JOINT CONFIGURATION AND DIMENSIONS: bolted end plate connection (unstiffened)

Resistances for the extended end plate connection.

Assumption: The design moments in the two beams are equal and opposite. No axial compression in the beams ($N_{Ed} = 0$).

Column: UC 254 x 254 x 107 in S 275 (EN 10025-2)

Beams: UB 533 x 210 x 92 in S 275 (EN 10025-2)

End plate: 670 x 250 x 25 in S 275 (EN 10025-2)

Bolts: M24 non preloaded class 8.8 bolts

Welds: Fillet welds. Assumed weld sized:
- $f_{fl} = 12$mm (flange)
- $f_{wb} = 8$ mm (web)

Bolt spacings
- $S = D =$ 75 mm
- $C = W =$ 100 mm
- $R_1 = e_1 =$ 50 mm
- $R_1 + x =$ 40 mm
- $R_2 =$ 60 mm
- $R_3 =$ 90 mm

RESISTANCE OF JOINT

Bending moment: $M_{fl} =$ 424 kNm

Vertical shear: $V_{fl} =$ 499 kNm

Technical References:
The Steel Construction Institute. www.steel-sci.com
The British Constructional Steelwork Association Limited. www.steelconstruction.org
DIMENSIONS AND SECTION PROPERTIES

**Column** From data tables for UC 254 x 254 x 107:

- Depth: \( h_c = 266.7 \text{ mm} \)
- Width: \( b_c = 258.8 \text{ mm} \)
- Web thickness: \( t_{wc} = 12.8 \text{ mm} \)
- Flange thickness: \( t_{fc} = 20.5 \text{ mm} \)
- Root radius: \( r_c = 12.7 \text{ mm} \)
- Depth between fillets: \( d_s = 200.3 \text{ mm} \)
- Area: \( A_c = 13640 \text{ mm}^2 = 136.4 \text{ cm}^2 \)

**Beams** From data tables for UB 533 x 210 x 92:

- Depth: \( h_b = 533.1 \text{ mm} \)
- Width: \( b_b = 209.3 \text{ mm} \)
- Web thickness: \( t_{wb} = 10.1 \text{ mm} \)
- Flange thickness: \( t_{fb} = 15.6 \text{ mm} \)
- Root radius: \( r_b = 12.7 \text{ mm} \)
- Depth between fillets: \( d_b = 476.5 \text{ mm} \)
- Area: \( A_b = 11740 \text{ mm}^2 = 117.4 \text{ cm}^2 \)

**End plates**

- Depth: \( h_p = 670 \text{ mm} \)
- Width: \( b_p = 250 \text{ mm} \)
- Thickness: \( t_p = 25 \text{ mm} \)

**Bolts**

- M24 non preloaded class 8.8 bolts
- Diameter of bolt shank: \( d = 24 \text{ mm} \)
- Diameter of hole: \( d_0 = 26 \text{ mm} \)
- Shear area: \( A_s = 353 \text{ mm}^2 \)
- Diameter of washer: \( d_w = 44 \text{ mm} \) (ISO 7089/7091)

**Bolts spacings**

**Column**

- End distance (row 1 from column edge): \( e_1 = 1000 \text{ mm} \)
- Spacing (guage): \( w = 100 \text{ mm} \)
- Edge distance: \( e_2 = 79.4 \text{ mm} \)
- Spacing row 1-2: \( p_{1-2} = 100 \text{ mm} \)
- Spacing row 2-3: \( p_{2-3} = 90 \text{ mm} \)

**End plate**

- End distance: \( e_3 = 50 \text{ mm} \)
- Spacing (guage): \( w = 100 \text{ mm} \)
- Edge distance: \( e_4 = 75 \text{ mm} \)
- Spacing row 1 above beam flange: \( x = 40 \text{ mm} \)
- Spacing row 1-2: \( p_{1-2} = 100 \text{ mm} \)
- Spacing row 2-3: \( p_{2-3} = 90 \text{ mm} \)

![Diagram of structural components](image-url)
MATERIAL STRENGTHS

Steel strength

For building the nominal values of the yield strength \( f_y \) and the ultimate strength \( f_u \) for structural steel should be those obtained from the product standard. Where a range is given, the lowest nominal value should be used.

