN  
**Nominal pile diameter** $PileD=450$ mm  
**Column dimension cx** $cx=500$ mm  
**Column dimension cy** $cy=500$ mm  
**Spc.of pile ctrs in X-direction** $AX=1350$ mm  
**Overhang: pile face to cap edge** $olap=150$ mm  
**Length of pile cap** $lx=3450$ mm  
**Spc.of pile ctrs in Y-direction** $AY=1169$ mm  
**Breadth of pile cap** $ly=3088$ mm  
**Concrete grade** $fcu=35$ N/mm$^2$  
**Steel strength** $fy=500$ N/mm$^2$  
**Cover to underside of main bars** $Cover=150$ mm  
**Side and top cover to main bars** $Scvr=75$ mm  
**Chosen depth of pilecap** $h=1500$ mm

### Design of main steel in X-direction

Main steel diameter $diaX=20$ mm  
Main steel spacing $SpcX=135$ mm

### Design of main steel in Y-direction

Main steel diameter $diaY=20$ mm  
Main steel spacing $SpcY=145$ mm

### Distribution steel

Diameter of bar $LSize=20$ mm  
Spacing of bars $LSpac=160$ mm  
Area of dist.steel provided $A2=\pi\times LSize^2\times 250/LSpac=1963.5$ mm$^2$/m
Check shear resistance in X-direction (Cl.3.11.4.4)

Total shear resistance provided: \( V_{\text{Prov}} = \frac{(V_{\text{Mod}} \times CP1Xs + vc \times CP2Xs) \times \text{EffDX}}{1E3} = 11006 \text{ kN} \)

Total shear resistance required: \( = 4200 \text{ kN} \)

Shear resistance in X-direction is satisfactory.

Check shear resistance in Y-direction - Clause 3.11.4.4

Total shear resistance provided: \( V_{\text{Prov}} = \frac{(V_{\text{Mod}} \times CP1Ys + vc \times CP2Ys) \times \text{EffDY}}{1E3} = 4354.9 \text{ kN} \)

Total shear resistance required: \( = 4200 \text{ kN} \)

Shear resistance in Y-direction is satisfactory.

Check punching shear at column face - Clause 3.11.4.5

Punching-shear resistance provided: \( V_{\text{pR1}} = V_{\text{max}} \times 2 \times (cx \times \text{EffDY} + cy \times \text{EffDX}) / 1E3 = 12589 \text{ kN} \)

Punching-shear resistance required: \( = 11200 \text{ kN} \)

Punching-shear stress on column perimeter is satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION**

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>135 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>22</td>
</tr>
<tr>
<td>Total area required</td>
<td>6689.8 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>6911.5 mm²</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>33348 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>261.78 kg</td>
</tr>
</tbody>
</table>

**MAIN REINFORCEMENT**

**IN Y-DIRECTION**

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>145 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>23</td>
</tr>
<tr>
<td>Total area required</td>
<td>7133.4 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>7225.7 mm²</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>32537 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>255.42 kg</td>
</tr>
</tbody>
</table>

**DISTRIBUTION REINFORCEMENT**

**SUMMARY**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
</tbody>
</table>

**IN X-DIRECTION:**

**In top:**

| Number of bars | 19        |
| Spacing of bars| 160 mm    |
| Area required   | 1950 mm²/m |
| Area provided   | 1963.5 mm²/m |
| Volume of steel | 19698 cm³ |
| Weight of steel | 154.63 kg |
### IN Y-DIRECTION:

**In top:**
- Number of bars: 21
- Spacing of bars: 160 mm
- Area required: 1950 mm²/m
- Area provided: 1963.5 mm²/m
- Volume of steel: 19383 cm³
- Weight of steel: 152.16 kg

**SIDE BARS SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 24
- Spacing of bars: 250 mm
- Volume of steel: 9785.2 cm³
- Weight of steel: 76.814 kg

**QUANTITIES**
- Total concrete volume: 15.98 m³
- Total area of formwork: 19.614 m²
- Total volume of steel: 114751 cm³
- Total weight of steel: 900.8 kg

Approx. self-weight of pile cap: 536.94 kN
Approx. total load on each pile: 1468 kN
Location: Cap for heavily-loaded column w. mild steel reinforcement

Design to BS8110(1997) with partial safety factor for steel $\gamma_S=1.15$
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for eight piles

Designed by Bending Theory. Calculations are in accordance with BS8110.
Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.

For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column $N=50000$ kN
Nominal pile diameter $PileD=450$ mm
Column dimension $cx=550$ mm
Column dimension $cy=550$ mm
Spc.of pile ctrs in X-direction $AX=1350$ mm
Overhang: pile face to cap edge $olap=475$ mm
Length of pile cap $lx=4100$ mm
Spc.of pile ctrs in Y-direction $AY=1169$ mm
Breadth of pile cap $ly=4100$ mm
Concrete grade $fcu=35$ N/mm$^2$
Steel strength $fy=250$ N/mm$^2$
Cover to underside of main bars $Cover=150$ mm
Side and top cover to main bars $Scvr=75$ mm
Chosen depth of pilecap $h=5000$ mm

Design of main steel in X-direction

Main steel diameter $diaX=40$ mm
Main steel spacing $SpcX=100$ mm

Design of main steel in Y-direction

Main steel diameter $diaY=40$ mm
Main steel spacing $SpcY=100$ mm

Distribution steel

Diameter of bar $LSize=50$ mm
Spacing of bars $LSpac=160$ mm
Area of dist.steel provided $A2=\pi*LSize^2*250/LSpac=12272$ mm$^2$/m
Check shear resistance in X-direction (Cl.3.11.4.4)

Total shear resistance provided \[ V_{\text{Prov}} = \frac{(V_{\text{Mod}} \cdot C_{P1Xs} + v_c \cdot C_{P2Xs}) \cdot \text{EffDX}}{1E3} = 93725 \text{ kN} \]

Total shear resistance required \[ = 18750 \text{ kN} \]
Shear resistance in X-direction is satisfactory.

Check shear resistance in Y-direction - Clause 3.11.4.4

Total shear resistance provided \[ V_{\text{Prov}} = \frac{(V_{\text{Mod}} \cdot C_{P1Ys} + v_c \cdot C_{P2Ys}) \cdot \text{EffDY}}{1E3} = 74552 \text{ kN} \]

Total shear resistance required \[ = 18750 \text{ kN} \]
Shear resistance in Y-direction is satisfactory.

Check punching shear at column face - Clause 3.11.4.5

Punching-shear resistance prov. \[ V_{\text{PR1}} = \frac{V_{\text{max}} \cdot 2 \cdot (c_x \cdot \text{EffDY} + c_y \cdot \text{EffDX})}{1E3} = 50083 \text{ kN} \]

Punching-shear resistance reqd. \[ = 50000 \text{ kN} \]
Punching-shear stress on column perimeter is satisfactory.

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN X-DIRECTION</td>
<td>Diameter of bars</td>
<td>40 mm</td>
</tr>
<tr>
<td></td>
<td>Spacing</td>
<td>100 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Total number of bars</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>49200 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>50265 mm²</td>
</tr>
<tr>
<td></td>
<td>Volume of steel</td>
<td>578556 cm³</td>
</tr>
<tr>
<td></td>
<td>Weight of steel</td>
<td>4541.7 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN Y-DIRECTION</td>
<td>Diameter of bars</td>
<td>40 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>100 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>49200 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>50265 mm²</td>
</tr>
<tr>
<td></td>
<td>Volume of steel</td>
<td>569508 cm³</td>
</tr>
<tr>
<td></td>
<td>Weight of steel</td>
<td>4470.6 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DISTRIBUTION REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>250 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
<td>Diameter of bars</td>
<td>50 mm</td>
</tr>
</tbody>
</table>

In X-DIRECTION:

<table>
<thead>
<tr>
<th>Number of bars</th>
<th>25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing of bars</td>
<td>160 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>12000 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>12272 mm²/m</td>
</tr>
<tr>
<td>Volume of steel</td>
<td>193895 cm³</td>
</tr>
<tr>
<td>Weight of steel</td>
<td>1522.1 kg</td>
</tr>
</tbody>
</table>
IN Y-DIRECTION:

In top:
- Number of bars: 25
- Spacing of bars: 160 mm
- Area required: 12000 mm²/m
- Area provided: 12272 mm²/m
- Volume of steel: 193895 cm³
- Weight of steel: 1522.1 kg

SIDE BARS SUMMARY
- Characteristic strength: 250 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 80
- Spacing of bars: 250 mm
- Volume of steel: 40082 cm³
- Weight of steel: 314.64 kg

QUANTITIES
- Total concrete volume: 84.05 m³
- Total area of formwork: 82 m²
- Total volume of steel: 1.5759E6 cm³
- Total weight of steel: 12371 kg

Approx. self-weight of pile cap: 2824.1 kN
Approx. total load on each pile: 6604 kN
Location: Cap for lightly-loaded column

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Bottom layer of main steel is assumed to run in X-direction.

Pilecap for eight piles

Designed by Bending Theory. Calculations are in accordance with BS8110. Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.

For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column N=500 kN
Nominal pile diameter PileD=250 mm
Column dimension cx cx=300 mm
Column dimension cy cy=300 mm
Spc.of pile ctrs in X-direction AX=750 mm
Overhang: pile face to cap edge olap=300 mm
Length of pile cap lx=2350 mm
Spc.of pile ctrs in Y-direction AY=650 mm
Breadth of pile cap ly=2350 mm
Concrete grade fcu=35 N/mm²
Steel strength fy=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=340 mm

Design of main steel in X-direction

Main steel diameter diaX=12 mm
Main steel spacing SpcX=195 mm

Design of main steel in Y-direction

Main steel diameter diaY=12 mm
Main steel spacing SpcY=180 mm

Distribution steel

Diameter of bar LSize=12 mm
Spacing of bars LSpac=255 mm
Area of dist.steel provided A2=PI*LSize^2*250/LSpac=443.52 mm²/m
Check shear resistance in X-direction (Cl.3.11.4.4)

Total shear resistance provided: \[ V_{Prov} = \frac{(V_{Mod} \cdot C_{P1Xs} + v_{c} \cdot C_{P2Xs}) \cdot \text{EffDX}}{1E3} = 572.11 \text{ kN} \]

Total shear resistance required: \[ = 187.5 \text{ kN} \]

Shear resistance in X-direction is satisfactory.

Check shear resistance in Y-direction - Clause 3.11.4.4

Total shear resistance provided: \[ V_{Prov} = \frac{(V_{Mod} \cdot C_{P1Ys} + v_{c} \cdot C_{P2Ys}) \cdot \text{EffDY}}{1E3} = 251.92 \text{ kN} \]

Total shear resistance required: \[ = 187.5 \text{ kN} \]

Shear resistance in Y-direction is satisfactory.

Check punching shear at column face - Clause 3.11.4.5

Punching-shear resistance prov.: \[ V_{pR1} = V_{\text{max}} \cdot 2 \times (c_{x} \cdot \text{EffDY} + c_{y} \cdot \text{EffDX}) / 1E3 = 1010.9 \text{ kN} \]

Punching-shear resistance reqd.: \[ = 500 \text{ kN} \]

Punching-shear stress on column perimeter is satisfactory.

Check punching shear at critical planes - Clause 3.11.4.5

At Critical Perimeter 1:
- Punching-shear resistance prov.: 582.95 kN
- Punching-shear resistance reqd.: 500 kN
Punching shear at Critical Perimeter 1 is satisfactory.

At Critical Perimeter 2:
- Punching-shear resistance prov.: 536.34 kN
- Punching-shear resistance reqd.: 375 kN
Punching shear at Critical Perimeter 2 is satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION**

**SUMMARY**

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Spacing: 195 mm
- Total number of bars: 12
- Total area required: 1268.4 mm²
- Total area provided: 1357.2 mm²
- Volume of steel: 2958.6 cm³
- Weight of steel: 23.225 kg

**MAIN REINFORCEMENT**

**IN Y-DIRECTION**

**SUMMARY**

- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Spacing: 180 mm
- Total number of bars: 13
- Total area required: 1442 mm²
- Total area provided: 1470.3 mm²
- Volume of steel: 2969.9 cm³
- Weight of steel: 23.314 kg
### DISTRIBUTION REINFORCEMENT

**SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm

#### IN X-DIRECTION:
- **In top:**
  - Number of bars: 9
  - Spacing of bars: 255 mm
  - Area required: 442 mm²/m
  - Area provided: 443.52 mm²/m
  - Volume of steel: 2239.3 cm³
  - Weight of steel: 17.579 kg

#### IN Y-DIRECTION:
- **In top:**
  - Number of bars: 9
  - Spacing of bars: 255 mm
  - Area required: 442 mm²/m
  - Area provided: 443.52 mm²/m
  - Volume of steel: 2239.3 cm³
  - Weight of steel: 17.579 kg

### SIDE BARS SUMMARY
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 4
- Spacing of bars: 250 mm
- Volume of steel: 1207.9 cm³
- Weight of steel: 9.4819 kg

### QUANTITIES
- Total concrete volume: 1.8777 m³
- Total area of formwork: 3.196 m²
- Total volume of steel: 11615 cm³
- Total weight of steel: 91.179 kg

- Approx. self-weight of pile cap: 63.089 kN
- Approx. total load on each pile: 71 kN
Location: Ex1 - Pilecap at G3

Bottom layer of main steel is assumed to run in X-direction.

Total design load on column \( N = 12000 \) kN
Nominal pile diameter \( PileD = 450 \) mm
Column dimension \( cx = 500 \) mm
Column dimension \( cy = 500 \) mm
Partial safety factor for steel \( \gamma_{ms} = 1.15 \)
Partial safety factor for conc. \( \gamma_{mc} = 1.5 \)
Spacing of pile ctrs in X-dir \( AX = 1350 \) mm
Overhang: pile face to cap edge \( olap = 150 \) mm
Length of pile cap \( lx = 3450 \) mm
Spacing of pile ctrs in Y-dir \( AY = 1169 \) mm
Breadth of pile cap \( ly = 3088 \) mm
Char yield strength of reinf. \( f_{yk} = 500 \) N/mm²
Cover to underside of main bars \( Cover = 150 \) mm
Side and top cover to main bars \( Scvr = 75 \) mm
Chosen depth of pilecap \( h = 1500 \) mm

Design of main steel in X-direction
Main steel diameter \( diaX = 16 \) mm
Main steel spacing \( SpcX = 85 \) mm

Design of main steel in Y-direction
Main steel diameter \( diaY = 16 \) mm
Main steel spacing \( SpcY = 85 \) mm

Distribution steel
Distribution steel diameter \( LSize = 25 \) mm
Spacing of bars \( LSpac = 215 \) mm
Area of dist. steel provided \( \frac{2 \pi \cdot LSize^2 \cdot 250}{LSpac} = 2283 \) mm²/m
Check shear resistance in X-direction

At Critical Plane 1:
Shear force due to ultimate load \( V_{Ed} = \frac{3N}{8} + Mx' * 1000 / AX = 4200 \text{ kN} \)
Shear resistance \( \frac{\beta'^*CR_{dc}*ks*v_{Rdc}*bw*d}{1000} = 4847 \text{ kN} \)
Modified shear resistance \( \frac{VR_{dc} = VR_{dm} = 5355 \text{ kN}}{1000} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>4200 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>5355 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

At Critical Plane 2:
Shear force due to ultimate load \( V_{Ed} = \frac{N}{4} + Mx' * 1000 / AX = 2800 \text{ kN} \)
Shear resistance \( \frac{\beta'^*CR_{dc}*ks*v_{Rdc}*bw*d}{1000} = 3370 \text{ kN} \)
Modified shear resistance \( \frac{VR_{dc} = VR_{dm} = 3724 \text{ kN}}{1000} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>2800 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>3724 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = 3N/8 + My' * 1000 / AY = 4200 \text{ kN} \)
Shear resistance \( \frac{\beta'*CR_{dc}*ks*v_{Rdc}*bw*d}{1000} = 4547 \text{ kN} \)
Modified shear resistance \( VR_{dc} = VR_{dm} = 5012 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>4200 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>5012 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face

Check punching shear at column face:
Eccentricity factor \( \beta = 1 \)
Shear stress at column perimeter \( \frac{v_{Ed} = \beta*V_{Ed}*1000}{(uo*d)} = 4.223 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5*v*f_{cd} = 5.581 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN X-DIRECTION SUMMARY</td>
<td></td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>85 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>35</td>
</tr>
<tr>
<td>Total area required</td>
<td>6948 mm$^2$</td>
</tr>
<tr>
<td>Total area provided</td>
<td>7037 mm$^2$</td>
</tr>
<tr>
<td>Section</td>
<td>Characteristic strength</td>
</tr>
<tr>
<td>--------------------------------</td>
<td>-------------------------</td>
</tr>
<tr>
<td><strong>MAIN REINFORCEMENT</strong></td>
<td></td>
</tr>
<tr>
<td><strong>IN Y-DIRECTION</strong></td>
<td></td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td></td>
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</tr>
<tr>
<td><strong>DISTRIBUTION REINFORCEMENT</strong></td>
<td></td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>IN X-DIRECTION:</strong></td>
<td></td>
</tr>
<tr>
<td>In top:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
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<tr>
<td></td>
<td>Spacing of bars</td>
</tr>
<tr>
<td></td>
<td>Area required</td>
</tr>
<tr>
<td></td>
<td>Area provided</td>
</tr>
<tr>
<td><strong>IN Y-DIRECTION:</strong></td>
<td></td>
</tr>
<tr>
<td>In top:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Number of bars</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
</tr>
<tr>
<td></td>
<td>Area required</td>
</tr>
<tr>
<td></td>
<td>Area provided</td>
</tr>
<tr>
<td><strong>SIDE BARS SUMMARY</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Characteristic strength</td>
</tr>
<tr>
<td></td>
<td>Diameter of bars</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
</tr>
<tr>
<td><strong>QUANTITIES</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total concrete volume</td>
</tr>
<tr>
<td></td>
<td>Total area of formwork</td>
</tr>
</tbody>
</table>
**Location: Ex2 - Cap for heavily-loaded column w. mild steel reinf.**

Bottom layer of main steel is assumed to run in X-direction.

**Pilecap for eight piles**

Designed using bending theory to EC2 Part 1-1. Main reinforcement is arranged in bands // to principal axes and is symmetrical about pile centres. Ultimate load & moment values to be used.

**Design of main steel in X-direction**

- Main steel diameter: \( \text{diaX}=40 \text{ mm} \)
- Main steel spacing: \( \text{SpcX}=160 \text{ mm} \)

**Design of main steel in Y-direction**

- Main steel diameter: \( \text{diaY}=40 \text{ mm} \)
- Main steel spacing: \( \text{SpcY}=160 \text{ mm} \)

**Distribution steel**

- Distribution steel diameter: \( \text{LSize}=50 \text{ mm} \)
- Spacing of bars: \( \text{LSpac}=260 \text{ mm} \)
- Area of dist. steel provided: \( A_2=\pi \cdot \text{LSize}^2 \cdot 250 / \text{LSpac}=7552 \text{ mm}^2 / \text{m} \)
Check shear resistance in X-direction

At Critical Plane 1:
Shear force due to ultimate load \( V_{Ed} = \frac{3}{8}N + Mx' \times 1000/AX = 15000 \) kN
Shear resistance
\[ V_{Rdc} = \beta' \times C_{Rdc} \times ks \times v_{Rdc} \times bw \times d / 1000 = 18805 \] kN
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 19369 \) kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Shear force</th>
<th>15000 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>19369 kN</td>
<td></td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

At Critical Plane 2:
Shear force due to ultimate load \( V_{Ed} = \frac{N}{4} + Mx' \times 1000/AX = 10000 \) kN
Shear resistance
\[ V_{Rdc} = \beta' \times C_{Rdc} \times ks \times v_{Rdc} \times bw \times d / 1000 = 18805 \] kN
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 19369 \) kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Shear force</th>
<th>10000 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>19369 kN</td>
<td></td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = \frac{3}{8}N + M_y' \times 1000/AY = 15000 \) kN
Shear resistance
\[ V_{Rdc} = \beta' \times C_{Rdc} \times ks \times v_{Rdc} \times bw \times d / 1000 = 18714 \] kN
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 19229 \) kN

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th></th>
<th>Shear force</th>
<th>15000 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>19229 kN</td>
<td></td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face

Check punching shear at column face:
Eccentricity factor \( \beta \)

\[ \beta = 1 \]

Shear stress at column perimeter \( v_{Ed} = \beta \times V_{Ed} \times 1000/(uo \times d) = 3.796 \) N/mm²

Maximum punching shear resistance \( v_{Rdmax} = 0.5 \times v \times fcd = 4.973 \) N/mm²

Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION**

<table>
<thead>
<tr>
<th></th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bars</td>
<td>40 mm</td>
<td></td>
</tr>
<tr>
<td>Spacing</td>
<td>160 mm</td>
<td></td>
</tr>
<tr>
<td>Total number of bars</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>Total area required</td>
<td>30750 mm²</td>
<td></td>
</tr>
<tr>
<td>Total area provided</td>
<td>31416 mm²</td>
<td></td>
</tr>
</tbody>
</table>
### MAIN REINFORCEMENT

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>IN Y-DIRECTION</strong></td>
<td></td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>40 mm</td>
</tr>
<tr>
<td>Spacing</td>
<td>160 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>25</td>
</tr>
<tr>
<td>Total area required</td>
<td>30750 mm²</td>
</tr>
<tr>
<td>Total area provided</td>
<td>31416 mm²</td>
</tr>
</tbody>
</table>

### DISTRICT REINFORCEMENT

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SUMMARY</strong></td>
<td></td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>50 mm</td>
</tr>
</tbody>
</table>

#### IN X-DIRECTION:

**In top:**

| Number of bars | 16 |
| Spacing of bars | 260 mm |
| Area required   | 7500 mm²/m |
| Area provided   | 7552 mm²/m |

#### IN Y-DIRECTION:

**In top:**

| Number of bars | 16 |
| Spacing of bars | 260 mm |
| Area required   | 7500 mm²/m |
| Area provided   | 7552 mm²/m |

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Diameter of bars</strong></td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>80</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

| Total concrete volume   | 84.05 m³ |
| Total area of formwork  | 82 m²    |
Location: Ex3 - Cap for lightly-loaded column

Bottom layer of main steel is assumed to run in X-direction.

---

Pilecap for eight piles

 Designed using bending theory to EC2 Part 1-1. Main reinforcement is arranged in bands // to principal axes and is symmetrical about pile centres. Ultimate load & moment values to be used.

---

Total design load on column $N = 500$ kN
Nominal pile diameter $PileD = 250$ mm
Column dimension $cx = 300$ mm
Column dimension $cy = 300$ mm
Partial safety factor for steel $\gamma_m = 1.15$
Partial safety factor for conc. $\gamma_c = 1.5$
Spacing of pile ctrs in X-dir $AX = 750$ mm
Overhang: pile face to cap edge $olap = 300$ mm
Length of pile cap $lx = 2350$ mm
Spacing of pile ctrs in Y-dir $AY = 650$ mm
Breadth of pile cap $ly = 2350$ mm
Char yield strength of reinft. $f_{yk} = 500$ N/mm$^2$
Cover to underside of main bars $Cover = 150$ mm
Side and top cover to main bars $Scvr = 75$ mm
Chosen depth of pilecap $h = 400$ mm

---

**Design of main steel in X-direction**

Main steel diameter $diaX = 12$ mm
Main steel spacing $SpcX = 180$ mm

**Design of main steel in Y-direction**

Main steel diameter $diaY = 12$ mm
Main steel spacing $SpcY = 180$ mm

**Distribution steel**

Distribution steel diameter $LSize = 12$ mm
Spacing of bars $LSpac = 185$ mm
Area of dist. steel provided $A2 = \pi * LSize^2 * 250 / LSpac = 611.3$ mm$^2$/m
Check shear resistance in X-direction

At Critical Plane 1:
Shear force due to ultimate load \( V_{Ed} = \frac{3}{8}N + M_x' \times 1000/AX = 187.5 \text{ kN} \)
Shear resistance \( V_{Rd\text{c}} = \beta' \times C_{Rd} \times k_s \times v_{Rd} \times bw \times d/1000 = 751.2 \text{ kN} \)
Modified shear resistance \( V_{Rd} = V_{Rd\text{m}} = 792.7 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>187.5 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>792.7 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

At Critical Plane 2:
Shear force due to ultimate load \( V_{Ed} = N/4 + M_x' \times 1000/AX = 125 \text{ kN} \)
Shear resistance \( V_{Rd\text{c}} = \beta' \times C_{Rd} \times k_s \times v_{Rd} \times bw \times d/1000 = 230.9 \text{ kN} \)
Modified shear resistance \( V_{Rd} = V_{Rd\text{m}} = 243.6 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>125 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>243.6 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = \frac{3}{8}N + M_y' \times 1000/AY = 187.5 \text{ kN} \)
Shear resistance \( V_{Rd\text{c}} = \beta' \times C_{Rd} \times k_s \times v_{Rd} \times bw \times d/1000 = 262.5 \text{ kN} \)
Modified shear resistance \( V_{Rd} = V_{Rd\text{m}} = 282.7 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>187.5 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>282.7 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear at column face

Check punching shear at column face:
Eccentricity factor \( \beta \) \( \beta = 1 \)
Shear stress at column perimeter \( v_{Ed} = \beta \times V_{Ed} \times 1000/(uo \times d) = 1.796 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{Rd\text{max}} = 0.5 \times v \times f_{cd} = 4.973 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

Check punching shear at 2d:
Shear stress at control perimeter \( v_{Ed} = \beta \times V_{Ed} \times 1000/(u_1 \times d) = 1.006 \text{ N/mm}^2 \)
Design punching shear resistance \( v_{Rd} = (0.18/g_{amc}) \times k \times (100*p_1 f_{ck})^{0.33} = 0.333 = 0.4594 \text{ N/mm}^2 \)
Enhanced punching shear resistance \( v_{Rd} = senh \times v_{Rd} = 1.421 \text{ N/mm}^2 \)
Design punching shear resistance \( v_{Rd} = senh \times 0.035 \times k \times 1.5 \times f_{ck}^{0.5} = 1.534 \text{ N/mm}^2 \)

Check punching shear at 2d:
Shear stress at control perimeter \( v_{Ed} = \beta \times V_{Ed} \times 1000/(u_1 \times d) = 0.4176 \text{ N/mm}^2 \)
Design punching shear resistance \( v_{Rd} = (0.18/g_{amc}) \times k \times (100*p_1 f_{ck})^{0.33} = 0.333 = 0.4594 \text{ N/mm}^2 \)
Enhanced punching shear resist. \( v_{\text{Rdc}} = s e n h \cdot v_{\text{Rdc}} = 0.5016 \, \text{N/mm}^2 \)

Design punching shear resistance \( v_{\text{Rdc}} = s e n h \cdot 0.035 \cdot k^1.5 \cdot f_{\text{ck}}^0.5 \) = 0.5415 \( \text{N/mm}^2 \)

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN X-DIRECTION</td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>180 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>1410 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>1470 mm²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN Y-DIRECTION</td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>180 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>1410 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>1470 mm²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DISTRIBUTION REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN X-DIRECTION:</td>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>In top:</td>
<td>Number of bars</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>Spacing of bars</td>
<td>185 mm</td>
</tr>
<tr>
<td></td>
<td>Area required</td>
<td>600 mm²/m</td>
</tr>
<tr>
<td></td>
<td>Area provided</td>
<td>611.3 mm²/m</td>
</tr>
</tbody>
</table>

| IN Y-DIRECTION:             | Number of bars          | 12        |
| In top:                     | Spacing of bars         | 185 mm    |
|                            | Area required           | 600 mm²/m |
|                            | Area provided           | 611.3 mm²/m |

| SIDE BARS SUMMARY          | Characteristic strength | 500 N/mm² |
|                            | Diameter of bars        | 12 mm     |
|                            | Total number of bars    | 4         |
|                            | Spacing of bars         | 250 mm    |

| QUANTITIES                 | Total concrete volume   | 2.209 m³  |
|                            | Total area of formwork  | 3.76 m²   |
Location: Pilecap at G3

Design to BS8110(1997) with partial safety factor for steel $\gamma_S = 1.15$
Bottom layer of main steel is assumed to run in X-direction.

**Pilecap for nine piles**

Based on Bending Theory. Calculations are in accordance with BS8110. Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.

For bond calculations, any high-yield bars are assumed Type 2: deformed.

---

Total ultimate load on column $N = 12600$ kN
Nominal pile diameter $PileD = 450$ mm
Column dimension $cx = 500$ mm
Column dimension $cy = 500$ mm
Spc.of pile ctrs in X-direction $AX = 1350$ mm
Overhang: pile face to cap edge $olap = 150$ mm
Length of pile cap $lx = 3450$ mm
Spc.of pile ctrs in Y-direction $AY = 1350$ mm
Breath of pile cap $ly = 3450$ mm
Concrete grade $fcu = 35$ N/mm²
Steel strength $fy = 500$ N/mm²
Cover to underside of main bars $Cover = 150$ mm
Side and top cover to main bars $Scvr = 75$ mm
Chosen depth of pilecap $h = 1690$ mm

**Design of main steel in X-direction**

Main steel diameter $diaX = 16$ mm
Main steel spacing $SpcX = 85$ mm

**Design of main steel in Y-direction**

Main steel diameter $diaY = 16$ mm
Main steel spacing $SpcY = 85$ mm

**Distribution steel**

Diameter of bar $LSIZE = 25$ mm
Spacing of bars $LSpac = 220$ mm
### MAIN REINFORCEMENT

#### IN X-DIRECTION:

**SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm
- Spacing: 85 mm
- Total number of bars: 39
- Total area required: 7579.7 mm²
- Total area provided: 7841.4 mm²
- Volume of steel: 40877 cm³
- Weight of steel: 320.89 kg

#### IN Y-DIRECTION:

**SUMMARY**
- Characteristic strength: 500 N/mm²
- Diameter of bars: 16 mm
- Spacing: 85 mm
- Total number of bars: 39
- Total area required: 7579.7 mm²
- Total area provided: 7841.4 mm²
- Volume of steel: 41191 cm³
- Weight of steel: 323.35 kg

### DISTRIBUTION REINFORCEMENT

#### SUMMARY
- Characteristic strength: 500 N/mm²
- Diameter of bars: 25 mm

#### IN X-DIRECTION:
- **In top:**
  - Number of bars: 15
  - Spacing of bars: 220 mm
  - Area required: 2197 mm²/m
  - Area provided: 2231.2 mm²/m
  - Volume of steel: 24298 cm³
  - Weight of steel: 190.74 kg

#### IN Y-DIRECTION:
- **In top:**
  - Number of bars: 15
  - Spacing of bars: 220 mm
  - Area required: 2197 mm²/m
  - Area provided: 2231.2 mm²/m
  - Volume of steel: 24298 cm³
  - Weight of steel: 190.74 kg

### SIDE BARS SUMMARY
- Characteristic strength: 500 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 24
- Spacing of bars: 250 mm
- Volume of steel: 10260 cm³
- Weight of steel: 80.542 kg

### QUANTITIES
- **Total volume of concrete needed:** 20.115 m³
- **Total area of formwork:** 23.322 m²
- **Total volume of steel:** 140925 cm³
- **Total weight of steel:** 1106.3 kg
- **Approx. self-weight of pile cap:** 675.87 kN
- **Approx. total load on each pile:** 1476 kN
Location: Cap for heavily-loaded column using mild steel reinf.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.15. Bottom layer of main steel is assumed to run in X-direction.

**Pilecap for nine piles**

Based on Bending Theory. Calculations are in accordance with BS8110. Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.

For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column \( N = 30000 \text{ kN} \)
Nominal pile diameter \( PileD = 500 \text{ mm} \)
Column dimension \( cx = 600 \text{ mm} \)
Column dimension \( cy = 600 \text{ mm} \)
Spc.of pile ctrs in X-direction \( AX = 1500 \text{ mm} \)
Overhang: pile face to cap edge \( olap = 300 \text{ mm} \)
Length of pile cap \( lx = 4100 \text{ mm} \)
Spc.of pile ctrs in Y-direction \( AY = 1500 \text{ mm} \)
Breadth of pile cap \( ly = 4100 \text{ mm} \)
Concrete grade \( fcu = 35 \text{ N/mm}^2 \)
Steel strength \( fy = 250 \text{ N/mm}^2 \)
Cover to underside of main bars \( Cover = 150 \text{ mm} \)
Side and top cover to main bars \( Scvr = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 2840 \text{ mm} \)

**Design of main steel in X-direction**

Main steel diameter \( diaX = 25 \text{ mm} \)
Main steel spacing \( SpcX = 70 \text{ mm} \)

**Design of main steel in Y-direction**

Main steel diameter \( diaY = 25 \text{ mm} \)
Main steel spacing \( SpcY = 70 \text{ mm} \)

**Distribution steel**

Diameter of bar \( LSize = 50 \text{ mm} \)
Spacing of bars \( LSpac = 285 \text{ mm} \)
### MAIN REINFORCEMENT

**IN X-DIRECTION:**
- Characteristic strength: 250 N/mm²
- Diameter of bars: 25 mm
- Spacing: 70 mm
- Total number of bars: 57
- Total area required: 27946 mm²
- Total area provided: 27980 mm²
- Volume of steel: 216564 cm³
- Weight of steel: 1700 kg

**SUMMARY**
- Total number of bars: 57
- Total area required: 27946 mm²
- Total area provided: 27980 mm²
- Volume of steel: 216564 cm³
- Weight of steel: 1700 kg

### MAIN REINFORCEMENT

**IN Y-DIRECTION:**
- Characteristic strength: 250 N/mm²
- Diameter of bars: 25 mm
- Spacing: 70 mm
- Total number of bars: 57
- Total area required: 27946 mm²
- Total area provided: 27980 mm²
- Volume of steel: 217963 cm³
- Weight of steel: 1711 kg

### DISTRIBUTION REINFORCEMENT

**SUMMARY**
- Characteristic strength: 250 N/mm²
- Diameter of bars: 50 mm

**IN X-DIRECTION:**
- In top:
  - Number of bars: 14
  - Spacing of bars: 285 mm
  - Area required: 6816 mm²/m
  - Area provided: 6889.5 mm²/m
  - Volume of steel: 108581 cm³
  - Weight of steel: 852.36 kg

**IN Y-DIRECTION:**
- In top:
  - Number of bars: 14
  - Spacing of bars: 285 mm
  - Area required: 6816 mm²/m
  - Area provided: 6889.5 mm²/m
  - Volume of steel: 108581 cm³
  - Weight of steel: 852.36 kg

### SIDE BARS SUMMARY

- Characteristic strength: 250 N/mm²
- Diameter of bars: 12 mm
- Total number of bars: 44
- Spacing of bars: 250 mm
- Volume of steel: 21896 cm³
- Weight of steel: 171.88 kg

### QUANTITIES

- Total volume of concrete needed: 47.74 m³
- Total area of formwork: 46.576 m²
- Total volume of steel: 673585 cm³
- Total weight of steel: 5287.6 kg
- Approx. self-weight of pile cap: 1604.1 kN
- Approx. total load on each pile: 3512 kN
Location: Cap for lightly-loaded column

Design to BS8110(1997) with partial safety factor for steel \( \gamma_S = 1.15 \). Bottom layer of main steel is assumed to run in X-direction.

**Pilecap for nine piles**

Based on Bending Theory. Calculations are in accordance with BS8110. Main reinforcement is arranged in bands parallel to principal axes and is symmetrical about pile centres.

For bond calculations, any high-yield bars are assumed Type 2: deformed.

Total ultimate load on column \( N = 500 \text{ kN} \)
Nominal pile diameter \( P = 250 \text{ mm} \)
Column dimension \( cx = 300 \text{ mm} \)
Column dimension \( cy = 300 \text{ mm} \)
Spc.of pile ctrs in X-direction \( AX = 750 \text{ mm} \)
Overhang: pile face to cap edge \( olap = 300 \text{ mm} \)
Length of pile cap \( lx = 2350 \text{ mm} \)
Spc.of pile ctrs in Y-direction \( AY = 750 \text{ mm} \)
Breadth of pile cap \( ly = 2350 \text{ mm} \)
Concrete grade \( f_{cu} = 35 \text{ N/mm}^2 \)
Steel strength \( f_y = 500 \text{ N/mm}^2 \)
Cover to underside of main bars \( Cover = 150 \text{ mm} \)
Side and top cover to main bars \( Scvr = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 300 \text{ mm} \)

**Design of main steel in X-direction**

Main steel diameter \( \text{diaX} = 16 \text{ mm} \)
Main steel spacing \( \text{SpcX} = 225 \text{ mm} \)

**Design of main steel in Y-direction**

Main steel diameter \( \text{diaY} = 12 \text{ mm} \)
Main steel spacing \( \text{SpcY} = 105 \text{ mm} \)

**Distribution steel**

Diameter of bar \( \text{LSize} = 12 \text{ mm} \)
Spacing of bars \( \text{LSpac} = 285 \text{ mm} \)
<table>
<thead>
<tr>
<th><strong>MAIN REINFORCEMENT</strong></th>
<th><strong>Characteristic strength</strong></th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>IN X-DIRECTION:</strong></td>
<td><strong>Diameter of bars</strong></td>
<td>16 mm</td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td><strong>Spacing</strong></td>
<td>225 mm</td>
</tr>
<tr>
<td><strong>Total number of bars</strong></td>
<td><strong>Total area required</strong></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td><strong>Total area provided</strong></td>
<td>1939.1 mm²</td>
</tr>
<tr>
<td></td>
<td><strong>Volume of steel</strong></td>
<td>4720.9 cm³</td>
</tr>
<tr>
<td></td>
<td><strong>Weight of steel</strong></td>
<td>37.059 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>MAIN REINFORCEMENT</strong></th>
<th><strong>Characteristic strength</strong></th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>IN Y-DIRECTION:</strong></td>
<td><strong>Diameter of bars</strong></td>
<td>12 mm</td>
</tr>
<tr>
<td><strong>SUMMARY</strong></td>
<td><strong>Spacing</strong></td>
<td>105 mm</td>
</tr>
<tr>
<td><strong>Total number of bars</strong></td>
<td><strong>Total area required</strong></td>
<td>21</td>
</tr>
<tr>
<td></td>
<td><strong>Total area provided</strong></td>
<td>2287 mm²</td>
</tr>
<tr>
<td></td>
<td><strong>Volume of steel</strong></td>
<td>5225.1 cm³</td>
</tr>
<tr>
<td></td>
<td><strong>Weight of steel</strong></td>
<td>41.017 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>DISTRIBUTION REINFORCEMENT</strong></th>
<th><strong>Characteristic strength</strong></th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SUMMARY</strong></td>
<td><strong>Diameter of bars</strong></td>
<td>12 mm</td>
</tr>
<tr>
<td><strong>IN X-DIRECTION:</strong></td>
<td><strong>Number of bars</strong></td>
<td>8</td>
</tr>
<tr>
<td><strong>In top:</strong></td>
<td><strong>Spacing of bars</strong></td>
<td>285 mm</td>
</tr>
<tr>
<td></td>
<td><strong>Area required</strong></td>
<td>390 mm²/m</td>
</tr>
<tr>
<td></td>
<td><strong>Area provided</strong></td>
<td>396.83 mm²/m</td>
</tr>
<tr>
<td></td>
<td><strong>Volume of steel</strong></td>
<td>1990.5 cm³</td>
</tr>
<tr>
<td></td>
<td><strong>Weight of steel</strong></td>
<td>15.626 kg</td>
</tr>
<tr>
<td><strong>IN Y-DIRECTION:</strong></td>
<td><strong>Number of bars</strong></td>
<td>8</td>
</tr>
<tr>
<td><strong>In top:</strong></td>
<td><strong>Spacing of bars</strong></td>
<td>285 mm</td>
</tr>
<tr>
<td></td>
<td><strong>Area required</strong></td>
<td>390 mm²/m</td>
</tr>
<tr>
<td></td>
<td><strong>Area provided</strong></td>
<td>396.83 mm²/m</td>
</tr>
<tr>
<td></td>
<td><strong>Volume of steel</strong></td>
<td>1990.5 cm³</td>
</tr>
<tr>
<td></td>
<td><strong>Weight of steel</strong></td>
<td>15.626 kg</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>SIDE BARS SUMMARY</strong></th>
<th><strong>Characteristic strength</strong></th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Diameter of bars</strong></td>
<td>12 mm</td>
</tr>
<tr>
<td></td>
<td><strong>Total number of bars</strong></td>
<td>4</td>
</tr>
<tr>
<td></td>
<td><strong>Spacing of bars</strong></td>
<td>250 mm</td>
</tr>
<tr>
<td></td>
<td><strong>Volume of steel</strong></td>
<td>1210.1 cm³</td>
</tr>
<tr>
<td></td>
<td><strong>Weight of steel</strong></td>
<td>9.4996 kg</td>
</tr>
</tbody>
</table>

| **QUANTITIES**               | **Total volume of concrete needed** | 1.6568 m³ |
|                              | **Total area of formwork**       | 2.82 m²   |
|                              | **Total volume of steel**        | 15137 cm³ |
|                              | **Total weight of steel**        | 118.83 kg |
| Approx.self-weight of pile cap | **Approx.total load on each pile** | 55.667 kN | 62 kN |
Location: Ex1 - Pilecap at G3

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for nine piles

Designed using bending theory to EC2 Part 1-1. Main reinforcement is arranged in bands // to principal axes and is symmetrical about pile centres. Ultimate load and moment values to be used. Ninth pile is located under column.

Total design load on column N=10600 kN
Nominal pile diameter PileD=450 mm
Column dimension cx cx=500 mm
Column dimension cy cy=500 mm
Partial safety factor for steel gamS=1.15
Partial safety factor for conc. gamC=1.5
Spacing of pile ctrs in X-dir AX=1350 mm
Overhang: pile face to cap edge olap=150 mm
Length of pile cap lx=3450 mm
Spacing of pile ctrs in Y-dir AY=1350 mm
Breadth of pile cap ly=3450 mm
Char yield strength of reinft. fyk=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=1690 mm

Design of main steel in X-direction

Main steel diameter diaX=20 mm
Main steel spacing SpcX=120 mm

Design of main steel in Y-direction

Main steel diameter diaY=20 mm
Main steel spacing SpcY=120 mm

Distribution steel

Distribution steel diameter LSize=25 mm
Spacing of bars LSpac=190 mm
Area of dist.steel provided A2=PI*LSize^2*250/LSpac=2584 mm²/m
Check shear resistance in X-direction

At Critical Plane 1:
Shear force due to ultimate load \( V_{Ed} = \frac{N}{3} + M_x' \times 1000 / A_X = 3533 \text{ kN} \)
Shear resistance \( V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d / 1000 = 4778 \text{ kN} \)
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 5265 \text{ kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>3533 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear resistance</td>
<td>5265 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = \frac{N}{3} + M_y' \times 1000 / A_Y = 3533 \text{ kN} \)
Shear resistance \( V_{Rdc} = \beta' \times C_{Rdc} \times k_s \times v_{Rdc} \times b_w \times d / 1000 = 4682 \text{ kN} \)
Modified shear resistance \( V_{Rdc} = V_{Rdm} = 5142 \text{ kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>3533 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear resistance</td>
<td>5142 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor \( \beta \) \( \beta = 1 \)
Shear stress at column perimeter \( v_{Ed} = \beta \times V_{Ed} \times 1000 / (u_0 \times d) = 3.51 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5 \times v \times f_{cd} = 5.581 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN X-DIRECTION:</td>
</tr>
<tr>
<td>SUMMARY</td>
</tr>
<tr>
<td>Characteristic strength</td>
</tr>
<tr>
<td>Diameter of bars</td>
</tr>
<tr>
<td>Spacing</td>
</tr>
<tr>
<td>Total number of bars</td>
</tr>
<tr>
<td>Total area required</td>
</tr>
<tr>
<td>Total area provided</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN Y-DIRECTION:</td>
</tr>
<tr>
<td>SUMMARY</td>
</tr>
<tr>
<td>Characteristic strength</td>
</tr>
<tr>
<td>Diameter of bars</td>
</tr>
<tr>
<td>Spacing</td>
</tr>
<tr>
<td>Total number of bars</td>
</tr>
<tr>
<td>Total area required</td>
</tr>
<tr>
<td>Total area provided</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DISTRIBUTION REINFORCEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUMMARY</td>
</tr>
<tr>
<td>Characteristic strength</td>
</tr>
<tr>
<td>Diameter of bars</td>
</tr>
</tbody>
</table>

In top:
| Number of bars | 18 |
| Spacing of bars| 190 mm |
| Area required | 2535 mm²/m |
| Area provided | 2584 mm²/m |
### IN Y-DIRECTION:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>18</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>190 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>2535 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>2584 mm²/m</td>
</tr>
</tbody>
</table>

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>24</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total concrete volume</td>
<td>20.12 m³</td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>23.32 m²</td>
</tr>
</tbody>
</table>
Location: Ex2 - Cap for heavily-loaded column w. mild steel reinf.

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for nine piles

Designed using bending theory to EC2 Part 1-1. Main reinforcement is arranged in bands // to principal axes and is symmetrical about pile centres. Ultimate load and moment values to be used. Ninth pile is located under column.

Total design load on column N=30000 kN
Nominal pile diameter PileD=500 mm
Column dimension cx cx=600 mm
Column dimension cy cy=600 mm
Partial safety factor for steel gam_s=1.15
Partial safety factor for conc. gam_c=1.5
Spacing of pile ctrs in X-dir AX=1500 mm
Overhang: pile face to cap edge olap=300 mm
Length of pile cap lx=4100 mm
Spacing of pile ctrs in Y-dir AY=1500 mm
Breadth of pile cap ly=4100 mm
Char yield strength of reinft. fyk=500 N/mm²
Cover to underside of main bars Cover=150 mm
Side and top cover to main bars Scvr=75 mm
Chosen depth of pilecap h=3200 mm

Design of main steel in X-direction

Main steel diameter diaX=16 mm
Main steel spacing SpcX=40 mm

Design of main steel in Y-direction

Main steel diameter diaY=16 mm
Main steel spacing SpcY=40 mm

Distribution steel

Distribution steel diameter LSize=40 mm
Spacing of bars LSpc=260 mm
Area of dist.steel provided A2=PI*LSize^2*250/LSpc=4833 mm²/m
Check shear resistance in X-direction

At Critical Plane 1:
Shear force due to ultimate load \( V_{Ed} = \frac{N}{3} + Mx' \times 1000/AX = 10000 \text{ kN} \)
Shear resistance \( VR_{dc} = \beta' \times CR_{dc} \times ks \times v_{Rdc} \times bw \times d/1000 = 12389 \text{ kN} \)
Modified shear resistance \( VR_{dc} = VR_{dm} = 13012 \text{ kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>Shear resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10000 kN</td>
<td>13012 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = \frac{N}{3} + My' \times 1000/AY = 10000 \text{ kN} \)
Shear resistance \( VR_{dc} = \beta' \times CR_{dc} \times ks \times v_{Rdc} \times bw \times d/1000 = 12352 \text{ kN} \)
Modified shear resistance \( VR_{dc} = VR_{dm} = 12954 \text{ kN} \)

<table>
<thead>
<tr>
<th>SHEAR SUMMARY</th>
<th>Shear force</th>
<th>Shear resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10000 kN</td>
<td>12954 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor \( \beta \) \( \beta = 1 \)
Shear stress at column perimeter \( v_{Ed} = \beta \times V_{Ed} \times 1000/(uo \times d) = 4.131 \text{ N/mm}^2 \)
Maximum punching shear resistance \( v_{Rdmax} = 0.5 \times v \times f_{cd} = 4.973 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN X-DIRECTION:</td>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>40 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>99</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>19680 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>19905 mm²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MAIN REINFORCEMENT</th>
<th>Characteristic strength</th>
<th>500 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>IN Y-DIRECTION:</td>
<td>Diameter of bars</td>
<td>16 mm</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>Spacing</td>
<td>40 mm</td>
</tr>
<tr>
<td></td>
<td>Total number of bars</td>
<td>99</td>
</tr>
<tr>
<td></td>
<td>Total area required</td>
<td>19680 mm²</td>
</tr>
<tr>
<td></td>
<td>Total area provided</td>
<td>19905 mm²</td>
</tr>
</tbody>
</table>

| DISTRIBUTION REINFORCEMENT | Characteristic strength | 500 N/mm² |
|                           | Diameter of bars         | 40 mm     |
| IN X-DIRECTION:           |                         |           |
| In top:                   | Number of bars           | 16        |
|                           | Spacing of bars          | 260 mm    |
|                           | Area required            | 4800 mm²/m|
|                           | Area provided            | 4833 mm²/m|
### IN Y-DIRECTION:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>16</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>260 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>4800 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>4833 mm²/m</td>
</tr>
</tbody>
</table>

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>48</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total concrete volume</td>
<td>53.79 m³</td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>52.48 m²</td>
</tr>
</tbody>
</table>
Location: Ex3 - Cap for lightly-loaded column

Bottom layer of main steel is assumed to run in X-direction.

Pilecap for nine piles

Designed using bending theory to EC2 Part 1-1. Main reinforcement is arranged in bands // to principal axes and is symmetrical about pile centres. Ultimate load and moment values to be used. Ninth pile is located under column.

Total design load on column \( N = 500 \text{ kN} \)
Nominal pile diameter \( \text{PileD} = 250 \text{ mm} \)
Column dimension \( cx = 300 \text{ mm} \)
Column dimension \( cy = 300 \text{ mm} \)
Partial safety factor for steel \( gams = 1.15 \)
Partial safety factor for conc. \( gamc = 1.5 \)
Spacing of pile ctrs in X-dir \( AX = 750 \text{ mm} \)
Overhang: pile face to cap edge \( olap = 300 \text{ mm} \)
Length of pile cap \( lx = 2350 \text{ mm} \)
Spacing of pile ctrs in Y-dir \( AY = 750 \text{ mm} \)
Breadth of pile cap \( ly = 2350 \text{ mm} \)
Char yield strength of reinft. \( fyk = 500 \text{ N/mm}^2 \)
Cover to underside of main bars \( Cover = 150 \text{ mm} \)
Side and top cover to main bars \( Scvr = 75 \text{ mm} \)
Chosen depth of pilecap \( h = 300 \text{ mm} \)

Design of main steel in X-direction

Main steel diameter \( \text{diaX} = 16 \text{ mm} \)
Main steel spacing \( \text{SpcX} = 225 \text{ mm} \)

Design of main steel in Y-direction

Main steel diameter \( \text{diaY} = 12 \text{ mm} \)
Main steel spacing \( \text{SpcY} = 105 \text{ mm} \)

Distribution steel

Distribution steel diameter \( \text{LSize} = 12 \text{ mm} \)
Spacing of bars \( \text{LSpac} = 250 \text{ mm} \)
Area of dist.steel provided \( A2 = \pi \times \text{LSize}^2 \times 2 \times 250 / \text{LSpac} = 452.4 \text{ mm}^2 / \text{m} \)
Check shear resistance in X-direction

At Critical Plane 1:
Shear force due to ultimate load \( V_{Ed} = \frac{N}{3} + M_x' \times 1000/AX = 166.7 \text{ kN} \)
Shear resistance \( \frac{V_{Rdc}}{\beta'^*CR_{dc}*k_s*v_{Rdc}*b_w*d}{1000} \approx 205.4 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>166.7 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>205.4 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check shear resistance in Y-direction

Shear force due to ultimate load \( V_{Ed} = \frac{N}{3} + M_y' \times 1000/AY = 166.7 \text{ kN} \)
Shear resistance \( \frac{V_{Rdc}}{\beta'^*CR_{dc}*k_s*v_{Rdc}*b_w*d}{1000} \approx 202.6 \text{ kN} \)

**SHEAR SUMMARY**

<table>
<thead>
<tr>
<th>Shear force</th>
<th>166.7 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear resistance</td>
<td>202.6 kN</td>
</tr>
</tbody>
</table>

Hence section is satisfactory, no shear reinforcement is required.

Check punching shear

Check punching shear at column face:
Eccentricity factor \( \beta \) \( \beta = 1 \)
Shear stress at column perimeter \( \frac{V_{Ed}}{\beta *V_{Ed} *1000/(u_o*d)} = 3.255 \text{ N/mm}^2 \)
Maximum punching shear resistance \( \frac{V_{Rdmax}}{0.5*v*f_{cd}} = 4.973 \text{ N/mm}^2 \)
Shear stress at column perimeter is within the maximum punching shear resistance, hence satisfactory.