S 275 (EN 10025-2)

- For \( t \leq 16 \text{ mm} \) \( f_y = f_{\text{ReH}} = 275 \text{ N/mm}^2 \)
- For \( 16 \text{ mm} < t \leq 40 \text{ mm} \) \( f_y = f_{\text{ReH}} = 265 \text{ N/mm}^2 \)
- For \( 3 \text{ mm} \leq t \leq 100 \text{ mm} \) \( f_u = f_{\text{ReH}} = 410 \text{ N/mm}^2 \)

Hence:

- Beam yield strength \( f_{y,b} = 275 \text{ N/mm}^2 \)
- Beam ultimate strength \( f_{u,b} = 410 \text{ N/mm}^2 \)
- Column yield strength \( f_{y,c} = 265 \text{ N/mm}^2 \)
- Column ultimate strength \( f_{u,c} = 410 \text{ N/mm}^2 \)
- End plate yield strength \( f_{y,p} = 265 \text{ N/mm}^2 \)
- End plate ultimate strength \( f_{u,p} = 410 \text{ N/mm}^2 \)

Bolt strength

- Nominal yield strength \( f_{\text{yb}} = 640 \text{ N/mm}^2 \)
- Nominal ultimate strength \( f_{u,b} = 800 \text{ N/mm}^2 \)

PARTIAL FACTORS FOR RESISTANCE

Structural steel

\( \gamma_{M0} = 1.00 \) (see NA)
\( \gamma_{M1} = 1.00 \)
\( \gamma_{M2} = 1.10 \)

Parts in connection

\( \gamma_{M0} = 1.25 \) (bolts, welds, plates in bearing)
TENSION ZONE T-STUBS

When prying forces may develop, the design tension resistance \( F_{\text{T,unst}} \) of a T-stub flange should be taken as the smallest value for the 3 possible failure models in Table 6.2 (EN 1993-1-8).

6.2.4.1(6)

BOLT ROW 1

Column flange in bending (no backing plate)

Consider bolt row 1 to be acting alone. The key dimensions are shown below.

Determine \( e_{\text{max}} \), \( m \) and \( l_{\text{eff}} \) for the unstiffed column flange

\[
m = m_c = \frac{w - \frac{t_{\text{wc}}}{2} - 0.8r_c}{2} = 0.5 \times (100 - 12.8 - 2 \times 0.8 \times 12.7) = 33.4 \text{ mm}
\]

\[
e_{\text{max}} = \min([e_c; e_{\text{nc}}]) = \min(75; 79.4) = 75 \text{ mm}
\]

For Mode 1, \( l_{\text{eff,1}} \) is the lesser of \( l_{\text{eff,cn}} \) and \( l_{\text{eff,cp}} \)

\[
l_{\text{eff,cp}} = 2 \text{ mm} = 2 \times 3.14 \times 33.4 = 210 \text{ mm}
\]

\[
l_{\text{eff,nc}} = 4 \text{ mm} + 1.25e_{\text{c}} = 4 \times 33.4 + 1.25 \times 79.4 = 233 \text{ mm}
\]

\[
l_{\text{eff,1}} = \min([l_{\text{eff,cn}}; l_{\text{eff,cp}}]) = \min(233; 210) = 210 \text{ mm}
\]

For failure Mode 2, \( l_{\text{eff,2}} = l_{\text{eff,nc}} \)

Therefore \( l_{\text{eff,2}} = 233 \text{ mm} \)

Mode 1 resistance

For Mode 1, without backing plates, using Method 2:

\[
F_{\text{T,1}} = \frac{(8n - 2e_{\text{c}})M_{\text{ub,unst}}}{2mn - e_{\text{c}}(m + n)}
\]

where:

\[
m = m_c = 33.4 \text{ mm}
\]

\[
n = \min([e_{\text{nc}}, 1.25m]) = \min([75; (1.25 \times 33.4)]) = \min(75; 41.8) = 41.8 \text{ mm}
\]

(cont'd)
Diameter of the washer (for M24 non preloaded class 8.8 bolts):

\[ d_a = 44 \text{ mm} \]

\[ e_a = d_a/4 = (44)/4 = 11 \text{ mm} \]

Therefore,

\[ F_{t,1,Rd} = \frac{(8n - 2e_a)M_{b,1,Rd}}{2mn - e_a(m - n)} = [(8 \times 41.8 - 2 \times 11) \times 5847 \times 10^3/(10^3 \times [2 \times 33.4 \times 41.8 - 11 \times (33.4 + 41.8)])] = 928.1 \text{ kN} \]

Mode 2 resistance

For Mode 2

\[ F_{t,2,Rd} = \frac{2M_{b,2,Rd} + \sum F_{t,1,Rd}}{m + n} = \frac{2 \times (6487 \times 10^3) + 41.8 \times 406.7 \times 10^3/(33.4 + 41.8)}{398.4 \text{ kN}} \]

where:

\[ M_{b,2,Rd} = 0.25 \sum \frac{\mu \cdot f_{u1}}{\gamma} = \frac{[0.25 \times 233 \times (20.5)^2 \times 265]}{1} = 6487 \times 10^3 \text{ Nmm} = 6.49 \text{ kNm} \]

\[ \sum F_{t,1,Rd} \text{ is the total value of } F_{t,1,Rd} \text{ for all the bolts in the row, where:} \]

\[ F_{t,1,Rd} = k \frac{f_{y,c}}{\gamma} \frac{A_{1}}{\gamma} = (0,9 \times 800 \times 353)/1.25 = 203.3 \times 10^3 \text{ N} = 203.3 \text{ kN} \text{ (for a single bolt)} \]

For 2 bolts in the row \[ \sum F_{t,1,Rd} = 2 \times 203.3 \times 10^3 = 406.7 \times 10^3 \text{ N} = 406.7 \text{ kN} \]

Mode 3 resistance (bolt failure)

\[ F_{t,3,Rd} = \sum F_{t,1,Rd} = 406.7 \text{ kN} \]

Resistance of column flange in bending

\[ F_{c,b,Rd} = \min(F_{t,1,Rd}; F_{t,2,Rd}; F_{t,3,Rd}) = \min(928.1; 398.4; 406.7) = 398.4 \text{ kN} \]

Column web in transverse tension

The design resistance of an unstiffened column web to transverse tension is determined from:

\[ F_{w,c,Rd} = \frac{c_n b_{w,c}}{\gamma} \frac{f_{y,c}}{\gamma} \]

where \( \gamma \) is a reduction factor that allows for the interaction with shear in the column web panel.
Transformation parameter $\beta = 0.0 \, [-] \quad \rightarrow \quad \text{Reduction factor: } 0 \leq \beta \leq 0.5 \quad \rightarrow \quad \omega = 1.00$

Here, as $\min(F_{T,1,Rd}; F_{T,2,Rd}) = [928.1; 398.4] = 398.4 \, kN$

with:

$\ell_{\text{eff},1} = 210 \, \text{mm}$

$\ell_{\text{eff},2} = 233 \, \text{mm}$

for a bolted connection the effective width of the column web in tension is considered to be:

$\beta_{\text{eff,wc}} = 233 \, \text{mm}$.  

$f_{\text{y,wc}} = f_{\text{y,c}} = 265 \, \text{N/mm}^2$

$\omega = \frac{1.00 \, [-]}{1.00}$

Thus,

$F_{\text{eff,Rd}} = \frac{(1.00 \times 233 \times 12.8 \times 265)/1.00}{10^3} = 790.4 \, kN$

End plate in bending

Bolt row 1 is outside the tension flange of the beam. The key dimensions for the T-stub are shown below.