**MAIN REINFORCEMENT**

**IN X-DIRECTION:**

**SUMMARY**

| Characteristic strength | 500 N/mm² |
| Diameter of bars | 16 mm |
| Spacing | 225 mm |
| Total number of bars | 10 |
| Total area required | 1938 mm² |
| Total area provided | 2011 mm² |

**MAIN REINFORCEMENT**

**IN Y-DIRECTION:**

**SUMMARY**

| Characteristic strength | 500 N/mm² |
| Diameter of bars | 12 mm |
| Spacing | 105 mm |
| Total number of bars | 21 |
| Total area required | 2284 mm² |
| Total area provided | 2375 mm² |

**DISTRIBUTION REINFORCEMENT**

**SUMMARY**

| Characteristic strength | 500 N/mm² |
| Diameter of bars | 12 mm |

**IN X-DIRECTION:**

| Number of bars | 9 |
| Spacing of bars | 250 mm |
| Area required | 450 mm²/m |
| Area provided | 452.4 mm²/m |
### IN Y-DIRECTION:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>9</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
<tr>
<td>Area required</td>
<td>450 mm²/m</td>
</tr>
<tr>
<td>Area provided</td>
<td>452.4 mm²/m</td>
</tr>
</tbody>
</table>

### SIDE BARS SUMMARY

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic strength</td>
<td>500 N/mm²</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>12 mm</td>
</tr>
<tr>
<td>Total number of bars</td>
<td>4</td>
</tr>
<tr>
<td>Spacing of bars</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

### QUANTITIES

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total concrete volume</td>
<td>1.657 m³</td>
</tr>
<tr>
<td>Total area of formwork</td>
<td>2.82 m²</td>
</tr>
</tbody>
</table>
Location: Example in Ch 22 of 'Reinforced Concrete Design' by Bray

RC retaining wall to BS8002

Using either Rankine or Coulomb theory, the following assumptions are made:

- surface of rupture is a plane
- point of resultant pressure P on back of wall is at one third of the distance up from the base of the wall.

Weight/unit vol. of retained soil \( w = 18.8 \text{ kN/m}^3 \)
Height of wall \( h = 4.38 \text{ m} \)
Load above wall supported on stem \( v_o = 0 \text{ kN/m} \)
Angle of slope of soil (if any) \( \theta = 0^\circ \)
Internal friction angle of soil \( \phi = 30^\circ \)
Angle of wall-to-soil friction \( \delta = 20^\circ \)
Friction factor on underside base \( f_b = 0.4 \)
Thickness of wall \( t = 0.22 \text{ m} \)
Density of wall (stem) \( d_t = 24 \text{ kN/m}^3 \)

**Forces and moments on wall**

Maximum B.M. at base of stem \( M = Ph/h/3 + Psh/h/2 = 87.762 \text{ kNm/m} \)
Maximum S.F. at base of stem \( S = Ph + Psh = 60.111 \text{ kN/m} \)

**Base**

Thickness of base \( d = 0.19 \text{ m} \)
Length of base \( L = 3.124 \text{ m} \)
Distance from A to centre of stem \( x = 1.86 \text{ m} \)
Depth of soil over base at toe \( d_b = 0 \text{ m} \)
Depth of soil for passive resist. \( d_p = 1.5 \text{ m} \)
Allowable ground pressure \( p = 75 \text{ kN/m}^2 \)
Centroid of load lies within middle third. Pressure varies from pa at A to pb at B.

\[ \text{Max pressure is at B \quad p_{\text{max}} = pb = 62.75 \, \text{kN/m}^2} \]

As p\textsubscript{max} does not exceed \( p \) (62.75 kN/m\(^2\) \leq 75 kN/m\(^2\)), pressure beneath base is within specified limit.

**Shear forces and bending moments on base**

- Total S.F. on toe (plus is up) \( \text{Sw}_{b} = \frac{(p_{wb}+pb)}{2-dt\cdot d} \cdot (xl-t/2)-wf \)
  \[= 65.165 \, \text{kN/m} \]
- Total S.F. on heel (plus is up) \( \text{Sw}_{a} = \text{Sau} - \text{Sad} = -54.011 \, \text{kN/m} \)
- Total B.M. on toe (plus is up) \( \text{M}_{wb} = \frac{(2\cdot pb+p_{wb})\cdot (xl-t/2)^2}{6-M_{bd}} \)
  \[= 37.982 \, \text{kNm/m} \]
- Total B.M. on heel (plus is up) \( \text{M}_{wa} = \frac{(2\cdot pa+p_{wa})\cdot (x-t/2)^2}{6-M_{ad}} \)
  \[= -48.592 \, \text{kNm/m} \]

**Resistance to sliding**

- FoS sliding (frict.resist.only) \( \text{FOS}_{f} = \frac{F}{St} = 1.1093 \)
- FoS sliding (friction/cohesion + passive) \( \text{FOS}_{f2} = \frac{(F+R)}{St} = 2.0789 \)

**Resistance to overturning**

Rotation assumed to occur about lowest forward edge of base (i.e. toe).
Safety factors against overturning:
- Including vert.comp.of pressure \( \text{FOS}_{O} = \frac{Mr_{1}}{Mot} = 3.7675 \)
- Ignoring vert.comp.of pressure \( \text{FOS}_{O2} = \frac{Mr_{2}}{Mot} = 3.7675 \)
**Design Summary**

- Moment at base of stem: 87.762 kNm
- Shear at base of stem: 60.111 kN
- Moment at c-c (toe): 37.982 kNm
- Shear at c-c (toe): 65.165 kN
- Moment at c-c (heel): -48.592 kNm
- Shear at c-c (heel): -54.011 kN

FoS for Overturning:
1. With vertical component of earth pressure considered: 3.7675
2. Ignoring vertical component of earth pressure: 3.7675

FoS for Sliding:
1. Friction only: 1.1093
2. Friction+passive: 2.0789

Pressure at A: 53.43 kN/m²
Pressure at B: 62.75 kN/m²
Allowable GBP: 75 kN/m²

NOTE: The sliding resistance calculation check assumes that a heel beam/key will be provided to the underside of base.
Location: Massive, heavily-loaded wall with cohesionless soil

RC retaining wall to BS8002

Using either Rankine or Coulomb theory, the following assumptions are made:

- surface of rupture is a plane
- point of resultant pressure P on back back of wall is at one third of the distance up from the base of the wall.

Weight/unit vol.of retained soil \( w = 25 \) kN/m\(^3\)
Height of wall \( h = 20 \) m
Depth of soil surcharge \( ds = 5 \) m
Load above wall supported on stem \( vo = 1000 \) kN/m
Angle of slope of soil (if any) \( \theta = 15^\circ \)
Internal friction angle of soil \( \phi = 35^\circ \)
Angle of wall-to-soil friction \( \delta = 23.333^\circ \)
Friction factor on underside base \( f_b = 0.4 \)
Thickness of wall \( t = 1.5 \) m
Density of wall (stem) \( d_t = 24 \) kN/m\(^3\)

Forces and moments on wall

Maximum B.M. at base of stem \( M = Ph^h/3 + Psh^h/2 = 16100 \) kNm/m
Maximum S.F. at base of stem \( S = Ph + Psh = 2087.8 \) kN/m

Base

Thickness of base \( d = 2 \) m
Length of base \( L = 20 \) m
Distance from A to centre of stem \( x = 15 \) m
Depth of soil over base at toe \( db = 0 \) m
Depth of soil for passive resist. \( dp = 2 \) m
Allowable ground pressure \( p = 700 \text{ kN/m}^2 \)

Centroid of load lies within middle third. Pressure varies from \( p_a \) at A to \( p_b \) at B.

\[
\begin{array}{c}
\text{A} \\
\text{pa} \\
\text{B} \\
\text{pb}
\end{array}
\]

Maximum pressure is at B \( p_{\text{max}} = p_b = 680.73 \text{ kN/m}^2 \)

As \( p_{\text{max}} \) does not exceed \( p \) (\( 680.73 \text{ kN/m}^2 \leq 700 \text{ kN/m}^2 \)), pressure beneath base is within specified limit.

**Shear forces and bending moments on base**

Total S.F. on toe (plus is up) \( S_{\text{wb}} = \frac{(p_{\text{wb}} + p_b)}{2} - d_t \cdot d \cdot (x_1 - t/2) - w_f \)
\[= 2657.2 \text{ kN/m} \]

Total S.F. on heel (plus is up) \( S_{\text{wa}} = S_{\text{au}} - S_{\text{a}} = -1218 \text{ kN/m} \)

Total B.M. on toe (plus is up) \( M_{\text{wb}} = \frac{(2 \cdot p_b + p_{\text{wb}}) \cdot (x_1 - t/2)^2}{6} - M_{\text{bd}} \)
\[= 5669.2 \text{ kNm/m} \]

Total B.M. on heel (plus is up) \( M_{\text{wa}} = \frac{(2 \cdot p_a + p_{\text{wa}}) \cdot (x - t/2)^2}{6} - M_{\text{a}} \)
\[= -11146 \text{ kNm/m} \]

**Resistance to sliding**

FoS sliding (friction resistance only) \( F_{\text{OSF}} = \frac{F}{S_t} = 2.1557 \)

FoS sliding (friction/cohesion + passive) \( F_{\text{OSF2}} = \frac{(F + R)}{S_t} = 2.2328 \)

**Resistance to overturning**

Rotation assumed to occur about lowest forward edge of base (i.e. toe).

Safety factors against overturning:

Including vert. comp. of pressure \( F_{\text{OSOT1}} = \frac{M_r}{M_{\text{ot}}} = 7.2725 \)

Ignoring vert. comp. of pressure \( F_{\text{OSOT2}} = \frac{M_r}{M_{\text{ot}}} = 7.0898 \)
Design Summary

- Moment at base of stem: 16100 kNm
- Shear at base of stem: 2087.8 kN
- Moment at c-c (toe): 5669.2 kNm
- Shear at c-c (toe): 2657.2 kN
- Moment at c-c (heel): -11146 kNm
- Shear at c-c (heel): -1218 kN

For Overturning:
1. With vertical component of earth pressure considered: 7.2725
2. Ignoring vertical component of earth pressure: 7.0898

For Sliding:
1. Friction only: 2.1557
2. Friction+passive: 2.2328

Pressure at A: 610.08 kN/m²
Pressure at B: 680.73 kN/m²
Allowable GBP: 700 kN/m²
Location: Cohesive soil, triangular soil pressure dist.

RC retaining wall to BS8002

Using either Rankine or Coulomb theory, the following assumptions are made:

- surface of rupture is a plane
- point of resultant pressure P on back back of wall is at one third of the distance up from the base of the wall.

Weights

- Weight/unit vol. of retained soil: $w=20\text{ kN/m}^3$
- Height of wall: $h=6\text{ m}$
- Pressure due to surcharge: $press=100\text{ kN/m}^2$
- Load above wall supported on stem: $vo=0\text{ kN/m}$
- Angle of slope of soil (if any): $\theta=-10^\circ$
- Internal friction angle of soil: $\phi=35^\circ$
- Angle of wall-to-soil friction: $\delta=23.333^\circ$
- Cohesion of clay under base: $fc=100\text{ kN/m}^2$
- Thickness of wall: $t=0.22\text{ m}$
- Density of wall (stem): $dt=24\text{ kN/m}^3$

Forces and moments on wall

- Maximum B.M. at base of stem: $M=Ph^3/3+Psh^3/2=594.42\text{ kNm/m}$
- Maximum S.F. at base of stem: $S=Ph+Psh=222.56\text{ kN/m}$

Base

- Thickness of base: $d=0.4\text{ m}$
- Length of base: $L=5.25\text{ m}$
- Distance from A to centre of stem: $x=0.11\text{ m}$
- Depth of soil over base at toe: $db=2\text{ m}$
- Depth of soil for passive resist: $dp=4\text{ m}$
Allowable ground pressure \( p = 500 \text{ kN/m}^2 \)

Centroid of load outside middle third giving triangular distribution of pressure.
Pressure varies from 0 at distance \( dp \) from A to a maximum of \( pb \) at B.

\[
\begin{align*}
A & \quad \text{dp} \quad \text{lb} \quad \text{B} \\
\text{\textbackslash} \text{\textbackslash} \text{\textbackslash} \text{\textbackslash} \text{\textbackslash} \text{\textbackslash} \\
\text{pb}
\end{align*}
\]

Maximum pressure is at B \( p_{\text{max}} = pb = 150.9 \text{ kN/m}^2 \)
As \( p_{\text{max}} \) does not exceed \( p \) (\( 150.9 \text{ kN/m}^2 \leq 500 \text{ kN/m}^2 \)), pressure beneath base is within specified limit.

**Shear forces and bending moments on base**

Total S.F. on toe (plus is up) \( S_{wb} = \frac{(p_{wb} + pb)}{2} \cdot d \cdot (x_1 - t/2) - w_f = 137.74 \text{ kN/m} \)

Total S.F. on heel (plus is up) \( S_{wa} = -dt \cdot d \cdot (x - t/2) - w_{be} - w_{bs} = 0 \text{ kN/m} \)

Total B.M. on toe (plus is up) \( M_{wb} = \frac{2 \cdot pb + p_{wb}}{6} \cdot (x_1 - t/2)^2 - M_{bd} = 658.12 \text{ kNm/m} \)

Total B.M. on heel (plus is up)

**Resistance to sliding**

FoS sliding (cohes. resist. only) \( F_{OSF} = \frac{F}{St} = 1.0637 \)

FoS sliding (friction/cohesion + passive) \( F_{OSF2} = \frac{F + R}{St} = 3.5101 \)

**Resistance to overturning**

Rotation assumed to occur about lowest forward edge of base (i.e. toe).
Safety factors against overturning:

Including vert. comp. of pressure \( F_{OSOT1} = \frac{Mr1}{Mot} = 1.9682 \)

Ignoring vert. comp. of pressure \( F_{OSOT2} = \frac{Mr2}{Mot} = 1.17 \)
**Design Summary**

- Moment at stem base: 594.42 kNm
- Shear at stem base: 222.56 kN
- Moment on toe: 658.12 kNm
- Shear on toe: 137.74 kN

**FoS for Overturning**

1. With vertical component of earth pressure considered: 1.9682
2. Ignoring vertical component of earth pressure: 1.17

**FoS for Sliding**

1. Friction only: 1.0637
2. Friction + passive: 3.5101

Maximum pressure at B: 150.9 kN/m²
Allowable GBP: 500 kN/m²

NOTE: The sliding resistance calculation check assumes that a heel beam/key will be provided to the underside of base.
Location: Ex1 - RC retaining wall

Reinforced concrete retaining wall to EC7

Using either Rankine or Coulomb theory, the following assumptions are made:

- the surface of rupture is assumed to be plane
- point of resultant pressure P on back of wall is at one third of the distance up from the base of the wall.

Weight/unit vol. of retained soil \( w = 18.8 \text{ kN/m}^3 \)
Wall height \( h = 4 \text{ m} \)
Load above wall supported on stem \( v_0 = 0 \text{ kN/m} \)
Angle of slope of soil (if any) \( \theta = 0^\circ \)
Angle of wall-to-soil friction \( \delta = 0^\circ \)
Friction factor @ underside base \( f_b = 0.5 \)
Thickness of wall \( t = 0.3 \text{ m} \)
Base thickness \( d = 0.3 \text{ m} \)
Density of wall (stem) \( \text{den} = 25 \text{ kN/m}^3 \)

Forces and moments on wall - SLS

compute \( M_1 = Ph^*(d/2+h/3) = 74.364 \text{ kNm/m} \)
compute \( M_2 = Psh^*(d/2+h/2) = 0 \text{ kNm/m} \)
Maximum stem bending moment \( M = M_1 + M_2 = 74.364 \text{ kNm/m} \)
Horizontal shear at base of stem \( V = Ph + Psh = 50.133 \text{ kNm/m} \)

Check bearing pressures - SLS

Length of base \( L = 3.124 \text{ m} \)
Distance from A to centre of stem \( x = 1.86 \text{ m} \)
Depth of soil over base at toe \( d_b = 0 \text{ m} \)
Depth of soil for passive resist. \( d_p = 1.2 \text{ m} \)
Allowable ground pressure \( p = 75 \text{ kN/m}^2 \)
Centroid of load lies within middle third. Pressure varies from pa at A to pb at B.

\[
\begin{array}{c}
A \quad \text{pa} \quad \_ \quad \_ \quad \_ \quad B
\end{array}
\]

Maximum pressure is at B  \( p_{\text{max}} = pb = 58.921 \text{ kN/m}^2 \)

As \( p_{\text{max}} \leq p \) (58.921 kN/m\(^2\) \leq 75 \text{ kN/m}^2), the pressure beneath base is within the specified limit.

**Partial factors - ULS load combination 1**

Partial load factor (backfill)  \( g_1 = 1 \)
Partial load factor (v.surcharge)  \( g_2 = 0 \)

**Forces and moments on wall - ULS load combination 1**

Stem design moment  \( M_{Ed1} = g_4 * M_1 + g_5 * M_2 = 100.39 \text{ kNm/m} \)
Horizontal shear at base of stem  \( V_{Ed1} = g_4 * P_h + g_5 * P_{sh} = 67.679 \text{ kN/m} \)

**Bearing pressures - ULS load combination 1**

Centroid of load lies within middle third. Pressure varies from pal at A to pbl at B.

\[
\begin{array}{c}
A \quad \text{pal} \quad \_ \quad \_ \quad \_ \quad B
\end{array}
\]

Maximum pressure is at B  \( p_{\text{max}} = pb = 84.699 \text{ kN/m}^2 \)

**Shear forces and bending moments on base - ULS load combination 1**

Total S.F. on toe (plus is up)  \( Sw_b = ((p_{wb} + p_{b1})/2 - \text{den}*d) \times (x_1-t/2) - w_f 1 = 77.877 \text{ kN/m} \)
Total S.F. on heel (plus is up)  \( Sw_a = S_{au} - S_{ad} = -47.372 \text{ kN/m} \)
Total B.M. on toe (plus is up)  \( M_{wb} = (2*p_{b1} + p_{wb}) \times (x_1-t/2)^2/6 - M_{bd} = 44.885 \text{ kNm/m} \)
Total B.M. on heel (plus is up)  \( M_{wa} = (2*p_{b1} + p_{a1}) \times (x-t/2)^2/6 - M_{ad} = -45.958 \text{ kNm/m} \)
Resistance to sliding - ULS (load combinations 1 & 2)

Combination 1:
Sliding force \( S_{For} = S_{t1}' = 114.38 \text{ kN} \)
Friction plus passive resistance \( S_{Res} = F + R = 131.62 \text{ kN} \)
Thus the criterion for sliding is satisfied.

Combination 2:
Coefficient of passive pressure \( K_{p}' = 2.4442 \)
Sliding force \( S_{For} = K_{a2}/K_{a}*(g_{7}P_{1}+g_{8}P_{2}) = 103.99 \text{ kN} \)
Friction plus passive resistance \( S_{Res} = F + R = 124.09 \text{ kN} \)
Thus the criterion for sliding is satisfied.

Resistance to overturning - ULS (load combinations 1 & 2)

Combination 1:
Total restraining mt. (with Mrv) \( Mr_{1} = g_{9}*(M_{re}+M_{rw}+M_{rv}) = 329.66 \text{ kNm/m} \)
ditto (without Mrv) \( Mr_{2} = g_{9}*(M_{re}+M_{rw}) = 329.66 \text{ kNm/m} \)
Total overturning moment \( M_{ot} = Mt_{1} + Mt_{2} = 91.344 \text{ kNm/m} \)

Combination 2:
Total restraining mt. (with Mrv) \( Mr_{1} = g_{9}'*(M_{re}+M_{rw}+M_{rv}) = 329.66 \text{ kNm/m} \)
ditto (without Mrv) \( Mr_{2} = g_{9}'*(M_{re}+M_{rw}) = 329.66 \text{ kNm/m} \)
Total overturning moment \( M_{ot} = Mt_{1} + Mt_{2} = 101.92 \text{ kNm/m} \)

Partial factors - ULS load combination 2

Forces and moments on wall - ULS load combination 2

Stem design moment \( M_{Ed2} = g_{4}M_{1} + g_{5}M_{2} = 74.364 \text{ kNm/m} \)
Horizontal shear at base of stem \( V_{Ed2} = g_{4}Ph + g_{5}P_{sh} = 50.133 \text{ kN/m} \)

Bearing pressures - ULS load combination 2

\[
Ka_{2} = \frac{1-\sin(\text{rad}(\phi'd_{1}'))}{1+\sin(\text{rad}(\phi'd_{1}'))}
\]
Centroid of load lies within middle third. Pressure varies from $\overline{pa_2}$ at A to $\overline{pb_2}$ at B.

$$\overline{pa_2} \quad \overline{-}\quad \overline{-}\quad \overline{pb_2}$$

Maximum pressure is at B $\quad p_{max_2}=\overline{pb_2}=70.53 \text{ kN/m}^2$

**Shear forces and bending moments on base - ULS load combination 2**

Total S.F. on toe (plus is up) $\quad S_{wb_2}=\left(\frac{(pwb+pb_2)/2-den*d}{2}\right)*(x_1-t/2) - w_{f_2}=65.343 \text{ kN/m}$

Total S.F. on heel (plus is up) $\quad S_{wa_2}=S_{a_u}-S_{a_d}=-51.275 \text{ kN/m}$

Total B.M. on toe (plus is up) $\quad M_{wb_2}=\left(2*pb_2+pwb\right)*(x_1-t/2)^2/6-M_{b_d} = 37.301 \text{ kNm/m}$

Total B.M. on heel (plus is up) $\quad M_{wa_2}=\left(2*pa_2+pwa\right)*(x-t/2)^2/6-M_{a_d} = -47.112 \text{ kNm/m}$

**Design Summary - ULS load combinations 1 & 2 to EC7**

- Moment at base of stem: 100.39 kNm
- Shear at base of stem: 67.679 kN
- Moment at b-b (toe): 44.885 kNm
- Shear at b-b (toe): 77.877 kN
- Moment at a-a (heel): -47.112 kNm
- Shear at a-a (heel): -51.275 kN

**Pressures under base - SLS**

- Pressure @ A: 57.611 kN/m²
- Pressure @ B: 58.921 kN/m²
- Allowable GBP: 75 kN/m²

NOTE: The sliding resistance calculation check assumes that a heel beam/key will be provided to the underside of base.
Location: Ex2 - Massive, heavily-loaded wall with cohesionless soil

Reinforced concrete retaining wall to EC7

Using either Rankine or Coulomb theory, the following assumptions are made:

- the surface of rupture is assumed to be plane
- point of resultant pressure $P$ on back of wall is at one third of the distance up from the base of the wall.

Weight/unit vol. of retained soil $w=25$ kN/m$^3$
Wall height $h=20$ m
Depth of soil surcharge $ds=5$ m
Load above wall supported on stem $vo=1000$ kN/m
Angle of slope of soil (if any) $\theta=15^\circ$
Angle of wall-to-soil friction $\delta=0^\circ$
Friction factor @ underside base $fb=0.4$
Thickness of wall $t=1.5$ m
Base thickness $d=2$ m
Density of wall (stem) $den=25$ kN/m$^3$

Forces and moments on wall - SLS

$M1=Ph*(d/2+h/3)=10989$ kNm/m
$M2=Psh*(d/2+h/2)=7198.3$ kNm/m
Maximum stem bending moment $M=M1+M2=18188$ kNm/m
Horizontal shear at base of stem $V=Ph+Fsh=2087.8$ kN/m

Check bearing pressures - SLS

Length of base $L=20$ m
Distance from A to centre of stem $x=15$ m
Depth of soil over base at toe $db=0$ m
Depth of soil for passive resist. $dp=3$ m
Allowable ground pressure \( p = 700 \text{ kN/m}^2 \)

Centroid of load lies within middle third. Pressure varies from \( p_a \) at A to \( p_b \) at B.

\[
\begin{array}{c}
A \\
p_a & - & - & - \\
\hline \\
& & & & B \\
\end{array} \quad \begin{array}{c}
p_b \\
\end{array}
\]

Maximum pressure is at B \( \text{p}_{\text{max}} = 686.48 \text{ kN/m}^2 \)

As \( \text{p}_{\text{max}} \leq p \ (686.48 \text{ kN/m}^2 \leq 700 \text{ kN/m}^2) \), the pressure beneath base is within the specified limit.

**Partial factors - ULS load combination 1**

Partial load factor (backfill) \( g_1 = 1 \)
Partial load factor (v.surcharge) \( g_2 = 0 \)

**Forces and moments on wall - ULS load combination 1**

Stem design moment \( \text{M}_{\text{Ed}1} = g_4 \cdot M_1 + g_5 \cdot M_2 = 25633 \text{ kNm/m} \)

Horizontal shear at base of stem \( \text{V}_{\text{Ed}1} = g_4 \cdot P_h + g_5 \cdot P_{sh} = 2916.7 \text{ kN/m} \)

**Bearing pressures - ULS load combination 1**

Centroid of load lies within middle third. Pressure varies from \( p_{a1} \) at A to \( p_{b1} \) at B.

\[
\begin{array}{c}
A \\
p_{a1} & - & - & - \\
\hline \\
& & & & B \\
\end{array} \quad \begin{array}{c}
p_{b1} \\
\end{array}
\]

Maximum pressure is at B \( \text{p}_{\text{max}1} = 921.03 \text{ kN/m}^2 \)
Shear forces and bending moments on base - ULS load combination 1

Total S.F. on toe (plus is up) \[ S_{wb} = \frac{(p_{wb}+p_{b1})}{2} - d \times (x_1 - t/2) - w_{f1} \]
= 3430.6 kN/m

Total S.F. on heel (plus is up) \[ S_{wa} = S_{a1} - S_{a2} = -904.01 \text{ kN/m} \]

Total B.M. on toe (plus is up) \[ M_{wb} = \frac{(2*p_{b1} + p_{wb})}{2} \times (x_1 - t/2)^2/6 - M_{bd} \]
= 7482.1 kNm/m

Total B.M. on heel (plus is up) \[ M_{wa} = \frac{(2*p_{a1} + p_{wa})}{2} \times (x - t/2)^2/6 - M_{ad} \]
= -15300 kNm/m

Resistance to sliding - ULS (load combinations 1 & 2)

Combination 1:
Sliding force \[ S_{For} = S_{t1}' = 3818.1 \text{ kN} \]
Friction plus passive resistance \[ S_{Res} = F + R = 4816.8 \text{ kN} \]
Thus the criterion for sliding is satisfied.

Combination 2:
Coefficient of passive pressure \[ K_p' = 2.9117 \]
Sliding force \[ S_{For} = K_a2/K_a \times (g7 * P1 + g8 * P2) = 3443 \text{ kN} \]
Friction plus passive resistance \[ S_{Res} = F + R = 4729.2 \text{ kN} \]
Thus the criterion for sliding is satisfied.

Resistance to overturning - ULS (load combinations 1 & 2)

Combination 1:
Total restraining mt. (with Mrv) \[ M_{r1} = g9 \times (M_{re} + M_{rw} + M_{rv}) = 111094 \text{ kNm/m} \]
Total overturning moment \[ M_{ot} = M_{t1} + M_{t2} = 25391 \text{ kNm/m} \]

Combination 2:
Total restraining mt. (with Mrv) \[ M_{r1} = g9' \times (M_{re} + M_{rw} + M_{rv}) = 111094 \text{ kNm/m} \]
Total overturning moment \[ M_{ot} = M_{t1} + M_{t2} = 26128 \text{ kNm/m} \]

Partial factors - ULS load combination 2

Forces and moments on wall - ULS load combination 2

Stem design moment \[ M_{Ed2} = g4 \times M1 + g5 \times M2 = 20347 \text{ kNm/m} \]
Horizontal shear at base of stem \[ V_{Ed2} = g4 \times Ph + g5 \times Psh = 2284.1 \text{ kN/m} \]
**Bearing pressures - ULS load combination 2**

Centroid of load lies within middle third. Pressure varies from $p_a2$ at $A$ to $p_b2$ at $B$.

Maximum pressure is at $B$  \[ p_{max2} = p_b2 = 802.35 \, \text{kN/m}^2 \]

**Shear forces and bending moments on base - ULS load combination 2**

| Total S.F. on toe (plus is up) | $S_{wb2} = \left( \frac{p_{wb2} + p_b2}{2} - d \cdot d \right) \cdot \left( x_1 - \frac{t}{2} \right)$  | $w_f2 = 2987.3 \, \text{kN/m}$ |
| Total S.F. on heel (plus is up) | $S_{wa2} = S_{au} - S_{ad} = -1354 \, \text{kN/m}$ |
| Total B.M. on toe (plus is up)  | $M_{wb2} = \left( 2 \cdot p_b2 + p_{wb} \right) \cdot \left( x_1 - \frac{t}{2} \right)^2 - M_{bd}$  | $6496.9 \, \text{kNm/m}$ |
| Total B.M. on heel (plus is up) | $M_{wa2} = \left( 2 \cdot p_a2 + p_{wa} \right) \cdot \left( x - \frac{t}{2} \right)^2 - M_{ad}$  | $-16875 \, \text{kNm/m}$ |
Design Summary - ULS load

combinations 1 & 2 to EC7

- Moment at base of stem: 25633 kNm
- Shear at base of stem: 2916.7 kN
- Moment at b-b (toe): 7482.1 kNm
- Shear at b-b (toe): 3430.6 kN
- Moment at a-a (heel): -16875 kNm
- Shear at a-a (heel): -1354 kN

Pressures under base - SLS

- Pressure @ A: 611.33 kN/m²
- Pressure @ B: 686.48 kN/m²
- Allowable GBP: 700 kN/m²

NOTE: The sliding resistance calculation check assumes that a heel beam/key will be provided to the underside of base.
Reinforced concrete retaining wall to EC7

Using either Rankine or Coulomb theory, the following assumptions are made:

- the surface of rupture is assumed to be plane
- point of resultant pressure P on back of wall is at one third of the distance up from the base of the wall.

**Location:** Ex3 - Cohesive soil, triangular soil pressure dist.

- Weight/unit vol. of retained soil: \( w = 20 \text{ kN/m}^3 \)
- Wall height: \( h = 6 \text{ m} \)
- Pressure due to surcharge: \( \text{press} = 50 \text{ kN/m}^2 \)
- Load above wall supported on stem: \( \text{vo} = 0 \text{ kN/m} \)
- Angle of slope of soil (if any): \( \theta = -10^\circ \)
- Angle of wall-to-soil friction \( \delta = 23.33^\circ \)
- Undrained shear strength: \( C_u = 50 \text{ kN/m}^2 \)
- Thickness of wall: \( t = 0.4 \text{ m} \)
- Base thickness: \( d = 0.4 \text{ m} \)
- Density of wall (stem): \( \text{den} = 25 \text{ kN/m}^3 \)

**Forces and moments on wall - SLS**

\[
M_1 = P_h \left( \frac{d}{2} + \frac{h}{3} \right) = 161.18 \text{ kNm/m}
\]

\[
M_2 = P_{sh} \left( \frac{d}{2} + \frac{h}{2} \right) = 238.87 \text{ kNm/m}
\]

Maximum stem bending moment: \( M = M_1 + M_2 = 400.06 \text{ kNm/m} \)

Horizontal shear at base of stem: \( V = P_h + P_{sh} = 147.91 \text{ kN/m} \)

**Check bearing pressures - SLS**

- Length of base: \( L = 5.25 \text{ m} \)
- Distance from A to centre of stem: \( x = 0.2 \text{ m} \)
- Depth of soil over base at toe: \( d_b = 2 \text{ m} \)
- Depth of soil for passive resist.: \( d_p = 4 \text{ m} \)
Allowable ground pressure $p = 500 \text{kN/m}^2$

Centroid of load lies within middle third. Pressure varies from $p_a$ at $A$ to $p_b$ at $B$.

Maximum pressure is at $B$ $p_{max} = p_b = 102.17 \text{kN/m}^2$

As $p_{max} \leq p$ ($102.17 \text{kN/m}^2 \leq 500 \text{kN/m}^2$), the pressure beneath base is within the specified limit.

**Partial factors - ULS load combination 1**

Partial load factor (backfill) $g_1 = 1$
Partial load factor (v.surcharge) $g_2 = 0$

**Forces and moments on wall - ULS load combination 1**

Stem design moment $M_{Ed1} = g_4 M_1 + g_5 M_2 = 575.91 \text{kNm/m}$
Horizontal shear at base of stem $V_{Ed1} = g_4 P_h + g_5 P_{sh} = 210.88 \text{kN/m}$

**Bearing pressures - ULS load combination 1**

Centroid of load lies within middle third. Pressure varies from $p_{a1}$ at $A$ to $p_{b1}$ at $B$.

Maximum pressure is at $B$ $p_{max1} = p_{b1} = 128.4 \text{kN/m}^2$
Shear forces and bending moments on base - ULS load combination 1

Total S.F. on toe (plus is up) \[ S_{wb} = \frac{(p_{wb}+p_{b1})}{2} - \text{den} \cdot d \cdot (x_1-t/2) - w_f \]
= 184.89 kN/m

Total S.F. on heel (plus is up) \[ S_{wa} = S_{a}-S_{d} = 0 \text{ kN/m} \]

Total B.M. on toe (plus is up) \[ M_{wb} = \frac{(2 \cdot p_{b1}+p_{wb})}{2} \cdot (x_1-t/2)^2/6 - M_{bd} \]
= 606.26 kNm/m

Total B.M. on heel (plus is up) \[ M_{wa} = \frac{(2 \cdot p_{a1}+p_{wa})}{2} \cdot (x-t/2)^2/6 - M_{ad} \]
= 0 kNm/m

Resistance to sliding - ULS (load combinations 1 & 2)

Combination 1:
Sliding force \[ S_{For} = S_{t1}' = 502.46 \text{ kN} \]
Cohesion plus passive resistance \[ S_{Res} = F + R = 829.84 \text{ kN} \]
Thus the criterion for sliding is satisfied.

Combination 2:
Coefficient of passive pressure \[ K_p' = 2.9117 \]
Sliding force \[ S_{For} = K_a \cdot g_7 \cdot P_1 + g_8 \cdot P_2 = 616.38 \text{ kN} \]
Cohesion plus passive resistance \[ S_{Res} = F + R = 653.37 \text{ kN} \]
Thus the criterion for sliding is satisfied.

Resistance to overturning - ULS (load combinations 1 & 2)

Combination 1:
Total restraining mt. (with Mrv) \[ M_{r1} = g_9 \cdot (M_{re}+M_{rw}+M_{rv}) = 987.45 \text{ kNm/m} \]
ditto (without Mrv) \[ M_{r2} = g_9 \cdot (M_{re}+M_{rw}) = 820.14 \text{ kNm/m} \]
Total overturning moment \[ M_{ot} = M_{t1} + M_{t2} = 574.84 \text{ kNm/m} \]

Combination 2:
Total restraining mt. (with Mrv) \[ M_{r1} = g_9' \cdot (M_{re}+M_{rw}+M_{rv}) = 987.45 \text{ kNm/m} \]
ditto (without Mrv) \[ M_{r2} = g_9' \cdot (M_{re}+M_{rw}) = 820.14 \text{ kNm/m} \]
Total overturning moment \[ M_{ot} = M_{t1} + M_{t2} = 704.64 \text{ kNm/m} \]

Partial factors - ULS load combination 2

Forces and moments on wall - ULS load combination 2

Stem design moment \[ M_{Ed2} = g_4 \cdot M_1 + g_5 \cdot M_2 = 471.72 \text{ kNm/m} \]
Horizontal shear at base of stem \[ V_{Ed2} = g_4 \cdot P_h + g_5 \cdot P_{sh} = 170.31 \text{ kN/m} \]
Bearing pressures - ULS load combination 2

Centroid of load lies within middle third. Pressure varies from $p_{a2}$ at A to $p_{b2}$ at B.

$$\begin{align*}
\text{Maximum pressure is at B} & \quad p_{\text{max}2} = p_{b2} = 143.15 \text{ kN/m}^2 \\
\end{align*}$$

Shear forces and bending moments on base - ULS load combination 2

$$\begin{align*}
\text{Total S.F. on toe (plus is up)} & \quad Sw_{b2} = \left(\frac{p_{wb} + p_{b2}}{2} - \text{den} \cdot d\right) \cdot (x_1 - t/2) \\
\text{Total S.F. on heel (plus is up)} & \quad Sw_{a2} = Sw_{a1} - Sw_{d1} = 0 \text{ kN/m} \\
\text{Total B.M. on toe (plus is up)} & \quad M_{wb2} = \left(2 \cdot p_{b2} + p_{wb}\right) \cdot (x_1 - t/2)^2 / 6 - M_{bd} \\
\text{Total B.M. on heel (plus is up)} & \quad M_{wa2} = \left(2 \cdot p_{a2} + p_{wa}\right) \cdot (x - t/2)^2 / 6 - M_{ad} = 0 \text{ kNm/m} \\
\end{align*}$$
Design Summary - ULS load

combinations 1 & 2 to EC7

- Moment at stem base: 575.91 kNm
- Shear at stem base: 210.88 kN
- Moment on toe: 635.33 kNm
- Shear on toe: 184.89 kN

Pressures under base - SLS

- Pressure @ A: 41.166 kN/m²
- Pressure @ B: 102.17 kN/m²
- Allowable GBP: 500 kN/m²

NOTE: The sliding resistance calculation check assumes that a heel beam/key will be provided to the underside of base.
Location: Ex4 - Ex 10.6 of 'Reinforced Concrete Design' by B Mosley

Reinforced concrete retaining wall to EC7

Using either Rankine or Coulomb theory, the following assumptions are made:

- the surface of rupture is assumed to be plane
- point of resultant pressure \( P \) on back of wall is at one third of the distance up from the base of the wall.

Weight/unit vol. of retained soil \( w = 17 \) kN/m³
Wall height \( h = 4.5 \) m
Pressure due to surcharge \( \text{press} = 5 \) kN/m²
Load above wall supported on stem \( \text{vo} = 0 \) kN/m
Angle of slope of soil (if any) \( \theta = 0^\circ \)
Angle of wall-to-soil friction \( \delta = 0^\circ \)
Friction factor \( f_b \) at underside base \( f_b = 0.5 \)
Thickness of wall \( t = 0.315 \) m
Base thickness \( d = 0.4 \) m
Density of wall (stem) \( \text{den} = 25 \) kN/m³

Forces and moments on wall - SLS

\[
M_1 = P_h \left( \frac{d}{2} + h/3 \right) = 97.537 \text{ kNm/m}
\]
\[
M_2 = P_{sh} \left( \frac{d}{2} + h/2 \right) = 18.375 \text{ kNm/m}
\]
Maximum stem bending moment \( M = M_1 + M_2 = 115.91 \) kNm/m
Horizontal shear at base of stem \( V = P_h + P_{sh} = 64.874 \) kN/m

Check bearing pressures - SLS

Length of base \( L = 3.4 \) m
Distance from A to centre of stem \( x = 2.4 \) m
Depth of soil over base at toe \( d_b = 0 \) m
Depth of soil for passive resist. \( d_p = 0.6 \) m
Allowable ground pressure \( p = 500 \text{ kN/m}^2 \)
Centroid of load lies within middle third. Pressure varies from \( p_a \) at A to \( p_b \) at B.

\[
\begin{array}{c}
\text{A} \\
\text{p}_a \\
\text{---} \quad \text{---} \quad \text{---} \\
\text{---} \quad \text{---} \quad \text{---} \\
\text{B} \\
\text{pb}
\end{array}
\]

Maximum pressure is at B \( p_{\text{max}} = 100.21 \text{ kN/m}^2 \)
As \( p_{\text{max}} \leq p \ (100.21 \text{ kN/m}^2 \leq 500 \text{ kN/m}^2) \), the pressure beneath base is within the specified limit.

**Partial factors - ULS load combination 1**

Partial load factor (backfill) \( g_1 = 1 \)
Partial load factor (v.surcharge) \( g_2 = 0 \)

**Forces and moments on wall - ULS load combination 1**

Stem design moment \( M_{E1} = g_4*M_1 + g_5*M_2 = 159.24 \text{ kNm/m} \)
Horizontal shear at base of stem \( V_{E1} = g_4*P_h + g_5*P_{sh} = 88.705 \text{ kN/m} \)

**Bearing pressures - ULS load combination 1**

\[
\begin{array}{c}
\text{Soil side} \\
\text{W} \\
\text{x} \\
\text{x}_1 \\
M_{E1}
\end{array}
\]

Centroid of load lies within middle third. Pressure varies from \( p_{a1} \) at A to \( p_{b1} \) at B.

\[
\begin{array}{c}
\text{A} \\
\text{p}_{a1} \\
\text{---} \quad \text{---} \quad \text{---} \\
\text{---} \quad \text{---} \quad \text{---} \\
\text{B} \\
\text{p}_{b1}
\end{array}
\]

Maximum pressure is at B \( p_{\text{max}1} = 137.31 \text{ kN/m}^2 \)
Shear forces and bending moments on base - ULS load combination 1

Total S.F. on toe (plus is up)  \( Swb=((pwb+pb1)/2-den\cdot d)\cdot(x1-t/2)-wf1 \)
\[ =94.882 \text{ kN/m} \]
Total S.F. on heel (plus is up)  \( Swa=Sau-Sad=-64.259 \text{ kN/m} \)
Total B.M. on toe (plus is up)  \( Mwb=(2\cdot pb1+pwb)\cdot(x1-t/2)^2/6-Mbd \)
\[ =41.707 \text{ kNm/m} \]
Total B.M. on heel (plus is up)  \( Mwa=(2\cdot pa1+pwa)\cdot(x-t/2)^2/6-Mad \)
\[ =-104.82 \text{ kNm/m} \]

Resistance to sliding - ULS (load combinations 1 & 2)

Combination 1:
Sliding force  \( SFor=Stl'=112.24 \text{ kN} \)
Friction plus passive resistance  \( SRes=F+R=129.67 \text{ kN} \)
Thus the criterion for sliding is satisfied.

Combination 2:
Coefficient of passive pressure  \( Kp'=2.4442 \)
Sliding force  \( SFor=Ka2/Ka\cdot(g7\cdot P1+g8\cdot P2)=104.02 \text{ kN} \)
Friction plus passive resistance  \( SRes=F+R=127.97 \text{ kN} \)
Thus the criterion for sliding is satisfied.

Resistance to overturning - ULS (load combinations 1 & 2)

Combination 1:
Total restraining mt. (with Mrv)  \( Mr1=g9\cdot (Mre+Mrw+Mrv)=435.74 \text{ kNm/m} \)
ditto (without Mrv)  \( Mr2=g9\cdot (Mre+Mrw)=435.74 \text{ kNm/m} \)
Total overturning moment  \( Mot=Mt1+Mt2=152.24 \text{ kNm/m} \)

Combination 2:
Total restraining mt. (with Mrv)  \( Mr1=g9'\cdot (Mre+Mrw+Mrv)=435.74 \text{ kNm/m} \)
ditto (without Mrv)  \( Mr2=g9'\cdot (Mre+Mrw)=435.74 \text{ kNm/m} \)
Total overturning moment  \( Mot=Mt1+Mt2=168.3 \text{ kNm/m} \)

Partial factors - ULS load combination 2

Forces and moments on wall - ULS load combination 2

Stem design moment  \( MEd2=g4\cdot M1+g5\cdot M2=121.42 \text{ kNm/m} \)
Horizontal shear at base of stem  \( VEd2=g4\cdot Ph+g5\cdot Psh=67.124 \text{ kN/m} \)
Bearing pressures - ULS load combination 2

Soil side

\[ W \]

\[ x \]

\[ x_1 \]

\[ VEd2 \]

\[ A \]

\[ L \]

\[ B \]

\[ d_b \]

\[ d \]

\[ Ka_2 = \frac{1 - \sin(\text{RAD}(\phi d'1))}{1 + \sin(\text{RAD}(\phi d'1))} \]

Centroid of load lies within middle third. Pressure varies from \( p_{a2} \) at A to \( p_{b2} \) at B.

\[ p_{a2} - \_] - \_ - \_ - \_ - \_ \]

\[ p_{b2} \]

Maximum pressure is at B \( p_{\text{max}2} = p_{b2} = 119.58 \text{ kN/m}^2 \)

Shear forces and bending moments on base - ULS load combination 2

Total S.F. on toe (plus is up)

\[ Swb_2 = \frac{(p_{wb} + p_{b2})/(2 - \text{den} \times d)}{x_1 - t/2} \]

- \( w_{f2} = 82.152 \text{ kN/m} \)

Total S.F. on heel (plus is up)

\[ Swa_2 = S_{Au} - S_{Ad} = -72.208 \text{ kN/m} \]

Total B.M. on toe (plus is up)

\[ M_{wb2} = \frac{(2 \times p_{b2} + p_{wb}) \times (x_1 - t/2)^2}{6 - M_{bd}} \]

- \( M_{bd} = 36.034 \text{ kNm/m} \)

Total B.M. on heel (plus is up)

\[ M_{wa2} = \frac{(2 \times p_{a2} + p_{wa}) \times (x - t/2)^2}{6 - M_{ad}} \]

- \( M_{ad} = -107.88 \text{ kNm/m} \)
Design Summary - ULS load

combinations 1 & 2 to EC7

Moment at base of stem 159.24 kNm
Shear at base of stem 88.705 kN
Moment at b-b (toe) 41.707 kNm
Shear at b-b (toe) 94.882 kN
Moment at a-a (heel) -107.88 kNm
Shear at a-a (heel) -72.208 kN

Pressures under base - SLS

Pressure @ A 48.146 kN/m²
Pressure @ B 100.21 kN/m²
Allowable GBP 500 kN/m²

NOTE: The sliding resistance calculation check assumes that a heel beam/key will be provided to the underside of base.
Reinforced concrete retaining wall to EC7

Using either Rankine or Coulomb theory, the following assumptions are made:

- the surface of rupture is assumed to be plane
- point of resultant pressure $P$ on back of wall is at one third of the distance up from the base of the wall.

Weight/unit vol. of retained soil $w = 20 \text{kN/m}^3$
Wall height $h = 3.5 \text{ m}$
Pressure due to surcharge $\text{press} = 5 \text{kN/m}^2$
Load above wall supported on stem $\text{vo} = 0 \text{kN/m}$
Angle of slope of soil (if any) $\theta = 0^\circ$
Angle of wall-to-soil friction $\delta = 0^\circ$
Undrained shear strength $C_u = 50 \text{kN/m}^2$
Thickness of wall $t = 0.4 \text{ m}$
Base thickness $d = 0.4 \text{ m}$
Density of wall (stem) $\text{den} = 25 \text{kN/m}^3$

Forces and moments on wall - SLS

$
\text{compute}
M_1 = Ph^*(d/2+h/3) = 45.368 \text{ kNm/m}

\text{compute}
M_2 = Psh^*(d/2+h/2) = 9.2475 \text{ kNm/m}

\text{Maximum stem bending moment}
M = M_1 + M_2 = 54.616 \text{ kNm/m}

\text{Horizontal shear at base of stem}
V = Ph + Psh = 37.939 \text{ kN/m}

Check bearing pressures - SLS

Length of base $L = 3 \text{ m}$
Distance from $A$ to centre of stem $x = 2 \text{ m}$
Depth of soil over base at toe $db = 0 \text{ m}$
Depth of soil for passive resist. $dp = 0.9 \text{ m}$
Allowable ground pressure $p = 150 \text{ kN/m}^2$

Centroid of load lies within middle third. Pressure varies from $p_a$ at A to $p_b$ at B.

Maximum pressure is at B $p_{max} = p_b = 66.925 \text{ kN/m}^2$

As \( p_{max} \leq p \) (66.925 kN/m$^2$ $\leq$ 150 kN/m$^2$), the pressure beneath base is within the specified limit.

**Partial factors - ULS load combination 1**

Partial load factor (backfill) $g_1 = 1$
Partial load factor (v.surcharge) $g_2 = 0$

**Forces and moments on wall - ULS load combination 1**

Stem design moment $M_{Ed1} = g_4 \times M_1 + g_5 \times M_2 = 75.118 \text{ kNm/m}$
Horizontal shear at base of stem $V_{Ed1} = g_4 \times P_h + g_5 \times P_{sh} = 51.928 \text{ kN/m}$

**Bearing pressures - ULS load combination 1**

Centroid of load lies within middle third. Pressure varies from $p_{a1}$ at A to $p_{b1}$ at B.

Maximum pressure is at B $p_{max1} = p_{b1} = 95.129 \text{ kN/m}^2$
SCALE 5.48                     Office 1007                 Proforma 750

Sample output for SCALE Proforma 750. (ans=5) Page: 3
Loadings and foundations Made by: IFB
Concrete retaining wall Date: 02/12/19
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Shear forces and bending moments on base - ULS load combination 1

Total S.F. on toe (plus is up) Swb=((pwb+pb1)/2-den*d)*(x1-t/2)-wf1 =63.009 kN/m
Total S.F. on heel (plus is up) Swa=Sau-Sad=-32.943 kN/m
Total B.M. on toe (plus is up) Mwb=(2*pb1+pwb)*(x1-t/2)^2/6-Mbd =25.883 kNm/m
Total B.M. on heel (plus is up) Mwa=(2*pa1+pwa)*(x-t/2)^2/6-Mad =-37.385 kNm/m

Resistance to sliding - ULS (load combinations 1 & 2)

Combination 1:
  Sliding force SFor=St1'=79.769 kN
  Cohesion plus passive resistance SRes=F+R=109.02 kN
  Thus the criterion for sliding is satisfied.

Combination 2:
  Coefficient of passive pressure Kp'=2.0747
  Sliding force SFor=Ka2/Ka*(g7*P1+g8*P2)=76.312 kN
  Cohesion plus passive resistance SRes=F+R=88.058 kN
  Thus the criterion for sliding is satisfied.

Resistance to overturning - ULS (load combinations 1 & 2)

Combination 1:
  Total restraining mt.(with Mrv) Mr1=g9*(Mre+Mrw+Mrv)=310.14 kNm/m
     ditto (without Mrv) Mr2=g9*(Mre+Mrw)=310.14 kNm/m
  Total overturning moment Mot=Mt1+Mt2=74.398 kNm/m

Combination 2:
  Total restraining mt.(with Mrv) Mr1=g9'**(Mre+Mrw+Mrv)=310.14 kNm/m
     ditto (without Mrv) Mr2=g9'**(Mre+Mrw)=310.14 kNm/m
  Total overturning moment Mot=Mt1+Mt2=84.885 kNm/m

Partial factors - ULS load combination 2

Forces and moments on wall - ULS load combination 2

Stem design moment MEd2=g4*M1+g5*M2=57.39 kNm/m
Horizontal shear at base of stem VEd2=g4*Ph+g5*Psh=39.361 kN/m
Bearing pressures - ULS load combination 2

Centroid of load lies within middle third. Pressure varies from pa2 at A to pb2 at B.

Maximum pressure is at B pmax2=pb2=81.524 kN/m²

Shear forces and bending moments on base - ULS load combination 2

Total S.F. on toe (plus is up) Swb2=((pwb+pb2)/2-den*d)*(x1-t/2) -wf2=53.409 kN/m
Total S.F. on heel (plus is up) Swa2=Sau-Sad=-42.257 kN/m
Total B.M. on toe (plus is up) Mwb2=(2*pb2+pwb)*(x1-t/2)^2/6-Mbd =21.872 kNm/m
Total B.M. on heel (plus is up) Mwa2=(2*pa2+pwa)*(x-t/2)^2/6-Mad =-43.817 kNm/m
Design Summary - ULS load

combinations 1 & 2 to EC7

- Moment at base of stem: 75.118 kNm
- Shear at base of stem: 51.928 kN
- Moment at b-b (toe): 25.883 kNm
- Shear at b-b (toe): 63.009 kN
- Moment at a-a (heel): -43.817 kNm
- Shear at a-a (heel): -42.257 kN

Pressures under base - SLS

- Pressure @ A: 66.409 kN/m²
- Pressure @ B: 66.925 kN/m²
- Allowable GBP: 150 kN/m²

NOTE: The sliding resistance calculation check assumes that a heel beam/key will be provided to the underside of base.
Location: Ex6 - Ex from 'The Structural Engineer' Jan 2014 Vol.92

Reinforced concrete retaining wall to EC7

Using either Rankine or Coulomb theory, the following assumptions are made:

- the surface of rupture is assumed to be plane
- point of resultant pressure $P$ on back of wall is at one third of the distance up from the base of the wall.

- Weight/unit vol. of retained soil $w=18$ kN/m$^3$
- Wall height $h=5$ m
- Pressure due to surcharge $\text{press}=10$ kN/m$^2$
- Load above wall supported on stem $\text{vo}=0$ kN/m
- Angle of slope of soil (if any) $\theta=0^\circ$
- Angle of wall-to-soil friction $\delta=0^\circ$
- Friction factor @ underside base $f_b=0.455$
- Thickness of wall $t=0.45$ m
- Base thickness $d=0.45$ m
- Density of wall (stem) $\text{den}=25$ kN/m$^3$

Forces and moments on wall - SLS

$$M_1=Ph*(d/2+h/3)=141.87 \text{ kNm/m}$$
$$M_2=Psh*(d/2+h/2)=45.417 \text{ kNm/m}$$
$$M=M_1+M_2=187.29 \text{ kNm/m}$$
$$V=Ph+Psh=91.666 \text{ kN/m}$$

Check bearing pressures - SLS

$$L=4 \text{ m}$$
$$x=3.225 \text{ m}$$
$$\text{db}=0 \text{ m}$$
$$\text{dp}=0.55 \text{ m}$$
Allowable ground pressure \( p = 250 \text{ kN/m}^2 \)

Centroid of load lies within middle third. Pressure varies from \( p_a \) at A to \( p_b \) at B.

\[
\begin{array}{c}
\text{A} \quad \text{B} \\
p_a & \text{---} \quad \text{---} \quad \text{---} & p_b
\end{array}
\]

Maximum pressure is at B \( p_{\text{max}} = p_b = 149.17 \text{ kN/m}^2 \)

As \( p_{\text{max}} \leq p \ (149.17 \text{ kN/m}^2 \leq 250 \text{ kN/m}^2) \), the pressure beneath base is within the specified limit.

**Partial factors - ULS load combination 1**

Partial load factor (backfill) \( g_1 = 1 \)

Partial load factor (v.surcharge) \( g_2 = 0 \)

**Forces and moments on wall - ULS load combination 1**

Stem design moment \( M_{\text{Ed1}} = g_4 M_1 + g_5 M_2 = 259.65 \text{ kNm/m} \)

Horizontal shear at base of stem \( V_{\text{Ed1}} = g_4 P_h + g_5 P_{sh} = 126.25 \text{ kN/m} \)

**Bearing pressures - ULS load combination 1**

Centroid of load lies within middle third. Pressure varies from \( p_{a1} \) at A to \( p_{b1} \) at B.

\[
\begin{array}{c}
\text{A} \quad \text{B} \\
p_{a1} & \text{---} \quad \text{---} \quad \text{---} & p_{b1}
\end{array}
\]

Maximum pressure is at B \( p_{\text{max1}} = p_{b1} = 195.73 \text{ kN/m}^2 \)
Shear forces and bending moments on base - ULS load combination 1

Total S.F. on toe (plus is up)  
Swb=((pwb+pb1)/2-den\*d)*(x1-t/2)-wf1  
=94.349 kN/m

Total S.F. on heel (plus is up)  
Swa=Sau-Sad=-69.276 kN/m

Total B.M. on toe (plus is up)  
Mwb=(2*pb1+pwb)*(x1-t/2)^2/6-Mbd  
=26.598 kNm/m

Total B.M. on heel (plus is up)  
Mwa=(2*pa1+pwa)*(x-t/2)^2/6-Mad  
=-209.73 kNm/m

Resistance to sliding - ULS (load combinations 1 & 2)

Combination 1:  
Sliding force  
SFor=St1'=152.5 kN  
Friction plus passive resistance  
SRes=F+R=177.09 kN  
Thus the criterion for sliding is satisfied.

Combination 2:  
Coefficient of passive pressure  
Kp'=2.4442  
Sliding force  
SFor=Ka2/Ka*(g7*P1+g8*P2)=142.94 kN  
Friction plus passive resistance  
SRes=F+R=175.57 kN  
Thus the criterion for sliding is satisfied.

Resistance to overturning - ULS (load combinations 1 & 2)

Combination 1:  
Total restraining mt.(with Mrv)  
Mr1=g9*(Mre+Mrw+Mrv)=727.73 kNm/m  
ditto (without Mrv)  
Mr2=g9*(Mre+Mrw)=727.73 kNm/m  
Total overturning moment  
Mot=Mr1+Mr2=252.32 kNm/m

Combination 2:  
Total restraining mt.(with Mrv)  
Mr1=g9''*(Mre+Mrw+Mrv)=727.73 kNm/m  
ditto (without Mrv)  
Mr2=g9''*(Mre+Mrw)=727.73 kNm/m  
Total overturning moment  
Mot=Mr1+Mr2=277.68 kNm/m

Partial factors - ULS load combination 2

Forces and moments on wall - ULS load combination 2

Stem design moment  
MEd2=g4*M1+g5*M2=200.92 kNm/m  
Horizontal shear at base of stem  
VEd2=g4*Ph+g5*Psh=96.666 kN/m
Bearing pressures - ULS load combination 2

\[
\text{Soil side} \quad W \quad x \quad x_1
\]

\[
\begin{align*}
\text{VEd2} & \quad \text{MEd2} \\
\text{heel} & \quad \text{toe}
\end{align*}
\]

\[
\begin{align*}
\text{db} & \quad \text{Ka2} = \frac{1-\sin(\text{RAD(\phi'1)})}{1+\sin(\text{RAD(\phi'1)})} \\
d & \quad \text{d}
\end{align*}
\]

Centroid of load lies within middle third. Pressure varies from \( p_{a2} \) at A to \( p_{b2} \) at B.

\[
\text{pa2} - \quad - \quad - \quad - \quad \text{pb2}
\]

Maximum pressure is at B \( p_{\text{max2}} = p_{b2} = 172.16 \text{ kN/m}^2 \)

Shear forces and bending moments on base - ULS load combination 2

Total S.F. on toe (plus is up) \( \text{Swb2} = ((p_{wb}+p_{b2})/2-\text{den}\times d)\times(x_1-t/2) \)
- \( \text{wf2} = 82.498 \text{ kN/m} \)

Total S.F. on heel (plus is up) \( \text{Swa2} = \text{Sau} - \text{Sad} = -84.82 \text{ kN/m} \)

Total B.M. on toe (plus is up) \( \text{Mwb2} = (2\times p_{b2} + p_{wb})\times(x_1-t/2)^2/6-\text{Mbd} \)
- \( = 23.237 \text{ kNm/m} \)

Total B.M. on heel (plus is up) \( \text{Mwa2} = (2\times p_{a2} + p_{wa})\times(x-t/2)^2/6-\text{Mad} \)
- \( = -216.49 \text{ kNm/m} \)
Design Summary - ULS load

combinations 1 & 2 to EC7

- Moment at base of stem: 259.65 kNm
- Shear at base of stem: 126.25 kN
- Moment at b-b (toe): 26.598 kNm
- Shear at b-b (toe): 94.349 kN
- Moment at a-a (heel): -216.49 kNm
- Shear at a-a (heel): -84.82 kN

Pressures under base - SLS

- Pressure @ A: 51.455 kN/m²
- Pressure @ B: 149.17 kN/m²
- Allowable GBP: 250 kN/m²

NOTE: The sliding resistance calculation check assumes that a heel beam/key will be provided to the underside of base.
Location: Ex7 - Mass concrete wall with no toe

Reinforced concrete retaining wall to EC7

Using either Rankine or Coulomb theory, the following assumptions are made:

- the surface of rupture is assumed to be plane
- point of resultant pressure P on back of wall is at one third of the distance up from the base of the wall.

Weight/unit vol. of retained soil  \( w = 18.8 \) kN/m³
Wall height above any toe provided  \( h = 1 \) m
Load above wall supported on stem  \( v_o = 0 \) kN/m
Angle of slope of soil (if any)  \( \theta = 0^\circ \)
Angle of wall-to-soil friction  \( \delta = 0^\circ \)
Friction factor @ underside base  \( f_b = 0.4 \)
Thickness of wall  \( t = 0.5 \) m
Thickness of toe (if provided)  \( d = 0 \) m
Density of wall (stem)  \( \text{den} = 25 \) kN/m³

Forces and moments on wall - SLS

\[
\begin{align*}
\text{compute} & \quad M_1 = P_h \left( \frac{d}{2} + \frac{h}{3} \right) = 1.0444 \text{ kNm/m} \\
\text{compute} & \quad M_2 = P_{sh} \left( \frac{d}{2} + \frac{h}{2} \right) = 0 \text{ kNm/m} \\
\text{Maximum stem bending moment} & \quad M = M_1 + M_2 = 1.0444 \text{ kNm/m} \\
\text{Horizontal shear at base of stem} & \quad V = P_h + P_{sh} = 3.1333 \text{ kN/m}
\end{align*}
\]

Bearing pressures - SLS

Soil depth for passive resistance  \( \text{dp} = 0 \) m
Allowable ground pressure  \( p = 100 \) kN/m²
Centroid of load outside middle third giving triangular distribution of pressure.
Pressure varies from 0 at distance \( x \) from A to a maximum of \( p_b \) at B.

\[
\begin{align*}
\text{Maximum pressure is at B} & \quad p_{\text{max}} = p_b = 50.067 \text{ kN/m}² \\
\text{As } p_{\text{max}} \leq p \text{ (50.067 kN/m}² \leq 100 \text{ kN/m}²), \text{ the pressure beneath base is within the specified limit.}
\end{align*}
\]
Partial factors - ULS load combination 1

Partial load factor (backfill) \( g_1 = 1 \)
Partial load factor (v. surcharge) \( g_2 = 0 \)

Forces and moments on wall - ULS load combination 1

Stem design moment \( M_{E1} = g_4 M_1 + g_5 M_2 = 1.41 \text{ kNm/m} \)
Horizontal shear at base of stem \( V_{E1} = g_4 P_h + g_5 P_{sh} = 4.23 \text{ kN/m} \)

Bearing pressures - ULS load combination 1

Centroid of load outside middle third giving triangular distribution of pressure.
Pressure varies from 0 at distance \( x \) from A to a maximum of \( p_{b1} \) at B.

\[
\begin{array}{c}
A \\
\downarrow \\
\downarrow \\
\downarrow \\
\downarrow \\
\downarrow \\
\downarrow \\
B
\end{array}
\]

Maximum pressure is at B \( p_{max1} = p_{b1} = 67.59 \text{ kN/m}^2 \)

Resistance to sliding - ULS (load combinations 1 & 2)

Combination 1:
Sliding force \( S_{For} = S_{t1} = 4.23 \text{ kN} \)
Frictional resistance \( S_{Res} = F = 5 \text{ kN} \)
Thus the criterion for sliding is satisfied.

Combination 2:
Sliding force \( S_{For} = K_2 / K_1 (g_7 P_{abh1} / g_4 + g_8 P_{sbh1} / g_5) = 3.8458 \text{ kN} \)
Frictional resistance \( S_{Res} = F = 5 \text{ kN} \)
Thus the criterion for sliding is satisfied.

Resistance to overturning - ULS (load combinations 1 & 2)

Combination 1:
Total restraining mt. (with Mrv) \( Mr_1 = g_9 (M_{re} + M_{rw} + M_{rv}) = 2.8125 \text{ kNm/m} \)
 ditto (without Mrv) \( Mr_2 = g_9 (M_{re} + M_{rw}) = 2.8125 \text{ kNm/m} \)
Total overturning moment \( M_{ot} = M_{t1} + M_{t2} = 1.1489 \text{ kNm/m} \)

Combination 2:
Total restraining mt. (with Mrv) \( Mr_1 = g_9' (M_{re} + M_{rw} + M_{rv}) = 2.8125 \text{ kNm/m} \)
 ditto (without Mrv) \( Mr_2 = g_9' (M_{re} + M_{rw}) = 2.8125 \text{ kNm/m} \)
Total overturning moment \( M_{ot} = M_{t1} + M_{t2} = 1.2819 \text{ kNm/m} \)
Design Summary - ULS load

combinations 1 & 2 to EC7

- Wall thickness: 500 mm
- Sliding force: 4.23 kN
- Sliding resistance: 5 kN

Pressures under base - SLS

- Max pressure @ B: 50.067 kN/m²
- Allowable GBP: 100 kN/m²

NOTE: Wall has no heel or toe and the total wall height h=1 m.
Active Pressure on a Retaining Wall to BS8002:1994 using Coulomb’s “Wedge Theory” for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal $\theta = 0^\circ$
Height of wall under consideration $H = 5.4 \text{ m}$

**Forces on soil wedge**

Surcharge load $q_s = 0 \text{ kN/m}^2$
Angle of internal friction $\phi = 30^\circ$
Angle of friction between wall and soil $\delta = 0^\circ$

**Wedge theory will be adopted**

Dry density of soil $D_s = 20 \text{ kN/m}^3$
Cohesion of soil $c = 0 \text{ kN/m}^2$
Coefficient of wall cohesion $c_w = 0 \text{ kN/m}^2$
Angle of shearing resistance $\phi = \phi = 30^\circ$
Coefficient of active soil pressure $K_a = (\tan(\text{RAD}(45-\phi/2)))^2 = 0.33333$
$K_1 = K_a = 0.33333$
Depth of tension cracks $Z_0 = (2cK_1^{0.5}q_sK_a)/(D_sK_a) = 0 \text{ m}$
Horiz. force from water in tension cracks $P_w = 0 \text{ kN}$
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))
* TAN(RAD(ap)) = 168.36

den=SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap))=1.7321

Total active force
Pa=num/den=97.2 kN

\[ A \quad \begin{array}{c}
\text{Horizontal component of active force} \\
\text{Vertical component of active force}
\end{array} \]

\[ \text{Distance} \\
\text{Distance} \\
\text{Distance}
\]

Distance
L1=H-2*Zo/3=5.4 m
L2=(H+qs/Ds-Zo)/3=1.8 m
L3=(Pw*L1+(hPa-Pw)*L2)/hPa=1.8 m

Diagram shows forces on wall due to active pressure
Active Pressure on a Retaining Wall to BS8002:1994 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

The angle of friction between wall and soil "delta" will be taken as zero as the soil at the top of the wall is assumed to be horizontal.

Soil 1

- Surcharge pressure: \( q = 0 \text{ kN/m}^2 \)
- Depth of soil (1): \( d_1 = 2 \text{ m} \)
- Cohesive strength of soil (1): \( c'_1 = 0 \text{ kN/m}^2 \)
- Coefficient of wall cohesion: \( c_{w1} = 0 \text{ kN/m}^2 \)
- Friction angle of soil (1): \( \phi'_1 = 35^\circ \)
- Dry unit weight of soil (1): \( \gamma_1 = 17 \text{ kN/m}^3 \)

Soil 2

- Depth of soil (2): \( d_2 = 3 \text{ m} \)
- Cohesive strength of soil (2): \( c'_2 = 0 \text{ kN/m}^2 \)
- Coefficient of wall cohesion: \( c_{w2} = 0 \text{ kN/m}^2 \)
- Friction angle of soil (2): \( \phi'_2 = 35^\circ \)
- Saturated unit weight soil (2): \( \gamma_2 = 20 \text{ kN/m}^3 \)

For soil 1

\[
K_1 = \frac{(1 - \sin(\text{RAD}(\phi'_1)))}{(1 + \sin(\text{RAD}(\phi'_1)))} = 0.27099
\]

For soil 2

\[
K_2 = \frac{(1 - \sin(\text{RAD}(\phi'_2)))}{(1 + \sin(\text{RAD}(\phi'_2)))} = 0.27099
\]

Horiz. force from water in tension crack: \( P_w = 0 \text{ kN} \)

No tension crack will develop: \( Z_0 = 0 \text{ m} \)

Active forces on wall (soil 1)

\[
P_1 = p_{at1} \cdot d_1 = 0 \text{ kN}
\]
\[
P_2 = 0.5 \cdot (p_{at1} - p_{at1}) \cdot d_1 = 9.2137 \text{ kN}
\]

Active forces on wall (soil 2)

\[
P_3 = p_{at2} \cdot d_2 = 27.641 \text{ kN}
\]
\[
P_4 = 0.5 \cdot (p_{at2} - p_{at2}) \cdot d_2 = 56.571 \text{ kN}
\]
Active force from water in crack \( P_5 = P_{w} = 0 \) kN

Point of application of total active pressure will be at a distance "L3" from the bottom of the wall.

\[
L_3 = \frac{(P_1L_1 + P_2L_2 + P_3L_3 + P_4L_4 + P_5L_5)}{\text{Pa}} = 1.4109 \text{ m}
\]

**Summary of results**

Diagram shows forces on wall and point of application

Horizontal component of active force \( h_{Pa} = \text{Pa} = 93.426 \) kN
Vertical component of active force \( R_v = 0 \) kN
Point of application of active force \( L_3 = L_3 = 1.4109 \) m
Active Pressure on a Retaining Wall to BS8002:1994 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal \( \theta = 0^\circ \)
Height of wall under consideration \( H = 5.4 \text{ m} \)

**Forces on soil wedge**

\[
\begin{align*}
\text{Surcharge load} & \quad q_s = 0 \text{ kN/m}^2 \\
\text{Angle of internal friction} \varphi & \quad \phi = 30^\circ \\
\text{Angle of friction between wall and soil} \delta & \quad \delta = 0^\circ \\
\end{align*}
\]

**Wedge theory will be adopted**

- **Dry density of soil** \( D_s = 20 \text{ kN/m}^3 \)
- **Cohesion of soil** \( c = 0 \text{ kN/m}^2 \)
- **Coefficient of wall cohesion** \( c_w = 0 \text{ kN/m}^2 \)
- **Angle of shearing resistance \( \phi \)** \( \phi = \phi = 30^\circ \)
- **Coefficient of active soil pressure** \( K_a = (\tan(\text{RAD}(45-\phi/2)))^2 = 0.33333 \)
- **Coefficient of tension cracks** \( K_1 = K_a = 0.33333 \)
- **Depth of tension cracks** \( Z_0 = (2cK1^0.5-q_sK_a)/(DsK_a) = 0 \text{ m} \)
- **Horizontal force from water in tension cracks** \( P_w = 0 \text{ kN} \)
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))*TAN(RAD(ap)) = 168.36

den=SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap)) = 1.7321

Total active force \( Pa = \frac{num}{den} = 97.2 \) kN

**Summary of results derived from wedge theory**

Horizontal component of active force \( hPa = Pa \cdot \cos(RAD(delta)) = 97.2 \) kN

Vertical component of active force \( vPa = Pa \cdot \sin(RAD(delta)) = 0 \) kN

Distance \( L1 = H - 2 \cdot Zo/3 = 5.4 \) m

Distance \( L2 = (H + qs/Ds - Zo)/3 = 1.8 \) m

Distance \( L3 = (Pw \cdot L1 + (hPa-Pw) \cdot L2)/hPa = 1.8 \) m

Diagram shows forces on wall due to active pressure
Active Pressure on a Retaining Wall to BS8002:1994 using Coulomb’s “Wedge Theory” for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal \( \theta = 0^\circ \)
Height of wall under consideration \( H = 5.4 \) m

**Forces on soil wedge**

\[
\text{surcharge} \\
\text{surcharge load} \quad q_s = 48 \text{ kN/m}^2 \\
\text{Angle of internal friction } \varphi = 30^\circ \\
\text{Angle of friction between wall and soil } \delta = 0^\circ \\
\]

**Wedge theory will be adopted**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry density of soil</td>
<td>( D_s = 20 ) kN/m(^3)</td>
</tr>
<tr>
<td>Cohesion of soil</td>
<td>( c = 0 ) kN/m(^2)</td>
</tr>
<tr>
<td>Coefficient of wall cohesion</td>
<td>( c_w = 0 ) kN/m(^2)</td>
</tr>
<tr>
<td>Angle of shearing resistance ( \varphi )</td>
<td>( \phi = \varphi = 30^\circ )</td>
</tr>
<tr>
<td>Coefficient of active soil pressure ( K_a )</td>
<td>( K_a = 0.33333 )</td>
</tr>
<tr>
<td>Depth of tension cracks ( z_o )</td>
<td>( z_o = 2 \times c \times K_l \times 0.5 - q_s \times K_a ) / (( D_s \times K_a )) = -2.4 m</td>
</tr>
<tr>
<td>As ( z_o ) is negative this will be ignored</td>
<td>( z_o = 0 ) m</td>
</tr>
<tr>
<td>Density of water</td>
<td>( D_w = 9.81 ) kN/m(^3)</td>
</tr>
<tr>
<td>Horiz.force from water in tension crack</td>
<td>( P_w = 0 ) kN</td>
</tr>
<tr>
<td>Horiz.force from water in tension cracks</td>
<td>( P_w = 0 ) kN</td>
</tr>
</tbody>
</table>
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))
  *TAN(RAD(ap))=318
den=SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap))=1.7321

Total active force \( Pa = \frac{\text{num}}{\text{den}} = 183.6 \) kN

### Summary of results derived from wedge theory

Horizontal component of active force \( hPa = Pa \times \cos(\text{RAD}(\delta)) = 183.6 \) kN
Vertical component of active force \( vPa = Pa \times \sin(\text{RAD}(\delta)) = 0 \) kN

Distance \( L1 = H - 2 \times Zo / 3 = 5.4 \) m
Distance \( L2 = (H + qs/\text{Ds} - Zo) / 3 = 2.6 \) m
Distance \( L3 = (Pw \times L1 + (hPa - Pw) \times L2) / hPa = 2.6 \) m

Diagram shows forces on wall due to active pressure
Location: Ex 8.7 Basic Soil Mechanics by Roy Whitlow, 4th Edtn.

Active Pressure on a Retaining Wall to BS8002:1994 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal $\theta = 0^{\circ}$
Height of wall under consideration $H = 8$ m

Forces on soil wedge

Surcharge load $q_s = 0$ kN/m$^2$
Angle of internal friction $\phi = 0^{\circ}$
Angle of friction between wall and soil $\delta = 0^{\circ}$

Wedge theory will be adopted

Saturated density of soil $D_s = 18$ kN/m$^3$
Cohesion of soil $c = 32$ kN/m$^2$
Coefficient of wall cohesion $c_w = 0$ kN/m$^2$
Angle of shearing resistance $\phi = \phi = 0^{\circ}$
Coefficient of active soil pressure $K_a = (\tan(\text{RAD}(45-\phi/2)))^2 = 1$
$K_1 = K_a = 1$
Depth of tension cracks $Z_0 = (2^cK_1^0.5-q_sK_a)/(D_sK_a) = 3.5556$ m
Rain water in tension cracks will be ignored.
Horiz. force from water in tension crack $P_w = 0$ kN
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))
*TAN(RAD(ap))=177.78

den=SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap))=1

Total active force Pa=num/den=177.78 kN

---

**Summary of results derived from wedge theory**

Horizontal component of active force hPa=Pa*COS(RAD(delta))=177.78 kN
Vertical component of active force vPa=Pa*SIN(RAD(delta))=0 kN

---

Distance L1=H-2*Zo/3=5.6296 m
Distance L2=(H+qs/Ds-Zo)/3=1.4815 m
Distance L3=(Pw*L1+(hPa-Pw)*L2)/hPa=1.4815 m

---

Diagram shows forces on wall due to active pressure
Active Pressure on a Retaining Wall to BS8002:1994 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall. The active pressure of cohesive soils could be evaluated using Coulomb's wedge theory. Bell's solution will be used for evaluating the depth of tension cracks (i.e. depth Zo).

Coulomb's theory assumes that there is a neutral or ineffective zone of depth (Zo) within which there is no adhesion or friction along the back of the wall or along the plane of rupture. This theory also assumes that the surface of rupture is plane. For cohesionless soils Zo will be taken as zero.

Coulomb's wedge theory takes into account two further forces acting on the wedge which have not previously been considered by "Rankine" or "Bell". These are as follows:

a) Cohesion along the failure plane;
b) Cohesion along the plane of the wall.

For cohesionless soils the above forces will be taken as zero.

It is worth pointing out that the "wedge theory" is much simpler to use for evaluating the active pressure when the soil surface is not horizontal. The back of the retaining wall will be assumed to be vertical in this program. Both cohesive and cohesionless soils can be considered using this theory.

Angle of earth surface to horizontal $\theta = 0^\circ$
Height of wall under consideration $H = 8\ m$
Bell's solution

Bell gives the formula for the active clay pressure at failure on the back of the wall as follows:

\[ p_a = K_a \cdot D_s \cdot Z - 2 \cdot c \cdot K_a^{0.5} \]

where \( K_a \) is the coefficient of active clay pressure which relates to the angle "phi" and is given by the following expression:

\[ K_a = \left( \tan \left( 45 - \frac{\phi}{2} \right) \right)^2 \]

By comparing two soils, one cohesive (say clay) and the other cohesionless, both having the same bulk density and angle of shearing resistance, the cohesive soil, being more self-supporting, exerts a smaller pressure on the retaining wall than does the cohesionless soil, the difference being the second component in "Bell's equation given above.

Forces on soil wedge

\[ \text{Surcharge load} \quad q_s = 75 \text{ kN/m}^2 \]
By resolving the forces shown in the figure above in two directions the following equations can be derived:

A) \[ P_a \cos(d) + C_s \sin(a) = R \cos(a + \phi) + P_w \]

B) \[ P_a \sin(d) + C_w - W - P_s + C_s \cos(a) + R \sin(a + \phi) = 0 \]

Substituting equation (A) into (B) the expression becomes

where,

- \( P_a \) = total active force on the retaining wall in kN
- \( W \) = weight of soil wedge in kN
- \( P_w \) = horizontal force from water in tension cracks in kN
- \( C_w \) = cohesion force at the back of wall \( (C_w = c_w \times \text{length KB}) \)
- \( c_w \) = coefficient of wall cohesion or adhesion at back of wall in kN/m²
- \( C_s \) = cohesion force acting along failure plane \( (C_s = c \times \text{length BD}) \)
- \( c \) = cohesion of the soil or shear strength of soil in kN/m²
- \( \phi \) = angle of internal friction (or angle of shearing resistance)
- \( \delta \) = angle of friction between wall and soil in degrees
- \( \alpha \) = angle between failure plane and the vertical in degrees
- \( \theta \) = angle between earth surface and horizontal in degrees
- \( P_s \) = surcharge force \( (P_s = q_s \times \text{length AE}) \)
- \( q_s \) = surcharge per unit area in kN/m²

As the plane of rupture (or failure) is not known a series of trial wedges will be considered by varying angle "alpha" and the maximum value active force on the wall \( (P_a) \).

**Consider the pressure distribution on a vertical wall**

\[ 2C(Ka-Ka.cw/c)^{0.5} \]

Theoretically, down to depth \( Z_o \) the active pressure is negative (i.e. tensile)

Angle of internal friction \( \phi = 0^\circ \)

Angle of friction between wall and soil \( \delta = 0^\circ \)
Wedge theory will be adopted

Saturated density of soil \( D_s = 18.5 \text{ kN/m}^3 \)
Cohesion of soil \( c = 30 \text{ kN/m}^2 \)
Coefficient of wall cohesion \( c_w = 0 \text{ kN/m}^2 \)
Angle of shearing resistance \( \phi = 0^\circ \)
Coefficient of active soil pressure \( K_a = (\tan(\text{RAD}(45-\phi/2)))^2 = 1 \)
\( K_1 = K_a = 1 \)
Depth of tension cracks \( Z_0 = \frac{(2cK_1^{0.5} - q_sK_a)}{D_sK_a} \)
\( = -0.81081 \text{ m} \)
As \( Z_0 \) is negative this will be ignored \( Z_0 = 0 \text{ m} \)
Density of water \( D_w = 9.81 \text{ kN/m}^3 \)
Horiz. force from water in tension crack \( P_w = 0 \text{ kN} \)

\[
\text{num}=W+P_s+P_w\tan(\text{RAD}(ap))-C_w-C_s\cos(\text{RAD}(a))-C_s\sin(\text{RAD}(a))
\]
\[
\text{den}=\sin(\text{RAD}(d))+\cos(\text{RAD}(d))\tan(\text{RAD}(ap))=712
\]
Total active force \( P_a = \frac{\text{num}}{\text{den}} = 712 \text{ kN} \)

Summary of results derived from wedge theory

Horizontal component of active force \( h_Pa = P_a\cos(\text{RAD}(\delta)) = 712 \text{ kN} \)
Vertical component of active force \( v_Pa = P_a\sin(\text{RAD}(\delta)) = 0 \text{ kN} \)

Distance \( L_1 = H - 2Z_0/3 = 8 \text{ m} \)
Distance \( L_2 = (H+q_s/D_s-Z_0)/3 = 4.018 \text{ m} \)
Distance \( L_3 = (P_wL_1+(h_Pa-P_w)L_2)/h_Pa = 4.018 \text{ m} \)
Diagram shows forces on wall due to active pressure.
Location: Ex 7.4 Problems in Engineering soils, PL Capper/WF Cassie

Active Pressure on a Retaining Wall to BS8002:1994 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall. The active pressure of cohesive soils could be evaluated using Coulomb's wedge theory. Bell's solution will be used for evaluating the depth of tension cracks (i.e. depth Zo).

Coulomb's theory assumes that there is a neutral or ineffective zone of depth (Zo) within which there is no adhesion or friction along the back of the wall or along the plane of rupture. This theory also assumes that the surface of rupture is plane. For cohesionless soils Zo will be taken as zero.

Coulomb's wedge theory takes into account two further forces acting on the wedge which have not previously been considered by "Rankine" or "Bell". These are as follows:

a) Cohesion along the failure plane;
b) Cohesion along the plane of the wall.

For cohesionless soils the above forces will be taken as zero.

It is worth pointing out that the "wedge theory" is much simpler to use for evaluating the active pressure when the soil surface is not horizontal. The back of the retaining wall will be assumed to be vertical in this program. Both cohesive and cohesionless soils can be considered using this theory.

Angle of earth surface to horizontal $\theta = 23.2^\circ$
Height of wall under consideration $H = 8$ m
Bell's solution

Bell gives the formula for the active clay pressure at failure on the back of the wall as follows:

\[ p_a = K_a . D_s . Z - 2 . c . K_a^{0.5} \]

where \( K_a \) is the coefficient of active clay pressure which relates to the angle "phi" and is given by the following expression:

\[ K_a = (\tan(45 - \frac{\phi}{2}))^2 \]

By comparing two soils, one cohesive (say clay) and the other cohesionless, both having the same bulk density and angle of shearing resistance, the cohesive soil, being more self-supporting, exerts a smaller pressure on the retaining wall than does the cohesionless soil, the difference being the second component in "Bell's equation given above.

Forces on soil wedge

Surcharge load \( q_s = 0 \text{ kN/m}^2 \)
By resolving the forces shown in the figure above in two directions
the following equations can be derived:

A) \[ P_a \cos(d) + C_s \sin(a) = R \cos(a + \phi) + P_w \]
B) \[ P_a \sin(d) + C_w - W - P_s + C_s \cos(a) + R \sin(a + \phi) = 0 \]

substituting equation ( A ) into ( B ) the expression becomes

where,

- \( P_a \) = total active force on the retaining wall in kN
- \( W \) = weight of soil wedge in kN
- \( P_w \) = horizontal force from water in tension cracks in kN
- \( C_w \) = cohesion force at the back of wall \(( C_w = c_w \cdot \text{length } KB )\)
- \( c_w \) = coefficient of wall cohesion or adhesion at back of wall in kN/m²
- \( C_s \) = cohesion force acting along failure plane \(( C_s = c \cdot \text{length } BD )\)
- \( c \) = cohesion of the soil or shear strength of soil in kN/m²
- \( \phi \) = angle of internal friction (or angle of shearing resistance)
- \( \delta \) = angle of friction between wall and soil in degrees
- \( \alpha \) = angle between failure plane and the vertical in degrees
- \( \theta \) = angle between earth surface and horizontal in degrees
- \( P_s \) = surcharge force \(( P_s = q_s \cdot \text{length } AE )\)
- \( q_s \) = surcharge per unit area in kN/m²

As the plane of rupture (or failure) is not known a series of trial wedges will be considered by varying angle "alpha" and the maximum value active force on the wall (\( P_a \)).

Consider the pressure distribution on a vertical wall

\[ 2.C.(K_a-K_a.c_c/c)^{0.5} \]

Theoretically, down to depth \( Z_0 \) the active pressure is negative (i.e. tensile)

Angle of internal friction \( \phi \) \( \phi = 30° \)
Angle of friction between wall and soil \( \delta = 20° \)
Wedge theory will be adopted

Dry density of soil \( D_s = 17.17 \text{ kN/m}^3 \)
Cohesion of soil \( c = 0 \text{ kN/m}^2 \)
Coefficient of wall cohesion \( c_w = 0 \text{ kN/m}^2 \)
Angle of shearing resistance \( \phi = 30^\circ \)
Coefficient of active soil pressure \( K_a = (\tan(\text{RAD}(45 - \phi/2)))^2 = 0.3333 \)
K_1 = K_a = 0.3333
Depth of tension cracks \( Z_o = \frac{2cK_1^{0.5} - q_sK_a}{DsK_a} = 0 \text{ m} \)
Horiz. force from water in tension cracks \( P_w = 0 \text{ kN} \)

\[
\text{num} = W + P_s + P_w \cdot \tan(\text{RAD}(\alpha)) - C_w - C_s \cdot \cos(\text{RAD}(\alpha)) - C_s \cdot \sin(\text{RAD}(\alpha)) \\
\text{den} = \sin(\text{RAD}(d)) + \cos(\text{RAD}(d)) \cdot \tan(\text{RAD}(\alpha)) = 3.6191
\]

Total active force \( P_a = \frac{\text{num}}{\text{den}} = 250.14 \text{ kN} \)

Summary of results derived from wedge theory

Horizontal component of active force \( h_{Pa} = P_a \cdot \cos(\text{RAD}(\delta)) = 235.05 \text{ kN} \)
Vertical component of active force \( v_{Pa} = P_a \cdot \sin(\text{RAD}(\delta)) = 85.552 \text{ kN} \)

Distance \( L_1 = H - 2Z_o/3 = 8 \text{ m} \)
Distance \( L_2 = (H + q_s/D_s - Z_o)/3 = 2.6667 \text{ m} \)
Distance \( L_3 = (P_wL_1 + (h_{Pa} - P_w)L_2)/h_{Pa} = 2.6667 \text{ m} \)
Active pressure on a retaining wall to BS8002:1994

Diagram shows forces on wall due to active pressure

\[ \theta \] (positive)
Active Pressure on a Retaining Wall to BS8002:1994 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal \( \theta = 0^\circ \)
Height of wall under consideration \( H = 8 \text{ m} \)

Forces on soil wedge

\[ \text{Surcharge load} \quad q_s = 0 \text{ kN/m}^2 \]
\[ \text{Angle of internal friction} \quad \phi = 20^\circ \]
\[ \text{Angle of friction between wall and soil} \quad \delta = 0^\circ \]

Wedge theory will be adopted

Dry density of soil \( D_s = 17.17 \text{ kN/m}^3 \)
Cohesion of soil \( c = 13 \text{ kN/m}^2 \)
Coefficient of wall cohesion \( c_w = 0 \text{ kN/m}^2 \)
Angle of shearing resistance \( \phi = \phi = 20^\circ \)
Coefficient of active soil pressure \( K_a = \left( \tan(\text{RAD}(45 - \phi/2)) \right)^2 = 0.49029 \)

Depth of tension cracks \( Z_o = \frac{2cK_1^0.5 - q_sK_a}{D_sK_a} = 2.1626 \text{ m} \)
Rain water in tension cracks will be ignored.
Horiz.force from water in tension crack \( P_w = 0 \text{ kN} \)
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))
  *TAN(RAD(ap))=204.84
den=SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap))=1.4281

Total active force                Pa=num/den=143.43 kN

--- Summary of results derived from wedge theory ---

Horizontal component of active force  hPa=Pa*COS(RAD(delta))=143.43 kN
Vertical component of active force    vPa=Pa*SIN(RAD(delta))=0 kN

Distance                          L1=H-2*Zo/3=6.5583 m
Distance                          L2=(H+qs/Ds-Zo)/3=1.9458 m
Distance                          L3=(Pw*L1+(hPa-Pw)*L2)/hPa=1.9458 m

--- Diagram shows forces on wall due to active pressure ---
Active Pressure on a Retaining Wall to BS8002:1994 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal \( \theta = 0^\circ \)
Height of wall under consideration \( H = 10 \) m

Forces on soil wedge

Surcharge load \( q_s = 0 \) kN/m²
Angle of internal friction \( \phi = 15^\circ \)
Angle of friction between wall and soil \( \delta = 15^\circ \)

Wedge theory will be adopted

Dry density of soil \( D_s = 18.6 \) kN/m³
Cohesion of soil \( c = 15 \) kN/m²
Coefficient of wall cohesion \( c_w = 15 \) kN/m²
Angle of shearing resistance \( \phi = \phi = 15^\circ \)
Coefficient of active soil pressure \( K_a = (\tan(radians(45-\phi/2)))^2 = 0.58879 \)

Depth of tension cracks \( z_o = (2cK_1^0.5 - q_sK_a)/(D_sK_a) = 2.9726 \) m
Rain water in tension cracks will be ignored.
Horiz. force from water in tension crack \( P_w = 0 \) kN
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))
*TAN(RAD(ap))=454.42

den=SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap))=1.9319

Total active force                  Pa=num/den=235.23 kN

Summary of results derived from wedge theory

Horizontal component of active force  hPa=Pa*COS(RAD(delta))=227.21 kN
Vertical component of active force    vPa=Pa*SIN(RAD(delta))=60.881 kN

Distance                          L1=H-2*Zo/3=8.0182 m
Distance                          L2=(H+qs/Ds-Zo)/3=2.3425 m
Distance                          L3=(Pw*L1+(hPa-Pw)*L2)/hPa=2.3425 m

Diagram shows forces on wall due to active pressure
Location: Ex 6.4 of Soil Mechanics by RF Craig, 6th Edtn.

**Active Pressure on a Retaining Wall to BS8002:1994 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils**

The angle of friction between wall and soil "delta" will be taken as zero as the soil at the top of the wall is assumed to be horizontal.

**Soil 1**

- Surcharge pressure $q=0$ kN/m²
- Depth of soil (1) $d_1=4$ m
- Cohesive strength of soil (1) $c_1'=8$ kN/m²
- Coefficient of wall cohesion $c_{w1}=4$ kN/m²
- Friction angle of soil (1) $\phi_1'=27^\circ$
- Dry unit weight of soil (1) $\gamma_{1}=20$ kN/m³

**Soil 2**

- Depth of soil (2) $d_2=4$ m
- Cohesive strength of soil (2) $c_2'=8$ kN/m²
- Coefficient of wall cohesion $c_{w2}=4$ kN/m²
- Friction angle of soil (2) $\phi_2'=27^\circ$
- Saturated unit weight soil (2) $\gamma_{2}=20$ kN/m³

For soil 1
$$K_a1 = \frac{1 - \sin(\text{RAD}(\phi_1'))}{1 + \sin(\text{RAD}(\phi_1'))} = 0.37552$$

For soil 2
$$K_a2 = \frac{1 - \sin(\text{RAD}(\phi_2'))}{1 + \sin(\text{RAD}(\phi_2'))} = 0.37552$$

Depth of tension cracks
$$Z_0 = \frac{2c_1'SQR(K_{ac})-q*K_{a1}}{(\gamma_{1}*K_{a1})} = 1.5989$$ m

Active forces on wall (soil 1)
$$P_1 = p_{at1}*d_1 = -39.219$$ kN
$$P_2 = 0.5*(p_{ab1}-p_{at1})*d_1 = 60.084$$ kN

Active forces on wall (soil 2)
$$P_3 = p_{at2}*d_2 = 80.949$$ kN
$$P_4 = 0.5*(p_{ab2}-p_{at2})*d_2 = 109.09$$ kN
Active force from water in crack $P_5 = P_{w} = 12.539 \text{ kN}$

Point of application of total active pressure will be at a distance "$L_3$" from the bottom of the wall.

$$L_3 = \frac{(P_1L_1 + P_2L_2 + P_3L_3 + P_4L_4 + P_5L_5)}{P_a} = 2.1456 \text{ m}$$

**Summary of results**

Diagram shows forces on wall and point of application

Horizontal component of active force $hP_a = P_a = 223.45 \text{ kN}$

Vertical component of active force $R_v = 0 \text{ kN}$

Point of application of active force $L_3 = L_3 = 2.1456 \text{ m}$
Location: Example 8.1 Basic Soil Mechanics by Roy Whitlow, 4th Edtn.

Active pressure on a retaining wall to BS EN 1997-1:2004 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal \( \theta = 0^\circ \)
Height of wall under consideration \( H = 5.4 \text{ m} \)

**Forces on soil wedge**

\[
\text{surcharge} \\
\text{surcharge load} \quad \phi_s = 0 \text{ kN/m}^2 \\
\text{Angle of shearing resistance} \quad \phi' = 30^\circ \\
\text{Angle of friction between wall and soil} \quad \delta = 0^\circ
\]

**Wedge theory will be adopted**

\[
\begin{align*}
\text{Dry density of soil} & \quad \rho_s = 20 \text{ kN/m}^3 \\
\text{Cohesion of soil} & \quad c = 0 \text{ kN/m}^2 \\
\text{Coefficient of wall cohesion} & \quad c_w = 0 \text{ kN/m}^2 \\
\text{Angle of shearing resistance} & \quad \phi' = 30^\circ \\
\text{Coefficient of active soil pressure} & \quad K_a = (\tan(45 - \phi'/2))^2 = 0.33333 \\
K_1 = K_a = 0.33333 \\
\text{Depth of tension cracks} & \quad Z_0 = (2cK_1^0.5 - \phi_s K_a)/(\rho_s K_a) = 0 \text{ m} \\
\text{Horiz.force from water in tension cracks} & \quad P_w = 0 \text{ kN}
\end{align*}
\]
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))
* TAN(RAD(ap))=168.36
den=SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap))=1.7321

Total active force Pa=num/den=97.2 kN

**Summary of results derived from wedge theory**

Horizontal component of active force hPa=Pa*COS(RAD(delta))=97.2 kN
Vertical component of active force vPa=Pa*SIN(RAD(delta))=0 kN

Distance L1=H-2*Zo/3=5.4 m
Distance L2=(H+qs/Ds-Zo)/3=1.8 m
Distance L3=(Pw*L1+(hPa-Pw)*L2)/hPa=1.8 m
Active pressure on a retaining wall to BS EN 1997-1:2004 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

The angle of friction between wall and soil "delta" will be taken as zero as the soil at the top of the wall is assumed to be horizontal.

Surcharge pressure \( q = 0 \text{ kN/m}^2 \)

**Design data for soil 1**
- Depth of soil \( d_1 = 2 \text{ m} \)
- Cohesive strength of soil \( c_1' = 0 \text{ kN/m}^2 \)
- Coefficient of wall cohesion \( c_{w1} = 0 \text{ kN/m}^2 \)
- Angle of shearing resistance \( \phi_1' = 35^\circ \)
- Dry unit weight of soil \( \gamma_1 = 17 \text{ kN/m}^3 \)

**Design data for soil 2**
- Depth of soil \( d_2 = 3 \text{ m} \)
- Cohesive strength of soil \( c_2' = 0 \text{ kN/m}^2 \)
- Coefficient of wall cohesion \( c_{w2} = 0 \text{ kN/m}^2 \)
- Angle of shearing resistance \( \phi_2' = 35^\circ \)
- Saturated unit weight soil \( \gamma_2 = 20 \text{ kN/m}^3 \)

Horiz. force from water in tension crack \( P_w = 0 \text{ kN} \)
No tension crack will develop \( Z_o = 0 \text{ m} \)
Active forces on wall (soil 1) \( P_1 = \gamma_1 d_1 = 0 \text{ kN} \)
\[ P_2 = 0.5 \times (\gamma_1 - \gamma_1) d_1 = 9.2137 \text{ kN} \]
Active forces on wall (soil 2) \( P_3 = \gamma_2 d_2 = 27.641 \text{ kN} \)
\[ P_4 = 0.5 \times (\gamma_2 - \gamma_1) d_2 = 56.571 \text{ kN} \]
Active force from water in crack \( P_5 = P_w = 0 \text{ kN} \)
Point of application of total active pressure will be at a distance "L3" from the bottom of the wall.

\[ L3 = (P_1*L_1 + P_2*L_2 + P_3*L_3 + P_4*L_4 + P_5*L_5) / \text{Pa} = 1.4109 \text{ m} \]

---

**Summary of results**

Diagram shows forces on wall and point of application

Horizontal component of active force \( h\text{Pa} = \text{Pa} = 93.426 \text{ kN} \)
Vertical component of active force \( R_v = 0 \text{ kN} \)
Point of application of active force \( L_3 = 1.4109 \text{ m} \)
Location: Example 8.1 Basic Soil Mechanics by Roy Whitlow, 4th Edtn.

Active pressure on a retaining wall to BS EN 1997-1:2004 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal $\theta = 0^\circ$
Height of wall under consideration $H = 5.4 \text{ m}$

Forces on soil wedge

Surcharge load $q_s = 0 \text{ kN/m}^2$
Angle of shearing resistance $\varphi$ $\phi' = 30^\circ$
Angle of friction between wall and soil $\delta = 0^\circ$

Wedge theory will be adopted

Dry density of soil $D_s = 20 \text{ kN/m}^3$
Cohesion of soil $c = 0 \text{ kN/m}^2$
Coefficient of wall cohesion $c_w = 0 \text{ kN/m}^2$
Angle of shearing resistance $\varphi$ $\phi' = \phi' = 30^\circ$
Coefficient of active soil pressure $K_a = (\tan \left(45 - \frac{\phi'}{2}\right))^2 = 0.33333$
$K_1 = K_a = 0.33333$
Depth of tension cracks $Z_o = (2cK_10.5-q_sK_a)/(D_sK_a) = 0 \text{ m}$
Horiz.force from water in tension cracks $P_w = 0 \text{ kN}$
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))
* TAN(RAD(ap))=168.36

den=SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap))=1.7321

Total active force \( Pa = \frac{num}{den} = 97.2 \) kN

**Summary of results derived from wedge theory**

Horizontal component of active force \( hPa = Pa \cdot \cos(\text{RAD}(\delta)) = 97.2 \) kN
Vertical component of active force \( vPa = Pa \cdot \sin(\text{RAD}(\delta)) = 0 \) kN

Distance \( L1 = H - 2 \cdot Zo/3 = 5.4 \) m
Distance \( L2 = (H + qS/Ds - Zo)/3 = 1.8 \) m
Distance \( L3 = (Pw \cdot L1 + (hPa - Pw) \cdot L2)/hPa = 1.8 \) m
Location: Ex 8.4 Basic Soil Mechanics by Roy Whitlow, 4th Edtn.

Active pressure on a retaining wall to BS EN 1997-1:2004 using Coulomb’s “Wedge Theory” for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.  
Angle of earth surface to horizontal  theta=0°  
Height of wall under consideration  H=5.4 m

Forces on soil wedge

Surcharge load  \( qs=48 \text{ kN/m}^2 \)
Angle of shearing resistance \( \phi \)  \( \phi'=30° \)
Angle of friction between wall and soil  \( \delta=0° \)

Wedge theory will be adopted

Dry density of soil  \( D_s=20 \text{ kN/m}^3 \)
Cohesion of soil  \( c=0 \text{ kN/m}^2 \)
Coefficient of wall cohesion  \( c_w=0 \text{ kN/m}^2 \)
Angle of shearing resistance \( \phi \)  \( \phi'=30° \)
Coefficient of active soil pressure  \( K_a=(\tan(\text{RAD}(45-\phi'/2)))^2=0.33333 \)
Coefficient of passive soil pressure  \( K_l=0.33333 \)
Depth of tension cracks  \( Z_o=(2*c*K_l^0.5-qs*K_a)/(D_s*K_a)=-2.4 \text{ m} \)
Density of water  \( D_w=9.81 \text{ kN/m}^3 \)
Horiz.force from water in tension crack  \( P_w=0 \text{ kN} \)
Horiz.force from water in tension cracks  \( P_w=0 \text{ kN} \)
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))  
*TAN(RAD(ap))=318  
den=SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap))=1.7321  

Total active force  
$$Pa = \frac{num}{den} = \frac{183.6}{kN}$$

**Summary of results derived from wedge theory**

Horizontal component of active force  
$$h_{Pa} = Pa \times \cos(\text{RAD}(\delta)) = 183.6 \text{ kN}$$

Vertical component of active force  
$$v_{Pa} = Pa \times \sin(\text{RAD}(\delta)) = 0 \text{ kN}$$

Distance  
$$L_1 = H - 2 \times Zo / 3 = 5.4 \text{ m}$$

Distance  
$$L_2 = (H + qs / Ds - Zo) / 3 = 2.6 \text{ m}$$

Distance  
$$L_3 = (Pw \times L_1 + (h_{Pa} - Pw) \times L_2) / h_{Pa} = 2.6 \text{ m}$$
Active pressure on a retaining wall to BS EN 1997-1:2004 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal \( \theta = 0^\circ \)
Height of wall under consideration \( H = 8 \text{ m} \)

Forces on soil wedge

\[
\text{Surcharge load} \quad q_{s} = 0 \text{ kN/m}^2
\]

Angle of shearing resistance \( \varphi \) \( \phi' = 0^\circ \)
Angle of friction between wall and soil \( \delta = 0^\circ \)

Wedge theory will be adopted

Saturated density of soil \( D_s = 18 \text{ kN/m}^3 \)
Cohesion of soil \( c = 32 \text{ kN/m}^2 \)
Coefficient of wall cohesion \( c_w = 0 \text{ kN/m}^2 \)
Angle of shearing resistance \( \varphi \) \( \phi' = \phi' = 0^\circ \)
Coefficient of active soil pressure \( K_a = (\tan(\text{RAD}(45 - \phi')/2))^2 = 1 \)
\( K_1 = K_a = 1 \)
Depth of tension cracks \( Z_o = (2cK_1^{0.5} - q_{s}K_a)/(D_sK_a) = 3.5556 \text{ m} \)
Rain water in tension cracks will be ignored.
Horiz.force from water in tension crack \( P_w = 0 \text{ kN} \)
num = W + Ps + Pw * TAN(RAD(ap)) - Cw - Cs * COS(RAD(a)) - Cs * SIN(RAD(a)) * TAN(RAD(ap)) = 177.78

Total active force

\[ Pa = \frac{num}{den} = 177.78 \text{ kN} \]

**Summary of results derived from wedge theory**

Horizontal component of active force

\[ hPa = Pa \times \text{COS(RAD(delta))} = 177.78 \text{ kN} \]

Vertical component of active force

\[ vPa = Pa \times \text{SIN(RAD(delta))} = 0 \text{ kN} \]

Distance

\[ L1 = H - 2 \times ZO / 3 = 5.6296 \text{ m} \]

\[ L2 = (H + qS / Ds - ZO) / 3 = 1.4815 \text{ m} \]

\[ L3 = (Pw \times L1 + (hPa - Pw) \times L2) / hPa = 1.4815 \text{ m} \]
Active pressure on a retaining wall to BS EN 1997-1:2004 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall. The active pressure of cohesive soils could be evaluated using Coulomb's wedge theory. Bell's solution will be used for evaluating the depth of tension cracks (i.e. depth Zo).

Coulomb's theory assumes that there is a neutral or ineffective zone of depth (Zo) within which there is no adhesion or friction along the back of the wall or along the plane of rupture. This theory also assumes that the surface of rupture is plane. For cohesionless soils Zo will be taken as zero.

Coulomb's wedge theory takes into account two further forces acting on the wedge which have not previously been considered by "Rankine" or "Bell". These are as follows:

a) Cohesion along the failure plane;
b) Cohesion along the plane of the wall.

For cohesionless soils the above forces will be taken as zero.

It is worth pointing out that the "wedge theory" is much simpler to use for evaluating the active pressure when the soil surface is not horizontal. The back of the retaining wall will be assumed to be vertical in this program. Both cohesive and cohesionless soils can be considered using this theory.

Angle of earth surface to horizontal  \( \theta = 0^\circ \)
Height of wall under consideration  \( H = 8 \text{ m} \)
Bell's solution

Bell gives the formula for the active clay pressure at failure on the back of the wall as follows:

$$pa = Ka .Ds .Z - 2 .c .Ka^{0.5}$$

where Ka is the coefficient of active clay pressure which relates to the angle "phi'" and is given by the following expression:

$$Ka = ( \tan (45 - \phi'/2) )^2$$

By comparing two soils, one cohesive (say clay) and the other cohesionless, both having the same bulk density and angle of shearing resistance, the cohesive soil, being more self-supporting, exerts a smaller pressure on the retaining wall than does the cohesionless soil, the difference being the second component in "Bell's equation given above.