The values of $m_x$, $e_x$, and $e$ for the T-stub are:

$e = e_x = 75 \, \text{mm}$

$e_x = 50 \, \text{mm}$

$m_x = x - 0.8e_x = 40 - 0.8 \times 12 = 30.4 \, \text{mm}$

For Mode 1, $\ell_{\text{eff}}$ is the lesser of $\ell_{\text{eff,nc}}$ and $\ell_{\text{eff,cp}}$

$\ell_{\text{eff,cp}} = \min(4m_x + 1.25e_x; e + 2m_x + 0.625e_x; 0.5w + 2m_x + 0.625e_x) = $

$\min(4 \times 30.4 + 1.25 \times 75; 75 + 2 \times 30.4 + 0.625 \times 50; 0.5 \times 250; 0.5 \times 100 + 2 \times 30.4 + 0.625 \times 50) = $  

$\ell_{\text{eff,cp}} = \min(184; 167; 125; 142) = 125 \, \text{mm}$
Mode 1 resistance

For Mode 1 failure, using Method 2:

\[ F_{T,1,UL} = \frac{(8n - 2e_m)M_{A,URd}}{2mn - e_m(m + n)} \]

where:
\[ n = e_{min} = \min\{e_p, 1.25m_x\} = \min\{50 \times 1.25 \times 30.4\} = 38.3 \text{ mm} \]
\[ m = m_x = 30.4 \text{ mm} \]
\[ e_m = 11.0 \text{ mm} \]
\[ f_p = f_{y,p} = 265 \text{ N/mm}^2 \]
\[ t = t_p = 25 \text{ mm} \]
\[ M_{A,URd} = \frac{0.25 \sum l_{i,1} f_{y,p} t_i}{7_{UR}} = \frac{(0.25 \times 125 \times 25^2 \times 265)\times 1}{7_{UR}} = 5.176 \times 10^4 = N\text{mm} = 5.2 \text{ kNm} \]
\[ F_{T,1,UL} = \frac{(8n - 2e_m)M_{A,URd}}{2mn - e_m(m + n)} = \frac{(8 \times 38.3 - 2 \times 11.0) \times 5.176 \times 10^4 \times (2 \times 30.4 \times 38.3 - 11.0 \times (30.4 + 38.3))}{= 936 \text{ kN} \]

Mode 2 resistance

\[ F_{T,2,UL} = \frac{2M_{A,URd} + n \sum F_{i,URd}}{m + n} \]

where:
\[ M_{A,URd} = \frac{0.25 \sum l_{i,2} f_{y,p} t_i}{7_{UR}} = \frac{(0.25 \times 125 \times 25^2 \times 265)\times 1}{7_{UR}} = 5.176 \times 10^4 = N\text{mm} = 5.2 \text{ kNm} \]
\[ l_{i,2} = 125 \text{ mm} \]
\[ \sum F_{i,URd} = 407 \times 10^4 N = 407 \text{ kN} \]
Therefore, \( F_{T,2,UL} = (2 \times 5.176 \times 10^4 + 38.3 \times 407 \times 10^4 \times (10^4 \times (30.4 + 38.3)) = 377 \text{ kN} \)

Mode 3 resistance (bolt failure)

\[ F_{T,3,UL} = \sum F_{i,URd} = 407 \text{ kN} \]

Resistance of end plate in bending

\[ F_{lep,UL} = \min\{F_{T,1,UL}; F_{T,2,UL}; F_{T,3,UL}\} = \min\{377; 407\} = 377 \text{ kN} \]

Beam web in tension

As bolt row 1 is in the extension of the end plate, the resistance of the beam web in tension is not applicable to this bolt row.
SUMMARY: RESISTANCE OF T-STUB FOR BOLT ROW 1

Resistance of bolt row 1 is the smallest value of:

- Column flange in bending: $F_{t,fc,Rd} = 398\ \text{kN}$
- Column web in tension: $F_{t,wc,Rd} = 790\ \text{kN}$
- End plate in bending: $F_{t,ep,Rd} = 377\ \text{kN}$

Therefore, the resistance of bolt row 1 is:

$$F_{t,Rd} = \min\{F_{t,fc,Rd}; F_{t,wc,Rd}; F_{t,ep,Rd}\} = \min\{398; 790; 377\} = 377\ \text{kN}$$

BOLT ROW 2

Firstly, consider row 2 alone.

**Column flange in bending**

The resistance of the column flange in bending is as calculated for bolt row 1 (Mode 2):

$$F_{t,fc,Rd} = 398\ \text{kN}$$

**Column web in transverse tension**

The column web resistance to transverse tension will also be as calculated for bolt row 1.

Therefore:

$$F_{t,wc,Rd} = 790\ \text{kN}$$

**End plate in bending**

Bolt row 2 is the first bolt row below the beam flange, considered as "first bolt-row below tension flange of beam" in Table 6.6. The key dimensions for the T-stub are as shown for the column flange T-stub for row 1 and as shown below (in elevation) for row 2.

$$m = m_0 = \frac{w - \ell_{te} - 2 \cdot 0.8s}{2} = \frac{(100 - 10,1 - 2 \times 0.8 \times 8)}{2} = 38.6\ \text{mm}$$

$$e = e_p = 75\ \text{mm}$$

$$m_0 = R_2 - b_0 - 0.8s = 60 - 15,6 - (0.8 \times 12) = 34.8\ \text{mm}$$

$\alpha$ is obtained from Figure 6.11 (reproduced in EN 1993-1-8, Appendix G as Figure G.1)

Parameters required to determine $\alpha$: $\lambda_1 = \frac{m_0}{m + e}$ and $\lambda_2 = \frac{m_0}{m + e}$
\[ \lambda_1 = \frac{38.6}{(38.6 + 75)} = 0.34 \]
\[ \lambda_2 = \frac{34.8}{(38.6 + 75)} = 0.31 \]

Thus, by interpolation (see EN 1993-1-8, Figure 6.11), \( \alpha = 7.3 \) [\( \text{°} \)]

\[ l_{w,c,1} = 2 \text{ mm} = 2 \times \pi \times 38.6 = 242 \text{ mm} \]
\[ l_{w,c,2} = \alpha \times m = 7.3 \times 38.6 = 283 \text{ mm} \]
\[ l_{w,1} = l_{w,2,c} = 283 \text{ mm} \]
\[ l_{w,1} = \min\{l_{w,c,1}; l_{w,2,c}\} = \min\{242; 283\} = 242 \text{ mm} \]

### Mode 1 resistance

For Mode 1, using Method 2:

\[ F_{T,1,Rd} = \frac{(8n - 2e_u)M_{pl,1,Rd}}{2mn - e_u(m + n)} \]

where:

\[ n = \min\{e_u + 1.25m\} = \min\{75; (1.25 \times 38.6)\} = 48.2 \text{ mm} \]
\[ e_u = 11.0 \text{ mm} \]
\[ b_0 = 25 \text{ mm} \]
\[ f_{pl} = 265 \text{ N/mm}^2 \]

\[ M_{pl,1,Rd} = \frac{0.25 \sum |I_{pl} l_{w,1} l_{w,1} | f_{pl}}{7 \mu c} = (0.25 \times 242 \times 25^2 \times 265) / 1.00 = 10.0 \times 10^3 \times 10^3 \text{ Nmm} \]

\[ F_{T,1,Rd} = \frac{(8n - 2e_u)M_{pl,1,Rd}}{2mn - e_u(m + n)} = [(8 \times 48.2 - 2 \times 11.0) \times 10^3] / [(2 \times 38.6 \times 48.2) - 11.0 \times (38.6 + 48.2)] = 1.320 \text{ kN} \]

### Mode 2 resistance

\[ F_{T,2,Rd} = \sum F_{T,2,Rd} = 203 \text{ kN} \]

\[ M_{pl,2,Rd} = \frac{0.25 \sum |I_{pl} l_{w,2} l_{w,1} | f_{pl}}{7 \mu c} = (0.25 \times 283 \times 25^2 \times 265) / 1.00 = 11.7 \times 10^3 \times 10^3 \text{ Nmm} \]

\[ \sum F_{T,2,Rd} = 2 \times 203 = 407 \text{ kN} \]

\[ F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + \sum F_{T,2,Rd}}{m + n} = (2 \times 11.7 \times 10^3 + 48.2 \times 407) / [(38.6 + 48.2)] = 496 \text{ kN} \]