Forces on soil wedge

Surcharge load $qs=75$ kN/m²
By resolving the forces shown in the figure above in two directions
the following equations can be derived:

A) \( P_a \cos(d) + C_s \sin(a) = R \cos(a + \phi') + P_w \)
B) \( P_a \sin(d) + C_w - W - P_s + C_s \cos(a) + R \sin(a + \phi') = 0 \)

substituting equation (A) into (B) the expression becomes

where,

\( P_a \) = total active force on the retaining wall in kN
\( W \) = weight of soil wedge in kN
\( P_w \) = horizontal force from water in tension cracks in kN
\( C_w \) = cohesion force at the back of wall (\( C_w = c_w . \) length KB)
\( C_s \) = cohesion force acting along failure plane (\( C_s = c . \) length BD)
\( c \) = cohesion of the soil or shear strength of soil in kN/m²
\( \phi' \) = angle of internal friction (or angle of shearing resistance)
\( \delta \) = angle of friction between wall and soil in degrees
\( \alpha \) = angle between failure plane and the vertical in degrees
\( \theta \) = angle between earth surface and horizontal in degrees
\( P_s \) = surcharge force (\( P_s = q_s . \) length AE)
\( q_s \) = surcharge per unit area in kN/m²

As the plane of rupture (or failure) is not known a series of trial
wedges will be considered by varying angle "\( \alpha \)" and the maximum
value active force on the wall (\( P_a \)).

**Consider the pressure distribution on a vertical wall**

\[ 2.C.(K_a-K_c.cw/c)^{0.5} \]

Theoretically, down to
depth \( Z_0 \) the active
pressure is negative
(i.e. tensile)

Angle of shearing resistance \( \phi \) = \( 0^\circ \)
Angle of friction between wall and soil \( \delta = 0^\circ \)
Wedge theory will be adopted

Saturated density of soil \( D_s = 18.5 \text{ kN/m}^3 \)

Cohesion of soil \( c = 30 \text{ kN/m}^2 \)

Coefficient of wall cohesion \( c_w = 0 \text{ kN/m}^2 \)

Angle of shearing resistance \( \phi' = 0^\circ \)

Coefficient of active soil pressure \( K_a = (\tan(\text{RAD}(45-\phi'/2)))^2 = 1 \)

\( K_1 = K_a = 1 \)

Depth of tension cracks \( Z_o = (2cK_1^{0.5} - q_sK_a)/(D_sK_a) \approx -0.81081 \text{ m} \)

As \( Z_o \) is negative this will be ignored \( Z_o = 0 \text{ m} \)

Density of water \( D_w = 9.81 \text{ kN/m}^3 \)

Horiz. force from water in tension crack \( P_w = 0 \text{ kN} \)

\[
\text{num} = W + P_s + P_w \tan(\text{RAD}(\alpha)) - C_w \cos(\text{RAD}(\alpha)) - C_s \sin(\text{RAD}(\alpha)) * \tan(\text{RAD}(\alpha)) = 712
\]

\[
\text{den} = \sin(\text{RAD}(\delta)) + \cos(\text{RAD}(\delta)) * \tan(\text{RAD}(\alpha)) = 1
\]

Total active force \( P_a = \frac{\text{num}}{\text{den}} = 712 \text{ kN} \)

Summary of results derived from wedge theory

Horizontal component of active force \( hP_a = P_a \cos(\text{RAD}(\delta)) = 712 \text{ kN} \)

Vertical component of active force \( vP_a = P_a \sin(\text{RAD}(\delta)) = 0 \text{ kN} \)

Distance \( L_1 = H - 2Z_o/3 = 8 \text{ m} \)

Distance \( L_2 = (H + q_s/D_s - Z_o)/3 = 4.018 \text{ m} \)

Distance \( L_3 = (P_wL_1 + (hP_a - P_w)L_2)/hP_a = 4.018 \text{ m} \)
Diagram shows forces on wall due to active pressure.
Active pressure on a retaining wall to BS EN 1997-1:2004 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
The active pressure of cohesive soils could be evaluated using Coulomb's wedge theory. Bell's solution will be used for evaluating the depth of tension cracks (i.e. depth Zo).

Coulomb's theory assumes that there is a neutral or ineffective zone of depth (Zo) within which there is no adhesion or friction along the back of the wall or along the plane of rupture. This theory also assumes that the surface of rupture is plane. For cohesionless soils Zo will be taken as zero.

Coulomb's wedge theory takes into account two further forces acting on the wedge which have not previously been considered by "Rankine" or "Bell". These are as follows:

a) Cohesion along the failure plane;
b) Cohesion along the plane of the wall.

For cohesionless soils the above forces will be taken as zero.

It is worth pointing out that the "wedge theory" is much simpler to use for evaluating the active pressure when the soil surface is not horizontal. The back of the retaining wall will be assumed to be vertical in this program. Both cohesive and cohesionless soils can be considered using this theory.

Angle of earth surface to horizontal \( \theta = 23.2^\circ \)
Height of wall under consideration \( H = 8 \text{ m} \)
**Bell's solution**

Bell gives the formula for the active clay pressure at failure on the back of the wall as follows:

\[ p_a = K_a \cdot D_s \cdot Z - 2 \cdot c \cdot K_a^{0.5} \]

where \( K_a \) is the coefficient of active clay pressure which relates to the angle "phi'" and is given by the following expression:

\[ K_a = (\tan(45 - \phi'/2))^2 \]

By comparing two soils, one cohesive (say clay) and the other cohesionless, both having the same bulk density and angle of shearing resistance, the cohesive soil, being more self-supporting, exerts a smaller pressure on the retaining wall than does the cohesionless soil, the difference being the second component in "Bell's equation given above.

**Forces on soil wedge**

\[
\begin{align*}
\text{surcharge} & \uparrow \\
\text{Ps} & \\
\text{Vo} & \\
\text{Vo} & \\
\text{Pa} & \\
\text{B} & \\
\text{t} & = \text{angle} \ "\thetaeta" \\
\text{phi'} & = \text{angle} \ "\phi'" \\
\text{Zo} & = \text{depth of cracks} \\
a & = \text{angle} \ "\alphaalpha" \\
d & = \text{angle} \ "\deltaelt" \\
qs & = 0 \text{ kN/m}^2
\end{align*}
\]
By resolving the forces shown in the figure above in two directions the following equations can be derived:

A) \[ Pa \cos(d) + Cs \sin(a) = R \cos(a + \phi') + Pw \]
B) \[ Pa \sin(d) + Cw - W - Ps + Cs \cos(a) + R \sin(a + \phi') = 0 \]

substituting equation (A) into (B) the expression becomes

where,

\[
\begin{align*}
Pa &= \text{total active force on the retaining wall in kN} \\
W &= \text{weight of soil wedge in kN} \\
Pw &= \text{horizontal force from water in tension cracks in kN} \\
Cw &= \text{cohesion force at the back of wall (Cw = cw. length KB)} \\
Cs &= \text{cohesion force acting along failure plane (Cs = c . length BD)} \\
c &= \text{cohesion of the soil or shear strength of soil in kN/m}^2 \\
\phi' &= \text{angle of internal friction (or angle of shearing resistance)} \\
delta &= \text{angle of friction between wall and soil in degrees} \\
alpha &= \text{angle between failure plane and the vertical in degrees} \\
theta &= \text{angle between earth surface and horizontal in degrees} \\
Ps &= \text{surcharge force (Ps = qs. length AE)} \\
qs &= \text{surcharge per unit area in kN/m}^2
\end{align*}
\]

As the plane of rupture (or failure) is not known a series of trial wedges will be considered by varying angle "alpha" and the maximum value active force on the wall (Pa).

**Consider the pressure distribution on a vertical wall**

\[ 2.0 \cdot (K_a - K_a \cdot cw/c)^{0.5} \]

![Pressure distribution diagram](image)

Theoretically, down to depth Zo the active pressure is negative (i.e. tensile)

Angle of shearing resistance \( \phi' = 30^\circ \)
Angle of friction between wall and soil \( \delta = 20^\circ \)
Wedge theory will be adopted

Dry density of soil \( D_s = 17.17 \text{ kN/m}^3 \)
Cohesion of soil \( c = 0 \text{ kN/m}^2 \)
Coefficient of wall cohesion \( c_w = 0 \text{ kN/m}^2 \)
Angle of shearing resistance \( \phi' = \phi' = 30^\circ \)
Coefficient of active soil pressure \( K_a = (\tan(\text{RAD}(45-\phi'/2)))^2 = 0.33333 \)
\( K_1 = K_a = 0.33333 \)
Depth of tension cracks \( Z_o = (2cK_1^{0.5} - q_s K_a)/(D_s K_a) = 0 \text{ m} \)
Horiz. force from water in tension cracks \( P_w = 0 \text{ kN} \)

\[
\text{num} = W + P_s + P_w \tan(\text{RAD}(\delta)) - C_w - C_s \cos(\text{RAD}(\alpha)) - C_s \sin(\text{RAD}(\alpha)) \\
\text{den} = \sin(\text{RAD}(d)) + \cos(\text{RAD}(d)) \tan(\text{RAD}(\alpha)) = 3.6191
\]

Total active force \( P_a = \text{num}/\text{den} = 250.14 \text{ kN} \)

Summary of results derived from wedge theory

Horizontal component of active force \( h_P = P_a \cos(\text{RAD}(\delta)) = 235.05 \text{ kN} \)
Vertical component of active force \( v_P = P_a \sin(\text{RAD}(\delta)) = 85.552 \text{ kN} \)

Distance \( L_1 = H - 2Z_o/3 = 8 \text{ m} \)
Distance \( L_2 = (H + q_s/D_s - Z_o)/3 = 2.6667 \text{ m} \)
Distance \( L_3 = (P_wL_1 + (h_P - P_w)L_2)/h_P = 2.6667 \text{ m} \)
Diagram shows forces on wall due to active pressure.

\[ \text{vPa} \]

\[ \text{hPa} \]

\[ \text{H} \]

\[ \text{L3} \]
Location: Ex 7.5 Problems in Engineering soils, PL Capper/WF Cassie

Active pressure on a retaining wall to BS EN 1997-1:2004 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal  \( \theta = 0^\circ \)
Height of wall under consideration  \( H = 8 \) m

**Forces on soil wedge**

Surcharge load  \( q_s = 0 \) kN/m²
Angle of shearing resistance  \( \phi' = 20^\circ \)
Angle of friction between wall and soil  \( \delta = 0^\circ \)

Wedge theory will be adopted

Dry density of soil  \( D_s = 17.17 \) kN/m³
Cohesion of soil  \( c = 13 \) kN/m²
Coefficient of wall cohesion  \( c_w = 0 \) kN/m²
Angle of shearing resistance  \( \phi' = \phi'' = 20^\circ \)
Coefficient of active soil pressure  \( K_a = (\tan(RAD(45-\phi'/2)))^2 = 0.49029 \)

Depth of tension cracks  \( z_o = (2cK_1^0.5-q_sK_a)/(D_sK_a) = 2.1626 \) m
Rain water in tension cracks will be ignored.
Horiz.force from water in tension crack  \( P_w = 0 \) kN
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))
*ΤΑΝ(RAD(ap))=204.84

den=SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap))=1.4281

Total active force $Pa=num/den=143.43$ kN

Summary of results derived from wedge theory

Horizontal component of active force $hPa=Pa*COS(RAD(delta))=143.43$ kN
Vertical component of active force $vPa=Pa*SIN(RAD(delta))=0$ kN

Distance $L1=H-2*Zo/3=6.5583$ m
Distance $L2=(H+qs/Ds-Zo)/3=1.9458$ m
Distance $L3=(Pw*L1+(hPa-Pw)*L2)/hPa=1.9458$ m
Active pressure on a retaining wall to BS EN 1997-1:2004 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

Water level is assumed to be below the base of the wall.
Angle of earth surface to horizontal \( \theta = 0^\circ \)
Height of wall under consideration \( H = 10 \text{ m} \)

**Forces on soil wedge**

![Soil wedge diagram]

Surcharge load \( q_s = 0 \text{ kN/m}^2 \)
Angle of shearing resistance \( \phi' = 15^\circ \)
Angle of friction between wall and soil \( \delta = 15^\circ \)

**Wedge theory will be adopted**

Dry density of soil \( D_s = 18.6 \text{ kN/m}^3 \)
Cohesion of soil \( c = 15 \text{ kN/m}^2 \)
Coefficient of wall cohesion \( c_w = 15 \text{ kN/m}^2 \)
Angle of shearing resistance \( \phi' = \phi' = 15^\circ \)
Coefficient of active soil pressure \( K_a = (\tan(\text{RAD}(45-\phi'/2)))^2 = 0.58879 \)
\( K_1 = K_a + K_a c_w / c = 1.1776 \)
Depth of tension cracks \( z_o = (2cK_1^0.5 - q_sK_a) / (D_sK_a) = 2.9726 \text{ m} \)
Rain water in tension cracks will be ignored.
Horiz. force from water in tension crack \( P_w = 0 \text{ kN} \)
num=W+Ps+Pw*TAN(RAD(ap))-Cw-Cs*COS(RAD(a))-Cs*SIN(RAD(a))
* TAN(RAD(ap))=454.42

den= SIN(RAD(d))+COS(RAD(d))*TAN(RAD(ap))=1.9319

Total active force Pa=num/den=235.23 kN

\[\text{Horizontal component of active force } hPa=Pa \times \cos(RAD(delta))=227.21 \text{ kN}\]

\[\text{Vertical component of active force } vPa=Pa \times \sin(RAD(delta))=60.881 \text{ kN}\]

Distance \(L1=H-2*Zo/3=8.0182 \text{ m}\)

Distance \(L2=(H+qs/Ds-Zo)/3=2.3425 \text{ m}\)

Distance \(L3=(Pw*L1+(hPa-Pw)*L2)/hPa=2.3425 \text{ m}\)

Diagram shows forces on wall due to active pressure
Active pressure on a retaining wall to BS EN 1997-1:2004 using Coulomb's "Wedge Theory" for cohesive and cohesionless soils

The angle of friction between wall and soil "delta" will be taken as zero as the soil at the top of the wall is assumed to be horizontal.

Surcharge pressure \( q = 0 \) kN/m²

**Design data for soil 1**

- Depth of soil \( d_1 = 4 \) m
- Cohesive strength of soil \( c_1' = 8 \) kN/m²
- Coefficient of wall cohesion \( c_{w1} = 4 \) kN/m²
- Angle of shearing resistance \( \phi_{1'} = 27° \)
- Dry unit weight of soil \( \gamma_1 = 20 \) kN/m³

**Design data for soil 2**

- Depth of soil \( d_2 = 4 \) m
- Cohesive strength of soil \( c_2' = 8 \) kN/m²
- Coefficient of wall cohesion \( c_{w2} = 4 \) kN/m²
- Angle of shearing resistance \( \phi_{2'} = 27° \)
- Saturated unit weight soil \( \gamma_2 = 20 \) kN/m³

Depth of tension cracks \( Z_o = \frac{2 \cdot c_1' \cdot \sqrt{K_{ac}} - q \cdot K_{a1}}{\gamma_1 \cdot K_{a1}} = 1.5989 \) m

Active forces on wall (soil 1)

- \( P_1 = p_{a1} \cdot d_1 = -39.219 \) kN
- \( P_2 = 0.5 \cdot (p_{a1} - p_{a1}) \cdot d_1 = 60.084 \) kN

Active forces on wall (soil 2)

- \( P_3 = p_{a2} \cdot d_2 = 80.949 \) kN
- \( P_4 = 0.5 \cdot (p_{a2} - p_{a2}) \cdot d_2 = 109.09 \) kN

Active force from water in crack \( P_5 = P_{w} = 12.539 \) kN
Point of application of total active pressure will be at a distance "L3" from the bottom of the wall.

\[
L3 = \frac{(P1 \cdot L1 + P2 \cdot L2 + P3 \cdot L3 + P4 \cdot L4 + P5 \cdot L5)}{Pa} = 2.1456 \text{ m}
\]

---

**Summary of results**

- **Horizontal component of active force**: \(hPa = Pa = 223.45 \text{ kN}\)
- **Vertical component of active force**: \(Rv = 0 \text{ kN}\)
- **Point of application of active force**: \(L3 = 2.1456 \text{ m}\)
Location: Example 8.10 of 'Basic soil mechanics' by Roy Whitlow

Retaining wall base to BS8002

vo = axial load on wall
d = depth of base
h = height of stem
L3 = lever arm from u/s of base
Pabh = horizontal component of active force
Pabv = vertical component of active force
theta = angle of slope of soil

Horiz.component of active force Pabh=138.9 kN/m
Distance (as above) L3=2.667 m
Vertical component active force Pabv=0 kN/m
Weight/unit vol.of retained soil w=20 kN/m³
Height of wall h=7.25 m
Load above wall supported on stem vo=0 kN/m
Angle of shearing resistance phi'=40°
Friction factor @ underside base fb=0.67
Thickness of wall t=0.3 m
Density of wall (stem) dt=24 kN/m³
Tickness of base d=0.75 m
Angle of slope of soil (if any) theta=0°

Forces and moments on wall

Maximum B.M. at base of stem M=Pabh*(L3-d)=266.27 kNm/m
Maximum S.F. at base of stem S=Pabh=138.9 kN/m

Check bearing pressure

Length of base L=5.2 m
Distance from A to centre of stem x=3.35 m
Depth of soil over base at toe db=0 m
Depth of soil for passive resist. \( dp = 0 \) m
Allowable ground pressure \( p = 150 \) kN/m²

**WARNING:**

*Eccentricity is negative, i.e. pressure at heel (A) is greater than pressure at toe (B).*

Centroid of load lies within middle third. Pressure varies from \( p_a \) at A to \( p_b \) at B.

\[
\begin{array}{c}
A \\
p_a \\
\_ \\
- \\
- \\
\_ \\
p_b \\
B
\end{array}
\]

Maximum pressure is at A \( p_{max} = p_a = 129.34 \) kN/m²
As \( p_{max} \leq p \) (129.34 kN/m² \( \leq 150 \) kN/m²), the pressure beneath the base is within specified limit.

**Shear forces and bending moments on base**

- Total S.F. on toe (plus is up) \( S_{wb} = \left( \frac{p_w + p_b}{2} - d_t d \right) \times \left( x_1 - \frac{t}{2} \right) - w_f \)
  \[ = 154.94 \] kN/m

- Total S.F. on heel (plus is up) \( S_{wa} = S_{au} - S_{ad} = -131.48 \) kN/m

- Total B.M. on toe (plus is up) \( M_{wb} = (2 \times p_b + p_w) \times \left( x_1 - \frac{t}{2} \right)^2 / 6 - M_{bd} \)
  \[ = 129.8 \] kNm/m

- Total B.M. on heel (plus is up) \( M_{wa} = (2 \times p_a + p_w) \times \left( x - \frac{t}{2} \right)^2 / 6 - M_{ad} \)
  \[ = -197.69 \] kNm/m

**Resistance to sliding**

- FoS sliding (frict.resist.only) \( F_{OSF} = F / St = 2.9414 \)

**Resistance to overturning**

Rotation assumed to occur about lowest forward edge of base (i.e. toe). Safety factors against overturning:

- Including vert.comp.of pressure \( F_{OST1} = M_{r1} / M_{ot} = 5.4268 \)
- Ignoring vert.comp.of pressure \( F_{OST2} = M_{r2} / M_{ot} = 5.4268 \)
Loadings and foundations  Made by: IFB
Stability calculations for a retaining wall  Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.  Ref No: SC752 BS

Location: Example 6.5 of 'Soil mechanics' by R.F.Craig

Retaining wall base to BS8002

\[ \text{vo} = \text{axial load on wall} \]
\[ d = \text{depth of base} \]
\[ h = \text{height of stem} \]
\[ L_3 = \text{lever arm from u/s of base} \]
\[ \text{Pabh} = \text{horizontal component of active force} \]
\[ \text{Pabv} = \text{vertical component of active force} \]
\[ \theta = \text{angle of slope of soil} \]

Horiz.component of active force  \( \text{Pabh}=78.4 \text{ kN/m} \)
Distance (as above)  \( L_3=1.962 \text{ m} \)
Vertical component active force  \( \text{Pabv}=0 \text{ kN/m} \)
Weight/unit vol.of retained soil  \( w=17 \text{ kN/m}^3 \)
Height of wall  \( h=5 \text{ m} \)
Pressure due to surcharge  \( \text{press}=10 \text{ kN/m}^2 \)
Load above wall supported on stem  \( \text{vo}=0 \text{ kN/m} \)
Angle of shearing resistance  \( \phi'=36^\circ \)
Friction factor @ underside base  \( f_b=0.509 \)
Thickness of wall  \( t=0.3 \text{ m} \)
Density of wall (stem)  \( dt=23.5 \text{ kN/m}^3 \)
Thickness of base  \( d=0.4 \text{ m} \)
Angle of slope of soil (if any)  \( \theta=0^\circ \)

Forces and moments on wall

Maximum B.M. at base of stem  \( M=\text{Pabh}*(L_3-d)=122.46 \text{ kNm/m} \)
Maximum S.F. at base of stem  \( S=\text{Pabv}=78.4 \text{ kN/m} \)

Check bearing pressure

Length of base  \( L=3 \text{ m} \)
Distance from A to centre of stem  \( x=1.9 \text{ m} \)
Depth of soil over base at toe \( db = 0 \text{ m} \)
Depth of soil for passive resist. \( dp = 0 \text{ m} \)
Allowable ground pressure \( p = 150 \text{ kN/m}^2 \)

Centroid of load lies within middle third. Pressure varies from \( pa \) at A to \( pb \) at B.

\[
\begin{array}{c}
A \\
\text{pa} \\
B \\
\text{pb}
\end{array}
\]

Maximum pressure is at B \( \text{pmax} = pb = 119.24 \text{ kN/m}^2 \)
As \( \text{pmax} \leq p \) (119.24 kN/m\(^2\) \leq 150 kN/m\(^2\)), the pressure beneath the base is within specified limit.

**Shear forces and bending moments on base**

- Total S.F. on toe (plus is up) \( Swb = \frac{(pwb+pb)/2-dt*d}{2}*(x1-t/2)-wf \)  
  \[= 91.512 \text{ kN/m} \]
- Total S.F. on heel (plus is up) \( Swa = Sau - Sad = -79.827 \text{ kN/m} \)
- Total B.M. on toe (plus is up) \( Mwb = \frac{2*pb+pwb}{x1-t/2}^2/6-Mbd \)  
  \[= 45.501 \text{ kNm/m} \]
- Total B.M. on heel (plus is up) \( Mwa = \frac{2*pa+pwa}{x-t/2}^2/6-Mad \)  
  \[= -82.555 \text{ kNm/m} \]

**Resistance to sliding**

- FoS sliding (frict.resist.only) \( FOSF = F/St = 1.4913 \)

**WARNING:**

*Factor of safety against sliding (considering frictional resistance only) is less than 1.5.*

**Resistance to overturning**

Rotation assumed to occur about lowest forward edge of base (i.e. toe).
Safety factors against overturning:
- Including vert.comp.of pressure \( \text{FOSOT1} = Mr1/Mot = 2.8238 \)
- Ignoring vert.comp.of pressure \( \text{FOSOT2} = Mr2/Mot = 2.8238 \)
Retaining wall base to BS8002

vo = axial load on wall

\( \theta \)

\( d = \) depth of base
\( h = \) height of stem

\( L_3 = \) lever arm from u/s of base

\( P_{abv} = \) horizontal component of active force

\( P_{abh} = \) vertical component of active force

\( \theta = \) angle of slope of soil

Horiz. component of active force \( P_{abh} = 65.439 \text{ kN/m} \)
Distance (as above) \( L_3 = 1.5233 \text{ m} \)
Vertical component active force \( P_{abv} = 0 \text{ kN/m} \)
Weight/unit vol. of retained soil \( w = 18.8 \text{ kN/m}^3 \)
Height of wall \( h = 4.38 \text{ m} \)
Load above wall supported on stem \( v_o = 0 \text{ kN/m} \)
Angle of shearing resistance \( \phi' = 30^\circ \)
Friction factor \( f_b = 0.4 \)
Thickness of wall \( t = 0.22 \text{ m} \)
Density of wall (stem) \( d_t = 24 \text{ kN/m}^3 \)
Thickness of base \( d = 0.19 \text{ m} \)
Angle of slope of soil (if any) \( \theta = 0^\circ \)

Forces and moments on wall

Maximum B.M. at base of stem \( M = P_{abh}(L_3 - d) = 87.25 \text{ kNm/m} \)
Maximum S.F. at base of stem \( S = P_{abh} = 65.439 \text{ kN/m} \)

Check bearing pressure

Length of base \( L = 3.124 \text{ m} \)
Distance from A to centre of stem \( x = 1.86 \text{ m} \)
Depth of soil over base at toe \( d_b = 0 \text{ m} \)
Depth of soil for passive resist. \( dp = 1.5 \text{ m} \)
Allowable ground pressure \( p = 75 \text{ kN/m}^2 \)

Centroid of load lies within middle third. Pressure varies from \( p_a \) at A to \( p_b \) at B.

\[
\begin{array}{c}
A \\
- \\
p_a \\
- \\
B \\
\end{array}
\]

Maximum pressure is at B \( p_{\text{max}} = p_b = 62.748 \text{ kN/m}^2 \)
As \( p_{\text{max}} \leq p \) (62.748 kN/m² ≤ 75 kN/m²), the pressure beneath the base is within specified limit.

**Shear forces and bending moments on base**

Total S.F. on toe (plus is up) \( S_{wb} = \frac{(p_{wb} + p_b)}{2 - d_t d} * (x_1 - t/2) - w_f \)
= 65.164 kN/m

Total S.F. on heel (plus is up) \( S_{wa} = S_{au} - S_{ad} = -54.009 \text{ kN/m} \)

Total B.M. on toe (plus is up) \( M_{wb} = \frac{(2p_b + p_{wb})}{6} * (x_1 - t/2)^2 - M_{bd} \)
= 37.981 kNm/m

Total B.M. on heel (plus is up) \( M_{wa} = \frac{(2p_a + p_{wa})}{6} * (x - t/2)^2 - M_{ad} \)
= -48.59 kNm/m

**Resistance to sliding**

\( \text{FoS sliding (frict. resist. only)} \) \( \frac{F_{OSF}}{F/st} = 1.1093 \)

**WARNING:**

Factor of safety against sliding (considering frictional resistance only) is less than 1.5.

\( \text{FoS sliding (friction/cohesion + passive)} \) \( \frac{F_{OSS}}{(F+R)/st} = 2.0789 \)

**Resistance to overturning**

Rotation assumed to occur about lowest forward edge of base (i.e. toe).
Safety factors against overturning:

Including vert. comp. of pressure \( \frac{F_{OS1}}{M_{r1}/M_{ot}} = 3.7676 \)

Ignoring vert. comp. of pressure \( \frac{F_{OS2}}{M_{r2}/M_{ot}} = 3.7676 \)
Location: Ex1 - 'Soil mechanics' by R.F.Craig (Ex. 6.5)

Retaining wall base to EC7

\[ \text{vo} = \text{axial load on wall} \]
\[ d = \text{depth of base} \]
\[ h = \text{height of stem} \]
\[ L_3 = \text{lever arm from u/s of base} \]

Pabh and Pabv are the unfactored horizontal & vertical components of the total active force. Angle \( \theta \) is the soil slope.

**Lever arm (as per diagram)**

\[ L_3 = 1.962 \text{ m} \]

**Weight/unit vol. of retained soil**

\[ w = 17 \text{ kN/m}^3 \]

**Height of stem**

\[ h = 5 \text{ m} \]

**Pressure due to surcharge**

\[ \text{press} = 0 \text{ kN/m}^2 \]

**Load above wall supported on stem**

\[ \text{vo} = 0 \text{ kN/m} \]

**Friction factor @ underside base**

\[ f_b = 0.509 \]

**Thickness of wall**

\[ t = 0.3 \text{ m} \]

**Thickness of base**

\[ d = 0.4 \text{ m} \]

**Density of wall (stem)**

\[ \text{den} = 25 \text{ kN/m}^3 \]

**Angle of slope of soil (if any)**

\[ \theta = 0^\circ \]

**Forces and moments on wall - SLS**

\[ M = \text{Pabh}(L_3-d) = 122.46 \text{ kNm/m} \]

\[ V = \text{Pabh} = 78.4 \text{ kN/m} \]

**Check bearing pressures - SLS**

\[ L = 3 \text{ m} \]
\[ x = 1.9 \text{ m} \]
\[ db = 0 \text{ m} \]
\[ dp = 0 \text{ m} \]
\[ p = 150 \text{ kN/m}^2 \]
Centroid of load lies within middle third. Pressure varies from pa at A to pb at B.

\[ \text{Maximum pressure is at B } \quad p_{\text{max}} = pb = 122.65 \text{ kN/m}^2 \]

As \( p_{\text{max}} \leq p \) (122.65 kN/m\(^2\) ≤ 150 kN/m\(^2\)), the pressure beneath base is within specified limit.

**Partial factors - ULS load combinations 1**

- Partial load factor (backfill) \( g_1 = 1 \)
- Partial load factor (v. surcharge) \( g_2 = 0 \)
- Partial load factor (wall & base) \( g_3 = 1.35 \)
- Partial load factor (earth press) \( g_4 = 1.35 \) (horiz. pressure)
- Partial load factor (surcharge) \( g_5 = 1.5 \) (horiz. pressure)

**Forces and moments on wall - ULS load combination 1**

- Stem design moment \( M_{Ed1} = (g_4 \times Pe + g_5 \times Ps) \times (L_3 - d/2) = 186.49 \text{ kNm/m} \)
- Horizontal shear at base of stem \( V_{Ed1} = g_4 \times Pe + g_5 \times Ps = 105.84 \text{ kN/m} \)

**Bearing pressures - ULS load combination 1**

Centroid of load outside middle third giving triangular distribution of pressure.

Pressure varies from 0 at distance x from A to a maximum of pb1 at B.
Maximum pressure is at B  \( p_{\text{max1}} = p_{b1} = 170.59 \text{ kN/m}^2 \)

**Shear forces and bending moments on base - ULS load combination 1**

- Total S.F. on toe (plus is up)  \( S_{wb} = \frac{(p_{wb} + p_{b1})}{2} - \text{den} \cdot d \cdot (x_1 - t/2) - w_{f1} = 125.19 \text{ kN/m} \)
- Total S.F. on heel (plus is up)  \( S_{wa} = S_{au} - S_{ad} = -92.22 \text{ kN/m} \)
- Total B.M. on toe (plus is up)  \( M_{wb} = \frac{(2 \cdot p_{b1} + p_{wb})}{6} \cdot (x_1 - t/2)^2 = 63.797 \text{ kNm/m} \)
- Total B.M. on heel (plus is up)  \( M_{wa} = S_{au} \cdot \left( \frac{lb - (x_1 + t/2)}{3} \right) = -106.91 \text{ kNm/m} \)

**Resistance to sliding - ULS (load combinations 1 & 2)**

- **Combination 1:**
  - Sliding force  \( S_{For} = S_{t1} = 105.84 \text{ kN} \)
  - Frictional resistance  \( S_{Res} = F = 110.07 \text{ kN} \)
  - Thus the criterion for sliding is satisfied.

- **Combination 2:**
  - Sliding force  \( S_{For} = \frac{K_a \cdot (g_7 \cdot P_e + g_8 \cdot P_s)}{K_a} = 99.987 \text{ kN} \)
  - Frictional resistance  \( S_{Res} = F = 110.07 \text{ kN} \)
  - Thus the criterion for sliding is satisfied.

**Resistance to overturning - ULS (load combination 1 & 2)**

- **Combination 1:**
  - Total restraining mt. (with Mrv)  \( M_{r1} = g_9 \cdot (M_{re} + M_{rw} + M_{rv}) = 362.11 \text{ kNm/m} \)
  - ditto (without Mrv)  \( M_{r2} = g_9 \cdot (M_{re} + M_{rw}) = 362.11 \text{ kNm/m} \)
  - Total overturning moment  \( M_{ot} = M_{t1} + M_{t2} = 169.2 \text{ kNm/m} \)

- **Combination 2:**
  - Total restraining mt. (with Mrv)  \( M_{r1} = g_9' \cdot (M_{re} + M_{rw} + M_{rv}) = 362.11 \text{ kNm/m} \)
  - ditto (without Mrv)  \( M_{r2} = g_9' \cdot (M_{re} + M_{rw}) = 362.11 \text{ kNm/m} \)
  - Total overturning moment  \( M_{ot} = M_{t1} + M_{t2} = 196.17 \text{ kNm/m} \)

**Partial factors - ULS load combinations 2**

- Partial load factor (backfill)  \( g_1 = 1.0 \)
- Partial load factor (v.surcharge)  \( g_2' = 0 \)
- Partial load factor (wall & base)  \( g_3 = 1.0 \)
- Partial load factor (earth press)  \( g_4 = 1.0 \) (horiz.pressure)
- Partial load factor (surcharge)  \( g_5 = 1.3 \) (horiz.pressure)

**Forces and moments on wall - ULS load combination 2**

- Stem design moment  \( M_{Ed2} = (g_4 \cdot P_e + g_5 \cdot P_s) \cdot (L_3 - d/2) = 138.14 \text{ kNm/m} \)
- Horizontal shear at base of stem  \( V_{Ed2} = g_4 \cdot P_e + g_5 \cdot P_s = 78.4 \text{ kN/m} \)
Bearing pressures - ULS load combination 2

Centroid of load lies within middle third. Pressure varies from \( p_a^2 \) at A to \( p_b^2 \) at B.

Maximum pressure is at B \( p_{\text{max}}^2 = p_b^2 = 122.65 \text{ kN/m}^2 \)

Shear forces and bending moments on base - ULS load combination 2

Total S.F. on toe (plus is up):
\[
S_{wb}^2 = (\frac{p_wb + p_b^2}{2} - \text{den} * d) * (x_1 - t/2) - w_f^2 = 91.806 \text{ kN/m}
\]

Total S.F. on heel (plus is up):
\[
S_{wa}^2 = S_{a2} - S_{a1} = -76.977 \text{ kN/m}
\]

Total B.M. on toe (plus is up):
\[
M_{wb}^2 = (2*p_b^2 + p_wb) * (x_1 - t/2)^2/6 - M_{b1} = 46.017 \text{ kNm/m}
\]

Total B.M. on heel (plus is up):
\[
M_{wa}^2 = (2*p_a^2 + p_wa) * (x - t/2)^2/6 - M_{a1} = -82.411 \text{ kNm/m}
\]
Design Summary - ULS load

combination 1 & 2 to EC7

- Moment at base of stem: 186.49 kNm
- Shear at base of stem: 105.84 kN
- Moment at b-b (toe): 63.797 kNm
- Shear at b-b (toe): 125.19 kN
- Moment at a-a (heel): -106.91 kNm
- Shear at a-a (heel): -92.22 kN

Pressures under base - SLS

- Pressure @ A: 21.515 kN/m²
- Pressure @ B: 122.65 kN/m²
- Allowable GBP: 150 kN/m²
Location: Ex2 - 'Basic soil mechanics' by Roy Whitlow (Ex. 8.10)

Retaining wall base to EC7

\[ \text{vo} = \text{axial load on wall} \]
\[ d = \text{depth of base} \]
\[ h = \text{height of stem} \]
\[ L_3 = \text{lever arm from u/s of base} \]

Pabh and Pabv are the unfactored horizontal & vertical components of the total active force. Angle \( \theta \) is the soil slope.

Lever arm (as per diagram) \( L_3 = 2.667 \) m
Weight/unit vol. of retained soil \( w = 20 \) kN/m\(^3\)
Height of stem \( h = 7.25 \) m
Load above wall supported on stem \( \text{vo} = 0 \) kN/m
Friction factor @ underside base \( \text{fb} = 0.67 \)
Thickness of wall \( t = 0.3 \) m
Thickness of base \( d = 0.75 \) m
Density of wall (stem) \( \text{den} = 25 \) kN/m\(^3\)
Angle of slope of soil (if any) \( \theta = 0^\circ \)

Forces and moments on wall - SLS

Maximum B.M. at base of stem \( M = \text{Pabh} \times (L_3 - d) = 266.27 \) kNm/m
Maximum S.F. at base of stem \( V = \text{Pabh} = 138.9 \) kN/m

Check bearing pressures - SLS

Length of base \( L = 5.2 \) m
Distance from A to centre of stem \( x = 3.35 \) m
Depth of soil over base at toe \( db = 0 \) m
Depth of soil for passive resist. \( dp = 0 \) m
Allowable ground pressure \( p = 150 \) kN/m\(^2\)
WARNING:

Eccentricity is negative, i.e. pressure at heel (A) is greater than pressure at toe (B).

Centroid of load lies within middle third. Pressure varies from pa at A to pb at B.

\[
\begin{array}{c|c|c}
A & \_ & B \\
\_ & \_ & \_ \\
\hline
pa & \_ & pb
\end{array}
\]

Maximum pressure is at A \( p_{\text{max}} = p_a = 130.15 \text{ kN/m}^2 \)

As \( p_{\text{max}} \leq p \) \((130.15 \text{ kN/m}^2 \leq 150 \text{ kN/m}^2)\), the pressure beneath base is within specified limit.

**Partial factors - ULS load combinations 1**

- Partial load factor (backfill) \( g_1 = 1 \)
- Partial load factor (v.surcharge) \( g_2 = 0 \)
- Partial load factor (wall & base) \( g_3 = 1.35 \)
- Partial load factor (earth press) \( g_4 = 1.35 \) (horiz. pressure)
- Partial load factor (surcharge) \( g_5 = 1.5 \) (horiz. pressure)

**Forces and moments on wall - ULS load combination 1**

- Stem design moment \( M_{\text{Ed1}} = (g_4 \cdot P_e + g_5 \cdot P_s) \cdot (L_3 - d/2) \)
  \[ = 429.78 \text{ kNm/m} \]

- Horizontal shear at base of stem \( V_{\text{Ed1}} = g_4 \cdot P_e + g_5 \cdot P_s = 187.52 \text{ kN/m} \)

**Bearing pressures - ULS load combination 1**
Centroid of load lies within middle third. Pressure varies from pal at A to pbl at B.

\[
A \quad \text{pal} \quad \text{B} \quad \text{pbl}
\]

Maximum pressure is at B \( p_{max1}=p_{b1}=148.89 \text{ kN/m}^2 \)

**Shear forces and bending moments on base - ULS load combination 1**

- Total S.F. on toe (plus is up) \( S_{wb}=(p_{wb}+p_{b1})/(2-den*d)*(x_1-t/2)-w_{f1} \)
  \(=209.99 \text{ kN/m} \)
- Total S.F. on heel (plus is up) \( S_{wa}=S_{uw}-S_{ad}=-137.18 \text{ kN/m} \)
- Total B.M. on toe (plus is up) \( M_{wb}=(2*p_{b1}+p_{wb})*(x_1-t/2)^2/6-M_{bd} \)
  \(=181.68 \text{ kNm/m} \)
- Total B.M. on heel (plus is up) \( M_{wa}=(2*p_{a1}+p_{wa})*(x-t/2)^2/6-M_{ad} \)
  \(=-240.74 \text{ kNm/m} \)

**Resistance to sliding - ULS (load combinations 1 & 2)**

Combination 1:
- Sliding force \( S_{For}=S_{t1}=187.52 \text{ kN} \)
- Frictional resistance \( S_{Res}=F=412.64 \text{ kN} \)
  Thus the criterion for sliding is satisfied.

Combination 2:
- Sliding force \( S_{For}=K_a^2/K_a*(g_7*P_e+g_8*P_s)=181.56 \text{ kN} \)
- Frictional resistance \( S_{Res}=F=412.64 \text{ kN} \)
  Thus the criterion for sliding is satisfied.

**Resistance to overturning - ULS (load combination 1 & 2)**

Combination 1:
- Total restraining mt. (with Mrv) \( M_{r1}=g_9*(M_{re}+M_{rw}+M_{rv})=1822 \text{ kNm/m} \)
  ditto \( M_{r2}=g_9*(M_{re}+M_{rw})=1822 \text{ kNm/m} \)
- Total overturning moment \( M_{ot}=M_{t1}+M_{t2}=407.49 \text{ kNm/m} \)

Combination 2:
- Total restraining mt. (with Mrv) \( M_{r1}=g_9''*(M_{re}+M_{rw}+M_{rv})=1822 \text{ kNm/m} \)
  ditto \( M_{r2}=g_9''*(M_{re}+M_{rw})=1822 \text{ kNm/m} \)
- Total overturning moment \( M_{ot}=M_{t1}+M_{t2}=484.23 \text{ kNm/m} \)

**Partial factors - ULS load combinations 2**

- Partial load factor (backfill) \( g_1=1.0 \)
- Partial load factor (v.surcharge) \( g_2'=0 \)
- Partial load factor (wall & base) \( g_3=1.0 \)
- Partial load factor (earth press) \( g_4=1.0 \text{ (horiz.pressure)} \)
- Partial load factor (surcharge) \( g_5=1.3 \text{ (horiz.pressure)} \)
Forces and moments on wall - ULS load combination 2

Stem design moment \[ M_{Ed2} = (g4 \cdot P_e + g5 \cdot P_s) \cdot (L3 - d/2) \] \[ = 318.36 \text{ kNm/m} \]

Horizontal shear at base of stem \[ V_{Ed2} = g4 \cdot P_e + g5 \cdot P_s = 138.9 \text{ kN/m} \]

Bearing pressures - ULS load combination 2

WARNING:
Eccentricity is negative, i.e. pressure at heel (A) is greater than pressure at toe (B).

Centroid of load lies within middle third. Pressure varies from \( P_{a2} \) at A to \( P_{b2} \) at B.

\[ A \quad \frac{-}{-} \quad \frac{-}{-} \quad \frac{-}{-} \quad B \]

\[ P_{a2} \quad P_{b2} \]

Maximum pressure is at A \( P_{max2} = P_{a2} = 130.15 \text{ kN/m}^2 \)

Shear forces and bending moments on base - ULS load combination 2

Total S.F. on toe (plus is up):
\[ S_{wb2} = ((p_{wb} + p_{b2})/2 - \text{den} \cdot d) \cdot (x1 - t/2) \]
\[ -w_{f2} = 156.07 \text{ kN/m} \]

Total S.F. on heel (plus is up):
\[ S_{wa2} = S_{\text{a2}} - S_{\text{d}} = -130.59 \text{ kN/m} \]

Total B.M. on toe (plus is up):
\[ M_{wb2} = (2 \cdot p_{b2} + p_{wb}) \cdot (x1 - t/2)^2/6 - M_{bd} \]
\[ = 130.82 \text{ kNm/m} \]

Total B.M. on heel (plus is up):
\[ M_{wa2} = (2 \cdot p_{a2} + p_{wa}) \cdot (x - t/2)^2/6 - M_{ad} \]
\[ = -196.64 \text{ kNm/m} \]
Design Summary - ULS load

combination 1 & 2 to EC7

- Moment at base of stem: 429.78 kNm
- Shear at base of stem: 187.52 kN
- Moment at b-b (toe): 181.68 kNm
- Shear at b-b (toe): 209.99 kN
- Moment at a-a (heel): -240.74 kNm
- Shear at a-a (heel): -137.18 kN

Pressures under base - SLS

- Pressure @ A: 130.15 kN/m²
- Pressure @ B: 106.73 kN/m²
- Allowable GBP: 150 kN/m²
Retaining wall base to EC7

\[ \text{vo} = \text{axial load on wall} \]
\[ d = \text{depth of base} \]
\[ h = \text{height of stem} \]
\[ L_3 = \text{leverage arm from u/s of base} \]

Pabh and Pabv are the unfactored horizontal & vertical components of the total active force. Angle theta is the soil slope.

Lever arm (as per diagram) \( L_3 = 1.5233 \text{ m} \)
Weight/unit vol. of retained soil \( w = 18.8 \text{ kN/m}^3 \)
Height of stem \( h = 4.38 \text{ m} \)
Load above wall supported on stem \( \text{vo} = 0 \text{ kN/m} \)
Friction factor @ underside base \( \text{fb} = 0.4 \)
Thickness of wall \( t = 0.22 \text{ m} \)
Thickness of base \( d = 0.19 \text{ m} \)
Density of wall (stem) \( \text{den} = 25 \text{ kN/m}^3 \)
Angle of slope of soil (if any) \( \theta = 0^\circ \)

Forces and moments on wall - SLS

Maximum B.M. at base of stem \( M = \text{Pabh} \times (L_3 - d) = 87.25 \text{ kNm/m} \)
Maximum S.F. at base of stem \( V = \text{Pabh} = 65.439 \text{ kN/m} \)

Check bearing pressures - SLS

Length of base \( L = 3.124 \text{ m} \)
Distance from A to centre of stem \( x = 1.86 \text{ m} \)
Depth of soil over base at toe \( db = 0 \text{ m} \)
Depth of soil for passive resist. \( dp = 1.5 \text{ m} \)
Allowable ground pressure \( p = 75 \text{ kN/m}^2 \)
Centroid of load lies within middle third. Pressure varies from pa at A to pb at B.

\[ \text{A} \quad \text{pa} \quad - \quad - \quad - \quad \text{pb} \quad \text{B} \]

Maximum pressure is at B  \( p_{\text{max}} = \text{pb} = 63.423 \text{ kN/m}^2 \)

As \( p_{\text{max}} \leq p \) (63.423 kN/m\(^2\) \leq 75 kN/m\(^2\)), the pressure beneath base is within specified limit.

**Partial factors - ULS load combinations 1**

- Partial load factor (backfill)  \( g_1 = 1 \)
- Partial load factor (v.surcharge)  \( g_2 = 0 \)
- Partial load factor (wall & base)  \( g_3 = 1.35 \)
- Partial load factor (earth press)  \( g_4 = 1.35 \) (horiz.pressure)
- Partial load factor (surcharge)  \( g_5 = 1.5 \) (horiz.pressure)

**Forces and moments on wall - ULS load combination 1**

- Stem design moment  \( M_{\text{Ed1}} = (g_4 \times P_e + g_5 \times P_s) \times (L_3 - d/2) = 126.18 \text{ kNm/m} \)
- Horizontal shear at base of stem  \( V_{\text{Ed1}} = g_4 \times P_e + g_5 \times P_s = 88.343 \text{ kN/m} \)

**Bearing pressures - ULS load combination 1**

Centroid of load lies within middle third. Pressure varies from pal at A to pbl at B.

\[ \text{A} \quad \text{pal} \quad - \quad - \quad - \quad \text{pbl} \quad \text{B} \]

Maximum pressure is at B  \( p_{\text{max1}} = \text{pbl} = 90.779 \text{ kN/m}^2 \)
Shear forces and bending moments on base - ULS load combination 1

Total S.F. on toe (plus is up) \[ Swb=((pwb+pb1)/2-den*d)*(x1-t/2)-wf1 \] = 87.415 kN/m
Total S.F. on heel (plus is up) \[ Swa=Sau-Sad=-63.671 \text{ kN/m} \]
Total B.M. on toe (plus is up) \[ Mwb=(2*pb1+pwb)*(x1-t/2)^2/6-Mbd \] = 52.72 kNm/m
Total B.M. on heel (plus is up) \[ Mwa=(2*pa1+pwa)*(x-t/2)^2/6-Mad \] = -63.67 kNm/m

Resistance to sliding - ULS (load combinations 1 & 2)

Combination 1:
Sliding force \[ SFor=St1'=88.343 \text{ kN} \]
Friction plus passive resistance \[ SRes=F+R=136.66 \text{ kN} \]
Thus the criterion for sliding is satisfied.

Combination 2:
Coefficient of passive pressure \[ Kp'=2.4442 \]
Sliding force \[ SFor=Ka2/Ka*(g7*P1+g8*P2)=80.321 \text{ kN} \]
Friction plus passive resistance \[ SRes=F+R=124.91 \text{ kN} \]
Thus the criterion for sliding is satisfied.

Resistance to overturning - ULS (load combination 1 & 2)

Combination 1:
Total restraining mt. (with Mrv) \[ Mr1=g9*(Mre+Mrw+Mrv)=339.94 \text{ kNm/m} \]
ditto (without Mrv) \[ Mr2=g9*(Mre+Mrw)=339.94 \text{ kNm/m} \]
Total overturning moment \[ Mot=Mt1+Mt2=109.65 \text{ kNm/m} \]

Combination 2:
Total restraining mt. (with Mrv) \[ Mr1=g9'**(Mre+Mrw+Mrv)=339.94 \text{ kNm/m} \]
ditto (without Mrv) \[ Mr2=g9'**(Mre+Mrw)=339.94 \text{ kNm/m} \]
Total overturning moment \[ Mot=Mt1+Mt2=122.35 \text{ kNm/m} \]

Partial factors - ULS load combinations 2

Partial load factor (backfill) \[ g1=1.0 \]
Partial load factor (v.surcharge) \[ g2'=0 \]
Partial load factor (wall & base) \[ g3=1.0 \]
Partial load factor (earth press) \[ g4=1.0 \text{ (horiz.pressure)} \]
Partial load factor (surcharge) \[ g5=1.3 \text{ (horiz.pressure)} \]

Forces and moments on wall - ULS load combination 2

Stem design moment \[ MEd2=(g4*Pe+g5*Ps)*(L3-d/2) = 93.467 \text{ kNm/m} \]
Horizontal shear at base of stem \[ VEd2=g4*Pe+g5*Ps=65.439 \text{ kN/m} \]
Bearing pressures - ULS load combination 2

Centroid of load lies within middle third. Pressure varies from $p_{a2}$ at A to $p_{b2}$ at B.

Maximum pressure is at B \[ p_{\text{max}2}=p_{b2}=63.423 \text{ kN/m}^2 \]

Shear forces and bending moments on base - ULS load combination 2

Total S.F. on toe (plus is up):
\[ S_{\text{wb}2}=\frac{(p_{\text{wb}}+p_{b2})}{2}\text{den}d\times(x_{1}-t/2) \]
\[ -w_{f2}=65.648 \text{ kN/m} \]

Total S.F. on heel (plus is up):
\[ S_{\text{wa}2}=S_{\text{au}}-S_{\text{ad}}=-53.606 \text{ kN/m} \]

Total B.M. on toe (plus is up):
\[ M_{\text{wb}2}=\frac{(2p_{b2}+p_{\text{wb}})}{6}\times(x_{1}-t/2)^2-M_{\text{bd}} \]
\[ =38.275 \text{ kNm/m} \]

Total B.M. on heel (plus is up):
\[ M_{\text{wa}2}=\frac{(2p_{a2}+p_{\text{wa}})}{6}\times(x-t/2)^2-M_{\text{ad}} \]
\[ =-48.287 \text{ kNm/m} \]
Design Summary - ULS load

combination 1 & 2 to EC7

- Moment at base of stem: 126.18 kNm
- Shear at base of stem: 88.343 kN
- Moment at b-b (toe): 52.72 kNm
- Shear at b-b (toe): 87.415 kN
- Moment at a-a (heel): -63.67 kNm
- Shear at a-a (heel): -63.671 kN

Pressures under base - SLS

- Pressure @ A: 53.754 kN/m²
- Pressure @ B: 63.423 kN/m²
- Allowable GBP: 75 kN/m²
Location: Biaxial moments on pad base

Pressures due to biaxial bending on rectangular base

3               Corner 1
N   Mx
o    My
Ly

4      Lx

Base length in X-direction Lx=6 m
Base length in Y-direction Ly=3 m
Bending moment in X-direction Mx=150 kNm
Bending moment in Y-direction My=300 kNm
Axial load N=1000 kN

\[ P_3 = 80.556 \text{ kN/m}^2 \]
\[ P_{\text{max}} = 97.222 \text{ kN/m}^2 \]
\[ P_2 = 30.556 \text{ kN/m}^2 \]
\[ P_4 = 13.889 \text{ kN/m}^2 \]

Moments Mx & My are shown in directions X & Y (rather than the more usual way of showing them acting about the axes) to accord with the references.
Location: Bases on grids A2-A6, K2-K6

Pad base with vertical and horizontal loads (V & H) and overturning moment (M)

Characteristic bending moment \( M = 150 \text{ kNm} \)
Characteristic vertical force \( V = 250 \text{ kN} \)
Characteristic horizontal shear \( H = 50 \text{ kN} \)
Width of base (into figure) \( b = 2.4 \text{ m} \)
Depth of base \( d = 0.6 \text{ m} \)
Length of base \( L = 3 \text{ m} \)
Distance to load position \( x = 1.5 \text{ m} \)
Allowable ground pressure \( p = 100 \text{ kN/m}^2 \)

Overturning

Overturning moment \( \text{Mot}=M+H*d = 180 \text{ kNm} \)
Restraining moment \( \text{Mre}=Sw*L/2+V*(L-x) = 530.52 \text{ kNm} \)
F.O.S. against overturning \( \text{FOSOT} = \text{Mre}/\text{Mot} = 2.9473 \)
Location: Bases on grids A2-A6, K2-K6

Characteristic bending moment   \( M = 150 \text{ kNm} \)
Characteristic vertical load    \( V = 250 \text{ kN} \)
Characteristic horizontal load  \( H = 50 \text{ kN} \)
Width of base (into figure)     \( b = 2.4 \text{ m} \)
Depth of base                   \( d = 0.6 \text{ m} \)
Length of base                  \( L = 3 \text{ m} \)
Distance to load position       \( x = 1.5 \text{ m} \)
Allowable ground pressure       \( p = 100 \text{ kN/m}^2 \)

Resistance to overturning - ULS (load combination 1 & 2)

Combination 1:
Partial load factor (vert.load)  \( g_1 = 1 \)
Partial load factor (bend.moment) \( g_2 = 1.5 \)
Partial load factor (horiz.shear) \( g_3 = 1.5 \)
Restraining moment              \( M_{re} = g_1 \left( \frac{S_w L}{2} + V(L-x) \right) = 530.52 \text{ kNm} \)
Total overturning moment        \( M_{ot1} = M_{t1} + M_{t2} = 270 \text{ kNm/m} \)

Combination 2:
Partial load factor (vert.load)  \( g_4 = 1 \)
Partial load factor (bend.moment) \( g_5 = 1.3 \)
Partial load factor (horiz.shear) \( g_6 = 1.3 \)
Restraining moment              \( M_{re} = g_4 \left( \frac{S_w L}{2} + V(L-x) \right) = 530.52 \text{ kNm} \)
Total overturning moment        \( M_{ot2} = M_{t1} + M_{t2} = 234 \text{ kNm/m} \)
Location: Automatic sizing of trapezoidal base for typical loads.

Preliminary calculations for trapezoidal combined base

Distance between columns  dist=7.5 m
Thickness of base         h=1 m
Left column: vertical load FVleft=580 kN
horizontal shear         FHleft=150 kN
moment (clockwise)       Mleft=0 kNm
Right column: vertical load FVrt=400 kN
horizontal shear         FHrt=50 kN
moment (clockwise)       Mrt=0 kNm

No reversal of moments so moment-reversal multiplier revm=1 throughout.
Shear-reversal multiplier revs is set to 1 when shear forces act from
left to right, and to -1 when shear forces act from right to left.
Allowable bearing pressure press=80 kN/m²
Concrete density         den=24 kN/m³

Value of dimension x0    x0=0.82 m
Value of dimension x1    x1=0.82 m
Value of dimension x2    x2=1.38 m
Value of dimension x3    x3=1.38 m
Length of base           L=x1+dist+x3=9.7 m
Smaller width of base    bmin=2*x0=1.64 m
Greater width of base    bmax=2*x2=2.76 m
Area of base             areaP=L*(bmax+bmin)/2=21.34 m²
Self-weight of base      Swt=areaP*den=512.16 kN
Total vertical load on base V=FVleft+FVrt+Swt=1492.2 kN
Position of centroid of base xbarP=L/3*(2*x0+x2)/(x0+x2)=4.4385 m from left end.
x'=xbarP-x3=3.0585 m from left-hand column.
Moment of inertia of base Ixx=(x0^2+4*x0*x2+x2^2)/(x0+x2)*L^3/18=163.71 m⁴

With clockwise moments and left to right shears

Taking moments about position of left column:
clock=FVrt*dist+revs*(FHleft+FHrt)*h+revm*(Mleft+Mrt)+Swt*x'
   =4766.4 kNm
Centroid of load from left col. xbar=clock/V=3.1943 m
Eccentricity of upward pressure e=x'-xbar=-0.13583 m
Trapezoidal distribution of pressure beneath base

Pressure at left end of base
\[ \text{Plt} = V \left( \frac{1}{\text{areaP}} + e \frac{x_{barP}}{I_{xx}} \right) \]
\[ = 64.428 \text{ kN/m}^2 \]

Pressure at right end of base
\[ \text{Prt} = V \left( \frac{1}{\text{areaP}} - e \frac{(l-x_{barP})}{I_{xx}} \right) \]
\[ = 76.437 \text{ kN/m}^2 \]

Dist (m) Width Shearing force (kN) Bending moment (kNm)
from LHE (m) To left To right To left To right

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<th>Dist (m)</th>
<th>Width (m)</th>
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<th>Bending moment To left</th>
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Bending moments and shears at a specific point

Dist of point considered from left end of base x=4.85 m
\[ \text{Final shearing forces:} \]
To left \[ \text{SL} = \text{SL} - \text{FV}_{\text{left}} = -58.979 \text{ kN} \]
To right \[ \text{SR} = \text{SR} - \text{FV}_{\text{rt}} = 58.979 \text{ kN} \]
\[ \text{Final bending moments:} \]
To left \[ \text{ML} = \text{ML} - \text{FV}_{\text{left}} \times (x-x_3) + \text{revm} \times \text{M}_{\text{left}} + \text{revs} \times \text{FH}_{\text{left}} \times h = -580.64 \text{ kNm} \]
To right \[ \text{MR} = \text{MR} - \text{FV}_{\text{rt}} \times (l-x-x_1) - \text{revm} \times \text{M}_{\text{rt}} - \text{revs} \times \text{FH}_{\text{rt}} \times h = -580.64 \text{ kNm} \]

With clockwise moments and right to left shears

Taking moments about position of left column:
\[ \text{clock} = \text{FV}_{\text{rt}} \times \text{dist} + \text{revs} \times (\text{FH}_{\text{left}} + \text{FH}_{\text{rt}}) \times h + \text{revm} \times (\text{M}_{\text{left}} + \text{M}_{\text{rt}}) + \text{Swt} \times x' \]
\[ = 4366.4 \text{ kNm} \]
Centroid of load from left col. \[ \text{xbar} = \frac{\text{clock}}{V} = 2.9263 \text{ m} \]
Eccentricity of upward pressure \[ e = x' - \text{xbar} = 0.13223 \text{ m} \]

Trapezoidal distribution of pressure beneath base

Pressure at left end of base
\[ \text{Plt} = V \left( \frac{1}{\text{areaP}} + e \frac{x_{barP}}{I_{xx}} \right) \]
\[ = 75.273 \text{ kN/m}^2 \]

Pressure at right end of base
\[ \text{Prt} = V \left( \frac{1}{\text{areaP}} - e \frac{(l-x_{barP})}{I_{xx}} \right) \]
\[ = 63.582 \text{ kN/m}^2 \]
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<tr>
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**Bending moments and shears at a specific point**

Dist. of point considered from left end of base x=4.85 m

**Final shearing forces:**

- To left: \( SL = SL - FV_{left} = 2.8763 \) kN
- To right: \( SR = SR - FV_{rt} = -2.8763 \) kN

**Final bending moments:**

- To left: \( ML = ML - FV_{left} \times (x-x_3) + revm \times M_{left} + revs \times FH_{left} \times h = -667.63 \) kNm
- To right: \( MR = MR - FV_{rt} \times (l-x-x_1) - revm \times M_{rt} - revs \times FH_{rt} \times h = -667.63 \) kNm
Location: Manual check of trapezoidal base for typical loads.

Preliminary calculations for trapezoidal combined base

Distance between columns \( \text{dist}=7.5 \text{ m} \)
Thickness of base \( h=1 \text{ m} \)
Left column: vertical load \( \text{FV}_{\text{left}}=1500 \text{ kN} \)
  horizontal shear \( \text{FH}_{\text{left}}=400 \text{ kN} \)
  moment (clockwise) \( \text{M}_{\text{left}}=50 \text{ kNm} \)
Right column: vertical load \( \text{FV}_{\text{rt}}=400 \text{ kN} \)
  horizontal shear \( \text{FH}_{\text{rt}}=400 \text{ kN} \)
  moment (clockwise) \( \text{M}_{\text{rt}}=50 \text{ kNm} \)

No reversal of moments so moment-reversal multiplier \( \text{revm}=1 \) throughout.
Shear-reversal multiplier \( \text{revs} \) is set to 1 when shear forces act from left to right, and to -1 when shear forces act from right to left.
Allowable bearing pressure \( \text{press}=300 \text{ kN/m}^2 \)
Concrete density \( \text{den}=24 \text{ kN/m}^3 \)

\( \text{Value of dimension } x_0 \quad x_0=1 \text{ m} \)
\( \text{Value of dimension } x_1 \quad x_1=1 \text{ m} \)
\( \text{Value of dimension } x_2 \quad x_2=2 \text{ m} \)
\( \text{Value of dimension } x_3 \quad x_3=1 \text{ m} \)
Length of base \( l=x_1+\text{dist}+x_3=9.5 \text{ m} \)
Smaller width of base \( b_{\text{min}}=2*x_0=2 \text{ m} \)
Greater width of base \( b_{\text{max}}=2*x_2=4 \text{ m} \)
Area of base \( \text{areaP}=l*(b_{\text{max}}+b_{\text{min}})/2=28.5 \text{ m}^2 \)
Self-weight of base \( \text{Swt}=(\text{areaP}*h*\text{den})=684 \text{ kN} \)
Total vertical load on base \( V=\text{FV}_{\text{left}}+\text{FV}_{\text{rt}}+\text{Swt}=2584 \text{ kN} \)
Position of centroid of base \( \text{xbarP}=1/3*(2*x_0+x_2)/(x_0+x_2) \)
  \( =4.2222 \text{ m from left end.} \)
  \( x'=\text{xbarP}-x_3 \)
  \( =3.2222 \text{ m from left-hand column.} \)
Moment of inertia of base \( I_{xx}=(x_0^2+4*x_0*x_2+x_2^2)/(x_0+x_2)*l^3 \)
  \( /18=206.41 \text{ m}^4 \)

With clockwise moments and left to right shears

Taking moments about position of left column:
  \( \text{clock}=\text{FV}_{\text{rt}}*\text{dist}+\text{revs}*(\text{FH}_{\text{left}}+\text{FH}_{\text{rt}})*h*\text{revm}*(\text{M}_{\text{left}}+\text{M}_{\text{rt}})+\text{Swt}*x'=6104 \text{ kNm} \)
Centroid of load from left col. \( \text{xbar}=\text{clock}/V=2.3622 \text{ m} \)
Eccentricity of upward pressure \( e=x'-\text{xbar}=0.85999 \text{ m} \)
Trapezoidal distribution of pressure beneath base

Pressure at left end of base \[ \text{Plt} = \frac{V}{(1/\text{areaP}+e\times \bar{x}_P/I_{xx})} = 136.12 \text{ kN/m}^2 \]

Pressure at right end of base \[ \text{Prt} = \frac{V}{(1/\text{areaP}-e(1-\bar{x}_P)/I_{xx})} = 33.844 \text{ kN/m}^2 \]

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<th>Bending moment (kNm)</th>
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Bending moments and shears at a specific point

Dist. of point considered from left end of base \( x = 4.75 \) m

Final shearing forces:
- To left: \( \text{SL} = \text{SL-FVleft} = -40.789 \text{ kN} \)
- To right: \( \text{SR} = \text{SR-FVrt} = 40.789 \text{ kN} \)

Final bending moments:
- To left: \( \text{ML} = \text{ML-FVleft*(x-x3)+revm*Mleft+revs*(FHleft*h)} = -1210.1 \text{ kNm} \)
- To right: \( \text{MR} = \text{MR-FVrt*(1-x-x1)-revm*Mrt-revs*(FHrt*h)} = -1210.1 \text{ kNm} \)

With clockwise moments and right to left shears

Taking moments about position of left column:
- \( \text{clock} = \text{FVrt*dist+revs*(FHleft+FHrt)*h+revm*(Mleft+Mrt)+Swt*x'} = 4504 \text{ kNm} \)
- Centroid of load from left col. \( \bar{x} = \text{clock/V} = 1.743 \) m
- Eccentricity of upward pressure \( e = x'-\bar{x} = 1.4792 \) m

Triangular distribution of pressure beneath base.

**WARNING: Uplift occurs**

Point of zero upward pressure \( xz = 9.0918 \) m from left end of base.

Pressure at left end of base \[ \text{Plt} = \frac{V}{(xz*(wz+4*x2)/6)} = 169.07 \text{ kN/m}^2 \]
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Bending moments and shears at a specific point

Dist.of point considered from left end of base x=4.75 m

Final shearing forces:
- To left \( SL = SL - FV_{left} = 212.56 \) kN
- To right \( SR = SR - FV_{rt} = -212.56 \) kN

Final bending moments:
- To left \( ML = ML - FV_{left} \times (x - x_3) + \text{revm} \times M_{left} + \text{revs} \times FH_{left} \times h = -1136.6 \) kNm
- To right \( MR = MR - FV_{rt} \times (l - x - x_1) - \text{revm} \times M_{rt} - \text{revs} \times FH_{rt} \times h = -1136.6 \) kNm
Location: Manual check of trapezoidal base w. uplift on right column

Preliminary calculations for trapezoidal combined base

Distance between columns \( \text{dist}=7.5 \text{ m} \)
Thicknes of base \( h=1 \text{ m} \)
Left column: vertical load \( F_{Vleft}=1000 \text{ kN} \)
  horizontal shear \( F_{Hleft}=0 \text{ kN} \)
  moment (clockwise) \( M_{left}=0 \text{ kNm} \)
Right column: vertical load \( F_{Vrt}=-200 \text{ kN} \)
  horizontal shear \( F_{Hrt}=0 \text{ kN} \)
  moment (clockwise) \( M_{rt}=0 \text{ kNm} \)

No reversal of moments so moment-reversal multiplier \( \text{revm}=1 \) throughout.
No reversal of shears, so shear-reversal multiplier \( \text{revs}=1 \) throughout.
Allowable bearing pressure \( \text{press}=200 \text{ kN/m}^2 \)
Concrete density \( \text{den}=24 \text{ kN/m}^3 \)

Value of dimension \( x_0 \) \( x_0=1 \text{ m} \)
Value of dimension \( x_1 \) \( x_1=1 \text{ m} \)
Value of dimension \( x_2 \) \( x_2=2 \text{ m} \)
Value of dimension \( x_3 \) \( x_3=1 \text{ m} \)
Length of base \( l=x_1+\text{dist}+x_3=9.5 \text{ m} \)
Smaller width of base \( b_{\text{min}}=2\times x_0=2 \text{ m} \)
Greater width of base \( b_{\text{max}}=2\times x_2=4 \text{ m} \)
Area of base \( \text{area}_{P}=l\times (b_{\text{max}}+b_{\text{min}})/2=28.5 \text{ m}^2 \)
Self-weight of base \( \text{Swt} = \text{area}_{P}\times \text{den} = 684 \text{ kN} \)
Total vertical load on base \( V=F_{Vleft}+F_{Vrt}+\text{Swt}=1484 \text{ kN} \)
Position of centroid of base \( x_{\text{bar}_{P}}=l/3\times(2\times x_0+x_2)/(x_0+x_2) = 4.2222 \text{ m from left end.} \)
\( x'=x_{\text{bar}_{P}}-x_3 = 3.2222 \text{ m from left-hand column.} \)
Moment of inertia of base \( I_{xx} = (x_0^2+4\times x_0\times x_2+x_2^2)/(x_0+x_2)\times l^3/18 = 206.41 \text{ m}^4 \)
Taking moments about position of left column:
  clock\(=F_{Vrt}\times \text{dist}+\text{revs}\times (F_{Hleft}+F_{Hrt})\times h+\text{revm}\times (M_{left}+M_{rt})+\text{Swt}\times x'=704 \text{ kNm} \)
Centroid of load from left col. \( x_{\text{bar}} = \text{clock}/V = 0.47439 \text{ m} \)
Eccentricity of upward pressure \( e = x'-x_{\text{bar}} = 2.7478 \text{ m} \)
Triangular distribution of pressure beneath base.

WARNING: Uplift occurs

Point of zero upward pressure \( x_z = 4.6276 \) m from left end of base.
Pressure at left end of base \( P_{lt} = \frac{V}{(x_z^* (w_z + 4*x^2))} = 174.51 \) kN/m²

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Bending moments and shears at a specific point

Dist of point considered from left end of base \( x = 4.75 \) m

Final shearing forces:
To left \( SL = SL - F_{V left} = 85 \) kN
To right \( SR = SR - F_{V right} = -85 \) kN

Final bending moments:
To left \( ML = ML - F_{V left} (x - x_3) + revm^* M_{left} + revs \) \* \( FH_{left}^* h = 118.25 \) kNm
To right \( MR = MR - F_{V right} (l - x - x_1) - revm^* M_{right} - revs \) \* \( FH_{right}^* h = 118.25 \) kNm
Location: Typical calculation for optimum missing dimension.

Preliminary calculations for trapezoidal combined base

Distance between columns \( \text{dist} = 7.5 \text{ m} \)
Thickness of base \( h = 1 \text{ m} \)
Left column: vertical load \( F_{Vleft} = 1450 \text{ kN} \)
horizontal shear \( F_{Hleft} = 200 \text{ kN} \)
moment (clockwise) \( M_{left} = 0 \text{ kNm} \)
Right column: vertical load \( F_{Vrt} = 800 \text{ kN} \)
horizontal shear \( F_{Hrt} = 200 \text{ kN} \)
moment (clockwise) \( M_{rt} = 0 \text{ kNm} \)

No reversal of moments so moment-reversal multiplier \( \text{revm} = 1 \) throughout.
No reversal of shears, so shear-reversal multiplier \( \text{revs} = 1 \) throughout.
Allowable bearing pressure \( \text{press} = 200 \text{ kN/m}^2 \)
Concrete density \( \text{den} = 24 \text{ kN/m}^3 \)

Value of dimension \( x_0 = 0.5 \text{ m} \)
Value of dimension \( x_1 = 0.5 \text{ m} \)
Value of dimension \( x_2 = 0 \text{ m} \)
Value of dimension \( x_3 = 1.2 \text{ m} \)
Length of base \( l = x_1 + \text{dist} + x_3 = 9.2 \text{ m} \)
Smaller width of base \( b_{min} = 2x_0 = 1 \text{ m} \)
Greater width of base \( b_{max} = 2x_2 = 1.94 \text{ m} \)
Area of base \( \text{area}_P = l(x_{max} + b_{min})/2 = 13.524 \text{ m}^2 \)
Self-weight of base \( \text{Swt} = \text{area}_P \times \text{den} = 324.58 \text{ kN} \)
Total vertical load on base \( V = F_{Vleft} + F_{Vrt} + \text{Swt} = 2574.6 \text{ kN} \)
Position of centroid of base \( x_{bar} = \text{clock} / V = 2.8527 \text{ m} \)
Eccentricity of upward pressure \( e = x' - x_{bar} = 0.057073 \text{ m} \)

Taking moments about position of left column:

\[
\text{clock} = F_{Vrt} \times \text{dist} + \text{revs} \times (F_{Hleft} + F_{Hrt}) \times h + \text{revm} \times (M_{left} + M_{rt}) + \text{Swt} \times x' \\
= 7344.4 \text{ kNm}
\]

Position of centroid from left col. \( x_{bar} = \text{clock} / V = 2.8527 \text{ m} \)

Eccentricity of upward pressure \( e = x' - x_{bar} = 0.057073 \text{ m} \)
Trapezoidal distribution of pressure beneath base

Pressure at left end of base
\[ P_{lt} = V \times \left( \frac{1}{A_1} + e \times \bar{x}_{P} / I_{xx} \right) \]
\[ = 196.92 \text{ kN/m}^2 \]

Pressure at right end of base
\[ P_{rt} = V \times \left( \frac{1}{A_1} - e \times (l - \bar{x}_{P}) / I_{xx} \right) \]
\[ = 182.25 \text{ kN/m}^2 \]

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Bending moments and shears at a specific point

Dist of point considered from left end of base x=4.6 m

Final shearing forces:
To left \( SL = SL - FV_{left} = -121.2 \text{ kN} \)
To right \( SR = SR - FV_{rt} = 121.2 \text{ kN} \)

Final bending moments:
To left \( ML = ML - FV_{left} \times (x-x_3) + revm \times M_{left} + revs \times FH_{left} \times h = -1511.4 \text{ kNm} \)
To right \( MR = MR - FV_{rt} \times (l-x-x_1) - revm \times M_{rt} - revs \times FH_{rt} \times h = -1511.4 \text{ kNm} \)
Preliminary calculations for T-shaped combined base

Distance between columns \( \text{dist}=7.5 \text{ m} \)
Thickness of base \( h=1 \text{ m} \)
Left column: vertical load \( \text{FV}_{\text{left}}=580 \text{ kN} \)
  \( \text{horizontal shear (->)} \) \( \text{FH}_{\text{left}}=150 \text{ kN} \)
  \( \text{moment (clockwise)} \) \( \text{M}_{\text{left}}=0 \text{ kNm} \)
Right column: vertical load \( \text{FV}_{\text{rt}}=400 \text{ kN} \)
  \( \text{horizontal shear (->)} \) \( \text{FH}_{\text{rt}}=150 \text{ kN} \)
  \( \text{moment (clockwise)} \) \( \text{M}_{\text{rt}}=0 \text{ kNm} \)

No reversal of moments so moment-reversal multiplier \( \text{revm}=1 \) throughout.
Shear-reversal multiplier \( \text{revs} \) is set to 1 when shear forces act from left to right, and to -1 when shear forces act from right to left.
Allowable bearing pressure \( \text{press}=80 \text{ kN/m}^2 \)
Concrete density \( \text{den}=24 \text{ kN/m}^3 \)

Location: Automatic sizing of T-shaped base with typical loads.

Value of dimension \( x_0 \) \( x_0=0.88 \text{ m} \)
Value of dimension \( x_1 \) \( x_1=0.88 \text{ m} \)
Value of dimension \( x_2 \) \( x_2=1.32 \text{ m} \)
Value of dimension \( x_3 \) \( x_3=1.32 \text{ m} \)
Length of base \( l=x_1+\text{dist}+x_3=9.7 \text{ m} \)
Smaller width of base \( b_{\text{min}}=2x_0=1.76 \text{ m} \)
Greater width of base \( b_{\text{max}}=2x_2=2.64 \text{ m} \)
Area of base \( \text{area}_P= l*b_{\text{min}}+(b_{\text{max}}-b_{\text{min}})*(x_3+\text{dist}/2) = 21.534 \text{ m}^2 \)
Self-weight of base \( \text{Swt}=\text{area}_P*h*\text{den}=516.81 \text{ kN} \)
Total vertical load on base \( \text{V}=\text{FV}_{\text{left}}+\text{FV}_{\text{rt}}+\text{Swt}=1496.8 \text{ kN} \)
\( x_{\text{bar}}P=((x_2-x_0)*(x_3+\text{dist}/2)^2+x_0*l^2)/(2*((x_2-x_0)*(x_3+\text{dist}/2)+x_0*l)) = 4.3703 \)
Position of centroid of base \( x_{\text{bar}}P=4.3703 \text{ m} \) from left end.
\( x'=x_{\text{bar}}P-x_3 = 3.0503 \text{ m} \) from left-hand column.
\( I_{xx}=2*(x_2*x_{\text{bar}}P^3+3+x_0*(1-x_{\text{bar}}P)^3-(x_{\text{bar}}P-x_3-\text{dist}/2)^3*(x_2-x_0))/3=162.37 \text{ m}^4 \)
Moment of inertia of base \( I_{xx}=162.37 \text{ m}^4 \)
Dist. from left end of base to change in width \( x_4=x_3+\text{dist}/2=5.07 \text{ m} \)
With clockwise moments and left to right shears

Taking moments about position of left column:
\[
\text{clock} = FV_{rt} \cdot \text{dist} + \text{revs} \cdot (F_{Hleft} + F_{Hrt}) \cdot h + \text{revm} \cdot (M_{left} + M_{rt}) + Swt' \cdot x' \]
\[
= 4776.4 \text{ kNm}
\]
Centroid of load from left col. \(x_{bar} = \text{clock} / V = 3.1911 \text{ m}\)
Eccentricity of upward pressure \(e = x' - x_{bar} = -0.14074 \text{ m}\)

Trapezoidal distribution of pressure beneath base

Pressure at left end of base
\[
\text{Pl}_l = V \cdot (1 / \text{area}_P + e' \cdot x_{bar} / I_{xx})
\]
\[
= 63.84 \text{ kN/m}
\]
Pressure at right end of base
\[
\text{Pr}_r = V \cdot (1 / \text{area}_P - e \cdot (l - x_{bar}) / I_{xx})
\]
\[
= 76.425 \text{ kN/m}
\]

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Bending moments and shears at a specific point

Dist. of point considered from left end of base \(x = 4.85 \text{ m}\)
Final shearing forces:
To left \(SL = SL - FV_{left} = -29.602 \text{ kN}\)
To right \(SR = SR - FV_{rt} = 29.602 \text{ kN}\)
Final bending moments:
To left \(ML = ML - FV_{left} \cdot (x - x3) + \text{revm} \cdot M_{left} + \text{revs} \cdot F_{Hleft} \cdot h = -595.25 \text{ kNm}\)
To right \(MR = MR - FV_{rt} \cdot (1 - x - x1) - \text{revm} \cdot M_{rt} - \text{revs} \cdot F_{Hrt} \cdot h = -595.25 \text{ kNm}\)

With clockwise moments and right to left shears

Taking moments about position of left column:
\[
\text{clock} = FV_{rt} \cdot \text{dist} + \text{revs} \cdot (F_{Hleft} + F_{Hrt}) \cdot h + \text{revm} \cdot (M_{left} + M_{rt}) + Swt' \cdot x' \]
\[
= 4376.4 \text{ kNm}
\]
Centroid of load from left col. \(x_{bar} = \text{clock} / V = 2.9239 \text{ m}\)
Eccentricity of upward pressure \(e = x' - x_{bar} = 0.1265 \text{ m}\)
Trapezoidal distribution of pressure beneath base

Pressure at left end of base

\[ Pl_t = V \left( \frac{1}{\text{area}_P} + e x_{\text{bar}_P} / I_{xx} \right) \]

\[ = 74.607 \text{ kN/m}^2 \]

Pressure at right end of base

\[ Pr_t = V \left( \frac{1}{\text{area}_P} - e (l - x_{\text{bar}_P}) / I_{xx} \right) \]

\[ = 63.295 \text{ kN/m}^2 \]

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Bending moments and shears at a specific point

Dist. of point considered from left end of base \( x = 4.85 \) m

Final shearing forces:
- To left: \( SL = SL - F_{V_{left}} = 31.759 \) kN
- To right: \( SR = SR - F_{V_{right}} = -31.759 \) kN

Final bending moments:
- To left: \( ML = ML - F_{V_{left}} \times (x - x_3) + rev_{M_{left}} + rev_{F_{H_{left}}} \times h = -684.62 \) kNm
- To right: \( MR = MR - F_{V_{right}} \times (l - x - x_1) - rev_{M_{right}} - rev_{F_{H_{right}}} \times h = -684.62 \) kNm
**Location:** Manual check of T-shaped base with typical loading.

**Preliminary calculations for T-shaped combined base**

- **Distance between columns:** $\text{dist}=7.5 \text{ m}$
- **Thickness of base:** $h=1 \text{ m}$
- **Left column:**
  - Vertical load: $F_{V_{\text{left}}}=1500 \text{ kN}$
  - Horizontal shear: $F_{H_{\text{left}}}=400 \text{ kN}$
  - Moment (clockwise): $M_{\text{left}}=50 \text{ kNm}$
- **Right column:**
  - Vertical load: $F_{V_{\text{rt}}}=400 \text{ kN}$
  - Horizontal shear: $F_{H_{\text{rt}}}=400 \text{ kN}$
  - Moment (clockwise): $M_{\text{rt}}=50 \text{ kNm}$

No reversal of moments so moment-reversal multiplier $revm=1$ throughout.

Shear-reversal multiplier $revs$ is set to 1 when shear forces act from left to right, and to -1 when shear forces act from right to left.

- **Allowable bearing pressure:** $\text{press}=300 \text{ kN/m}^2$
- **Concrete density:** $\text{den}=24 \text{ kN/m}^3$

```
\begin{tikzpicture}
  \node at (0,0) {T \text{-} S \text{H} \text{A} \text{P} \text{E} \text{D}};
  \node at (0,1) {\text{BASE}};
  \node at (-1,-1) {dist/2};
  \node at (1,-1) {dist/2};
  \node at (0,2) {x0};
  \node at (0,-2) {x0};
  \node at (-2,-1) {$x_2$};
  \node at (2,-1) {$x_2$};
  \node at (-1,-3) {$x_3$};
  \node at (1,-3) {$x_3$};
\end{tikzpicture}
```

- **Value of dimension $x_0$:** $x_0=1 \text{ m}$
- **Value of dimension $x_1$:** $x_1=1 \text{ m}$
- **Value of dimension $x_2$:** $x_2=2 \text{ m}$
- **Value of dimension $x_3$:** $x_3=1 \text{ m}$
- **Length of base:** $l=x_1+\text{dist}+x_3=9.5 \text{ m}$
- **Smaller width of base:** $b_{\text{min}}=2x_0=2 \text{ m}$
- **Greater width of base:** $b_{\text{max}}=2x_2=4 \text{ m}$
- **Area of base:** $\text{area}_P=l\cdot b_{\text{min}}+(b_{\text{max}}-b_{\text{min}})\cdot (x_3+\text{dist}/2)=28.5 \text{ m}^2$
- **Self-weight of base:** $\text{Swt}=\text{area}_P\cdot h\cdot \text{den}=684 \text{ kN}$
- **Total vertical load on base:** $V=F_{V_{\text{left}}}+F_{V_{\text{rt}}}+\text{Swt}=2584 \text{ kN}$
- **Position of centroid of base:** $x_{\text{bar}}=3.9583 \text{ m}$ from left end.
  - $x'=x_{\text{bar}}-x_3=2.9583 \text{ m}$ from left-hand column.
- **Moment of inertia of base:** $I_{xx}=196.48 \text{ m}^4$
- **Dist. from left end of base to change in width:** $x_4=x_3+\text{dist}/2=4.75 \text{ m}$
With clockwise moments and left to right shears

Taking moments about position of left column:
\[
\text{clock} = FV_{rt} \times \text{dist} + \text{revs} \times (FH_{left} + FH_{rt}) \times h + \text{revm} \times (M_{left} + M_{rt}) + Swt \times x'
\]
\[= 5923.5 \text{ kNm} \]

Centroid of load from left col.
\[x_{bar} = \frac{\text{clock}}{V} = 2.2924 \text{ m} \]

Eccentricity of upward pressure
\[e = x' - x_{bar} = 0.66596 \text{ m} \]

Trapezoidal distribution of pressure beneath base

Pressure at left end of base
\[P_{lt} = V \times \left( \frac{1}{\text{areaP}} + e \times x_{barP} / I_{xx} \right) \]
\[= 125.33 \text{ kN/m}^2 \]

Pressure at right end of base
\[P_{rt} = V \times \left( \frac{1}{\text{areaP}} - e \times (l - x_{barP}) / I_{xx} \right) \]
\[= 42.131 \text{ kN/m}^2 \]

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<th>Width (m)</th>
<th>Shearing force (kN)</th>
<th>Bending moment (kNm)</th>
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Bending moments and shears at a specific point

Dist.of point considered from left end of base x=4.75 m

Final shearing forces:
\[SL = SL - FV_{left} = 30.144 \text{ kN} \]
\[SR = SR - FV_{rt} = -30.144 \text{ kN} \]

Final bending moments:
\[ML = ML - FV_{left} \times (x-x) + \text{revm} \times M_{left} + \text{revs} \times FH_{left} \times h = -1228 \text{ kNm} \]
\[MR = MR - FV_{rt} \times (l-x-x) - \text{revm} \times M_{rt} - \text{revs} \times FH_{rt} \times h = -1228 \text{ kNm} \]

With clockwise moments and right to left shears

Taking moments about position of left column:
\[
\text{clock} = FV_{rt} \times \text{dist} + \text{revs} \times (FH_{left} + FH_{rt}) \times h + \text{revm} \times (M_{left} + M_{rt}) + Swt \times x'
\]
\[= 4323.5 \text{ kNm} \]

Centroid of load from left col.
\[x_{bar} = \frac{\text{clock}}{V} = 1.6732 \text{ m} \]

Eccentricity of upward pressure
\[e = x' - x_{bar} = 1.2852 \text{ m} \]
### Triangular distribution of pressure beneath base.

**WARNING:** Uplift occurs

Point of zero upward pressure: $x_z = 9.3165$ m from left end of base.

Pressure at left end of base:

$$P_{lt} = \frac{V}{(x_z^2 \times 2 - (x_2 - x_0) \times (x_z - x_4)^2 / x_z} = 157.61 \text{ kN/m}^2$$

### Bending moments and shears at a specific point

Dist. of point considered from left end of base: $x = 4.75$ m

**Final shearing forces:**

To left: $SL = SL - FV_{\text{left}} = 275.22$ kN

To right: $SR = SR - FV_{\text{rt}} = -275.22$ kN

**Final bending moments:**

To left: $ML = ML - FV_{\text{left}} \times (x - x_3) + \text{revm} \times M_{\text{left}} + \text{revs} \times F_{\text{Hleft}} \times h = -1154.5$ kNm

To right: $MR = MR - FV_{\text{rt}} \times (l - x - x_1) - \text{revm} \times M_{\text{rt}} - \text{revs} \times F_{\text{Hrt}} \times h = -1154.5$ kNm
Location: Manual check of T-shaped base with uplift on right column.

Preliminary calculations for T-shaped combined base

Distance between columns \( \text{dist}=7.5 \text{ m} \)
Thickness of base \( h=1 \text{ m} \)
Left column: vertical load \( F_{V_{\text{left}}}=1000 \text{ kN} \)
  horizontal shear(\( -> \)) \( F_{H_{\text{left}}}=0 \text{ kN} \)
  moment (clockwise) \( M_{\text{left}}=0 \text{ kNm} \)
Right column: vertical load \( F_{V_{\text{rt}}}=440 \text{ kN} \)
  horizontal shear(\( -> \)) \( F_{H_{\text{rt}}}=0 \text{ kN} \)
  moment (clockwise) \( M_{\text{rt}}=0 \text{ kNm} \)
No reversal of moments so moment-reversal multiplier \( \text{revm}=1 \) throughout.
No reversal of shears so shear-reversal multiplier \( \text{revs}=1 \) throughout.
Allowable bearing pressure \( \text{press}=200 \text{ kN/m}^2 \)
Concrete density \( \text{den}=24 \text{ kN/m}^3 \)

```
\[ T - S H A P E D \]
\[ B A S E \]
```

Value of dimension \( x_0 \) \( x_0=1 \text{ m} \)
Value of dimension \( x_1 \) \( x_1=4 \text{ m} \)
Value of dimension \( x_2 \) \( x_2=2 \text{ m} \)
Value of dimension \( x_3 \) \( x_3=2 \text{ m} \)
Length of base \( l=x_1+\text{dist}+x_3=13.5 \text{ m} \)
Smaller width of base \( b_{\text{min}}=2\times x_0=2 \text{ m} \)
Greater width of base \( b_{\text{max}}=2\times x_2=4 \text{ m} \)
Area of base \( \text{areaP}=l\times b_{\text{min}}+(b_{\text{max}}-b_{\text{min}})\times(x_3+\text{dist}/2) =38.5 \text{ m}^2 \)
Self-weight of base \( Swt=\text{areaP}\times h\times \text{den}=924 \text{ kN} \)
Total vertical load on base \( V=F_{V_{\text{left}}}+F_{V_{\text{rt}}}+\text{Swt}=2364 \text{ kN} \)
\( \text{xbarP}=((x_2-x_0)\times(x_3+\text{dist}/2)^2+x_0\times 1^2)/(2\times((x_2-x_0)\times(x_3+\text{dist}/2)+x_0\times 1)) =5.5925 \)
Position of centroid of base \( \text{xbarP}=5.5925 \text{ m from left end.} \)
\( x'=\text{xbarP}-x_3 =3.5925 \text{ m from left-hand column.} \)
\( I_{xx}=2\times x_2\times \text{xbarP}^3+x_0\times(1-\text{xbarP})^3-(\text{xbarP}-x_3-\text{dist}/2)^3\times(x_2-x_0)/3=562.85 \text{ m}^4 \)
Moment of inertia of base \( I_{xx}=562.85 \text{ m}^4 \)
Dist. from left end of base to change in width \( x_4=x_3+\text{dist}/2=5.75 \text{ m} \)
Taking moments about position of left column:
\( \text{clock}=F_{V_{\text{rt}}}\times\text{dist}+\text{revs}\times(F_{H_{\text{left}}}+F_{H_{\text{rt}}})\times h+\text{revm}\times(M_{\text{left}}+M_{\text{rt}})+\text{Swt}\times x' =6619.5 \text{ kNm} \)
Centroid of load from left col. \( x_{\text{bar}}=\text{clock}/V=2.8001 \text{ m} \)
Eccentricity of upward pressure \( e=x'-x_{\text{bar}}=0.79241 \text{ m} \)
Trapezoidal distribution of pressure beneath base

Pressure at left end of base
\[ P_{lt} = V \left( \frac{1}{\text{area}_P + e \cdot \text{xbar}_P / I_{xx}} \right) \]
\[ = 80.015 \text{ kN/m}^2 \]

Pressure at right end of base
\[ P_{rt} = V \left( \frac{1}{\text{area}_P - e \cdot (l - \text{xbar}_P) / I_{xx}} \right) \]
\[ = 35.085 \text{ kN/m}^2 \]

<table>
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<tr>
<th>Dist (m)</th>
<th>Width</th>
<th>Shearing force (kN)</th>
<th>Bending moment (kNm)</th>
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Bending moments and shears at a specific point

Dist. of point considered from left end of base \[ x = 6.75 \text{ m} \]

Final shearing forces:
To left \[ \text{SL} = \text{SL} - \text{FV}_{\text{left}} = 138.71 \text{ kN} \]
To right \[ \text{SR} = \text{SR} - \text{FV}_{\text{rt}} = -138.71 \text{ kN} \]

Final bending moments:
To left \[ \text{ML} = \text{ML} - \text{FV}_{\text{left}} \cdot (x-x_3) + \text{revm} \cdot \text{M}_{\text{left}} + \text{revs} \cdot \text{FH}_{\text{left}} \cdot h = -363.74 \text{ kNm} \]
To right \[ \text{MR} = \text{MR} - \text{FV}_{\text{rt}} \cdot (l-x-x_1) - \text{revm} \cdot \text{M}_{\text{rt}} - \text{revs} \cdot \text{FH}_{\text{rt}} \cdot h = -363.74 \text{ kNm} \]
Location: Typical calculation for optimum missing dimension.

Preliminary calculations for T-shaped combined base

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tbody>
<tr>
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<td>dist=7.5 m</td>
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<tr>
<td>Thickness of base</td>
<td>h=1 m</td>
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<tr>
<td>Left column: vertical load</td>
<td>( F_{V_{left}}=1450 \text{ kN} )</td>
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<tr>
<td>horizontal shear(-&gt;)</td>
<td>( F_{H_{left}}=200 \text{ kN} )</td>
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<tr>
<td>moment (clockwise)</td>
<td>( M_{left}=0 \text{ kNm} )</td>
</tr>
<tr>
<td>Right column: vertical load</td>
<td>( F_{V_{rt}}=800 \text{ kN} )</td>
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<td>horizontal shear(-&gt;)</td>
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<tr>
<td>moment (clockwise)</td>
<td>( M_{rt}=0 \text{ kNm} )</td>
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</table>

No reversal of moments so moment-reversal multiplier \( \text{revm}=1 \) throughout.

No reversal of shears so shear-reversal multiplier \( \text{revs}=1 \) throughout.

Allowable bearing pressure          \( \text{press}=200 \text{ kN/m}^2 \)

Concrete density                    \( \text{den}=24 \text{ kN/m}^3 \)

```
\[
\text{T - S H A P E D B A S E}
\]
```

Value of dimension \( x_0 \) \( x_0=0.5 \text{ m} \)

Value of dimension \( x_1 \) \( x_1=0.5 \text{ m} \)

Value of dimension \( x_2 \) \( x_2=0 \text{ m} \)

Value of dimension \( x_3 \) \( x_3=1.2 \text{ m} \)

Length of base \( l=x_1+\text{dist}+x_3=9.2 \text{ m} \)

Smaller width of base \( b_{min}=2\times x_0=1 \text{ m} \)

Greater width of base \( b_{max}=2\times x_2=2.08 \text{ m} \)

Area of base \( \text{area}_P=l\times b_{min}+(b_{max}-b_{min})\times(x_3+\text{dist}/2) \)
\( =14.546 \text{ m}^2 \)

Self-weight of base \( \text{Swt}=\text{area}_P\times h\times \text{den}=349.1 \text{ kN} \)

Total vertical load on base \( \text{V}=F_{V_{left}}+F_{V_{rt}}+\text{Swt}=2599.1 \text{ kN} \)

\( \text{xbar}_P=((x_2-x_0)\times(x_3+\text{dist}/2)^2+x_0\times l^2)/(2\times(x_2-x_0)\times(x_3+\text{dist}/2)+x_0\times l) \)
\( =3.819 \)

Position of centroid of base \( \text{xbar}_P=3.819 \text{ m} \) from left end.

\( x'=\text{xbar}_P-x_3 \)
\( =2.619 \text{ m} \) from left-hand column.

\( I_{xx}=2\times(x_2\times\text{xbar}_P^3+2\times0\times1^2)-(\text{xbar}_P-x_3-\text{dist}/2)^3\times(x_2-x_0))/3=91.075 \text{ m}^4 \)

Dist. from left end of base to change in width \( x_4=x_3+\text{dist}/2=4.95 \text{ m} \)

Taking moments about position of left column:

\( \text{clock}=F_{V_{rt}}\times\text{dist}\times\text{xbar}_P\times(\text{xbar}_P^2+\text{xbar}_P\times\text{h})\times\text{revm}\times(M_{left}+M_{rt})+\text{Swt}\times x' \)
\( =7314.3 \text{ kNm} \)

Centroid of load from left col. \( \text{xbar}=\text{clock}/V=2.8142 \text{ m} \)
Eccentricity of upward pressure \( e = x' - x_{\text{bar}} = -0.19515 \text{ m} \)

**Trapezoidal distribution of pressure beneath base**

Pressure at left end of base
\[
Plt = V \times \left( \frac{1}{\text{areaP}} + e \times x_{\text{barP}} / I_{xx} \right)
\]
\[
= 157.41 \text{ kN/m}^2
\]

Pressure at right end of base
\[
Prt = V \times \left( \frac{1}{\text{areaP}} - e \times (l - x_{\text{barP}}) / I_{xx} \right)
\]
\[
= 208.65 \text{ kN/m}^2
\]

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<th>Shearing force (kN)</th>
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**Bending moments and shears at a specific point**

Dist. of point considered from left end of base \( x = 4.6 \text{ m} \)

Final shearing forces:
- To left \( SL = SL - FV_{\text{left}} = -50.949 \text{ kN} \)
- To right \( SR = SR - FV_{\text{rt}} = 50.949 \text{ kN} \)

Final bending moments:
- To left \( ML = ML - FV_{\text{left}} \times (x - x_3) + \text{revm} \times M_{\text{left}} + \text{revs} \times FH_{\text{left}} \times h = -1606.1 \text{ kNm} \)
- To right \( MR = MR - FV_{\text{rt}} \times (l - x - x_1) - \text{revm} \times M_{\text{rt}} - \text{revs} \times FH_{\text{rt}} \times h = -1606.1 \text{ kNm} \)

---

**SCALE 5.48**

**Office 1007**

**Proforma 757**
Preliminary calculations for combined rectangular base

Distance between columns \( \text{dist}=7.5 \text{ m} \)
Thickmess of base \( h=1 \text{ m} \)
Left column: vertical load \( F_{V\text{left}}=580 \text{ kN} \)
  horizontal shear \( F_{H\text{left}}=150 \text{ kN} \)
  moment (clockwise) \( M_{\text{left}}=0 \text{ kNm} \)
Right column: vertical load \( F_{V\text{rt}}=400 \text{ kN} \)
  horizontal shear \( F_{H\text{rt}}=50 \text{ kN} \)
  moment (clockwise) \( M_{\text{rt}}=0 \text{ kNm} \)

No reversal of moments so moment-reversal multiplier \( \text{revm}=1 \) throughout.
Shear-reversal multiplier \( \text{revs} \) is set to 1 when shear forces act from left to right, and to -1 when shear forces act from right to left.
Allowable bearing pressure \( \text{press}=80 \text{ kN/m}^2 \)
Concrete density \( \text{den}=24 \text{ kN/m}^3 \)

Value of dimension \( x_0 \) \( x_0=1.11 \text{ m} \)
Value of dimension \( x_1 \) \( x_1=0.4 \text{ m} \)
Value of dimension \( x_2 \) \( x_2=1.78 \text{ m} \)
Length of base \( l=x_1+\text{dist}+x_2=9.68 \text{ m} \)
Width of base \( b=2\times x_0=2.22 \text{ m} \)
Area of base \( \text{area}=l\times b=21.49 \text{ m}^2 \)
Self-weight of base \( \text{Swt}=l\times b\times h\times \text{den}=515.75 \text{ kN} \)
Total vertical load on base \( V=F_{V\text{left}}+F_{V\text{rt}}+\text{Swt}=1495.8 \text{ kN} \)
Base centroid from left column \( x'=l/2-x_2=3.06 \text{ m} \)

With clockwise moments and left->right shears:

Taking moments about position of left column:
\[ \text{clock}=F_{V\text{rt}}\times \text{dist}+(\text{revs})\times (F_{H\text{left}}+F_{H\text{rt}})\times h+(\text{revm})\times (M_{\text{left}}+M_{\text{rt}})+\text{Swt}\times x' \]
\[ =4778.2 \text{ kNm} \]

Centroid of load from left col. \( x_{\text{bar}}=\text{clock}/V=3.1945 \text{ m} \)
Eccentricity of upward pressure \( e=x'-x_{\text{bar}}=-0.13451 \text{ m} \)

As \( e<1/6 \), distribution of pressure beneath base is trapezoidal

Pressure at left end of base \( P_{\text{lt}}=V\times(1+6\times e/l)/(b\times l)=63.8 \text{ kN/m}^2 \)
Pressure at right end of base \( P_{\text{rt}}=V\times(1-6\times e/l)/(b\times l)=75.407 \text{ kN/m}^2 \)
Dist. (m) | Shearing force (kN) | Bending moment (kNm)
from LHE | To left | To right | To left | To right
---|---|---|---|---
0 | 0 | 0 | 0 | 0
0.968 | 86.776 | -86.776 | 41.798 | 41.798
1.78 | 161.49 | 418.51 | 142.48 | 292.48
1.936 | -403.95 | 312.19 | -118.49 | -118.49
2.904 | -312.19 | 217.93 | -375.27 | -375.27
3.872 | -217.93 | 121.18 | -539.6 | -539.6
4.84 | -121.18 | 21.931 | -609.07 | -609.07
5.808 | -21.931 | 0 | -611.38 | -611.38
6.0187 | 0 | 0 | -611.38 | -611.38
7.744 | 184.05 | -184.05 | -453.75 | -453.75
8.712 | 354.56 | 45.436 | -40.899 | 9.1014
9.28 | 0 | 0 | 0 | 0
9.68 | 79.811 | -79.811 | -581.25 | -581.25

Dist. of point considered from left end of base x=4.84 m

Final shearing forces:
- To left: SL = SL - FVleft = -121.18 kN
- To right: SR = SR - FVright = 121.18 kN

Final bending moments:
- To left: ML = ML - FVleft \cdot (x - x_2) + rev_m \cdot M_{left} + rev_s \cdot FH_{left} \cdot h = -539.6 kNm
- To right: MR = MR - FVright \cdot (l - x - x_1) - rev_m \cdot M_{right} - rev_s \cdot FH_{right} \cdot h = -539.6 kNm

With clockwise moments and right->left shears:

Taking moments about position of left column:
\[ \text{clock} = FV_{right} \cdot \text{dist} + (revs) \cdot (FH_{left} + FH_{right}) \cdot h + (revm) \cdot (M_{left} + M_{right}) + Swt \cdot x' = 4378.2 \text{ kNm} \]

Centroid of load from left col. \( xbar = \text{clock}/V = 2.9271 \text{ m} \)
Eccentricity of upward pressure \( e = x' - xbar = 0.13291 \text{ m} \)

As \( e<1/6 \), distribution of pressure beneath base is trapezoidal

Pressure at left end of base \( P_{left} = V \cdot (1 + 6 \cdot e/l) / (b \cdot l) = 75.338 \text{ kN/m}^2 \)
Pressure at right end of base \( P_{right} = V \cdot (1 - 6 \cdot e/l) / (b \cdot l) = 63.869 \text{ kN/m}^2 \)
Dist (m) | Shearing force (kN) | Bending moment (kNm)
---|---|---
| from LHE | To left | To right | To left | To right |
0 | 0 | 0 | 0 | 0 |
0.968 | 109.09 | -109.09 | 52.998 | 52.998 |
1.78 | 198.7 | 381.3 | 178.08 | 28.078 |
1.936 | -364.28 | 364.28 | -30.077 | -30.077 |
2.904 | -260.12 | 260.12 | -332.09 | -332.09 |
3.872 | -158.43 | 158.43 | -534.47 | -534.47 |
4.84 | -59.194 | 59.194 | -639.6 | -639.6 |
5.4293 | 0 | 0 | -656.99 | -656.99 |
5.808 | 37.574 | -37.574 | -649.87 | -649.87 |
6.776 | 131.88 | -131.88 | -567.65 | -567.65 |
7.744 | 223.72 | -223.72 | -395.35 | -395.35 |
8.712 | 313.09 | -313.09 | -135.33 | -135.33 |
9.28 | 364.39 | 35.614 | 57.109 | 7.1089 |
9.68 | 0 | 0 | 0 | 0 |

Dist. of point considered from left end of base x=4.84 m

Final shearing forces:
- To left: \( SL = SL - FV_{left} = -59.194 \text{ kN} \)
- To right: \( SR = SR - FV_{rt} = 59.194 \text{ kN} \)

Final bending moments:
- To left: \( ML = ML - FV_{left}(x-x2) + revm*M_{left} + revs*FH_{left}h = -639.6 \text{ kNm} \)
- To right: \( MR = MR - FV_{rt}(l-x-x1) - revm*M_{rt} - revs*F_{Hrt}h = -639.6 \text{ kNm} \)
Location: Manual check of rectangular base with typical loading

Preliminary calculations for combined rectangular base

Distance between columns  \( \text{dist}=7.5 \text{ m} \)
Thickness of base  \( h=1 \text{ m} \)
Left column: vertical load  \( FV_{\text{left}}=1500 \text{ kN} \)
  horizontal shear  \( FH_{\text{left}}=400 \text{ kN} \)
  moment (clockwise)  \( M_{\text{left}}=50 \text{ kNm} \)
Right column: vertical load  \( FV_{\text{rt}}=400 \text{ kN} \)
  horizontal shear  \( FH_{\text{rt}}=400 \text{ kN} \)
  moment (clockwise)  \( M_{\text{rt}}=50 \text{ kNm} \)

No reversal of moments so moment-reversal multiplier \( \text{revm}=1 \) throughout.
Shear-reversal multiplier \( \text{revs} \) is set to 1 when shear forces act from left to right, and to -1 when shear forces act from right to left.
Allowable bearing pressure  \( \text{press}=300 \text{ kN/m}^2 \)
Concrete density  \( \text{den}=24 \text{ kN/m}^3 \)

\[
\begin{array}{c}
\text{Value of dimension } x_0 \quad x_0=1 \text{ m} \\
\text{Value of dimension } x_1 \quad x_1=1 \text{ m} \\
\text{Value of dimension } x_2 \quad x_2=2 \text{ m} \\
\text{Length of base} \quad l=x_1+\text{dist}+x_2=10.5 \text{ m} \\
\text{Width of base} \quad b=2\times x_0=2 \text{ m} \\
\text{Area of base} \quad \text{area}=l\times b=21 \text{ m}^2 \\
\text{Self-weight of base} \quad \text{Swt}=l\times b\times h\times \text{den}=504 \text{ kN} \\
\text{Total vertical load on base} \quad V=FV_{\text{left}}+FV_{\text{rt}}+\text{Swt}=2404 \text{ kN} \\
\text{Base centroid from left column} \quad x'=l/2-x_2=3.25 \text{ m} \\
\end{array}
\]

With clockwise moments and left->right shears:

Taking moments about position of left column:
\[
\text{clock}=FV_{\text{rt}}\times \text{dist}+(\text{revs})\times (FH_{\text{left}}+FH_{\text{rt}})\times h+(\text{revm})\times (M_{\text{left}}+M_{\text{rt}})+\text{Swt}\times x' =5538 \text{ kNm}
\]
Centroid of load from left col.  \( x_{\text{bar}}=\text{clock}/V=2.3037 \text{ m} \)
Eccentricity of upward pressure  \( e=x'-x_{\text{bar}}=0.94634 \text{ m} \)

As \( e<1/6 \), distribution of pressure beneath base is trapezoidal

Pressure at left end of base  \( P_{\text{lt}}=V\times (1+6\times e/l)/(b\times l)=176.38 \text{ kN/m}^2 \)
Pressure at right end of base  \( P_{\text{rt}}=V\times (1-6\times e/l)/(b\times l)=52.571 \text{ kN/m}^2 \)
Sample output for SCALE Proforma 758. (ans=2)  Page: 2
Loadings and foundations  Made by: IFB
Preliminary sizing of rectangular combined base  Date: 02/12/19
Copyright 1986-2019 Fitzroy Systems Ltd.  Ref No: SC758

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<th>Shearing force (kN)</th>
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Dist. of point considered from left end of base x=5.25 m

Final shearing forces:
- To left  SL=SL-FVleft=-225 kN
- To right SR=SR-FVrt=225 kN

Final bending moments:
- To left  ML=ML-FVleft*(x-x2)+revm*Mleft+revs*FHleft*h=-793.75 kNm
- To right MR=MR-FVrt*(l-x-x1)-revm*Mrt-revs*FHrt*h=-793.75 kNm

With clockwise moments and right->left shears:

Taking moments about position of left column:
- clock=FVrt*dist+(revs)*(FHleft+FHrt)*h+(revm)*(Mleft+Mrt)+Swt*x'  
  =3938 kNm
- Centroid of load from left col.  xbar=clock/V=1.6381 m
- Eccentricity of upward pressure  e=x'-xbar=1.6119 m

As e<1/6, distribution of pressure beneath base is trapezoidal

Pressure at left end of base  PLt=V*(1+6*e/l)/(b*l)=219.92 kN/m²
Pressure at right end of base  Prt=V*(1-6*e/l)/(b*l)=9.034 kN/m²
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<th>Dist (m) from LHE</th>
<th>Shearing force (kN) To left</th>
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Dist. of point considered from left end of base x=5.25 m

Final shearing forces:
- To left: $SL = SL - FV_{left} = 3.5714$ kN
- To right: $SR = SR - FV_{right} = -3.5714$ kN

Final bending moments:
- To left: $ML = ML - FV_{left}*(x-x2) + revm*M_{left} + revs*FH_{left} = -793.75$ kNm
- To right: $MR = MR - FV_{right}*(l-x-x1) - revm*M_{right} - revs*FH_{right} = -793.75$ kNm
Location: Manual check of rectangular base w. uplift on right column

Preliminary calculations for combined rectangular base

Distance between columns dist=7.5 m
Thickness of base h=1 m
Left column: vertical load FVleft=1000 kN
  horizontal shear FHleft=0 kN
  moment (clockwise) Mleft=0 kNm
Right column: vertical load FVrt=-150 kN
  horizontal shear FHrt=0 kN
  moment (clockwise) Mrt=0 kNm
No reversal of moments so moment-reversal multiplier revm=1 throughout.
No reversal of shears, so shear-reversal multiplier revs=1 throughout.
Allowable bearing pressure press=200 kN/m²
Concrete density den=24 kN/m³

RECTANGULAR BASE POSITIONED ECCENTRICALLY

Value of dimension x0 x0=1 m
Value of dimension x1 x1=1 m
Value of dimension x2 x2=2 m
Length of base l=x1+dist+x2=10.5 m
Width of base b=2*x0=2 m
Area of base area=l*b*h*den=504 kN
Self-weight of base Swt=l*b*h*den=504 kN
Total vertical load on base V=FVleft+FVrt+Swt=1354 kN
Base centroid from left column x'=l/2-x2=3.25 m
Taking moments about position of left column:
  clock=FVrt*dist+(revs)*(FHleft+FHrt)*h+(revm)*(Mleft+Mrt)+Swt*x'
  =513 kNm
Centroid of load from left col. xbar=3*(1/2-e)
Eccentricity of upward pressure e=x'-xbar=2.8711 m

As e>=1/6, distribution of pressure beneath base is triangular

***WARNING: Uplift occurs beneath base ***
Point of zero upward pressure xz=3*(1/2-e)
  =7.1366 m from left end of base.
Pressure at left end of base Plt=V/(x0*xz)=189.73 kN/m²
### Shearing Forces and Bending Moments

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<tr>
<th>Dist (m) from LHE</th>
<th>Shearing force (kN) To left</th>
<th>Shearing force (kN) To right</th>
<th>Bending moment (kN) To left</th>
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**Dist. of point considered from left end of base x=5.25 m**

**Final shearing forces:**
- To left: \( SL = SL - FV_{\text{left}} = 7.3749 \text{ kN} \)
- To right: \( SR = SR - FV_{\text{rt}} = -7.3749 \text{ kN} \)

**Final bending moments:**
- To left: \( ML = ML - FV_{\text{left}}(x-x_2) + revm * M_{\text{left}} + revs * FH_{\text{left}} * h = 35.508 \text{ kNm} \)
- To right: \( MR = MR - FV_{\text{rt}}(l-x-x_1) - revm * M_{\text{rt}} - revs * FH_{\text{rt}} * h = 35.508 \text{ kNm} \)
Location: Optimum calculation for missing dimension for Example 2

Preliminary calculations for combined rectangular base

Distance between columns \( \text{dist}=7.5 \text{ m} \)
Thickness of base \( h=1 \text{ m} \)
Left column: vertical load \( F_{V_{\text{left}}} = 1500 \text{ kN} \)
  horizontal shear \( F_{H_{\text{left}}} = 0 \text{ kN} \)
  moment (clockwise) \( M_{\text{left}} = 0 \text{ kNm} \)
Right column: vertical load \( F_{V_{\text{rt}}} = 400 \text{ kN} \)
  horizontal shear \( F_{H_{\text{rt}}} = 0 \text{ kN} \)
  moment (clockwise) \( M_{\text{rt}} = 0 \text{ kNm} \)

No reversal of moments so moment-reversal multiplier \( \text{revm}=1 \) throughout.
No reversal of shears, so shear-reversal multiplier \( \text{revs}=1 \) throughout.
Allowable bearing pressure \( \text{press}=100 \text{ kN/m}^2 \)
Concrete density \( \text{den}=24 \text{ kN/m}^3 \)

\[ \begin{align*}
\text{RECTANGULAR} & \quad \begin{array}{c}
\text{BASE POSITIONED} \\
\text{ECCENTRICALLY}
\end{array} \\
\text{x2} & \quad \text{dist} & \quad \text{x1}
\end{align*} \]

Value of dimension \( x_0 = 1 \text{ m} \)
Value of dimension \( x_1 = 1 \text{ m} \)
Value of dimension \( x_2 = 5.35 \text{ m} \)
Length of base \( l=x_1+\text{dist}+x_2=13.85 \text{ m} \)
Width of base \( b=2\times x_0=2 \text{ m} \)
Area of base \( \text{area}=l\times b=27.7 \text{ m}^2 \)
Self-weight of base \( \text{Swt}=l\times b\times h\times \text{den}=664.8 \text{ kN} \)
Total vertical load on base \( V=F_{V_{\text{left}}}+F_{V_{\text{rt}}}+\text{Swt}=2564.8 \text{ kN} \)
Base centroid from left column \( x'_{\text{left}}=\frac{l}{2}-x_2=1.575 \text{ m} \)
Taking moments about position of left column:
  \[ \text{clock}=F_{V_{\text{rt}}}\times \text{dist}+(\text{revs})\times(F_{H_{\text{left}}}+F_{H_{\text{rt}}})\times h+(\text{revm})\times (M_{\text{left}}+M_{\text{rt}})+\text{Swt}\times x'_{\text{left}} = 4047.1 \text{ kNm} \]
Centroid of load from left col. \( x_{\text{bar}}=\text{clock}/V=1.5779 \text{ m} \)
Eccentricity of upward pressure \( e=x'_{\text{left}}-x_{\text{bar}}=-0.0029242 \text{ m} \)

As \( e<1/6 \), distribution of pressure beneath base is trapezoidal

Pressure at left end of base \( \text{Plt}=V\times(1+6\times e/l)/(b\times l)=92.475 \text{ kN/m}^2 \)
Pressure at right end of base \( \text{Prt}=V\times(1-6\times e/l)/(b\times l)=92.709 \text{ kN/m}^2 \)
## Loadings and foundations

### Preliminary sizing of rectangular combined base

<table>
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<tr>
<th>Dist (m) from LHE</th>
<th>Shearing force (kN) To left</th>
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<th>Bending moment (kNm) To left</th>
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Dist. of point considered from left end of base x=6.925 m

**Final shearing forces:**

- To left: \( SL = SL - FV_{left} = -550.81 \) kN
- To right: \( SR = SR - FV_{rt} = 550.81 \) kN

**Final bending moments:**

- To left: \( ML = ML - FV_{left} \times (x-x^2) + revm \times M_{left} + revs \times FH_{left} \times h = 923.13 \) kNm
- To right: \( MR = MR - FV_{rt} \times (l-x-x_1) - revm \times M_{rt} - revs \times FH_{rt} \times h = 923.13 \) kNm
Location: Heavy load on left base: no moment or shear reversal

If pairs of bases supporting a trestle, gantry or similar structure are subjected to moments and horizontal shearing forces acting in the same sense on each base simultaneously, by linking the pairs of bases by tie beams the bases are relieved of the effects resulting from eccentricity and may be then designed as concentrically loaded. In such a case the size, flexibility and arrangement of the tie beam must be such that it does not provide load support itself nor does it transmit load from one base to the other. To reduce the risk of differential settlement, the bases must also be so dimensioned that the bearing pressure beneath each is similar under all possible loading conditions. If the applied moments and/or the horizontal shearing forces can reverse in direction, remember to check these situations also.

Distance between column centres: dist=5.5 m
Concrete density: den=24 kN/m³
Width: bleft=4 m
Length: lleft=2 m
Depth: hleft=0.8 m
Vertical applied load: FVleft=4500 kN
Horizontal load (left to right): FHleft=100 kN
Applied moment (clockwise): Mleft=50 kNm
Width: bright=4 m
Length: lright=2 m
Depth: hright=0.8 m
Vertical applied load: FVright=1000 kN
Horizontal load (left to right): FHright=100 kN
Applied moment (clockwise): Mright=50 kNm

No reversal of moments or shearing forces

Moment at centreline of left base: BMleft=-(Mleft+FHleft*hleft)=-130 kNm
Moment at centreline of right base: BMright=Mright+FHright*hright=130 kNm
BM at left end of connecting beam: Mlend=BMleft-(BMleft-BMright)*lleft/(2*dist) =-82.727 kNm
BM at right end of connecting beam: Mrend=BMright+(BMleft-BMright)*lright/(2*dist) =82.727 kNm
Shear force on connecting beam: Sbeam=(-BMleft+BMright)/dist =47.273 kN
**Location:** Same as Example No.1 but with moment and shear reversal

If pairs of bases supporting a trestle, gantry or similar structure are subjected to moments and horizontal shearing forces acting in the same sense on each base simultaneously, by linking the pairs of bases by tie beams the bases are relieved of the effects resulting from eccentricity and may be then designed as concentrically loaded. In such a case the size, flexibility and arrangement of the tie beam must be such that it does not provide load support itself nor does it transmit load from one base to the other. To reduce the risk of differential settlement, the bases must also be so dimensioned that the bearing pressure beneath each is similar under all possible loading conditions. If the applied moments and/or the horizontal shearing forces can reverse in direction, remember to check these situations also.

Distance between column centres \( \text{dist}=5.5 \text{ m} \)
Concrete density \( \text{den}=24 \text{ kN/m}^3 \)
Width \( b_{\text{left}}=4 \text{ m} \)
Length \( l_{\text{left}}=2 \text{ m} \)
Depth \( h_{\text{left}}=0.8 \text{ m} \)
Vertical applied load \( F_{V_{\text{left}}}=4500 \text{ kN} \)
Horizontal load (left to right) \( F_{H_{\text{left}}}=100 \text{ kN} \)
Applied moment (clockwise) \( M_{\text{left}}=50 \text{ kNm} \)
Width \( b_{\text{right}}=4 \text{ m} \)
Length \( l_{\text{right}}=2 \text{ m} \)
Depth \( h_{\text{right}}=0.8 \text{ m} \)
Vertical applied load \( F_{V_{\text{right}}}=1000 \text{ kN} \)
Horizontal load (left to right) \( F_{H_{\text{right}}}=100 \text{ kN} \)
Applied moment (clockwise) \( M_{\text{right}}=50 \text{ kNm} \)

**No reversal of moments or shearing forces**

Moment at centreline of left base \( B_{M_{\text{left}}}=-\left(M_{\text{left}}+F_{H_{\text{left}}}h_{\text{left}}\right)=-130 \text{ kNm} \)
Moment at centreline of right base \( B_{M_{\text{right}}}=M_{\text{right}}+F_{H_{\text{right}}}h_{\text{right}}=130 \text{ kNm} \)
BM at left end of connecting beam:
\[
M_{\text{left}} = B_{M_{\text{left}}} - (B_{M_{\text{left}}}-B_{M_{\text{right}}})l_{\text{left}}/(2\text{*dist})
\]
\[
= -82.727 \text{ kNm}
\]
BM at right end of connecting beam:
\[
M_{\text{right}} = B_{M_{\text{right}}} + (B_{M_{\text{left}}}-B_{M_{\text{right}}})l_{\text{right}}/(2\text{*dist})
\]
\[
= 82.727 \text{ kNm}
\]
Shear force on connecting beam \( S_{\text{beam}}=(B_{M_{\text{left}}}+B_{M_{\text{right}}})/\text{dist} \)
\[
= 47.273 \text{ kN}
\]
Location: Uplift on right base: moment reversal only

If pairs of bases supporting a trestle, gantry or similar structure are subjected to moments and horizontal shearing forces acting in the same sense on each base simultaneously, by linking the pairs of bases by tie beams the bases are relieved of the effects resulting from eccentricity and may be then designed as concentrically loaded. In such a case the size, flexibility and arrangement of the tie beam must be such that it does not provide load support itself nor does it transmit load from one base to the other. To reduce the risk of differential settlement, the bases must also be so dimensioned that the bearing pressure beneath each is similar under all possible loading conditions. If the applied moments and/or the horizontal shearing forces can reverse in direction, remember to check these situations also.

Distance between column centres \( \text{dist} = 5.5 \text{ m} \)
Concrete density \( \text{den}=24 \text{ kN/m}^3 \)
Width \( b_{\text{left}}=4 \text{ m} \)
Length \( l_{\text{left}}=2 \text{ m} \)
Depth \( h_{\text{left}}=0.8 \text{ m} \)
Vertical applied load \( F_{V_{\text{left}}}=4500 \text{ kN} \)
Horizontal load (left to right) \( F_{H_{\text{left}}}=100 \text{ kN} \)
Applied moment (clockwise) \( M_{\text{left}}=50 \text{ kNm} \)
Width \( b_{\text{right}}=4 \text{ m} \)
Length \( l_{\text{right}}=2 \text{ m} \)
Depth \( h_{\text{right}}=0.8 \text{ m} \)
Vertical applied load \( F_{V_{\text{right}}}=-100 \text{ kN} \)
Horizontal load (left to right) \( F_{H_{\text{right}}}=100 \text{ kN} \)
Applied moment (clockwise) \( M_{\text{right}}=50 \text{ kNm} \)

**No reversal of moments or shearing forces**

Moment at centreline of left base \( B_{M_{\text{left}}} = -(M_{\text{left}} + F_{H_{\text{left}}} \times h_{\text{left}}) = -130 \text{ kNm} \)
Moment at centreline of right base \( B_{M_{\text{right}}} = M_{\text{right}} + F_{H_{\text{right}}} \times h_{\text{right}} = 130 \text{ kNm} \)
BM at left end of connecting beam:
\[
M_{\text{lend}} = B_{M_{\text{left}}} - (B_{M_{\text{left}}} - B_{M_{\text{right}}}) \times l_{\text{left}} / (2 \times \text{dist})
\]
\[
= -82.727 \text{ kNm}
\]
BM at right end of connecting beam:
\[
M_{\text{rend}} = B_{M_{\text{right}}} + (B_{M_{\text{left}}} - B_{M_{\text{right}}}) \times l_{\text{right}} / (2 \times \text{dist})
\]
\[
= 82.727 \text{ kNm}
\]
Shear force on connecting beam \( S_{\text{beam}} = (-B_{M_{\text{left}}} + B_{M_{\text{right}}}) / \text{dist} \)
\[
= 47.273 \text{ kN}
\]
Analysis of combined footing

Tapered base of constant thickness supporting two point loads L1 and L2.

The point loads are on the centre-line of base.

Distance to load 1: a = 0.457 m
Distance to load 2: b = 4.72 m
Length of base: c = 5.13 m
Width of base at left end: d = 2.688 m
Width of base at right end: e = 1.295 m
Thickness of base: f = 0.9 m
Load L1: L1 = 1690 kN
Load L2: L2 = 1245 kN

Ground pressures

Average pressure: AP = TL/ta = 308.88 kN/m²
Eccentric pressure left end: PL = TL*(eb-EL)/zl = 0.20115 kN/m²
Eccentric pressure right end: PR = TL*(EL-eb)/zr = -0.25423 kN/m²
Maximum pressure at left end: SL = AP + PL = 309.08 kN/m²
Maximum pressure at right end: SR = AP + PR = 308.63 kN/m²
Maximum pressure (excluding self-weight) at left end: LS = SL - f*24 = 287.48 kN/m²
Maximum pressure (excluding self-weight) at right end: RS = SR - f*24 = 287.03 kN/m²
Distance right end to zero shear: x = SX = 2.65 m

Bending moments

For moment at load L1, with and
D = b - a = 4.263 m
B1 = e*RS*l^2/2 = 4058.4 kNm
B2 = (e*(LS-RS) + RS*(d-e)) * l^3 / (6*c)
   = 1327.5 kNm
B3 = (d-e) * (LS-RS) * l^4 / (12*c^2) - L2*D
   = -5306.5 kNm
Then bending moment at Load L1: BM = B1 + B2 + B3 = 79.449 kNm

For moment at load L2, with and
D = 0
B1 = e*RS*l^2/2 = 31.242 kNm
B2 = (e*(LS-RS) + RS*(d-e)) * l^3 / (6*c)
   = 0.8966 kNm

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Then bending moment at Load L2

\[ B3 = (d-e)(LS-RS) \frac{l^4}{(12c^2)} - L2D \]

\[ = 56.76 \times 10^{-6} \text{ kNm} \]

Then maximum moment in span

\[ BM = B1 + B2 + B3 = 32.138 \text{ kNm} \]

For maximum moment in span, with

\[ l = x = 2.65 \text{ m} \]

\[ D = x + b - c = 2.24 \text{ m} \]

\[ B1 = eRS \frac{l^2}{2} = 1305.1 \text{ kNm} \]

\[ B2 = e(LS-RS) + RS(d-e) \frac{l^3}{6c} = 242.1 \text{ kNm} \]

\[ B3 = (d-e)(LS-RS) \frac{l^4}{(12c^2)} - L2D = -2788.7 \text{ kNm} \]

Then maximum moment in span

\[ BM = B1 + B2 + B3 = -1241.5 \text{ kNm} \]

**Base widths**

Width of base at load L1

\[ D1 = e + (c-a)(d-e) / c = 2.5639 \text{ m} \]

Width of base at load L2

\[ D2 = e + (c-b)(d-e) / c = 1.4063 \text{ m} \]

Width of base at max. span moment

\[ D3 = e + x(d-e) / c = 2.0146 \text{ m} \]
**Location:** Large, heavily-loaded trapezoidal base

**Analysis of combined footing**

Tapered base of constant thickness supporting two point loads $L_1$ and $L_2$.

The point loads are on the centre-line of base.

Distance to load 1 $a=0.3$ m  
Distance to load 2 $b=9.8$ m  
Length of base $c=10$ m  
Width of base at left end $d=5$ m  
Width of base at right end $e=2$ m  
Thickness of base $f=1.5$ m  
Load $L_1$ $L_1=3000$ kN  
Load $L_2$ $L_2=1500$ kN

**Ground pressures**

Average pressure $AP=TL/ta=164.57$ kN/m²  
Eccentric pressure left end $PL=TL)*(eb-EL)/z1=57.689$ kN/m²  
Eccentric pressure right end $PR=TL*(EL-eb)/zr=-76.919$ kN/m²  
Maximum pressure at left end $SL=AP+PL=222.26$ kN/m²  
Maximum pressure at right end $SR=AP+PR=87.652$ kN/m²  
Maximum pressure (excluding self-weight) at left end $LS=SL-f*24=186.26$ kN/m²  
Maximum pressure (excluding self-weight) at right end $RS=SR-f*24=51.652$ kN/m²  
Distance right end to zero shear $x=SX=5.65$ m

**Bending moments**

For moment at load $L_1$, with $l=c-a=9.7$ m and $D=b-a=9.5$ m  
$B_1=e*RS*1^2/2=4860$ kNm  
$B_2=(e*(LS-RS)+RS*(d-e))*1^3/(6*c)$  
$=6452.2$ kNm  
$B_3=(d-e)*(LS-RS)*1^4/(12*c^2)-L_2*D$  
$=-11271$ kNm  
Then bending moment at Load $L_1$ $BM=B_1+B_2+B_3=41.357$ kNm

For moment at load $L_2$, with $l=c-b=0.2$ m and $D=0$  
$B_1=e*RS*1^2/2=2.0661$ kNm  
$B_2=(e*(LS-RS)+RS*(d-e))*1^3/(6*c)$  
$=0.056557$ kNm
B3 = (d-e) * (LS-RS) * l^4 / (12 * c^2) - L2 * D
= 0.53843E-3 kNm

Then bending moment at Load L2
BM = B1 + B2 + B3 = 2.1232 kNm

For maximum moment in span, with
l = x = 5.65 m
D = x + b - c = 5.45 m
B1 = e * RS * l^2 / 2 = 1648.9 kNm
B2 = (e * (LS-RS) + RS * (d-e)) * l^3 / (6 * c)
= 1275.1 kNm
B3 = (d-e) * (LS-RS) * l^4 / (12 * c^2) - L2 * D
= -7832.1 kNm

Then maximum moment in span
BM = B1 + B2 + B3 = -4908.1 kNm

**Base widths**

Width of base at load L1
D1 = e + (c-a) * (d-e) / c = 4.91 m

Width of base at load L2
D2 = e + (c-b) * (d-e) / c = 2.06 m

Width of base at max. span moment
D3 = e + x * (d-e) / c = 3.695 m
Location: Small, rectangular base

Analysis of combined footing

Tapered base of constant thickness supporting two Point loads L1 and L2. The point loads are on the centre-line of base.

Distance to load 1: a = 0.5 m
Distance to load 2: b = 2.5 m
Length of base: c = 3 m
Width of base at left end: d = 1.5 m
Width of base at right end: e = 1.5 m
Thickness of base: f = 0.5 m
Load L1: L1 = 1500 kN
Load L2: L2 = 1000 kN

Ground pressures

Average pressure: \( AP = \frac{TL}{ta} = 567.56 \text{ kN/m}^2 \)
Eccentric pressure left end: \( PL = \frac{TL*(eb-EL)}{zl} = 222.22 \text{ kN/m}^2 \)
Eccentric pressure right end: \( PR = \frac{TL*(EL-eb)}{zr} = -222.22 \text{ kN/m}^2 \)
Maximum pressure at left end: \( SL = AP + PL = 789.78 \text{ kN/m}^2 \)
Maximum pressure at right end: \( SR = AP + PR = 345.33 \text{ kN/m}^2 \)
Maximum pressure (excluding self-weight) at left end: \( LS = SL - f*24 = 777.78 \text{ kN/m}^2 \)
Maximum pressure (excluding self-weight) at right end: \( RS = SR - f*24 = 333.33 \text{ kN/m}^2 \)
Distance right end to zero shear: \( x = SX = 1.5 \text{ m} \)

Bending moments

For moment at load L1, with l = c-a=2.5 m and
\[
D = b-a = 2 \text{ m}
\]
\[
B1 = e*RS*1^2/2 = 1562.5 \text{ kNm}
\]
\[
B2 = (e*(LS-RS)+RS*(d-e))*1^3/(6*c)
\]
\[
= 578.7 \text{ kNm}
\]
\[
B3 = (d-e)*(LS-RS)*1^4/(12*c^2)-L2*D
\]
\[
= -2000 \text{ kNm}
\]
Then bending moment at Load L1: \( BM = B1 + B2 + B3 = 141.2 \text{ kNm} \)

For moment at load L2, with l = c-b=0.5 m and
\[
D = 0
\]
\[
B1 = e*RS*1^2/2 = 62.5 \text{ kNm}
\]
\[
B2 = (e*(LS-RS)+RS*(d-e))*1^3/(6*c)
\]
\[
= 4.6296 \text{ kNm}
\]

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B3 = (d-e) * (LS-RS) * l^4 / (12 * c^2) - L2 * D
= 0 kNm

Then bending moment at Load L2
BM = B1 + B2 + B3 = 67.13 kNm

For maximum moment in span, with
l = x = 1.5 m
D = x + b - c = 1 m
B1 = e * RS * l^2 / 2 = 562.5 kNm
B2 = (e * (LS-RS) + RS * (d-e)) * l^3 / (6 * c) = 125 kNm
B3 = (d-e) * (LS-RS) * l^4 / (12 * c^2) - L2 * D
= -1000 kNm

Then maximum moment in span
BM = B1 + B2 + B3 = -312.5 kNm

**Base widths**

Width of base at load L1
D1 = e + (c-a) * (d-e) / c = 1.5 m

Width of base at load L2
D2 = e + (c-b) * (d-e) / c = 1.5 m

Width of base at max. span moment
D3 = e + x * (d-e) / c = 1.5 m
Location: Example 17.5 from Design of Reinforced Concrete Buildings

Length of base \( L = 20 \text{ m} \)
Width of base \( B = 2 \text{ m} \)
Number of point loads \( n = 5 \)
Point load 1 \( W_1 = 500 \text{ kN} \)
Distance from left end of found \( x_1 = 2 \text{ m} \)
Point load 2 \( W_2 = 450 \text{ kN} \)
Distance from left end of found \( x_2 = 5 \text{ m} \)
Point load 3 \( W_3 = 400 \text{ kN} \)
Distance from left end of found \( x_3 = 8.5 \text{ m} \)
Point load 4 \( W_4 = 350 \text{ kN} \)
Distance from left end of found \( x_4 = 12.5 \text{ m} \)
Point load 5 \( W_5 = 300 \text{ kN} \)
Distance from left end of found \( x_5 = 18 \text{ m} \)

**SUMMARY**

<table>
<thead>
<tr>
<th>LOAD REF. NO.</th>
<th>MAGNITUDE TO LOAD</th>
<th>DISTANCE TO LOAD</th>
<th>GROUND PRESSURE</th>
<th>SHEAR TO LEFT</th>
<th>SHEAR TO RIGHT</th>
<th>BENDING MOMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>500 kN</td>
<td>2 m</td>
<td>71.45 kN/m²</td>
<td>296.52 kN</td>
<td>-203.48 kN</td>
<td>300.1 kNm</td>
</tr>
<tr>
<td>2</td>
<td>450 kN</td>
<td>5 m</td>
<td>63.406 kN/m²</td>
<td>201.09 kN</td>
<td>-248.91 kN</td>
<td>308.59 kNm</td>
</tr>
<tr>
<td>3</td>
<td>400 kN</td>
<td>8.5 m</td>
<td>54.022 kN/m²</td>
<td>162.09 kN</td>
<td>-237.91 kN</td>
<td>175.83 kNm</td>
</tr>
<tr>
<td>4</td>
<td>350 kN</td>
<td>12.5 m</td>
<td>43.297 kN/m²</td>
<td>151.37 kN</td>
<td>-198.63 kN</td>
<td>31.348 kNm</td>
</tr>
<tr>
<td>5</td>
<td>300 kN</td>
<td>18 m</td>
<td>28.55 kN/m²</td>
<td>196.53 kN</td>
<td>-103.47 kN</td>
<td>99.9 kNm</td>
</tr>
</tbody>
</table>
Location: Beam on grid lines A1-A5

\[ x \text{ varies from } \text{com to ext} \quad s(2) \quad n=\text{number of load points} \quad n'\text{th load} \]

---

Overall beam length \( L = 11 \text{ m} \)
No. of load points in load train \( n = 3 \)
Initial distance to first load \( \text{com} = 1.5 \text{ m} \)
Final distance to first load \( \text{ext} = 1.5 \text{ m} \)
Spacing of increments \( \text{spc} = 2 \text{ m} \)
Number of sections considered \( \text{nx} = 1 \)
Beam width \( B = 1 \text{ m} \)
Beam depth \( D = 0.5 \text{ m} \)
Elastic modulus for beam \( E = 14E6 \text{ kN/m}^2 \)
Modulus of subgrade reaction \( k' = 10000 \text{ kN/m}^3 \)

Distances from left load to each load in order, \( s(1) \) to \( s(3) \)
First load is at zero distance \( s(1) = 0 \text{ m} \)
\( s(2) = 4 \text{ m} \)
\( s(3) = 8 \text{ m} \)

Load magnitudes – from left hand end – in order, \( p(1) \) to \( p(3) \)
\( p(1) = 500 \text{ kN} \)
\( p(2) = 500 \text{ kN} \)
\( p(3) = 500 \text{ kN} \)

Positions of sections of interest, measured from left hand end, at which deflections etc. required – in order, \( ax(1) \) to \( ax(1) \)
\( ax(1) = 5.5 \text{ m} \)
<table>
<thead>
<tr>
<th>Distance to 1st load (m)</th>
<th>Distance to section (m)</th>
<th>Deflection down (m)</th>
<th>Rotation (degrees)</th>
<th>Moment (kNm)</th>
<th>Shear (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>5.5</td>
<td>0.01294</td>
<td>0</td>
<td>132.8</td>
<td>250</td>
</tr>
</tbody>
</table>

- Maximum deflection: 0.01294 m
- Corresponding soil pressure: 129.4 kN/m²
**Location: Beam on elastic foundation, after Roark**

- Beam depth: $h = 0.5$ m
- Beam width: $b = 1$ m
- Modulus of elasticity of beam: $E = 9 \times 10^6$ kN/m²
- Point load: $P = 2000$ kN
- Distance between reference points: $x' = 1$ m
- Modulus of subgrade reaction: $k_0 = 4000$ kN/m³
Location: Example 1

Active thrust on wall - cohesionless soil with horizontal surface

Height of wall $h=5\text{ m}$
Depth to water table $d=2\text{ m}$
Surcharge $q=0\text{ kN/m}^2$
Shearing resist. angle (degrees) $\phi'=35^\circ$
Dry unit weight $\gamma_d=17\text{ kN/m}^3$
Saturated unit weight $\gamma_{sm}=20\text{ kN/m}^3$
Unit weight of water $\gamma_w=9.8\text{ kN/m}^3$
Location: Example 2

Active thrust on wall - cohesionless soil with horizontal surface

![Diagram of wall and surcharge with labels for height h, surcharge, active thrust, and surcharge area.](image)

Height of wall \( h = 5 \text{ m} \)
Surcharge \( q = 0 \text{ kN/m}^2 \)
Unit weight \( \gamma = 18 \text{ kN/m}^3 \)
Shearing resist. angle (degrees) \( \phi' = 35^\circ \)
Location: Example 1

Active and passive pressures on sheet pile wall - with two soils

Unsupported height of wall \( h = 4.5 \) m
Surcharge pressure \( q = 50 \) kN/m²

Design data for soil 1
- Depth of soil \( d_1 = 6 \) m
- Cohesive strength \( c_1' = 0 \) kN/m²
- Dry unit weight \( \gamma_1 = 18 \) kN/m³
- Shearing resist. angle (degrees) \( \phi_1' = 38° \)

Design data for soil 2
- Depth of soil \( d_2 = 3 \) m
- Cohesive strength of soil \( c_2' = 10 \) kN/m²
- Saturated unit weight soil \( \gamma_2 = 20 \) kN/m³
- Shearing resist. angle (degrees) \( \phi_2' = 28° \)
- Unit weight of water \( \gamma_w = 9.8 \) kN/m³

Active and passive pressures
- Active pressure top of soil 1 \( p_{at1} = K_{a1} q - 2 c_1' \sqrt{K_{a1}} = 11.894 \) kN/m²
- Active pressure bot of soil 1 \( p_{ab1} = p_{at1} + K_{a1} \gamma_1 d_1 = 37.586 \) kN/m²
- Active pressure top of soil 2 \( p_{at2} = K_{a2} (q + \gamma_1 d_1) - 2 c_2' \sqrt{K_{a2}} = 45.026 \) kN/m²
- Active pressure bot of soil 2 \( p_{ab2} = p_{at2} + K_{a2} (\gamma_2 - \gamma_w) d_2 = 56.074 \) kN/m²
- Passive pressure top of soil 1 \( p_{pt1} = 0 \) kN/m²
- Passive pressure bot of soil 1 \( p_{pb1} = K_{p1} \gamma_1 (d_1 - h) = 113.5 \) kN/m²
- Passive pressure top of soil 2 \( p_{pt2} = K_{p2} \gamma_1 (d_1 - h) + 2 c_2' \sqrt{K_{p2}} = 108.07 \) kN/m²
- Passive pressure bot of soil 2 \( p_{pb2} = p_{pt2} + K_{p2} (\gamma_2 - \gamma_w) d_2 = 192.83 \) kN/m²
Location: Grid B

Active/passive wall pressures (Rankine) retaining soil slope

Design assumptions:
- water table below wall
- Cohesion $c'=0$

Height of wall $H=6$ m
Unit weight of soil $\gamma=18$ kN/m$^3$
Soil slope in degrees $\beta=20^\circ$
Soil friction angle in degrees $\phi=40^\circ$
Total active thrust on wall $P_a=0.5*\gamma H^2 \cos(\beta')$

$=81.135$ kN/m
Location: Example from Capper & Cassie

Slope Stability

Factor of safety of soil slope using Taylor's stability numbers method.

Height of bank: H=12 m
Slope of soil (degrees): i=45°
Unit weight of soil: \( \gamma = 20.4 \, \text{kN/m}^3 \)
Soil cohesion: \( c = 28.5 \, \text{kN/m}^2 \)
Soil angle of friction: \( \phi = 20° \)
Factor of Safety (cohesion): \( F_c = c/(N \cdot \gamma \cdot H) = 1.8191 \)

The true factor of safety will be somewhat less than this as cohesion alone was considered.
Selected reduced F.O.S.: \( F_s = 1.5 \)
The actual F.O.S. must lie between these values (1.5 and 1.313).
Further iteration gives:
Factor of Safety will be around 1.37
Location: Capper & Cassie Example in Fig 90

Slope stability - the Swedish method

Coordinates of boundary line 1 of 5

X value LHE of boundary line $x_a(1)=-5000$ m
Y value LHE of boundary line $y_a(1)=-11$ m
X value RHE of boundary line $x_b(1)=-26.7$ m
Y value RHE of boundary line $y_b(1)=-11$ m

Material properties

Density of material below line $w(1)=18.536$ kN/m\(^3\)
Angle of friction of soil above $f_a(1)=0^\circ$
Cohesion of soil above the line $c_a(1)=0$ kN/m\(^2\)
Angle of friction of soil below $f_b(1)=12^\circ$
Cohesion of soil below the line $c_b(1)=21.546$ kN/m\(^2\)

Coordinates of boundary line 2 of 5

X value LHE of boundary line $x_a(2)=-26.7$ m
Y value LHE of boundary line $y_a(2)=-11$ m
X value RHE of boundary line $x_b(2)=-14.356$ m
Y value RHE of boundary line $y_b(2)=-5.967$ m

Material properties

Density of material below line $w(2)=18.536$ kN/m\(^3\)
Angle of friction of soil above $f_a(2)=0^\circ$
Cohesion of soil above the line $c_a(2)=0$ kN/m\(^2\)
Angle of friction of soil below $f_b(2)=12^\circ$
Cohesion of soil below the line $c_b(2)=21.546$ kN/m\(^2\)
Coordinates of boundary line 3 of 5

X value LHE of boundary line      xa(3)=-14.356 m
Y value LHE of boundary line      ya(3)=-5.967 m
X value RHE of boundary line      xb(3)=0 m
Y value RHE of boundary line      yb(3)=0 m

Material properties

Density of material below line    w(3)=17.28 kN/m³
Angle of friction of soil above   fa(3)=0°
Cohesion of soil above the line   ca(3)=0 kN/m²
Angle of friction of soil below   fb(3)=25°
Cohesion of soil below the line   cb(3)=0 kN/m²

Coordinates of boundary line 4 of 5

X value LHE of boundary line      xa(4)=0 m
Y value LHE of boundary line      ya(4)=0 m
X value RHE of boundary line      xb(4)=5000 m
Y value RHE of boundary line      yb(4)=0 m

Material properties

Density of material below line    w(4)=17.28 kN/m³
Angle of friction of soil above   fa(4)=0°
Cohesion of soil above the line   ca(4)=0 kN/m²
Angle of friction of soil below   fb(4)=25°
Cohesion of soil below the line   cb(4)=0 kN/m²

Coordinates of boundary line 5 of 5

X value LHE of boundary line      xa(5)=-14.356 m
Y value LHE of boundary line      ya(5)=-5.967 m
X value RHE of boundary line      xb(5)=5000 m
Y value RHE of boundary line      yb(5)=-5.967 m

Material properties

Density of material below line    w(5)=18.536 kN/m³
Angle of friction of soil above   fa(5)=25°
Cohesion of soil above the line   ca(5)=0 kN/m²
Angle of friction of soil below   fb(5)=12°
Cohesion of soil below the line   cb(5)=21.546 kN/m²
Coordinates of trial area

Coordinates of large crosses are given by the Engineer, those of small crosses are at midpoints. There are nine positions. (xmax & xmin should both be negative)

Minimum X coord of trial circle  \( x_{\text{min}} = -17.6 \) m
Maximum X coord of trial circle  \( x_{\text{max}} = -17.6 \) m
Minimum Y coord of trial circle  \( y_{\text{min}} = 14.171 \) m
Maximum Y coord of trial circle  \( y_{\text{max}} = 14.171 \) m
Minimum radius of trial circle  \( r_{\text{min}} = 26.737 \) m
Maximum radius of trial circle  \( r_{\text{max}} = 26.737 \) m
**Location:** London Borough of Croydon Example

**Slope stability - the Swedish method**

Coordinates of boundary line 1 of 5

- X value LHE of boundary line: $x_a(1) = -500$ m
- Y value LHE of boundary line: $y_a(1) = -9$ m
- X value RHE of boundary line: $x_b(1) = -14$ m
- Y value RHE of boundary line: $y_b(1) = -9$ m

**Material properties**

- Density of material below line: $w(1) = 17$ kN/m$^3$
- Angle of friction of soil above: $f_a(1) = 0^°$
- Cohesion of soil above the line: $c_a(1) = 0$ kN/m$^2$
- Angle of friction of soil below: $f_b(1) = 20^°$
- Cohesion of soil below the line: $c_b(1) = 20$ kN/m$^2$

Coordinates of boundary line 2 of 5

- X value LHE of boundary line: $x_a(2) = -14$ m
- Y value LHE of boundary line: $y_a(2) = -9$ m
- X value RHE of boundary line: $x_b(2) = -2$ m
- Y value RHE of boundary line: $y_b(2) = -3$ m

**Material properties**

- Density of material below line: $w(2) = 17$ kN/m$^3$
- Angle of friction of soil above: $f_a(2) = 0^°$
- Cohesion of soil above the line: $c_a(2) = 0$ kN/m$^2$
- Angle of friction of soil below: $f_b(2) = 20^°$
- Cohesion of soil below the line: $c_b(2) = 20$ kN/m$^2$
Coordinates of boundary line 3 of 5

X value LHE of boundary line \( x_a(3) = 0 \) m
Y value LHE of boundary line \( y_a(3) = -3 \) m
X value RHE of boundary line \( x_b(3) = 0.01 \) m
Y value RHE of boundary line \( y_b(3) = 0 \) m

Material properties

Density of material below line \( w(3) = 22 \) kN/m³
Angle of friction of soil above \( \alpha_a(3) = 0^\circ \)
Cohesion of soil above the line \( c_a(3) = 0 \) kN/m²
Angle of friction of soil below \( \alpha_b(3) = 35^\circ \)
Cohesion of soil below the line \( c_b(3) = 0 \) kN/m²

Coordinates of boundary line 4 of 5

X value LHE of boundary line \( x_a(4) = 0.01 \) m
Y value LHE of boundary line \( y_a(4) = 0 \) m
X value RHE of boundary line \( x_b(4) = 100 \) m
Y value RHE of boundary line \( y_b(4) = 0 \) m

Material properties

Density of material below line \( w(4) = 22 \) kN/m³
Angle of friction of soil above \( \alpha_a(4) = 0^\circ \)
Cohesion of soil above the line \( c_a(4) = 0 \) kN/m²
Angle of friction of soil below \( \alpha_b(4) = 35^\circ \)
Cohesion of soil below the line \( c_b(4) = 0 \) kN/m²

Coordinates of boundary line 5 of 5

X value LHE of boundary line \( x_a(5) = -2 \) m
Y value LHE of boundary line \( y_a(5) = -3 \) m
X value RHE of boundary line \( x_b(5) = 0 \) m
Y value RHE of boundary line \( y_b(5) = -3 \) m

Material properties

Density of material below line \( w(5) = 17 \) kN/m³
Angle of friction of soil above \( \alpha_a(5) = 35^\circ \)
Cohesion of soil above the line \( c_a(5) = 0 \) kN/m²
Angle of friction of soil below \( \alpha_b(5) = 20^\circ \)
Cohesion of soil below the line \( c_b(5) = 20 \) kN/m²
Coordinates of trial area

Coordinates of large crosses are given by the Engineer, those of small crosses are at midpoints. There are nine positions. (xmax & xmin should both be negative)

Minimum X coord of trial circle \( x_{\text{min}} = -20 \text{ m} \)
Maximum X coord of trial circle \( x_{\text{max}} = -15 \text{ m} \)
Minimum Y coord of trial circle \( y_{\text{min}} = 15 \text{ m} \)
Maximum Y coord of trial circle \( y_{\text{max}} = 20 \text{ m} \)
Minimum radius of trial circle \( r_{\text{min}} = 30.5 \text{ m} \)
Maximum radius of trial circle \( r_{\text{max}} = 32.5 \text{ m} \)
Location: Example 7.1

Coefficient of volume compressibility and compression index

Specific gravity - soil particles \( G_s = 2.73 \)
Initial thickness of specimen \( t_i = 19 \text{ mm} \)
Water content at end of test \( w_1 = 19.8 \% \)

Final thickness \( t_f = t_i - (d(1) - d(i)) = 15.48 \text{ mm} \)
Void ratio at end of test \( e_1 = w_1 \times G_s / 100 = 0.54054 \)
Change in thickness \( \delta H = t_i - t_f = 3.52 \text{ mm} \)
Change in void ratio \( \delta e' = (1 + e_1) \times \delta H / (t_i - \delta H) = 0.3503 \)

Void ratio at start of test \( e_0 = e_1 + \delta e' = 0.89084 \)

\[
\delta e' = \frac{1.8908}{19} \quad \text{i.e.} \quad \delta e' = 0.099518 \times \delta H \quad \text{and can be used}
\]

to obtain the void ratio at the end of each increment as below:

<table>
<thead>
<tr>
<th>Pressure (kN/m²)</th>
<th>Change in thickness (mm)</th>
<th>Change in void ratio</th>
<th>Void ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.89084</td>
</tr>
<tr>
<td>1.7324</td>
<td>0.253</td>
<td>0.025178</td>
<td>0.86567</td>
</tr>
<tr>
<td>2.0294</td>
<td>0.507</td>
<td>0.050456</td>
<td>0.84039</td>
</tr>
<tr>
<td>2.3304</td>
<td>0.892</td>
<td>0.08877</td>
<td>0.80207</td>
</tr>
<tr>
<td>2.6325</td>
<td>1.551</td>
<td>0.15435</td>
<td>0.73649</td>
</tr>
<tr>
<td>2.9335</td>
<td>2.392</td>
<td>0.23805</td>
<td>0.6528</td>
</tr>
<tr>
<td>3.2345</td>
<td>3.324</td>
<td>0.3308</td>
<td>0.56005</td>
</tr>
<tr>
<td>3.5355</td>
<td>4.263</td>
<td>0.42425</td>
<td>0.4666</td>
</tr>
<tr>
<td>0</td>
<td>3.52</td>
<td>0.3503</td>
<td>0.54054</td>
</tr>
</tbody>
</table>

Preconsolidation pressure

RF Craig in 'Soil Mechanics' states "Whenever possible the preconsolidation pressure for an overconsolidated clay should not be exceeded in construction. Compression will not usually be great if the effective vertical stress remains below \( \sigma'c \): only if \( \sigma'c \) is exceeded will compression be large."

Casagrande has suggested an empirical method to estimate the preconsolidation pressure from the curve of void ratio plotted against the effective pressure plotted to a LOG10 scale. A numerical procedure will be used to avoid having to plot void ratio against the LOG of pressure. Firstly 'curve fit' to fill out 5 intermediate values between each of the above values for void ratio and pressure.

Preconsolidation pressure \( \text{sig}'p_c = \exp(x/0.4342945) = 318.39 \)
Coefficient of volume compressibility and compression index

Void ratio for higher stress  \( e_1 = 0.57814 \)

Coeff. of volume compressibility  \( m_v = (e_0 - e_1) / (1 + e_0) / (\sigma_{'1} - \sigma_{'0}) \)

\[ = 66.574 \times 10^{-6} \text{ m}^3/\text{kN} \]

Compression index is as follows:

\[ C_c = \frac{(e_0 - e_1)}{(0.43429 \times \log(\frac{\sigma_{'1}}{\sigma_{'0}}))} = 0.30859 \]
Location: Example 7.2

Length of foundation \( X = 45 \, \text{m} \)
Breadth of foundation \( Y = 30 \, \text{m} \)
Thickness of clay layer \( H = 4 \, \text{m} \)
Depth to centre of clay layer \( z = 23.5 \, \text{m} \)
Coefficient of volume compress. - clay \( m_v = 0.35 \, \text{m}^2/\text{MN} \)
Pressure beneath foundation \( q = 125 \, \text{kN/m}^2 \)
Location: Safe bearing pressures by bearing capacity factors

Determination of soil safe bearing capacity

Determination of soil Safe Bearing Capacity assuming the removal of overburden pressure when calculating the net ultimate bearing capacity.

Soil type is assumed to be constant.

Foundation formation depth $D_f = 1.8$ m
Foundation breadth $B = 1.4$ m
Foundation length $L = 15$ m
Drained shear strength parameter $c' = 0$ kN/m$^2$
Soil angle of friction $\phi = 28^\circ$
Saturated unit weight of soil $\gamma_{sat} = 18.8$ kN/m$^3$
Dry unit weight of soil $\gamma_{dry} = 14.6$ kN/m$^3$
Depth to ground water table $GWT = 1.8$ m
Gross ultimate bearing capacity $q_{ult} = nc + nq + ng = 443.22$ kN/m$^2$
Factor of safety against failure $F_s = 3$
Gross safe bearing capacity $q_{max} = q_{ult}/F_s + q = 174.02$ kN/m$^2$
Foundation type under consideration is a strip foundation.
Location: Ex1 - Leisure centre car park

The structural design of bituminous roads

The TRRL Report 1132 published by the Department of Transport will be adopted for pavement design. This document recommends that any material within 450 mm of the road surface including the Subbase itself, must be frost resistant. The weight of vehicles on circulatory roads or service roads in a commercial car park is assumed to be greater than 1500 kg and less than this weight on car parking areas.

(A) Parking areas for cars only

The TRRL Report is not relevant for parking areas as vehicle weights are less than 1500 kg. Dense bitumen macadam to be in accordance with BS EN 13108 and/or PD 6691:2010 Guidance on the use of BS EN 13108 Bituminous mixtures - Materials Specifications.

For CBR values over 3% the following minimum layer thicknesses are recommended:

Wearing course (Surface course): 25 mm thick (6 mm size) Dense Bitumen Macadam (DBM)

Basecourse (Binder course) : 60 mm Dense Bitumen Macadam (DBM)

Roadbase (Base) : 225 mm thick layer of DOT type 1 granular fill

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>25mm Wearing course</td>
<td></td>
</tr>
<tr>
<td>60mm Basecourse</td>
<td></td>
</tr>
<tr>
<td>225mm Roadbase</td>
<td></td>
</tr>
<tr>
<td>Subgrade (soil) with CBR value</td>
<td></td>
</tr>
</tbody>
</table>

(B) Circulatory roads

Vehicles of weight greater than 1500 kg are assumed.
Design life of flexible pavement life=20 years
CBR value from site tests cbr=2 %

The total thickness of the bound layer entails the Wearing course, Basecourse and Roadbase.

Design life n=life=20 years
Initial daily flow of vehicles Fo=10
Growth rate r=r'/100=0.02
Below P’ represents the commercial vehicles using the slow lane.
Percentage of commercial vehicles P’=100 %
Proportion of commercial vehicles P=P’/100=1
The expression below is by Addis and Robinson (1983) and will be used to calculate the total number of commercial vehicles (Tn) using the slow lane over the design life of 20 years.

Total number of vehicles

\[ T_n = \frac{365 \times F_0 \times ((1+r)^{n-1}) \times P}{(r \times 1000000)} \]

= 0.088685 million

Year car park will be completed

Year = 2017

Mid-term year with a base of 1945

\[ t = \text{Year} + \frac{n}{2} - 1945 = 82 \]

Mid-term daily flow

\[ F = F_0 \times (1+r)^{\frac{n}{2}} = 12.19 \]

Expression

\[ C_1 = 0.082 + 0.93^{\frac{t}{2}} \times 0.93^{\frac{t}{2}} = 0.084604 \]

Expression

\[ C_2 = 0.082 + 0.92^{\frac{t}{2}} \times 0.92^{\frac{t}{2}} = 0.083073 \]

Expression

\[ C_3 = 3.9^{\left(\frac{F}{1550}\right)} = 1.0108 \]

Vehicle damage factor

\[ D = \frac{0.35}{C_1 - \left(\frac{0.26}{C_2}\right) \times (1.0/C_3)} = 1.0405 \]

Number of standard axles

\[ N_{sa} = T_n \times D = 0.092275 \text{ msa} \]

The following minimum thickness is from Fig D2 of TRRL Report 1132.

Minimum thickness of bound layer

\[ \text{tbl} = \text{TABLE 1 for } N_{sa} = 0.092275, \text{ cbr}=2 \]

= 150.68 mm

= 155 mm (rounded up)

Bound layer thickness

Hot rolled asphalt and dense bitumen macadam to be in accordance with BS EN 13108 and/or PD 6691:2010 Guidance on the use of BS EN 13108 Bituminous mixtures - Materials Specifications.

Wearing course (Surface course): 35 mm Hot Rolled Asphalt (HRA)
Basecourse (Binder course) : 55 mm Dense Bitumen Macadam (DBM)
Roadbase (Base) : 85 mm Dense Bitumen Macadam (DBM)

Subbase thickness

The thickness of the Subbase can be obtained from Figure C3 or Figure C5 of the TRRL Report 1132. The former utilises DOT type 1 granular material only and the latter a combination of DOT type 1 granular fill and a Capping layer.

Option 1 - DOT type 1 only

When a Capping layer is not employed for weak Subgrades of less than 5% CBR, the thickness of Subbase will exceed 225 mm and must be laid in two layers. The lower layer should be as thick as practically possible and preferably 225 mm, in order to avoid the danger of damaging the Subbase or Subgrade during compaction. The upper layer should have a minimum thickness of 50 mm. Fig C3 of the TRRL Report 1132 will be utilised to evaluate the Subbase thickness.

Length of road under construction L_{road} = 150 m

Subbase thickness (DOT type 1) Sub = 300 mm
Option 2 - (DOT type 1 + Capping layer)

From Figure C5 for CBR values of 2%, 3%, 4% and 5% use 150 mm Subbase of DOT type 1 over 350 mm of Capping layer. In order to reduce the possibility of failure within the Capping layer the latter should have a minimum CBR value of 15%. A suitable material for a Capping layer is 6F2 as defined in the specification for Highway Works: Volume 1: Tables 6/1 and 6/2.
The structural design of bituminous roads

The TRRL Report 1132 published by the Department of Transport will be adopted for pavement design. This document recommends that any material within 450 mm of the road surface including the Subbase itself, must be frost resistant. The weight of vehicles on circulatory roads or service roads in a commercial car park is assumed to be greater than 1500 kg and less than this weight on car parking areas.

(A) Parking areas for cars only

The TRRL Report is not relevant for parking areas as vehicle weights are less than 1500 kg. Dense bitumen macadam to be in accordance with BS EN 13108 and/or PD 6691:2010 Guidance on the use of BS EN 13108 Bituminous mixtures - Materials Specifications.

For CBR values over 3% the following minimum layer thicknesses are recommended:

Wearing course (Surface course): 25 mm thick (6 mm size) Dense Bitumen Macadam (DBM)

Basecourse (Binder course): 60 mm Dense Bitumen Macadam (DBM)

Roadbase (Base): 225 mm thick layer of DOT type 1 granular fill

(B) Circulatory roads

Vehicles of weight greater than 1500 kg are assumed.

Design life of flexible pavement  life=20 years
CBR value from site tests cbr=5 %

The total thickness of the bound layer entails the Wearing course, Basecourse and Roadbase.

Design life n=life=20 years
Initial daily flow of vehicles Fo=300
Growth rate r=r'/100=0.02
Below P' represents the commercial vehicles using the slow lane.
Percentage of commercial vehicles P' = 100% 
Proportion of commercial vehicles P = P'/100 = 1
The expression below is by Addis and Robinson (1983) and will be used to calculate the total number of commercial vehicles (Tn) using the slow lane over the design life of 20 years.

Total number of vehicles 
\[ Tn = \frac{365 \times F_0 \times ((1+r)^{n-1}) \times P}{(r \times 1000000)} \]
= 2.6606 million

Year car park will be completed
Year = 2017
Mid-term year with a base year of 1945
\[ t = Year + \frac{n}{2} - 1945 = 82 \]
Mid term daily flow 
\[ F = F_0 \times (1+r)^{n/2} = 365.7 \]
Expression 
\[ C_1 = 0.082 + 0.93^{(t/2)} \times 0.93^{(t/2)} = 0.084604 \]
Expression 
\[ C_2 = 0.082 + 0.92^{(t/2)} \times 0.92^{(t/2)} = 0.083073 \]
Expression 
\[ C_3 = 3.9^{(F/1550)} = 1.3786 \]
Vehicle damage factor 
\[ D = 0.35/C_1 - (0.26/C_2) \times (1.0/C_3) = 1.8667 \]
Number of standard axles 
\[ N_{sa} = Tn \times D = 4.9666 \text{ msa} \]
The following minimum thickness is from Fig D2 of TRRL Report 1132.
Minimum thickness of bound layer 
\[ tbl = \text{TABLE 1 for } N_{sa} = 4.9666, \text{ cbr} = 5 \]
= 249.67 mm
= 250 mm (rounded up)

Bound layer thickness

Bound layer thickness

The thickness of the Subbase can be obtained from Figure C3 or Figure C5 of the TRRL Report 1132. The former utilises DOT type 1 granular material only and the latter a combination of DOT type 1 granular fill and a Capping layer.

Option 1 - DOT type 1 only

When a Capping layer is not employed for weak Subgrades of less than 5% CBR, the thickness of Subbase will exceed 225 mm and must be laid in two layers. The lower layer should be as thick as practically possible and preferably 225 mm, in order to avoid the danger of damaging the Subbase or Subgrade during compaction. The upper layer should have a minimum thickness of 50 mm. Fig C3 of the TRRL Report 1132 will be utilised to evaluate the Subbase thickness.

Length of road under construction \( L_{\text{road}} = 150 \text{ m} \)
Subbase thickness (DOT type 1) \( \text{Sub} = 225 \text{ mm} \)
Option 2 - (DOT type 1 + Capping layer)

From Figure C5 for CBR values of 2%, 3%, 4% and 5% use 150 mm Subbase of DOT type 1 over 350 mm of Capping layer.
In order to reduce the possibility of failure within the Capping layer the latter should have a minimum CBR value of 15%. A suitable material for a Capping layer is 6F2 as defined in the specification for Highway Works: Volume 1: Tables 6/1 and 6/2.
Location: Ex1 - Leisure centre car park (Foundation Class 1)

The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1, 2 & 3) or Performance Foundation Design (Classes 1, 2, 3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is <50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs.

Upper pavement overall design thickness

Design life (flexible pavement) life=20 years
Design traffic in msa units dtr=20 msa
Upper pavement design thickness dt=TABLE 3 for dtr=20
=320 mm
=325 mm (rounded up)

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm High Density Macadam (HDM50)
Base 225 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 280 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm High Density Macadam (HDM50)
Base 245 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm Dense Bitumen Macadam (DBM50)
Base 225 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 280 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm Dense Bitumen Macadam (DBM50)
Base 245 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

Foundation design

Restricted Foundation Design with Capping only will be adopted i.e. no Subbase layer will be provided. The design traffic is ≤ 20 million standard axles (msa). Capping Specification (MCHW1) Series 600 (see Foundation materials section).

Subgrade CBR (%) CBR=5 %
Subgrade Stiffness Modulus E=17.6*(CBR)^0.64=49.301 MPa
Capping layer thickness clt=TABLE 3.11 for E=49.301
=395.59 mm
=400 mm (rounded up)
<table>
<thead>
<tr>
<th></th>
<th>Surface course or Wearing course</th>
</tr>
</thead>
<tbody>
<tr>
<td>Binder course</td>
<td></td>
</tr>
<tr>
<td>Base</td>
<td></td>
</tr>
<tr>
<td>Capping layer only</td>
<td></td>
</tr>
<tr>
<td>Subgrade (soil)</td>
<td></td>
</tr>
</tbody>
</table>

**Upper pavement to HD26**

**Foundation to HD25 and IAN 73/06 Rev1**

**NOTE:** Base could be in two layers (i.e. upper base & lower base)
Location: Ex2 - Leisure centre car park (Foundation Class 2)

The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1, 2 & 3) or Performance Foundation Design (Classes 1, 2, 3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is <50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs.

Upper pavement overall design thickness

- Design life (flexible pavement): life=20 years
- Design traffic in msa units: dtr=75 msa
- Upper pavement design thickness: dt=TABLE 4 for dtr=75 =357.5 mm =360 mm (rounded up)

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
- Surfacing (wearing course): 45 mm Hot Rolled Asphalt (HRA)
- Binder course: 55 mm High Density Macadam (HDM50)
- Base: 260 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 315 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm High Density Macadam (HDM50)
Base 280 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm Dense Bitumen Macadam (DBM50)
Base 260 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 315 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm Dense Bitumen Macadam (DBM50)
Base 280 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

Foundation design

Restricted Foundation Design with Subbase & Capping layers provided as the design traffic is ≤ 80 million standard axles (msa). Subbase to be MCHW1 803, 804, 805, 806 and Capping to be MCHW1 Series 600 (refer to Foundation materials section).

WARNING:
Clause 804 (Subbase Type 2) may only be used for pavement design traffic levels up to 5 msa.

Subgrade CBR (%) CBR=5 %
Subgrade Stiffness Modulus E=17.6*(CBR)^0.64=49.301 MPa
Subbase layer thickness slt=TABLE 3.21 for E=49.301
=243.5 mm
=245 mm (rounded up)
Capping layer thickness clt=TABLE 3.23 for E=49.301
=211.4 mm
=215 mm (rounded up)
**Surface course or Wearing course**

**Binder course**

**Base**

**Subbase**

**Capping layer**

**Subgrade (soil)**

**Upper pavement to HD26**

**Foundation to HD25 and IAN 73/06 Rev1**

**NOTE:** Base could be in two layers (i.e. upper base & lower base). Subbase could be in two layers (i.e. upper Subbase & lower Subbase)
The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1,2 & 3) or Performance Foundation Design (Classes 1,2,3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is <50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs.

Upper pavement overall design thickness

Design life (flexible pavement) life=20 years
Design traffic in msa units dtr=75 msa
Upper pavement design thickness dt=TABLE 4 for dtr=75
=357.5 mm
=360 mm (rounded up)

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm High Density Macadam (HDM50)
Base 260 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 315 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm High Density Macadam (HDM50)
Base 280 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm Dense Bitumen Macadam (DBM50)
Base 260 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 315 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm Dense Bitumen Macadam (DBM50)
Base 280 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

Foundation design

Restricted Foundation Design with Subbase & Capping layers provided as the design traffic is ≤ 80 million standard axles (msa). Subbase to be MCHW1 821, 822, or 840 soil cement: strength C3/4 or C5/6 and Capping to be MCHW1 Series 600 (see Foundation materials section).

Subgrade CBR (%) CBR=3.5 %
Subgrade Stiffness Modulus E=17.6*(CBR)^0.64=39.239 MPa
Subbase layer thickness slt=TABLE 3.22 for E=39.239
=253.81 mm
=255 mm (rounded up)
Capping layer thickness clt=TABLE 3.23 for E=39.239
=231.9 mm
=235 mm (rounded up)
### Flexible pavement design to HD26/06 & HD25 (IAN 73)

**Surface course or Wearing course**

**Binder course**

**Base**

**Subbase**

+ **Capping layer**

**Subgrade (soil)**

**Upper pavement to HD26**

**Foundation to HD25 and IAN 73/06 Rev1**

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NOTE: Base could be in two layers (i.e. upper base & lower base). Subbase could be in two layers (i.e. upper Subbase & lower Subbase).
Location: Ex4 - Leisure centre car park (Foundation Class 2)

The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1, 2 & 3) or Performance Foundation Design (Classes 1, 2, 3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is <50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs.

Upper pavement overall design thickness

Design life (flexible pavement) life=20 years
Design traffic in msa units dtr=75 msa
Upper pavement design thickness dt=TABLE 4 for dtr=75
=357.5 mm
=360 mm (rounded up)

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm High Density Macadam (HDM50)
Base 260 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 315 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm High Density Macadam (HDM50)
Base 280 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm Dense Bitumen Macadam (DBM50)
Base 260 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 315 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm Dense Bitumen Macadam (DBM50)
Base 280 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

Foundation design

Restricted Foundation Design with Subbase layer only provided. The design traffic is ≤ 80 million standard axles (msa). Subbase to be MCHW1 821, 822, or 840 soil cement: strength C3/4 or C5/6 (refer to Foundation materials section).

Subgrade CBR (%) CBR=3.5 %
Subgrade Stiffness Modulus E=17.6*(CBR)^0.64=39.239 MPa
Subbase layer thickness slt=TABLE 3.13 for E=39.239
  =312.85 mm
  =315 mm (rounded up)
Sample output for SCALE Proforma 777. (ans=4) Page: 3
Loadings and foundations Made by: IFB
Flexible pavement design to HD26/06 & HD25 (IAN 73) Date: 02/12/19
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<td>Foundation to HD25 and IAN 73/06 Rev1</td>
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<tr>
<td>Subgrade (soil)</td>
<td></td>
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</tbody>
</table>

NOTE: Base could be in two layers (i.e. upper base & lower base).
Subbase could be in two layers (i.e. upper Subbase & lower Subbase)
Location: Ex5 - Leisure centre car park (Foundation Class 2)

The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1,2 & 3) or Performance Foundation Design (Classes 1,2,3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is <50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs

Upper pavement overall design thickness

Design life (flexible pavement) life=20 years
Design traffic in msa units dtr=75 msa
Upper pavement design thickness dt=TABLE 4 for dtr=75
             =357.5 mm
             =360 mm (rounded up)

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm High Density Macadam (HDM50)
Base 260 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course)        45 mm Hot Rolled Asphalt (HRA)
Base                              315 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course)        50 mm Porous Asphalt (PA)
Binder course                     60 mm High Density Macadam (HDM50)
Base                              280 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course)        45 mm Hot Rolled Asphalt (HRA)
Binder course                     55 mm Dense Bitumen Macadam (DBM50)
Base                              260 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course)        45 mm Hot Rolled Asphalt (HRA)
Base                              315 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course)        50 mm Porous Asphalt (PA)
Binder course                     60 mm Dense Bitumen Macadam (DBM50)
Base                              280 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

Foundation design

Restricted Foundation Design with Subbase layer only provided. The design traffic is ≤ 80 million standard axles (msa). Subbase to be MCHW1 803, 804, 805 and 806 (see Foundation materials section).

WARNING:
Clause 804 (Subbase Type 2) may only be used for pavement design traffic levels up to 5 msa.

Subgrade CBR (%)            CBR=3.5 %
Subgrade Stiffness Modulus         E=17.6*(CBR)^0.64=39.239 MPa
Subbase layer thickness            slt=TABLE 3.12 for E=39.239
                                      =386.66 mm
                                      =390 mm (rounded up)
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<td>Binder course</td>
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<tr>
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<td>Foundation to HD25 and IAN 73/06 Rev1</td>
</tr>
<tr>
<td>Subgrade (soil)</td>
<td></td>
</tr>
</tbody>
</table>

NOTE: Base could be in two layers (i.e. upper base & lower base). Subbase could be in two layers (i.e. upper Subbase & lower Subbase).
The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1, 2 & 3) or Performance Foundation Design (Classes 1, 2, 3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is $\geq 50$ must be frost susceptible in the long-term. Where the MAFI is $<50$ the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs

Upper pavement overall design thickness

| Design life (flexible pavement) | life=20 years |
| Design traffic in msa units | dtr=75 msa |
| Upper pavement design thickness | dt=TABLE 5 for dtr=75 |
| | $=320$ mm |
| | $=325$ mm (rounded up) |

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm High Density Macadam (HDM50)
Base 225 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 280 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm High Density Macadam (HDM50)
Base 245 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm Dense Bitumen Macadam (DBM50)
Base 225 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 280 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm Dense Bitumen Macadam (DBM50)
Base 245 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

Foundation design

Restricted Foundation Design with Subbase only provided. The design traffic is > 80 million standard axles (msa). Subbase to be MCHWI 821 or 822: strength C8/10 (see Foundation materials section).

Subgrade CBR (%)  
CBR=3.5 %  

Subgrade Stiffness Modulus  
E=17.6*(CBR)^0.64=39.239 MPa  

Subbase layer thickness  
slt=TABLE 3.14 for E=39.239  
=312.85 mm  
=315 mm (rounded up)
NOTE: Base could be in two layers (i.e. upper base & lower base).
Subbase could be in two layers (i.e. upper Subbase & lower Subbase)
The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1,2 & 3) or Performance Foundation Design (Classes 1,2,3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is <50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs.

Upper pavement overall design thickness

Design life (flexible pavement) life=20 years
Design traffic in msa units dtr=75 msa
Upper pavement design thickness dt=TABLE 6 for dtr=75
=325 mm
=330 mm (rounded up)

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm High Density Macadam (HDM50)
Base 230 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 285 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm High Density Macadam (HDM50)
Base 250 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm Dense Bitumen Macadam (DBM50)
Base 230 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 285 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm Dense Bitumen Macadam (DBM50)
Base 250 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

**Foundation design**

Performance Foundation Design with Subbase only layer provided. The design traffic is > 80 million standard axles (msa). The foundation completed surface must meet all the relevant criteria in Series 700 of the specification.

Subgrade CBR (%) CBR=3 %

For performance design, the CBR must be converted to the Subgrade Stiffness Modulus (E) using the expression derived from work on certain soils (Powell et al, 1984), where CBR is a % value.

Subgrade Stiffness Modulus $E=17.6 \times (CBR)^{0.64}=35.553$ MPa

Assumed Subbase layer stiffness $Sst=1000$ MPa

Subbase layer thickness $clt=TABLE 4.51$ for $E=35.553$

=442.24 mm

=445 mm (rounded up)
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</tr>
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<td>Binder course</td>
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<td>Base</td>
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<td>Foundation to HD25 and IAN 73/06 Rev1</td>
</tr>
<tr>
<td>Subgrade (soil)</td>
<td></td>
</tr>
</tbody>
</table>

NOTE: Base could be in two layers (i.e. upper base & lower base). Subbase could be in two layers (i.e. upper Subbase & lower Subbase)
The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1, 2 & 3) or Performance Foundation Design (Classes 1, 2, 3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is < 50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs.

Upper pavement overall design thickness

Design life (flexible pavement) life=20 years
Design traffic in msa units dtr=75 msa
Upper pavement design thickness dt=TABLE 6 for dtr=75
=325 mm
=330 mm (rounded up)

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm High Density Macadam (HDM50)
Base 230 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 285 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm High Density Macadam (HDM50)
Base 250 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm Dense Bitumen Macadam (DBM50)
Base 230 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 285 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm Dense Bitumen Macadam (DBM50)
Base 250 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

Foundation design

Performance Foundation Design with Subbase only layer provided. The design traffic is > 80 million standard axles (msa). The foundation completed surface must meet all the relevant criteria in Series 700 of the specification.

Subgrade CBR (%) CBR=3 %

For performance design, the CBR must be converted to the Subgrade Stiffness Modulus (E) using the expression derived from work on certain soils (Powell et al, 1984), where CBR is a % value.

Subgrade Stiffness Modulus E=17.6*(CBR)^0.64=35.553 MPa

Assumed Subbase layer stiffness Stt=2000 MPa

Subbase layer thickness clt=TABLE 4.52 for E=35.553
=296.68 mm
=300 mm (rounded up)
NOTE: Base could be in two layers (i.e. upper base & lower base). Subbase could be in two layers (i.e. upper Subbase & lower Subbase)
The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1,2 & 3) or Performance Foundation Design (Classes 1,2,3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is <50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs.

Upper pavement overall design thickness

Design life (flexible pavement) life=20 years
Design traffic in msa units dtr=75 msa
Upper pavement design thickness dt=TABLE 5 for dtr=75

=320 mm
=325 mm (rounded up)

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm High Density Macadam (HDM50)
Base 225 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 280 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm High Density Macadam (HDM50)
Base 245 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm Dense Bitumen Macadam (DBM50)
Base 225 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Base 280 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course) 50 mm Porous Asphalt (PA)
Binder course 60 mm Dense Bitumen Macadam (DBM50)
Base 245 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

Foundation design

Performance Foundation Design with Subbase only layer provided. The design traffic is > 80 million standard axles (msa). The foundation completed surface must meet all the relevant criteria in Series 700 of the specification.

Subgrade CBR (%) CBR=3 %

For performance design, the CBR must be converted to the Subgrade Stiffness Modulus (E) using the expression derived from work on certain soils (Powell et al, 1984), where CBR is as a % value.

Subgrade Stiffness Modulus E=17.6*(CBR)^0.64=35.553 MPa

Assumed Subbase layer stiffness Sst=1000 MPa
Subbase layer thickness clt=TABLE 4.43 for E=35.553
=221.12 mm
=225 mm (rounded up)
NOTE: Base could be in two layers (i.e. upper base & lower base). Subbase could be in two layers (i.e. upper Subbase & lower Subbase)
Sample output for SCALE Proforma 777. (ans=10)
Loadings and foundations
Flexible pavement design to HD26/06 & HD25 (IAN 73)
Copyright 1986-2019 Fitzroy Systems Ltd.

Location: Ex10 - Performance Foundation Design (Foundation Class 2)

The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1, 2 & 3) or Performance Foundation Design (Classes 1, 2, 3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is <50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs.

Upper pavement overall design thickness

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<th>Parameter</th>
<th>Value</th>
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<td>Design life (flexible pavement)</td>
<td>life=20 years</td>
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<tr>
<td>Design traffic in msa units</td>
<td>dtr=75 msa</td>
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<tr>
<td>Upper pavement design thickness</td>
<td>dt=TABLE 4 for dtr=75</td>
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<tr>
<td></td>
<td>=357.5 mm</td>
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<td></td>
<td>=360 mm (rounded up)</td>
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Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
- Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
- Binder course 55 mm High Density Macadam (HDM50)
- Base 260 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course)  45 mm Hot Rolled Asphalt (HRA)
Base  315 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course)  50 mm Porous Asphalt (PA)
Binder course  60 mm High Density Macadam (HDM50)
Base  280 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course)  45 mm Hot Rolled Asphalt (HRA)
Binder course  55 mm Dense Bitumen Macadam (DBM50)
Base  260 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course)  45 mm Hot Rolled Asphalt (HRA)
Base  315 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course)  50 mm Porous Asphalt (PA)
Binder course  60 mm Dense Bitumen Macadam (DBM50)
Base  280 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

Foundation design

Performance Foundation Design with Subbase only layer provided. The design traffic is ≤ 80 million standard axles (msa). The foundation completed surface must meet all the relevant criteria in Series 700 of the specification.

Subgrade CBR (%)  CBR=3 %
For performance design, the CBR must be converted to the Subgrade Stiffness Modulus (E) using the expression derived from work on certain soils (Powell et al, 1984), where CBR is as a % value.
Subgrade Stiffness Modulus  E=17.6*(CBR)^0.64=35.553 MPa
Assumed Subbase layer stiffness  Sst=200 MPa
Subbase layer thickness  slt=TABLE 4.32 for E=35.553
  =293.36 mm
  =295 mm (rounded up)
### Flexible pavement design to HD26/06 & HD25 (IAN 73)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface course or Wearing course</td>
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<tr>
<td>Binder course</td>
<td></td>
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<td>Base</td>
<td></td>
</tr>
<tr>
<td>Subbase layer only</td>
<td></td>
</tr>
<tr>
<td>Subgrade (soil)</td>
<td></td>
</tr>
</tbody>
</table>

**Upper pavement to HD26**

**Foundation to HD25 and IAN 73/06 Rev1**

**NOTE:** Base could be in two layers (i.e. upper base & lower base). Subbase could be in two layers (i.e. upper Subbase & lower Subbase).
The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1,2 & 3) or Performance Foundation Design (Classes 1,2,3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is <50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs.

Upper pavement overall design thickness

| Design life (flexible pavement) | life=20 years |
| Design traffic in msa units     | dtr=75 msa   |
| Upper pavement design thickness | dt=TABLE 4 for dtr=75 |
|                                 | =357.5 mm    |
|                                 | =360 mm (rounded up) |

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm High Density Macadam (HDM50)
Base 260 mm High Density Macadam (HDM50)
Option 2:
Surfacing (wearing course)  45 mm Hot Rolled Asphalt (HRA)
Base                          315 mm High Density Macadam (HDM50)

Option 3:
Surfacing (wearing course)  50 mm Porous Asphalt (PA)
Binder course                 60 mm High Density Macadam (HDM50)
Base                          280 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
Surfacing (wearing course)  45 mm Hot Rolled Asphalt (HRA)
Binder course                 55 mm Dense Bitumen Macadam (DBM50)
Base                          260 mm Dense Bitumen Macadam (DBM50)

Option 5:
Surfacing (wearing course)  45 mm Hot Rolled Asphalt (HRA)
Base                          315 mm Dense Bitumen Macadam (DBM50)

Option 6:
Surfacing (wearing course)  50 mm Porous Asphalt (PA)
Binder course                 60 mm Dense Bitumen Macadam (DBM50)
Base                          280 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

**Foundation design**

Performance Foundation Design with Subbase and Capping layers provided. The design traffic is ≤ 80 million standard axles (msa). The foundation completed surface must meet all the relevant criteria in Series 700 of the specification.

Subgrade CBR (%)  
CBR=3 %

For performance design, the CBR must be converted to the Subgrade Stiffness Modulus (E) using the expression derived from work on certain soils (Powell et al, 1984), where CBR is as a % value.

Subgrade Stiffness Modulus  
E=17.6*(CBR)^0.64=35.553 MPa

Assumed Subbase layer stiffness  
Sst=350 MPa

Subbase layer thickness  
slt=TABLE 4.63 for E=35.553
  =150 mm
  =155 mm (rounded up)

Assumed Capping layer stiffness  
Cst=75 MPa

Capping layer thickness  
clt=TABLE 4.64 for E=35.553
  =241.12 mm
  =245 mm (rounded up)
<table>
<thead>
<tr>
<th>Surface course or Wearing course</th>
<th>Upper pavement to HD26</th>
</tr>
</thead>
<tbody>
<tr>
<td>Binder course</td>
<td></td>
</tr>
<tr>
<td>Base</td>
<td></td>
</tr>
<tr>
<td>Subbase</td>
<td>Foundation to HD25 and IAN 73/06 Rev1</td>
</tr>
<tr>
<td>+ Capping layer</td>
<td></td>
</tr>
<tr>
<td>Subgrade (soil)</td>
<td></td>
</tr>
</tbody>
</table>

NOTE: Base could be in two layers (i.e. upper base & lower base). Subbase could be in two layers (i.e. upper Subbase & lower Subbase)
Location: Ex12 - Performance Foundation Design (Foundation Class 1)

The structural design of bituminous roads

The Highways Agency Design Manual for Roads and Bridges HD26/06 will be adopted for the upper pavement design. This standard provides details of permitted materials and thickness for the construction of pavements for new trunk roads. The total thickness of the combined bituminous layer will be determined from Fig 2.1 - Design Thickness for Flexible Pavements. The upper pavement thickness entails the surface course, binder course and base. The pavement foundation entails the Subbase and Capping over the Subgrade (Soil). The overall thickness of the pavement foundation will be determined in accordance with HD25 assuming either Restricted Foundation Design (Classes 1, 2 & 3) or Performance Foundation Design (Classes 1, 2, 3 & 4). Interim Advice Note IAN 73/06 Rev 1 (2009), will be utilised by this proforma as currently HD25 is in DRAFT form. Standard HD26/06 (Chapter 3 under Materials section) states that all materials within 450mm of the road surface, where the mean annual frost index (MAFI) of the site is ≥ 50 must be frost susceptible in the long-term. Where the MAFI is <50 the thickness of non-frost susceptible material may be 350mm.

Design Criteria

The designs are based on the design manual for roads and bridges, which in turn is based on the work of the Transport Research Laboratory. This design manual is only applicable to roads to be built in the UK and only for trunk roads, motorways and other multi-laned roads.

When comparing the relative benefits of the types of road, it is necessary to compare both over a similar time period. This is normally the design life, which is typically 20 years for flexible pavements. As such, it is necessary to take into consideration all maintenance costs.

Upper pavement overall design thickness

Design life (flexible pavement) life=20 years
Design traffic in msa units dtr=20 msa
Upper pavement design thickness dt=TABLE 3 for dtr=20
                           =320 mm
                           =325 mm (rounded up)

Upper pavement design options

(A) The following three options use HDM50 for binder & base courses

Option 1:
Surfacing (wearing course) 45 mm Hot Rolled Asphalt (HRA)
Binder course 55 mm High Density Macadam (HDM50)
Base 225 mm High Density Macadam (HDM50)
**Flexible pavement design to HD26/06 & HD25 (IAN 73)**

Option 2:
- **Surfacing (wearing course)**: 45 mm Hot Rolled Asphalt (HRA)
- **Base**: 280 mm High Density Macadam (HDM50)

Option 3:
- **Surfacing (wearing course)**: 50 mm Porous Asphalt (PA)
- **Binder course**: 60 mm High Density Macadam (HDM50)
- **Base**: 245 mm High Density Macadam (HDM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

(B) The following three options use DBM50 for binder & base courses

Option 4:
- **Surfacing (wearing course)**: 45 mm Hot Rolled Asphalt (HRA)
- **Binder course**: 55 mm Dense Bitumen Macadam (DBM50)
- **Base**: 225 mm Dense Bitumen Macadam (DBM50)

Option 5:
- **Surfacing (wearing course)**: 45 mm Hot Rolled Asphalt (HRA)
- **Base**: 280 mm Dense Bitumen Macadam (DBM50)

Option 6:
- **Surfacing (wearing course)**: 50 mm Porous Asphalt (PA)
- **Binder course**: 60 mm Dense Bitumen Macadam (DBM50)
- **Base**: 245 mm Dense Bitumen Macadam (DBM50)

Choosing a PA surfacing with a standard 50mm thickness, the contribution to design thickness from PA will be taken to be 20mm only.

**Foundation design**

Performance Foundation Design with Capping only layer provided. The design traffic is ≤ 20 million standard axles (msa). The foundation completed surface must meet all the relevant criteria in Series 700 of the specification.

- Subgrade CBR (%): CBR=3 %
- For performance design, the CBR must be converted to the Subgrade Stiffness Modulus (E) using the expression derived from work on certain soils (Powell et al, 1984), where CBR is as a % value.
- Subgrade Stiffness Modulus: $E=17.6 \times (CBR)^{0.64}=35.553$ MPa
- Assumed Capping layer stiffness: $Cst=50$ MPa
- Subbase layer thickness: $clt=TABLE \ 4.21$ for $E=35.553$
  - $=462.26$ mm
  - $=465$ mm (rounded up)
NOTE: Base could be in two layers (i.e. upper base & lower base)
Location: Example of soil pressures due to coal stack

Soil pressures due to coal and other stacks

It is sometimes necessary to pile in the vicinity of coal and other stacks. The stack produces a vertical and horizontal stress and accompanying shear stress thus:

\[
\begin{align*}
\text{N.B. when } Y \text{ increases, } & \text{ stress } \text{ABS}(P_x) \text{ decreases.} \\
\text{Consider the pressure at point } G, & \text{ distance H below soil surface } \\
& \text{H varies from } H(1) \text{ to } H(nh). \\
\text{Density of stack } & \gamma = 11 \text{ kN/m}^3 \\
\text{Height of stack } & h_s = 6 \text{ m} \\
\text{Distance between A & B } & a_b = 7.2 \text{ m} \\
\text{Distance between B & C } & b_c = 15 \text{ m} \\
\text{Distance from stack c.l. to } G & x_1 = 24.6 \text{ m} \\
\text{Number of values of } H() & n_h = 2
\end{align*}
\]
**Location: Example 5.1(b)**

**Stresses due to concentrated load**

![Stress Diagram]

Concentrated load \( Q = 1500 \text{ kN} \)
Depth to point X \( z' = 5 \text{ m} \)
Horizontal distance \( r = 2 \text{ m} \)
Poisson's ratio (typically 0.5) \( \nu = 0.5 \)

Radial stress: 
\[
sigma_r = \frac{Q}{2\pi} \left( 3r^2z'/d^{2.5} - (1-2\nu)/(d+z'*d^{0.5}) \right)
\]
\[
= 3.1628 \text{ kN/m}^2
\]

Tangential stress: 
\[
sigma_t = -\frac{Q}{2\pi} \left( 1-2\nu \right) \left( z'/d^{1.5} - 1/(d+z'*d^{0.5}) \right)
\]
\[
= 0 \text{ kN/m}^2
\]

Shear stress: 
\[
tau_z = 3Q/(2\pi) \left( r^2z'^2/d^{2.5} \right)
\]
\[
= 7.9069 \text{ kN/m}^2
\]
Location: Example 5.1(b)

**Vertical stress due to general loading**

- General loading (positive into figure) \( q(1) \rightarrow q(n) \) at coordinates \( x(1), y(1), 0 \) to \( x(n), y(n), 0 \). Position where stresses required may be anywhere within positive quadrant at coordinates \( x, y, z \) (\( z \) is positive into figure).

General loading modelled by 'n' concentrated loads.

X coordinate where stress reqd: \( x = 1 \) m
Y coordinate where stress reqd: \( y = 1 \) m
Z coordinate where stress reqd: \( z = 5 \) m
Number of concentrated loads: \( n = 4 \)
Magnitude of load No. 1: \( q(1) = 375 \) kN
X coordinate of load position: \( x(1) = 0.5 \) m
Y coordinate of load position: \( y(1) = 1.5 \) m
Magnitude of load No. 2: \( q(2) = 375 \) kN
X coordinate of load position: \( x(2) = 1.5 \) m
Y coordinate of load position: \( y(2) = 1.5 \) m
Magnitude of load No. 3: \( q(3) = 375 \) kN
X coordinate of load position: \( x(3) = 0.5 \) m
Y coordinate of load position: \( y(3) = 0.5 \) m
Magnitude of load No. 4: \( q(4) = 375 \) kN
X coordinate of load position: \( x(4) = 1.5 \) m
Y coordinate of load position: \( y(4) = 0.5 \) m
Total vertical stress: \( \sigma_{zz} = 27.264 \) kN/m²
Location: Example 5.1

Elastic stresses resulting from UDL on rectangular area

Position where stresses required may be anywhere within positive quadrant at coordinates x, y, z (z is positive downwards).

Length of foundation X = 2 m
Breadth of foundation Y = 2 m
Total load on foundation Q = 1500 kN
X coordinate where stresses reqd. x = 1 m
Y coordinate where stresses reqd. y = 1 m
Depth where stresses required z = 5 m
Number of strips parallel to X nx = 4
Number of strips parallel to Y ny = 4
UDL from rectangular foundation q = Q / (X * Y) = 375 kN/m²
Load on each element delQ = Q / (nx * ny) = 93.75 kN
Vertical stress at coords (x, y, z) oz = 26.955 kN/m²
Location: Typical calc for straight shafted bored pile in clay

Vertical loads on straight bored piles in clay

Unfactored load on top of pile \( W = 550 \text{ kN} \)
Pile diameter \( D = 400 \text{ mm} \)
Concrete cube strength \( f_{cu} = 35 \text{ N/mm}^2 \)
Adhesion factor \( \alpha = 0.45 \)
Factor of safety for friction \( FOS_f = 2.5 \)
Factor of safety for end bearing \( FOS_b = 2.5 \)
Cut off level \( \text{cut} = 50.4 \text{ m} \)
Level at underside of made ground \( \text{lev}(0) = 48.4 \text{ m} \)
Shear strength u/side made ground \( C(0) = +C(0) \text{ kN/m}^2 \)
Number of soil increments \( n = 3 \)
Level at u/side increment 1 \( \text{lev}(1) = 40.4 \text{ m} \)
Shear strength at u/side inc 1 \( C(1) = 90 \text{ kN/m}^2 \)
Level at u/side increment 2 \( \text{lev}(2) = 30.4 \text{ m} \)
Shear strength at u/side inc 2 \( C(2) = 140 \text{ kN/m}^2 \)
Level at u/side increment 3 \( \text{lev}(3) = 24.4 \text{ m} \)
Shear strength at u/side inc 3 \( C(3) = 200 \text{ kN/m}^2 \)

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Pile load</th>
<th>550 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile capacity</td>
<td>550 kN</td>
</tr>
<tr>
<td>Pile diameter</td>
<td>400 mm</td>
</tr>
<tr>
<td>Cut off level</td>
<td>50.4 m</td>
</tr>
<tr>
<td>Pile length</td>
<td>22.2 m</td>
</tr>
<tr>
<td>Concrete cube strength</td>
<td>35 N/mm²</td>
</tr>
</tbody>
</table>
Vertical loads on straight bored piles in clay

Unfactored load on top of pile \( W = 550 \text{ kN} \)
Pile diameter \( D = 400 \text{ mm} \)
Char cylinder compress strength \( f_{ck} = 28 \text{ N/mm}^2 \) (concrete)
Concrete cube strength \( f_{cu} = 35 \text{ N/mm}^2 \)
Adhesion factor \( \alpha = 0.45 \)
Factor of safety for friction \( \text{FOS}_f = 2.5 \)
Factor of safety for end bearing \( \text{FOS}_b = 2.5 \)
Cut off level \( \text{cut} = 50.4 \text{ m} \)
Level at underside of made ground \( \text{lev}(0) = 48.4 \text{ m} \)
Shear strength u/side made ground \( C(0) = +C(0) \text{ kN/m}^2 \)
Number of soil increments \( n = 3 \)
Level at u/side increment 1 \( \text{lev}(1) = 40.4 \text{ m} \)
Shear strength at u/side inc 1 \( C(1) = 90 \text{ kN/m}^2 \)
Level at u/side increment 2 \( \text{lev}(2) = 30.4 \text{ m} \)
Shear strength at u/side inc 2 \( C(2) = 140 \text{ kN/m}^2 \)
Level at u/side increment 3 \( \text{lev}(3) = 24.4 \text{ m} \)
Shear strength at u/side inc 3 \( C(3) = 200 \text{ kN/m}^2 \)

DESIGN SUMMARY

<table>
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<tr>
<td>Cut off level</td>
<td>50.4 m</td>
</tr>
<tr>
<td>Pile length</td>
<td>22.2 m</td>
</tr>
<tr>
<td>Char cylinder strength</td>
<td>28 N/mm² (concrete)</td>
</tr>
<tr>
<td>Concrete cube strength</td>
<td>35 N/mm²</td>
</tr>
</tbody>
</table>
Location: Large diameter belled pile in clay

Large diameter belled piles in clay

Safe working load on pile is least of:

\[ \text{SWL}(1) = \frac{(U_f + U_b)}{2.5} \]
\[ \text{SWL}(2) = \frac{U_f + U_b}{3} \]
\[ \text{SWL}(3) = U_f + lws \times A_b \]

where

- \( A_b \) = area of bell
- base stress is limited to \( lws \) kN/m² working.

Shaft friction value alpha varies from 0.25 to 0.35. Advice sought from soils investigation specialist.

Ignore shaft cohesion for length of 2 x bell diameter.

Bell diameter BD normally 2.5 x shaft diameter max 3 x shaft diameter.

Bearing capacity factor typically 0.75x9 used for circular piles. Advice sought from soils investigation specialist.

Unfactored load on top of pile \( W = 2500 \text{ kN} \)

Pile diameter \( D = 0.9 \text{ m} \)

Bell diameter \( BD = 2.25 \text{ m} \)

Limiting working stress on base \( lws = 500 \text{ kN/m}^2 \)

Concrete cube strength \( f_{cu} = 35 \text{ N/mm}^2 \)

Shaft friction value \( \alpha = 0.35 \)

Bearing capacity factor \( N_c = 6.75 \)

Soil properties

Each soil layer must be greater than 4.5 m for design method used.

The levels at underside of each of 'n' soil increments are given with corresponding shear strengths of clay.

Cut off level \( \text{cut}=50 \text{ m} \)

Level at underside of made ground \( \text{lev}(0)=48 \text{ m} \)

Shear strength u/side made ground \( C(0)=70 \text{ kN/m}^2 \)

Number of soil increments \( n=3 \)

Level at u/side increment 1 \( \text{lev}(1)=40 \text{ m} \)

Shear strength at u/side inc 1 \( C(1)=90 \text{ kN/m}^2 \)

Level at u/side increment 2 \( \text{lev}(2)=30 \text{ m} \)
Shear strength at u/side inc 2 \( C(2)=140 \text{ kN/m}^2 \)
Level at u/side increment 3 \( lev(3)=20 \text{ m} \)
Shear strength at u/side inc 3 \( C(3)=250 \text{ kN/m}^2 \)

**DESIGN SUMMARY**

- Pile load: 2500 kN
- Pile capacity: 2501.9 kN
- Pile diameter: 0.9 m
- Pile bell diameter: 2.25 m
- Cut off level: 50 m
- Pile length: 23.1 m
- Concrete cube strength: 35 N/mm²
Location:

Large diameter belled piles in clay

Safe working load on pile is least of:  
\[
\text{SWL}(1) = \frac{(U_f + U_b)}{2.5} \\
\text{SWL}(2) = \frac{U_f + U_b}{3} \\
\text{SWL}(3) = U_f + \frac{l_{ws} \times A_b}{2}
\]
where 
\[
A_b = \text{area of bell and where base stress is limited to} \\
l_{ws} \text{ kN/m}^2 \text{ working.}
\]

Shaft friction value alpha varies from 0.25 to 0.35. Advice sought from soils investigation specialist.

Ignore shaft cohesion for length of 2 x bell diameter.

Bell diameter BD normally 2.5 x shaft diameter max 3 x shaft diameter.

Bearing capacity factor typically 0.75x9 used for circular piles. Advice sought from soils investigation specialist.

Unfactored load on top of pile \( W = 2300 \text{ kN} \)

Pile diameter \( D = 0.9 \text{ m} \)

Bell diameter \( BD = 2.25 \text{ m} \)

Limiting working stress on base \( l_{ws} = 500 \text{ kN/m}^2 \)

Overall pile length \( L_o = 22.5 \text{ m} \)

Concrete cube strength \( f_{cu} = 35 \text{ N/mm}^2 \)

Shaft friction value \( \alpha = 0.35 \)

Bearing capacity factor \( N_c = 6.75 \)

Soil properties

Each soil layer must be greater than 4.5 m for design method used.

The levels at underside of each of 'n' soil increments are given with corresponding shear strengths of clay.

Cut off level \( \text{cut} = 50 \text{ m} \)

Level at underside of made ground \( l_{e} = 48 \text{ m} \)

Shear strength u/side made ground \( C(0) = 70 \text{ kN/m}^2 \)

Number of soil increments \( n = 3 \)

Level at u/side increment 1 \( l_{e}(1) = 40 \text{ m} \)

Shear strength at u/side inc 1 \( C(1) = 90 \text{ kN/m}^2 \)
Level at u/side increment 2 \( \text{lev}(2) = 30 \text{ m} \)
Shear strength at u/side inc 2 \( C(2) = 140 \text{ kN/m}^2 \)
Level at u/side increment 3 \( \text{lev}(3) = 20 \text{ m} \)
Shear strength at u/side inc 3 \( C(3) = 250 \text{ kN/m}^2 \)

**DESIGN SUMMARY**

- **Pile load**: 2300 kN
- **Pile capacity**: 2399.9 kN
- **Pile diameter**: 0.9 m
- **Pile bell diameter**: 2.25 m
- **Cut off level**: 50 m
- **Pile length**: 22.5 m
- **Concrete cube strength**: 35 N/mm²
Location: Crane type 1

Reference to symbols

- **P1** = load on pile 1
- **P2** = load on pile 2
- **P3** = load on pile 3
- **P4** = load on pile 4
- **L1** = load on crane leg 1
- **L2** = load on crane leg 2
- **L3** = load on crane leg 3
- **L4** = load on crane leg 4
- **O** = pile position

Dimensions:
- **a** = 1 m
- **b** = 2 m
- **c** = 1 m
- **d** = 2 m
- **m** = 1 m
- **A** = a+b = 3 m
- **B** = c+d = 3 m
- **e** = c+m = 2 m
- **f** = a+m = 2 m
- **g** = B-e = 1 m
- **h** = A-f = 1 m

Loads:
- **L1** = 1200 kN
- **L2** = 50 kN
- **L3** = -10 kN
- **L4** = 50 kN

Design Summary:
- Load on pile 1 from crane: 554.44 kN
- Load on pile 2 from crane: 292.22 kN
- Load on pile 3 from crane: 151.11 kN
- Load on pile 4 from crane: 292.22 kN
Location: Crane type 2

Reference to symbols

- P1 = load on pile 1
- P2 = load on pile 2
- P3 = load on pile 3
- P4 = load on pile 4
- L1 = load on crane leg 1
- L2 = load on crane leg 2
- L3 = load on crane leg 3
- L4 = load on crane leg 4
- O = pile position

Dimensions
- \( a = 0.9 \) m
- \( b = 2.1 \) m
- \( c = 0.9 \) m
- \( d = 2.1 \) m
- \( m = 1.2 \) m
- \( A = a + b = 3 \) m
- \( B = c + d = 3 \) m
- \( e = c + m = 2.1 \) m
- \( f = a + m = 2.1 \) m
- \( g = B - e = 0.9 \) m
- \( h = A - f = 0.9 \) m

Loads
- \( L1 = 600 \) kN
- \( L2 = 600 \) kN
- \( L3 = -50 \) kN
- \( L4 = -50 \) kN

Design summary
- Load on pile 1 from crane: 405 kN
- Load on pile 2 from crane: 405 kN
- Load on pile 3 from crane: 145 kN
- Load on pile 4 from crane: 145 kN
Location: Pile group analysis

A simple elastic analysis of a vertical load on a group of piles can be determined from the formula:

\[ P_n = W/N + \text{or} - (AY_n) + \text{or} - (BX_n) \]

where

\[ A = W(ExIy - EyIxy) / (IxIy - Ixy^2) \]
\[ B = W(EyIx - ExIxy) / (IxIy - Ixy^2) \]

W is the resultant vertical load on the pile group
N is the number of piles in the group
Xn and Yn are the co-ordinates of the individual piles relative to the centroidal XX and YY axes

Number of piles                   N=5
Pile No | X Co-Ord (m) | Y Co-Ord (m) |
---|---|---|
1 | 0.5 | 0.5 |
2 | 2.5 | 0.5 |
3 | 0.5 | 2.5 |
4 | 2.5 | 2.5 |
5 | 1.5 | 1.5 |

Number of loads                   NL=1
Position | Weight (kN) | X Co-Ord (m) | Y Co-Ord (m) |
---|---|---|---|
1 | 1000 | 1.5 | 1.5 |

Sum of X Co ordinates             SX=7.5
Sum of Y Co ordinates             SY=7.5
Pile centroid X Co ord             XB=SX/N=1.5
Pile centroid Y Co ord             YB=SY/N=1.5

Co-ordinates of piles relative to XX and YY axes
Pile No | X Co-Ordinate (m) | Y Co-Ordinate (m) |
---|---|---|
1 | -1 | -1 |
2 | 1  | -1 |
3 | -1 | 1  |
4 | 1  | 1  |
5 | 0  | 0  |

2nd Moment of area of Pile Grp     Ix=4 m^4
                                  Iy=4 m^4
Product of Area moments about centroidal axes  XY=0

Forces in piles are as follows:

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<th>Pile No</th>
<th>Force (kN)</th>
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<td>2</td>
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<td>3</td>
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<td>5</td>
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</table>
**Location:** Example from Reinforced Concrete Designers Handbook

**Loads on group of vertical piles**

Notation is taken from Reinforced Concrete Designer's Handbook by Chas Reynolds 1957 edtn. 

- \( h \) is depth to full fixity, thus pile bends in double curvature due to load \( H \).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tr>
<td>Number of rows of piles</td>
<td>( n = 4 )</td>
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<tr>
<td>Vertical load on group</td>
<td>( W = 800 ) kN</td>
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<tr>
<td>Distance of line of ( W ) from datum</td>
<td>( e' = 5.25 ) m</td>
</tr>
<tr>
<td>Horizontal load on group</td>
<td>( H = 100 ) kN</td>
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<tr>
<td>Height from cap to full fixity</td>
<td>( h = 4.8 ) m</td>
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<tr>
<td>Weight of pile</td>
<td>( w_p = 0 ) kN</td>
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<tr>
<td>Number of piles in row 1</td>
<td>( N(1) = 1 )</td>
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<tr>
<td>Distance of row from datum</td>
<td>( a(1) = 0 ) m</td>
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<tr>
<td>Number of piles in row 2</td>
<td>( N(2) = 1 )</td>
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<tr>
<td>Distance of row from datum</td>
<td>( a(2) = 3 ) m</td>
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<tr>
<td>Number of piles in row 3</td>
<td>( N(3) = 1 )</td>
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<td>Distance of row from datum</td>
<td>( a(3) = 6 ) m</td>
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<td>Number of piles in row 4</td>
<td>( N(4) = 1 )</td>
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<td>Distance of row from datum</td>
<td>( a(4) = 9 ) m</td>
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<td>Moment on pile group</td>
<td>( M = W \cdot e - H \cdot h / 2 = 360 ) kNm</td>
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<tr>
<td>Shearing force on any pile</td>
<td>( S = H / \sigma N = 25 ) kN</td>
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<tr>
<td>Bending moment on any pile</td>
<td>( B = 0.5 \cdot S \cdot h = 60 ) kNm</td>
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Location: Ex. in Fig 39c Reinforced Concrete Designers Handbook 1957

Loads on group of piles raking in the loading plane

Notation is taken from Reinforced Concrete Designer's Handbook by Chas Reynolds 1957 edtn.

Theta is angle of pile with vertical (shown positive for P1 and -ve for Pn in figure).
W & H are positive in direction shown.

P1          P2           Pn-1        Pn
A1          A2           An-1        An
L1          L2           Ln-1        Ln

W=80 kN
H=10 kN
A(1)=1 m²
L(1)=40 m
sw(1)=0 kN
x(1)=0 m
th(1)=11.31°
A(2)=1 m²
L(2)=40 m
sw(2)=0 kN
x(2)=0 m
th(2)=-11.31°
A(3)=1 m²
L(3)=40 m
sw(3)=0 kN
x(3)=18.667 m
th(3)=11.31°
A(4)=1 m²
L(4)=40 m
sw(4)=0 kN
x(4)=18.667 m
th(4)=-11.31°
Moment about elastic centre \[ M = W(e - x_0) + (y_0 - f)H = 199.72 \text{ kNm} \]

Axial load on pile 1 \[ P_x = 27.688 \text{ kN} \]
Axial load on pile 2 \[ P_x = 2.1931 \text{ kN} \]
Axial load on pile 3 \[ P_x = 38.599 \text{ kN} \]
Axial load on pile 4 \[ P_x = 13.104 \text{ kN} \]
Location: Sheet retaining wall to South of new building

Single tied wall

- **d5 kN/m² surcharge**
  - Downhill (passive) side.
  - Uphill (active) side.
  - Surcharge **d6 kN/m²**
  - Passive water level
  - Active water level

Retained height **h1=5.5 m**

Free-earth support method: It assumes the wall is embedded a sufficient distance into the soil to prevent translation, but is able to rotate at toe providing the wall with a pinned support. With this method the factor of safety against rotation will be taken as 1. Once the pile embedment depth required for equilibrium is found this will be multiplied by a factor of safety on embedment (Fd) to obtain the design embedment depth.

- Safety factor on embedment **Fd=1.2**
- Inclination of active ground **b=0°**
- Active water level **p1=30.5 m**
- Passive water level **p2=28 m**

- Increment of calculation **d1=0.1 m**
- Number of horizontal line loads **d3=0**
- Minimum fluid pressure **d4=4.7 kN/m³**
- Active vertical surcharge **d5=10 kN/m²**
- Passive vertical surcharge **d6=0 kN/m²**
- Number of soil strata **N=4**

- **Soil data for stratum 1 of 4**
  - Top level **TL(1)=35 m**
  - Bulk density **Gam(1)=18 kN/m³**
  - Angle of internal friction **Phi(1)=25°**
  - Cohesion **C'(1)=0 kN/m²**
  - Angle of active wall friction **Da(1)=16.7°**
  - Angle of passive wall friction **Dp(1)=3°**
  - Wall adhesion **Cw(1)=0 kN/m²**

- **Soil data for stratum 2 of 4**
  - Top level **TL(2)=31 m**
  - Bulk density **Gam(2)=20 kN/m³**
  - Angle of internal friction **Phi(2)=6°**
  - Cohesion **C'(2)=0 kN/m²**
  - Angle of active wall friction **Da(2)=4°**
  - Angle of passive wall friction **Dp(2)=3°**
  - Wall adhesion **Cw(2)=0 kN/m²**
• Soil data for stratum 3 of 4
Top level TL(3)=25.5 m
Bulk density Gam(3)=20 kN/m²
Angle of internal friction Phi(3)=24°
Cohesion C'(3)=0 kN/m²
Angle of active wall friction Da(3)=16°
Angle of passive wall friction Dp(3)=3°
Wall adhesion Cw(3)=0 kN/m²

• Soil data for stratum 4 of 4
Top level TL(4)=24 m
Bulk density Gam(4)=20 kN/m²
Angle of internal friction Phi(4)=40°
Cohesion C'(4)=0 kN/m²
Angle of active wall friction Da(4)=26.7°
Angle of passive wall friction Dp(4)=3°
Wall adhesion Cw(4)=0 kN/m²

Tie level TiL=32.5 m

Earth pressure coefficients for both sides

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Summary

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- Unfactored force in tie at level 32.5: 262.66 kN/m
- Factored force in tie at level 32.5: 525.31 kN/m
  (factor is 2.0 - see CIRIA Report 104)
- Founding level for unity F.O.S.: 23.689 m
- Founding level for required F.O.S.: 22.3 m
- Overall length of sheet retaining wall: 12.7 m
- Unfactored BM for section design: 566.47 kNm/m
- Factored BM for section design: 849.71 kNm/m
  (factor is 1.5 - see CIRIA Report 104)
Location: Anchored sheet-pile wall Example 8.14 Basic Soil Mechanics

Single tied wall

\[ \text{d5 kN/m}^2 \text{ surcharge} \]

Downhill (passive) side. \hspace{1cm} Uphill (active) side.

\[ \text{Surcharge} \ d6 \ kN/m^2 \]

\[ \text{Passive water level} \]

Active water level

Retained height \[ h1=9 \text{ m} \]

Free-earth support method: It assumes the wall is embedded a sufficient distance into the soil to prevent translation, but is able to rotate at toe providing the wall with a pinned support. With this method the factor of safety against rotation will be taken as 1. Once the pile embedment depth required for equilibrium is found this will be multiplied by a factor of safety on embedment (Fd) to obtain the design embedment depth.

Safety factor on embedment \[ Fd=1.2 \]

Inclination of active ground \[ b=0^\circ \]

Active water level \[ p1=0 \text{ m} \]

Passive water level \[ p2=0 \text{ m} \]

Increment of calculation \[ d1=0.1 \text{ m} \]

Number of horizontal line loads \[ d3=0 \]

Minimum fluid pressure \[ d4=0 \text{ kN/m}^3 \]

Active vertical surcharge \[ d5=0 \text{ kN/m}^2 \]

Passive vertical surcharge \[ d6=0 \text{ kN/m}^2 \]

Number of soil strata \[ N=1 \]

- Soil data for stratum 1 of 1

Top level \[ TL(1)=100 \text{ m} \]

Bulk density \[ \text{Gam}(1)=20 \text{ kN/m}^3 \]

Angle of internal friction \[ \text{Phi}(1)=28^\circ \]

Cohesion \[ C'(1)=0 \text{ kN/m}^2 \]

Angle of active wall friction \[ Da(1)=18.67^\circ \]

Angle of passive wall friction \[ Dp(1)=14^\circ \]

Wall adhesion \[ Cw(1)=0 \text{ kN/m}^2 \]

K factors for stratum

\[ \text{Ka}(1)=0.30438 \]
\[ \text{Kac}(1)=1.1034 \]
\[ \text{Kp}(1)=4.292 \]
\[ \text{Kpc}(1)=4.0972 \]

Tie level \[ TiL=98.5 \text{ m} \]
# Earth pressure coefficients for both sides

(K factors specified directly)

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Unfactored force in tie at level 98.5: 131.97 kN/m
Factored force in tie at level 98.5: 263.93 kN/m
(factor is 2.0 - see CIRIA Report 104)

Founding level for unity F.O.S.: 88.484 m
Founding level for required F.O.S.: 87.76 m
Overall length of sheet retaining wall: 12.24 m
Unfactored BM for section design: 381.33 kNm/m
Factored BM for section design: 572 kNm/m
(factor is 1.5 - see CIRIA Report 104)
Location: Cantilever sheet-pile wall Ex 8.13 Basic Soil Mechanics

Cantilever wall

Downhill (passive) side.

\[ d_5 \text{ kN/m}^2 \text{ surcharge} \]

Surcharge \( d_6 \) kN/m²

\[ p_2 /\//\// \]

Passive water level

Active water level

Density of water=9.8067 kN/m³

Retained height \( h_1 = 3 \) m

Free-earth support method: It assumes the wall is embedded a sufficient distance into the soil to prevent translation, but is able to rotate at toe providing the wall with a pinned support. With this method the factor of safety against rotation will be taken as 1. Once the pile embedment depth required for equilibrium is found this will be multiplied by a factor of safety on embedment (Fd) to obtain the design embedment depth.

Safety factor on embedment \( F_d = 1.2 \)

Inclination of active ground \( b = 0^\circ \)

Active water level \( p_1 = 0 \) m

Passive water level \( p_2 = 0 \) m

Increment of calculation \( d_1 = 0.1 \) m

Number of horizontal line loads \( d_3 = 0 \)

Minimum fluid pressure \( d_4 = 0 \) kN/m³

Active vertical surcharge \( d_5 = 0 \) kN/m²

Passive vertical surcharge \( d_6 = 0 \) kN/m²

Number of soil strata \( N = 1 \)

- Soil data for stratum 1 of 1
  - Top level \( TL(1) = 100 \) m
  - Bulk density \( \Gamma_{\text{m}}(1) = 20 \) kN/m³
  - Angle of internal friction \( \Phi(1) = 30^\circ \)
  - Cohesion \( C'(1) = 0 \) kN/m²
  - Angle of active wall friction \( D_a(1) = 0^\circ \)
  - Angle of passive wall friction \( D_p(1) = 0^\circ \)
  - Wall adhesion \( C_w(1) = 0 \) kN/m²
  - K factors for stratum
    - \( K_a(1) = 0.448 \)
    - \( K_{ac}(1) = 1.338 \)
    - \( K_p(1) = 2.232 \)
    - \( K_{pc}(1) = 2.9879 \)
Earth pressure coefficients for both sides

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Summary

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Founding level for rotational stability 92.7 m
Resultant shear difference at this level 173.96 kN/m
Passive pressure available on active side 287.34 kN/m²
Additonal length required for toe shear 0.6054 m
Founding level required 91.12 m
Overall length of sheet retaining wall 8.88 m
Unfactored BM for section design 132.3 kNm/m
Factored BM for section design 198.44 kNm/m
(factor is 1.5 - see CIRIA Report 104)
Unfactored BM for stability 132.3 kNm/m
Factored stability BM for section design 132.3 kNm/m
(factor is 1.0 - see CIRIA Report 104)
NOTE: The overall length of the sheet retaining wall is the actual length required and includes the additional length required for toe shear.
Location: Tied Sheet Retaining Wall

Single tied wall

\[d_5 \text{ kN/m}^2 \text{ surcharge}\]

Downhill (passive) side.  \(d_5\) \[\text{ kN/m}^2\] surcharge

\(p_2\) \[\text{ kN/m}^2\]  \(p_1\) \[\text{ kN/m}^2\]  \(h_1\) \[\text{ m}\]

Uphill (active) side.

\(\text{Active water level}\)

\(\text{Density of water}=9.8067 \text{ kN/m}^3\)

Retained height \(h_1=2.2\) m

Free-earth support method: It assumes the wall is embedded a sufficient distance into the soil to prevent translation, but is able to rotate at toe providing the wall with a pinned support. With this method the factor of safety against rotation will be taken as 1. Once the pile embedment depth required for equilibrium is found this will be multiplied by a factor of safety on embedment \(F_d\) to obtain the design embedment depth.

Safety factor on embedment \(F_d=1.2\)

Inclination of active ground \(b=25^\circ\)

Active water level \(p_1=100.5\) m

Passive water level \(p_2=99.7\) m

Increment of calculation \(d_1=0.1\) m

Number of horizontal line loads \(d_3=0\)

Minimum fluid pressure \(d_4=4.7\) kN/m\(^3\)

Active vertical surcharge \(d_5=0\) kN/m\(^2\)

Passive vertical surcharge \(d_6=0\) kN/m\(^2\)

Number of soil strata \(N=1\)

- Soil data for stratum 1 of 1

Top level \(T_L(1)=102.2\) m

Bulk density \(\gamma_m(1)=19.25\) kN/m\(^3\)

Angle of internal friction \(\Phi(1)=30^\circ\)

Cohesion \(C'(1)=0\) kN/m\(^2\)

Angle of active wall friction \(D_a(1)=15^\circ\)

Angle of passive wall friction \(D_p(1)=0^\circ\)

Wall adhesion \(C_w(1)=0\) kN/m\(^2\)

Tie level \(T_{IL}=101.7\) m
Earth pressure coefficients for both sides

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(Passive coefficients from Muller-Breslau equations)

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Unfactored force in tie at level 101.7: 17.682 kN/m
Factored force in tie at level 101.7: 35.365 kN/m
(factor is 2.0 - see CIRIA Report 104)
Founding level for unity F.O.S.: 98.331 m
Founding level for required F.O.S.: 97.84 m
Overall length of sheet retaining wall: 4.36 m
Unfactored BM for section design                      14.477 kNm/m
Factored BM for section design                        21.715 kNm/m
(factor is 1.5 - see CIRIA Report 104)
Location: Cantilever sheet-pile wall

Cantilever wall

Downhill (passive) side.                  Uphill (active) side.

Surcharge \(d_6\) kN/m\(^2\)          h_1\]

\(p_2\)                             \(p_1\)

Passive water level                Active water level

Retained height \(h_1=3\) m

Free-earth support method: It assumes the wall is embedded a sufficient distance into the soil to prevent translation, but is able to rotate at toe providing the wall with a pinned support. The factor of safety against rotation will be taken as 2. With this method the calculations are carried out to a greater depth until the rotational factor of safety is achieved.

Factor of safety against rotation \(w=2\)
Inclination of active ground \(b=0^\circ\)
Active water level \(p_1=0\) m
Passive water level \(p_2=0\) m

Increment of calculation \(d_1=0.1\) m
Number of horizontal line loads \(d_3=0\)
Minimum fluid pressure \(d_4=0\) kN/m\(^3\)
Active vertical surcharge \(d_5=0\) kN/m\(^2\)
Passive vertical surcharge \(d_6=0\) kN/m\(^2\)
Number of soil strata \(N=1\)

• Soil data for stratum 1 of 1
Top level \(TL(1)=100\) m
Bulk density \(\rho(1)=20\) kN/m\(^3\)
Angle of internal friction \(\phi(1)=30^\circ\)
Cohesion \(C'(1)=0\) kN/m\(^2\)
Angle of active wall friction \(\phi_a(1)=0^\circ\)
Angle of passive wall friction \(\phi_p(1)=0^\circ\)
Wall adhesion \(C_w(1)=0\) kN/m\(^2\)
Earth pressure coefficients for both sides

( Passive coefficients from Muller-Breslau equations)

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Founding level for rotational stability 92.3 m  
Resultant shear difference at this level 133.72 kN/m  
Passive pressure available on active side 215.33 kN/m²  
Additional length required for toe shear 0.62098 m  
Founding level required 91.7 m  
Overall length of sheet retaining wall 8.3 m  
Unfactored BM for section design 67.5 kNm/m  
Factored BM for section design 101.25 kNm/m  
(factor is 1.5 - see CIRIA Report 104)  
Unfactored BM for stability 107.36 kNm/m  
Factored stability BM for section design 107.36 kNm/m  
(factor is 1.0 - see CIRIA Report 104)  

Note: The overall length of the sheet retaining wall is the actual length required and includes the additional length required for toe shear.
Location: Laterally loaded pile analysed by Brom's method

Laterally loaded pile

Density of water $\gamma_w=9.8067 \text{ kN/m}^3$

Water level (negative values) \( p_1=-3 \text{ m} \)

Increment of calculation \( d_1=0.05 \text{ m} \)

F.O.S applied to lateral pressure \( f_o_s=3 \)

Number of soil strata \( N=6 \)

- Soil data for stratum 1 of 6
  - Top level of stratum \( T_L(1)=0 \text{ m} \)
  - Bulk density \( \gamma(1)=18 \text{ kN/m}^3 \)
  - Angle of internal friction \( \Phi(1)=25^\circ \)
  - Cohesive shear strength \( C_u(1)=0 \text{ kN/m}^2 \)

- Soil data for stratum 2 of 6
  - Top level of stratum \( T_L(2)=-2 \text{ m} \)
  - Bulk density \( \gamma(2)=20 \text{ kN/m}^3 \)
  - Angle of internal friction \( \Phi(2)=33^\circ \)
  - Cohesive shear strength \( C_u(2)=0 \text{ kN/m}^2 \)

- Soil data for stratum 3 of 6
  - Top level of stratum \( T_L(3)=-3 \text{ m} \)
  - Bulk density \( \gamma(3)=20 \text{ kN/m}^3 \)
  - Angle of internal friction \( \Phi(3)=33^\circ \)
  - Cohesive shear strength \( C_u(3)=0 \text{ kN/m}^2 \)

- Soil data for stratum 4 of 6
  - Top level of stratum \( T_L(4)=-6 \text{ m} \)
  - Bulk density \( \gamma(4)=20 \text{ kN/m}^3 \)
  - Angle of internal friction \( \Phi(4)=36^\circ \)
  - Cohesive shear strength \( C_u(4)=0 \text{ kN/m}^2 \)

- Soil data for stratum 5 of 6
  - Top level of stratum \( T_L(5)=-10 \text{ m} \)
  - Bulk density \( \gamma(5)=22 \text{ kN/m}^3 \)
  - Angle of internal friction \( \Phi(5)=40^\circ \)
  - Cohesive shear strength \( C_u(5)=0 \text{ kN/m}^2 \)
Soil data for stratum 6 of 6

Top level of stratum TL(6) = -20 m  
Bulk density Gam(6) = 22 kN/m²
Angle of internal friction Phi(6) = 40°
Cohesive shear strength Cu(6) = 0 kN/m²

Unfactored vertical surcharge s = 18 kN/m²
Unfactored lateral load at top H = 50 kN
Unfactored moment applied to top M = 10 kNm
Pile dia. resisting lateral load D = 0.9 m

Summary

<table>
<thead>
<tr>
<th>Level of max</th>
<th>Level of bottom</th>
<th>Unfactored max</th>
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<tbody>
<tr>
<td>Level (m)</td>
<td>Pressure (kN/m²)</td>
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<td>Reaction (kN)</td>
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</table>
Location: Sheet piling w. anchor, cohesionless soil (Rowe reduction)

Design of sheet-pile wall

Drained soil density \( \Gamma = 15.9 \text{ kN/m}^3 \)
Saturated soil density \( \Gamma_{Sat} = 19.33 \text{ kN/m}^3 \)
Angle of internal friction \( \phi = 30^\circ \)
Angle of surface friction (delta) \( \delta = 20^\circ \)

For active pressure:
\[
K_{Act} = \left( \frac{CRP}{(\sqrt{SRP}(SRP+CNP*TRP))} \right)^2 = 0.27938
\]

For passive resistance (unfactored):
\[
K_{Pas} = \left( \frac{CRP}{1-\sqrt{SRP/CRP}} \right)^2 = 5.7372
\]
\[
K_{Pas} = K_{Pas}/SF1 = 3.8248
\]

Min. penetration length required \( D_m = L_3 + L_4 = 3.3498 \text{ m} \)
Actual penetration depth adopted \( D = 3.5 \text{ m} \)
Tensile force in anchor \( \text{Force} = P - ((K_{Pas} - K_{Act}) \times G_{Prime}) \times L_4^2 / 2 \)
\[
\text{Maximum bending moment} = 113.63 \text{ kN/m}
\]
\[
\text{Rowe moment-reduction method}
\]

Adopted pile length \( \text{Length} = 15 \text{ m} \)
Most-economic solution with Larssen Section

From Fig 6.10 of Das, for loose sand with log.rho=-2.6924
Limiting value of Md/Mmax CRatio=0.52826
Larssen Section 2 B is satisfactory.
Larssen Section 3 B will also suffice ( MRatio/CRatio=1.3919 )
Larssen Section 4 B will also suffice ( MRatio/CRatio=1.6639 )
Larssen Section 6 will also suffice ( MRatio/CRatio=2.2786 )

Most-economic solution with Frodingham Section

From Fig 6.10 of Das, for loose sand with log.rho=-2.5226
Limiting value of Md/Mmax CRatio=0.46424
Frodingham Section 2 is satisfactory.
Frodingham Section 3 will also suffice ( MRatio/CRatio=1.5 )
Frodingham Section 4 will also suffice ( MRatio/CRatio=1.8407 )
Frodingham Section 5 will also suffice ( MRatio/CRatio=2.1342 )
**Location:** Sheet piling with anchor in cohesive soil

**Design of sheet-pile wall**

- Drained soil density: $\Gamma = 20 \text{ kN/m}^3$
- Saturated soil density: $G_{Sat} = 20 \text{ kN/m}^3$
- Angle of internal friction: $\Phi = 5^\circ$
- Angle of surface friction (delta): $\Delta = 12^\circ$

- Height $L_1$ above water-line: $L_1 = 4 \text{ m}$
- Height from dredge to water-line: $L_2 = 6 \text{ m}$
- Cohesion factor: $\text{Coh} = 40 \text{ kN/m}^2$
- Depth to ground anchor: $A_{Depth} = 2 \text{ m}$
- Passive-resistance safety factor: $SF_1 = 1.5$
- Coulomb coefficients:
  - For active pressure: $K_{Act} = \frac{(CRP/(SQR(SRP*(SRP+CRP*TRD))) + 1)^2}{0.73574}$
  - Min. penetration length required: $MinP = 10.586 \text{ m}$
  - Actual penetration depth adopted: $D = 11 \text{ m}$
  - Tensile force in anchor: $\text{Force} = P - P_6 \times D = 91.922 \text{ kN per metre}$
  - Maximum bending moment: $M_{max} = 26.887 \text{ kNm per metre}$
Location: Sheet piling without anchor in cohesionless soil

Design of sheet-pile wall

Drained soil density \( \text{Gamma} = 15.9 \text{ kN/m}^3 \)
Saturated soil density \( \text{GSat} = 19.33 \text{ kN/m}^3 \)
Angle of internal friction \( \Phi = 30^\circ \)
Angle of surface friction (delta) \( \Delta = 20^\circ \)

COHESIONLESS PASSIVE SOIL: NO GROUND ANCHOR PROVIDED

Height \( L_1 \) above water-line \( L_1 = 4 \text{ m} \)
Height from dredge to water-line \( L_2 = 6 \text{ m} \)
Passive-resistance safety factor \( SF_1 = 1.5 \)

Coulomb coefficients:
For active pressure:
\[ K_{Act} = \left( \frac{CRP}{(\sqrt{SRP*(SRP+CRP*TRD)})} \right)^2 = 0.27938 \]

For passive resistance (unfactored):
\[ K_{Pas} = \left( \frac{CRP}{(1-\sqrt{SPD*SRP/CRD})} \right)^2 = 5.7372 \]
\[ K_{Pas} = K_{Pas}/SF_1 = 3.8248 \]

Min. penetration length required \( D_m = L_3 + L_4 = 9.2288 \text{ m} \)
Actual penetration depth adopted \( D = 10 \text{ m} \)
Location: Ex1 - Default example

Embedded depths of holding down bolts

Using the method adopted in 'Steel Designer's Manual (5th Edition)' and based on the assumptions listed below:

- bolts are arranged symmetrically about major axis
- the failure mode of bolts pulled from a concrete foundation is based on a conical pull-out, such that the surface area is $4.44D^2$, where $D$ is the depth of embedment of the bolts
- holding down bolts should be anchored into the foundation by a washer (anchor) plate or other load distributing member embedded in the concrete
- bolts have been previously checked for combined shear and tension.

Factored design tensile force $F_t=44$ kN

Length of baseplate $D_p=560$ mm
Breadth of baseplate $B_p=200$ mm
Edge distance to bolt centre line $ed=60$ mm

Steel Grade $S\ 275$
Bolt grade 4.6
Concrete strength $f_{cu}=20\ \text{N/mm}^2$

Holding down bolt details

Number of bolts to be used $n=4$
Number of bolts in tension $n_t=2$
Bolt diameter $b_d=20$ mm
Embedded depth of bolts $D=100$ mm
Half the bolts subject to tension.
<table>
<thead>
<tr>
<th><strong>SUMMARY OF</strong></th>
<th><strong>HOLDING</strong></th>
<th><strong>DOWN BOLT</strong></th>
<th><strong>REQUIREMENTS</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of H.D. bolts</td>
<td>Diameter of bolts</td>
<td>Grade of bolts</td>
<td>Edge distance to bolt C.L. 60 mm</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>20 mm</td>
<td>4.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>60 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Embedded depth 100 mm</td>
</tr>
</tbody>
</table>
Embedded depths of holding down bolts

Using the method adopted in 'Joints in Steel Construction - moment connections' and based on the assumptions listed below:

- Anchor plates are used
- The concrete base is checked for punching shear in accordance with BS8110.

Factored design tensile force \( F_t = 674.16 \text{ kN} \)

Length of baseplate \( D_p = 600 \text{ mm} \)
Breadth of baseplate \( B_p = 600 \text{ mm} \)
Edge distance to bolt centre line \( \text{ed} = 75 \text{ mm} \)

Steel Grade \( S \ 275 \)
Bolt grade \( 8.8 \)
Concrete strength \( f_{cu} = 30 \text{ N/mm}^2 \)

**Holding down bolt details**

<table>
<thead>
<tr>
<th>Number of bolts to be used</th>
<th>( n = 8 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bolts in tension</td>
<td>( nt = 4 )</td>
</tr>
<tr>
<td>Bolt diameter</td>
<td>( b_d = 24 \text{ mm} )</td>
</tr>
<tr>
<td>Embedded depth of bolts</td>
<td>( D = 450 \text{ mm} )</td>
</tr>
<tr>
<td>Half the bolts subject to tension.</td>
<td></td>
</tr>
</tbody>
</table>

**Anchor plates**

<table>
<thead>
<tr>
<th>Anchor plate size (8.8 bolts)</th>
<th>( 5d \times 5d \times 0.8d ) (approx)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Assumed cover to reinforcement</td>
<td>( c_v = 50 \text{ mm} )</td>
</tr>
<tr>
<td>Tension reinforcement</td>
<td>( A_{sp} = 0.15 % )</td>
</tr>
</tbody>
</table>

**SUMMARY OF REQUIREMENTS**

<table>
<thead>
<tr>
<th>Number of H.D. bolts</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bolts</td>
<td>24 mm</td>
</tr>
<tr>
<td>Grade of bolts</td>
<td>8.8</td>
</tr>
<tr>
<td>Edge distance to bolt C.L.</td>
<td>75 mm</td>
</tr>
<tr>
<td>Embedded depth</td>
<td>450 mm</td>
</tr>
</tbody>
</table>
Location: Ex3 - Example based on bond length

Embedded depths of holding down bolts

Using the method adopted in 'Joints in Steel Construction - moment connections' and based on the assumptions listed below:

- The anchorage is developed by the bond along the embedded length of the holding bolt
- The bolt is regarded as a reinforcing bar and the requirements are based on BS8110 clauses 3.12.8.3 and 3.12.8.4.

Factored design tensile force: $F_t = 120$ kN
Length of baseplate: $D_p = 600$ mm
Breadth of baseplate: $B_p = 600$ mm
Edge distance to bolt centre line: $e_d = 75$ mm
Steel Grade: S 275
Bolt grade: 4.6
Concrete strength: $f_{cu} = 30$ N/mm²

**Holding down bolt details**

- Number of bolts to be used: $n = 4$
- Number of bolts in tension: $n_t = 4$
- Bolt diameter: $b_d = 24$ mm
- Embedded depth of bolts: $D = 450$ mm
- All bolts subject to tension.

**Anchorage**

Assumed cover to reinforcement: $c_v = 50$ mm
Since $F_t' \leq P_t$ (30 kN ≤ 141.2 kN), tensile load per bolt is allowable.

**SUMMARY OF HOLDING DOWN BOLT REQUIREMENTS**

<table>
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<tr>
<th>Requirement</th>
<th>Value</th>
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<tbody>
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<td>Number of H.D. bolts</td>
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</tr>
<tr>
<td>Diameter of bolts</td>
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</tr>
<tr>
<td>Grade of bolts</td>
<td>4.6</td>
</tr>
<tr>
<td>Edge distance to bolt C.L.</td>
<td>75 mm</td>
</tr>
<tr>
<td>Embedded depth</td>
<td>450 mm</td>
</tr>
</tbody>
</table>

SCALE 5.48 Office 1007 Proforma 796
Location: Ex4 - Four bolts intersecting cones

Embedded depths of holding down bolts

Using the method adopted in 'Steel Designer's Manual (5th Edition)' and based on the assumptions listed below:

- bolts are arranged symmetrically about major axis
- the failure mode of bolts pulled from a concrete foundation is based on a conical pull-out, such that the surface area is $4.44D^2$, where $D$ is the depth of embedment of the bolts
- holding down bolts should be anchored into the foundation by a washer (anchor) plate or other load distributing member embedded in the concrete
- bolts have been previously checked for combined shear and tension.

Factored design tensile force $F_t=315$ kN

Length of baseplate $D_p=400$ mm
Breadth of baseplate $B_p=400$ mm
Edge distance to bolt centre line $e_d=50$ mm

Steel Grade $S$ 275
Bolt grade 4.6
Concrete strength $f_{cu}=40$ N/mm$^2$

Holding down bolt details

Number of bolts to be used $n=4$
Number of bolts in tension $n_{t}=4$
Bolt diameter $b_d=20$ mm
Embedded depth of bolts $D=200$ mm
All bolts subject to tension.
<table>
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<th>Number of H.D. bolts</th>
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</thead>
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<td>HOLDING</td>
<td>Diameter of bolts</td>
<td>20 mm</td>
</tr>
<tr>
<td>DOWN BOLT</td>
<td>Grade of bolts</td>
<td>4.6</td>
</tr>
<tr>
<td>REQUIREMENTS</td>
<td>Edge distance to bolt C.L.</td>
<td>50 mm</td>
</tr>
<tr>
<td></td>
<td>Embedded depth</td>
<td>200 mm</td>
</tr>
</tbody>
</table>
Embedded depths of holding down bolts

Length of baseplate \( \text{hp}=600 \text{ mm} \)
Breadth of baseplate \( \text{bp}=600 \text{ mm} \)
Edge distance to bolt centre line \( k=75 \text{ mm} \)

Steel Grade \( S \text{ 275} \)
The grade of anchor bolts is grade 8.8 (Table 3.1 EC3 Part 1-8).
Char cube compressive strength \( f_{cu}=30 \text{ N/mm}^2 \) (concrete)

Holding down bolt details

Number of bolts to be used \( n=8 \)
Number of bolts in tension \( nt=4 \)
Bolt diameter \( bd=24 \text{ mm} \)

Overall embedded depth \( Lo=450 \text{ mm} \)
Assumed cover to reinforcement \( cv=50 \text{ mm} \)
Assumed percentage area of tension reinforcement \( Asp=0.15 \% \)
Punching shear force \( V_{Ed}=ABS(F_{tlEd})=674.16 \text{ kN} \)
Shear stress at control perimeter \( v_{Ed}=\beta*v_{Ed}*1000/(u_1*d_\text{av}) \)
\( \text{=0.31363 N/mm}^2 \)
Design punching shear resistance \( v_{Rdc}=(0.18/gamc)*k*(100*p_1*f_{ck})^{0.333}=0.31812 \text{ N/mm}^2 \)
Shear enhancement \( 2d/av \) \( \text{senh}=1 \)
The min design punching shear resistance governs and will be adopted.
Design punching shear resistance \( v_{Rdc}=\text{senh}*(0.035*k^1.5*f_{ck})^{0.5} \)
\( \text{=0.39033 N/mm}^2 \)

As \( v_{Ed} \leq v_{Rdc} \) \( (0.31363 \text{ N/mm}^2 \leq 0.39033 \text{ N/mm}^2) \), the anchorage arrangement is considered suitable.

Tension per anchor bolt \( F_{td}=ABS(F_{tlEd})/nt=168.54 \text{ kN} \)
Tensile resistance per anchor \( F_{Rd}=0.9*f_{ub}*As/(\text{gamM2}*10^3)=203.33 \text{ kN} \)
As \( F_{td} \leq F_{Rd} \) \( (168.54 \text{ kN} \leq 203.33 \text{ kN}) \), tensile load per bolt is OK.

**SUMMARY OF**

<table>
<thead>
<tr>
<th>Number of H.D. bolts</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of bolts</td>
<td>24 mm</td>
</tr>
<tr>
<td>Grade of bolts</td>
<td>8.8</td>
</tr>
<tr>
<td>Edge distance to bolt C.L.</td>
<td>75 mm</td>
</tr>
<tr>
<td>Overall embedded depth</td>
<td>450 mm</td>
</tr>
</tbody>
</table>
Location: Ex2 - Example based on bond length

Embedded depths of holding down bolts

Length of baseplate hp=600 mm
Breadth of baseplate bp=600 mm
Edge distance to bolt centre line k=75 mm

Steel Grade S 275
Char cube compressive strength $f_{cu}=30 \text{ N/mm}^2$ (concrete)

Holding down bolt details

Number of bolts to be used $n=4$
Number of bolts in tension $nt=4$
Bolt diameter $bd=24 \text{ mm}$
All bolts are subject to tension.
Overall embedded depth $L_o=450 \text{ mm}$
Assumed cover to reinforcement $c_v=50 \text{ mm}$
Tensile resistance per anchor $Ft_{Rd}=0.9*f_{ub}*A_s/(\gamma M^2*10^3)=101.66 \text{ kN}$
As $Ft_{Ed} \leq Ft_{Rd}$ (30 kN $\leq 101.66 \text{ kN}$), tensile load per bolt is OK.

SUMMARY OF

<table>
<thead>
<tr>
<th>REQUIREMENTS</th>
<th>Number of H.D. bolts</th>
<th>Diameter of bolts</th>
<th>Grade of bolts</th>
<th>Edge distance to bolt C.L.</th>
<th>Overall embedded depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>HOLDING</td>
<td>4</td>
<td>24 mm</td>
<td>4.6</td>
<td>75 mm</td>
<td>450 mm</td>
</tr>
</tbody>
</table>
Location: Default example

I section beam


Design moment about z-z axis \(M_{zz}'=296.63 \text{ kNm}\)
Design shear force \(SF'=169.5 \text{ kN}\)
Design axial load (+ve comp) \(F'=0 \text{ kN}\)
Length of member \(L'=7 \text{ m}\)
533 x 210 x 82 UB.
Steel Grade Grade=43
Effec length for buckling abt yy \(l_{yy}'=0 \text{ m}\)
Effec length for buckling abt zz \(l_{zz}'=7 \text{ m}\)

UNIVERSAL BEAM 533 x 210 x 82 UB Grade 43
SECTION
SUMMARY Section is satisfactory for axial, bending, shear, and local capacity, and overall buckling checks.
STRESSES Design bending 164.96 N/mm²
SUMMARY Permissible bending 180 N/mm²
Design shear 33.421 N/mm²
Permissible shear 110 N/mm²
Bending & shear 0.77159 < 1.25
Location: Default example

I section column


Design moment about z-z axis \( M_{zz}' = 445 \text{ kNm} \)
Design shear force \( S_F' = 0 \text{ kN} \)
Design axial load (+ve comp) \( F' = 8250 \text{ kN} \)
Length of member \( L' = 4.5 \text{ m} \)
Design BM about minor axis yy \( M_{yy}' = 18 \text{ kNm} \)
356 x 406 x 467 UC.
Steel Grade Grade=50
Effective length about zz axis \( l_{zz}' = 4.5 \text{ m} \)
Effective length about yy axis \( l_{yy}' = 4.5 \text{ m} \)

UNIVERSAL COLUMN 356 x 406 x 467 UC Grade 50
SECTION Section is satisfactory for axial,
SUMMARY bending, shear, and buckling checks
STRESSES Design compressive \( 138.66 \text{ N/mm}^2 \)
SUMMARY Permissible comp. \( 191.06 \text{ N/mm}^2 \)
Design bending (z-z) \( 53.084 \text{ N/mm}^2 \)
Permissible bending \( 230 \text{ N/mm}^2 \)
Design bending (y-y) \( 5.4717 \text{ N/mm}^2 \)
Permissible bending \( 230 \text{ N/mm}^2 \)
Bending & axial \( 0.98031 \leq 1 \)
Location: Example 3, Design in SHS to BS449

RHS beam section design

Calculations in accordance with DESIGN IN SHS to BS449 published by BSC Tubes Division.

Maximum design BM about axis zz \( M_{zz}' = 19.33 \text{ kNm} \)
Maximum design SF in y direction \( V' = 14.5 \text{ kN} \)
Design load (comp. positive) \( F' = 0 \text{ kN} \)
Length of member \( L = 4 \text{ m} \)

200 x 100 x 5 RHS - Hot finished.

Properties (cm):
- \( A = 28.7 \) cm
- \( r_x = 7.21 \) cm
- \( Z_x = 149 \) cm
- \( S_x = 185 \) cm
- \( I_x = 1500 \) cm
- \( J = 1200 \) cm
- \( C = 172 \) cm
- \( Z_y = 101 \) cm
- \( S_y = 114 \) cm
- \( I_y = 505 \) cm
- \( r_y = 4.19 \) cm

Steel Grade (suffix C appended) Grade=43

Span of beam for deflection \( L_d = 4000 \text{ mm} \)
Load on beam \( W = 29 \text{ kN} \)
Modulus of elasticity \( E = 210 \text{ kN/mm}^2 \)
Deflection OK.

RECTANGULAR HOLLOW 200 x 100 x 5 RHS Grade 43
SECTION
- Section is satisfactory for axial,
bending, shear, and buckling checks
SUMMARY
STRESSES
- Design bending 129.73 N/mm²
- Permissible bending 180 N/mm²
- Design shear 7.25 N/mm²
- Permissible shear 110 N/mm²
- Bending & shear 0.40405 ≤ 1.25
- Bending, shear, bearing 139.96 N/mm²
- Permissible 250 N/mm²
- Bending, shear, bearing 0.48325 ≤ 1.25
Location: Example 7, Design in SHS to BS449

RHS Column

Calculations are in accordance with DESIGN IN SHS to BS449 published by BSC Tubes Division.

Maximum design BM about axis zz \( M_{zz}' = 3 \text{ kNm} \)
Maximum design SF in y direction \( V' = 0 \text{ kN} \)
Design load (comp. positive) \( F' = 50 \text{ kN} \)
Length of member \( L = 4.5 \text{ m} \)
Design BM about yy axis \( M_{yy}' = 0 \text{ kNm} \)
100 x 100 x 4 SHS - Hot finished.
Properties (cm): \( A = 15.2 \) \( r_x = 3.91 \) \( Z_x = 46.4 \) \( S_x = 54.4 \) \( I_x = 232 \) \( J = 361 \) \( C = 68.2 \)
Effective length about yy axis \( l_{yy} = 4500 \text{ mm} \)
Effective length about zz axis \( l_{zz} = 4500 \text{ mm} \)

SQUARE HOLLOW 100 x 100 x 4 SHS Grade 43
SECTION Section is satisfactory for axial, bending, shear, and buckling checks
SUMMARY
STRESSES Design compressive 32.895 N/mm²
SUMMARY Permissible comp. 66.419 N/mm²
Design bending z-z 64.655 N/mm²
Permissible bending 180 N/mm²
Bi-axial bending & axial compression 0.85445 ≤ 1
Location: Example 4.3.1 in 'Manual on Connections'

Bolt grade (4.6 or 8.8)  bgrade=8.8
Diameter of bolts          dia=20 mm
Centres of bolts          crs=70 mm
Grade of supported beam & cleats  grade=43
Web thickness of supported beam  t=14 mm
Grade of supporting member   sgrade=43
Total thick of supporting member  tt=15.2 mm
Available thick for this connect  ts=15.2 mm

Thickness of angle cleats  tcleat=10 mm
Length of angle cleat leg  lcleat=90 mm
Backmark                  bmk=50 mm
Vertical shear on bolt group  V=550 kN
Number of bolts in cleat leg  n=9
End distance              end=35 mm
838 x 292 x 176 UB.
Length of notch to cleat face  w=160 mm
Depth of notch             n'=65 mm
WEB CLEAT SUMMARY - BOLTS IN DOUBLE SHEAR

```
      35 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      35 mm
```

50 mm

Angle:
90 mm x 90 mm x 10 mm

WEB CLEAT SUMMARY - BOLTS IN SINGLE SHEAR

```
      35 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      70 mm
      +
      35 mm
```

114 mm

40 mm  40 mm
Location: Example 8.3.1 in 'Manual on Connections'

Angle seat connection

Analysis of connection follows treatment in 'Manual on Connections for Beam and Column Construction, Conforming with the requirements of BS 449 Part 2:1969', published by BCSA.

Vertical shear on seat \( V = 156 \text{ kN} \)

Serial length of angle seat \( A = 150 \text{ mm} \)
Serial width of angle cleat \( B = 90 \text{ mm} \)
Thickness of angle seat \( t_{\text{seat}} = 10 \text{ mm} \)
Grade of angle seat \( \text{agrade}=43 \)
Length of angle seat into paper \( l_s = 150 \text{ mm} \)
Bolt diameter \( \text{dia}=20 \text{ mm} \)
Bolt grade (4.6 or 8.8) \( \text{bgrade}=8.8 \)
Grade of supporting member \( \text{sgrade}=43 \)
Total thickness of support memb \( t_t = 14.2 \text{ mm} \)
Available thick this connection \( t_s = 14.2 \text{ mm} \)
Grade of supported beam \( \text{grade}=43 \)
533 x 210 x 92 UB.
Root radius of angle seat \( r_1 = 12 \text{ mm} \)
Stiff bearing length at top of seat for 30 degree spread from tangent to root radius:
Bearing stress at root of seat \( f_b = V \times 1000/(l_d \times t_{\text{seat}}) = 104 \text{ N/mm}^2 \)
Permissible bearing stress \( p_b = \text{TABLE 9 for agrade}=43 \)
\( = 210 \text{ N/mm}^2 \)

Since \( f_b \leq p_b \) ( \( 104 \text{ N/mm}^2 \leq 210 \text{ N/mm}^2 \) )
Bearing stress at root of seat within allowable in TABLE 9.

SEATING BRACKET 150 mm x 90 mm x 10 mm Angle
SUMMARY Grade 43 x 150 mm long
LEG CONNECTED Number of bolts 4
TO SUPPORTING Diameter of bolts M 20 Grade 8.8
SUPPORTED BEAM 533 x 210 x 92 U.B.
BEAM END CLEARANCE 10 mm
**Location: Example 5.3.1 in 'Manual on Connections'**

**Flexible end plate connection**

Analysis of connection follows treatment in 'Manual on Connections for Beam and Column Construction, Conforming with the requirements of BS 449 Part 2:1969', published by BCSA.

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical shear</td>
<td>$V = 380 \text{ kN}$</td>
</tr>
<tr>
<td>Diameter of bolts</td>
<td>$\text{dia} = 20 \text{ mm}$</td>
</tr>
<tr>
<td>Centres of bolts</td>
<td>$\text{crs} = 70 \text{ mm}$</td>
</tr>
<tr>
<td>Grade of sup beam and end plate</td>
<td>$\text{grade} = 43$</td>
</tr>
<tr>
<td>Grade of supporting member</td>
<td>$\text{sgrade} = 43$</td>
</tr>
<tr>
<td>533 x 210 x 92 UB.</td>
<td></td>
</tr>
<tr>
<td>Total thick of supporting memb</td>
<td>$tt = 13.1 \text{ mm}$</td>
</tr>
<tr>
<td>Available thickness for connect</td>
<td>$ts = 13.1 \text{ mm}$</td>
</tr>
<tr>
<td>Thickness of end plate</td>
<td>$t_{end} = 10 \text{ mm}$</td>
</tr>
<tr>
<td>Number of bolts in end plate</td>
<td>$n = 12$</td>
</tr>
<tr>
<td>End distance</td>
<td>$\text{end} = 35 \text{ mm}$</td>
</tr>
<tr>
<td>Length of notch to cleat face</td>
<td>$w = 120 \text{ mm}$</td>
</tr>
<tr>
<td>Depth of notch</td>
<td>$n' = 65 \text{ mm}$</td>
</tr>
<tr>
<td>Width of end plate</td>
<td>$\text{width} = 180 \text{ mm}$</td>
</tr>
<tr>
<td>Bolt cross centres</td>
<td>$\text{bcc} = 90 \text{ mm}$</td>
</tr>
<tr>
<td>Weld leg length</td>
<td>$\text{leg} = 6 \text{ mm}$</td>
</tr>
</tbody>
</table>
END PLATE SUMMARY

- 35 mm
- 70 mm
- 70 mm
- 70 mm
- 70 mm
- 70 mm
- 35 mm

90 mm

45 mm 45 mm

Thickness 10 mm
Location: Example 12.2.4.1 in 'Manual on Connections'

Extended end plate moment connection

Analysis of connection follows treatment in 'Manual on Connections for Beam and Column Construction, Conforming with the requirements of BS 449 Part 2:1969', published by BCSA.

![Diagram of extended end plate connection]

Bolt grade \( b_{grade}=8.8 \)
Grade of supported beam & plate \( grade=43 \)
Grade of supporting member \( s_{grade}=43 \)
406 x 178 x 54 UB.
Bending moment on connection \( M=95 \ \text{kNm} \)
Vertical shear on connection \( V=80 \ \text{kN} \)
Axial force (positive = compr) \( A=20 \ \text{kN} \)

Flange weld leg length \( \text{leg}=10 \ \text{mm} \)
Web weld leg length \( \text{legw}=6 \ \text{mm} \)
Diameter of bolts \( d=20 \ \text{mm} \)
Bolt cross centres \( g=100 \ \text{mm} \)
Vertical crs of tension bolts \( C'=120 \ \text{mm} \)
Edge dist for outer tens bolts \( ae=50 \ \text{mm} \)
Actual width of end plate \( Be=200 \ \text{mm} \)
Actual length of end plate \( De=560 \ \text{mm} \)
bottom of end plate \( ab=60 \ \text{mm} \)

Thick of supporting memb avail \( tsup=14.2 \ \text{mm} \)
## CONNECTION SUMMARY

<table>
<thead>
<tr>
<th></th>
<th>50</th>
<th>25</th>
<th>50</th>
<th>200</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>120</td>
<td></td>
<td>120</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>330</td>
<td></td>
<td>330</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>60</td>
<td></td>
<td>60</td>
<td></td>
</tr>
</tbody>
</table>

Number of bolts 6
Bolt diameter 20 mm
Bolt grade 8.8
Flange weld 10 mm
Web weld 6 mm
Beam / Rafter section 406 x 178 x 54 Universal Beam
Location: Example 12.2.4.1 in 'Manual on Connections'

Bolts in tension and shear

Calculations are in accordance with BS449 : Part 2 : 1969.

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bolts in tension</td>
<td>nt=4</td>
</tr>
<tr>
<td>Tension on group</td>
<td>F=232.5 kN</td>
</tr>
<tr>
<td>Number of bolts in shear</td>
<td>ns=6</td>
</tr>
<tr>
<td>Shear force on group</td>
<td>V=80 kN</td>
</tr>
<tr>
<td>Bolt diameter</td>
<td>d=20 mm</td>
</tr>
<tr>
<td>Allowable tensile stress in bolt</td>
<td>pt=281 N/mm²</td>
</tr>
<tr>
<td>Allowable shear stress in bolt</td>
<td>ps=187 N/mm²</td>
</tr>
</tbody>
</table>

Check bolts for shear

The calculated shear stress based on the nominal bolt diameter

Shear per bolt \( v = \frac{V \times 1000}{ns} = 13333 \) N

Gross area of bolt \( g\text{area} = \pi \times d^2 / 4 = 314.16 \) mm²

Since \( fs \leq ps \) (42.441 N/mm² ≤ 187 N/mm²) shear stress in bolts within allowable.

Check bolts for tension

Tension per bolt \( f = \frac{F \times 1000}{nt} = 58125 \) N

Approx. bolt tensile stress area \( t\text{area} = 0.75 \times \pi \times d^2 / 4 = 235.62 \) mm²

Tensile stress in bolts \( ft = \frac{f}{t\text{area}} = 246.69 \) N/mm²

Since \( ft \leq pt \) (246.69 N/mm² ≤ 281 N/mm²) tensile stress in bolts within allowable.

Check for combined shear and tension

The interaction formula of BS449, Subclause 50.d is used for this investigation

factor = \( \frac{fs}{ps} + \frac{ft}{pt} = 1.1049 \)

Combined stress factor does not exceed 1.4, therefore bolts are satisfactory.
Location: Ex1 - Bolts in tension and shear

Bolts in tension and shear

Calculations are in accordance with BS EN 1993-1-8:2005 and publication entitled 'Joints in Steel Construction: Simple Joints to EC3' by SCI.

Number of bolts in tension \( nt = 4 \)

Tension on group \( N_{Ed} = 232.5 \) kN

Number of bolts in shear \( ns = 6 \)

Shear force on group \( V_{Ed} = 80 \) kN

Bolt diameter \( bd = 20 \) mm

**DESIGN**

Number of bolts in shear \( 6 \)

Number of bolts in tension \( 4 \)

Design shear force per bolt \( 13.333 \) kN

Design shear resistance per bolt \( 94.08 \) kN

Design tension per bolt \( 58.125 \) kN

Design tension resistance per bolt \( 141.12 \) kN

Interaction factor \( 0.43593 \leq 1 \)
Location: Example 11.3.2. 'Manual on Connections'

Column base with moment

![Diagram showing column base with moment, edge distance, and baseplate dimensions]

Overturning moment \( Mb = 75 \text{ kNm} \)
Horizontal shear force \( Sb = 25 \text{ kN} \)
Axial compressive force \( Wb = 1800 \text{ kN} \)

305 x 305 x 198 UC.
Length of baseplate \( lp = 700 \text{ mm} \)
Breadth of baseplate \( bp = 700 \text{ mm} \)
Allowable pressure on bedding \( pg = 5 \text{ N/mm}^2 \)
Edge distance for bolts \( edge = 75 \text{ mm} \)
Grade of column and slab base \( \text{grade}=43 \)
Grade of bolts (4.6 or 8.8) \( b\text{grade}=4.6 \)

Weld leg length \( \text{leg}=8 \text{ mm} \)
Number of bolts \( n=4 \)
Diameter of holding down bolts \( \text{dia}=20 \text{ mm} \)

**SUMMARY OF REQUIREMENTS**

| Size 700 mm x 700 mm x 55 mm |
| Grade 43 steel             |
| Edge distance 75 mm        |
| Number of H.D. bolts 4     |
| Diameter of bolts M 20     |
| Grade 4.6                 |
| Size of fillet weld 8 mm all round |
Location: Default example

Design of Guardrailing

Material: Steel hollow tubular section or solid circular section.

Rail UDL

\[
\begin{align*}
0 & < wr \text{ (kN/m)} < \\
< & \\
< Wj \text{ (kN)} & < \text{ Infill UDL} \\
\text{Infill} & < wi \text{ (kN/m²)} \\
\text{Pt Load} & < \\
(\text{anywhere}) & < \text{ Datum Level}
\end{align*}
\]

\[
\begin{align*}
\text{Horizontal Spacing of Posts} & \quad sp=1.83 \text{ m} \\
\text{Limiting bending Stress} & \quad pb=165 \text{ N/mm²}
\end{align*}
\]

Loading

BS 6399-1:1996:Table 4: Horiz loads on parapets and balustrades.
Loading is deemed to act at 1.1 metres above the Datum level (Cl 10).
In these calculations the vertical Posts are assumed to be attached to the side of the platform or walkway, 100mm below Datum Level.

\[
\begin{align*}
\text{Horiz UD loading on top rail} & \quad wr=0.22 \text{ kN/m} \\
\text{Horiz UD loading on infill} & \quad wi=0 \text{ kN/m²} \\
\text{Horiz Point Load on infill} & \quad Wj=0 \text{ kN}
\end{align*}
\]

SUMMARY

Limiting deflection criterion \( \text{span/120} \)

POST : 40 mm Nominal Bore Medium-Duty Tubular Steel Post
RAIL : 20 mm Nominal Bore Medium-Duty Tubular Steel Rail
Location: all 3 loads applied

Design of Guardrailing

Material: Steel hollow tubular section or solid circular section.

<table>
<thead>
<tr>
<th>Horizontal Spacing of Posts</th>
<th>sp=1 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limiting bending Stress</td>
<td>pb=165 N/mm²</td>
</tr>
</tbody>
</table>

Loading

BS 6399-1:1996:Table 4: Horiz loads on parapets and balustrades.
Loading is deemed to act at 1.1 metres above the Datum level (Cl 10).
In these calculations the vertical Posts are assumed to be attached to the side of the platform or walkway, 100mm below Datum Level.

Horiz UD loading on top rail  wr=3 kN/m
Horiz UD loading on infill    wi=1.5 kN/m²
Horiz Point Load on infill   Wj=1.5 kN

SUMMARY

Limiting deflection criterion  span/120
POST : 88.9 mm Outside Diam CHS Post; Wall thickness 5 mm
RAIL : 25 mm Nominal Bore Medium-Duty Tubular Steel Rail
**Location: Ex1 - Default example**

**Design of Guardrailing**

Material: Steel hollow tubular section or solid circular section.

---

**ELEVATION**

**Section**

---

**Design assumptions**

Loading is deemed to act at 1.1 metres above the datum level. Vertical posts are assumed to be attached to the side of the platform or walkway, 100 mm below datum level.

**Characteristic loading**

- Horizontal UDL on top rail: $w_r = 0.22 \text{ kN/m}$
- Horizontal UDL on infill: $w_i = 0 \text{ kN/m}^2$
- Horizontal Point Load on infill: $W_j = 0 \text{ kN}$

**Design of top rail**

- Design BM due to rail UDL: $M_{Edr} = g Q w_r L_1 / 8 = 0.13814 \text{ kNm}$
- Maximum design bending moment: $M_{Edr\text{max}} = 0.13814 \text{ kNm}$
- Actual horiz deflection of rail: $D_{r\text{max}} = 9.4727 \text{ mm}$
- Permiss horiz deflection of rail: $D_{r\text{maxp}} = 15.25 \text{ mm}$
- Design bending moment (rail): $M_{Edr\text{max}} = 0.13814 \text{ kNm}$
- Design moment resistance: $M_{Crdr} = f_y W_{elr} 10^3 / g M_0 = 0.33589 \text{ kNm}$

**Design of vertical post**

- Unfactored horizontal load at top of post: $W_{wr} = 0.4026 \text{ kN}$
- Design BM at base of post due to UDL on rail: $M_{Edpwr} = g Q w_r 1.2 = 0.72468 \text{ kNm}$
- Max design BM is due to the horizontal UDL on the rail.
- Maximum design bending moment: $M_{Edp\text{max}} = 0.72468 \text{ kNm}$
Try 40 mm Nominal Bore Medium-Duty Tubular Steel Post
Actual deflection (post) \[ D_{pmax} = \frac{W_{max} \times 1.2^3}{3 \times E \times I_p} \times 1000 \]
\[ = 8.9895 \text{ mm} \]
Permissible deflection (post) \[ D_{pmaxp} = 10 \text{ mm} \]
Design bending moment (post) \[ M_{Edpmax} = 0.72468 \text{ kNm} \]
Design moment resistance \[ M_{Crdp} = f_y \times W_{elp} \times 10^3 / \gamma_M0 = 1.4243 \text{ kNm} \]

**DESIGN SUMMARY**

Limiting deflection criterion \[ \text{span}/120 \]

**POST**
40 mm Nominal Bore Medium-Duty Tubular Steel Post
Minimum 2nd Moment of area of post \[ I_p = 12.284 \text{ cm}^4 \]

**RAIL**
20 mm Nominal Bore Medium-Duty Tubular Steel Rail
Minimum 2nd Moment of area of rail \[ I_r = 1.615 \text{ cm}^4 \]
Location: Ex2 - All 3 loads applied

Design of Guardrailing

Material: Steel hollow tubular section or solid circular section.

Design assumptions

Loading is deemed to act at 1.1 metres above the datum level. Vertical posts are assumed to be attached to the side of the platform or walkway, 100 mm below datum level.

Characteristic loading

Horizontal UDL on top rail \( wr = 3 \text{ kN/m} \)
Horizontal UDL on infill \( wi = 1.5 \text{ kN/m}^2 \)
Horizontal Point Load on infill \( Wj = 1.5 \text{ kN} \)

Design of top rail

Design bending moments:
Due to rail UDL \( ME_{dr} = \gamma Q \cdot W_{wr} \cdot L_1 / 8 = 0.5625 \text{ kNm} \)
Due to infill UDL \( ME_{dzi} = \gamma Q \cdot W_{wi} \cdot L_1 / 8 = 0.15469 \text{ kNm} \)
Due to infill PL at mid-span \( ME_{dWj} = \gamma Q \cdot W_{j} \cdot L_1 / 4 = 0.5625 \text{ kNm} \)
Maximum design bending moment \( ME_{dmax} = 0.5625 \text{ kNm} \)
Actual horiz deflection of rail \( Dr_{max} = 6.1655 \text{ mm} \)
Permiss horiz deflection of rail \( Dr_{maxp} = 8.3333 \text{ mm} \)
Design bending moment (rail) \( ME_{drmax} = 0.5625 \text{ kNm} \)
Design moment resistance \( MC_{Rdr} = f_y \cdot W_{elr} \cdot 10^3 / \gamma M_0 = 0.781 \text{ kNm} \)
Design of vertical post

Unfactored horizontal load at top of post:
Due to horizontal UDL on rail \( W_{wr} = 3 \) kN
Due to horizontal UDL on Infill \( W_{wi} = 0.825 \) kN
Due to Point Load on infill \( W_j = 1.5 \) kN

Design bending moments at base of post:
Due to rail UDL \[ M_{Edpwr} = \gamma Q \cdot W_{wr} \cdot 1.2 = 5.4 \text{ kNm} \]
Due to infill UDL \[ B_{pwi} = \gamma Q \cdot W_{wi} \cdot 1.2 = 1.485 \text{ kNm} \]
Due to infill PL \[ B_{pWj} = \gamma Q \cdot W_j \cdot 1.2 = 2.7 \text{ kNm} \]
Max design BM is due to the horizontal UDL on the rail.
Maximum design bending moment \( M_{Edpmax} = 5.4 \text{ kNm} \)

Try 88.9 mm Outside Diam CHS Post, with Wall thickness 4 mm
Actual deflection (post) \[ D_{pmax} = \frac{W_{pmax} \cdot 1.2^3}{3 \cdot E \cdot I_p} \cdot 1000 = 8.5447 \text{ mm} \]
Permissible deflection (post) \( D_{pmaxp} = 10 \text{ mm} \)
Design bending moment (post) \( M_{Edpmax} = 5.4 \text{ kNm} \)
Design moment resistance \[ M_{Crdp} = f_y \cdot W_{elp} \cdot 10^3 / \gamma M_0 = 7.7035 \text{ kNm} \]

DESIGN SUMMARY

Limiting deflection criterion \( \text{span/120} \)

POST : 88.9 mm Outside Diam CHS Post
Post Wall thickness 4 mm
Minimum 2nd Moment of area of post \( I_p = 96.3 \text{ cm}^4 \)

RAIL : 28 mm Diameter Solid Circular-section Steel Rail
Minimum 2nd Moment of area of rail \( I_r = 3.017 \text{ cm}^4 \)