### Mode 3 resistance (bolt failure)

\[ F_{T,3,Rd} = \sum F_{T,3,Rd} = 407 \text{ kN} \]
Resistance of end plate in bending

\[ F_{\text{tep,Rd}} = \min(F_{T,7,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = \min([1.320; 496; 407]) = 407 \text{ kN} \]

Beam web in tension

The design tension resistance of the web is determined from:

\[ F_{\text{T,web, Rd}} = \frac{\cos \beta \cdot b_{\text{web}} \cdot t_{\text{web}} \cdot f_{\text{web}}}{\omega \cdot \sigma_{\text{y}}} \]

where:

- \( b_{\text{web}} = 242 \text{ mm} \) (conservatively, consider the smallest \( b_{\text{web}} \) from earlier calculations).
- \( t_{\text{web}} = 10.1 \text{ mm} \)
- \( \omega = 1.00 \) [\( \sigma_{\text{y}} \)]

Therefore:

\[ F_{\text{T,web, Rd}} = \frac{\cos \beta \cdot b_{\text{web}} \cdot t_{\text{web}} \cdot f_{\text{web}}}{\omega \cdot \sigma_{\text{y}}} = \frac{(1.00 \times 242 \times 10.1 \times 275)/1000}{10^3} = 673 \text{ kN} \]

The above resistances for row 2 all consider the resistance of the row acting alone. However, on the column side, the resistance may be limited by the resistance of the group of rows 1 and 2. That group resistance is now considered.

ROWS 1 AND 2 COMBINED

Column flange in bending

For bolt row 1 combined with row 2 in the column flange, both rows are considered as "end bolt row" in Table 6.4.

For bolt row 1:

\[ l_{\text{ef,nc}} = \min[(2m + 0.625e + 0.5p); (e + 0.5p)] = \min[(2 \times 33.4 + 0.625 \times 79.4 + 0.5 \times 100); (1000 + 0.5 \times 100)] = 167 \text{ mm} \]

\[ l_{\text{ef,cp}} = \min[(\pi m + p); (2e + p)] = \min[(\pi \times 33.4 + 100); (2 \times 100 + 100)] = 205 \text{ mm} \]

The effective lengths for bolt row 2, as a bottom row of a group, are the same as for row 1:

\[ \Sigma l_{\text{ef,nc}} = 2 \times 167 = 333 \text{ mm} \]

\[ \Sigma l_{\text{ef,cp}} = 2 \times 205 = 410 \text{ mm} \]
The effective lengths for the group of bolts is:

Mode 1
\[ \sum_{\text{eff},1} = \min(\sum_{\text{eff},1c}; \sum_{\text{eff},1p}) = \min[333; 410] = 333 \text{ mm} \]

Mode 2:
\[ \sum_{\text{eff},2} = \sum_{\text{eff},nc} = 333 \text{ mm} \]

Mode 1 resistance
\[ \left[ (8n - 2a_n)\frac{M_{\text{eff},1}}{2mn - e_n (m + n)} \right] \]
where:
\[ m = 33,4 \text{ mm} \]
\[ n = 41,8 \text{ mm} \]
\[ a_n = 11,0 \text{ mm} \]
\[ M_{\text{eff},1} = \frac{0.25\sum l_{\text{eff},1} f_{\text{pl},1} t_{\text{eff},1}}{\gamma_0} \]
\[ = \frac{(0.25 \times 333 \times 20.5^2 \times 265)/1.00 = 9.27 \times 10^4 \times 10^3 \text{ Nmm}} \]
\[ F_{t,1,\text{res}} = \left[ \frac{(8n - 2a_n)\frac{M_{\text{eff},1}(m + n)}{2mn - e_n (m + n)} \right] = 1.472 \text{ kN} \]

Mode 2 resistance
\[ F_{t,2,\text{res}} = \frac{2M_{\text{eff},2} + n\sum F_{t,\text{res}}}{m + n} \]
where:
\[ F_{t,\text{res}} = 203 \text{ kN} \]
\[ \sum F_{t,\text{res}} = 4 \times 203 = 813 \text{ kN} \]
\[ M_{\text{eff},2} = \frac{0.25\sum l_{\text{eff},2} f_{\text{pl},2} t_{\text{eff},2}}{\gamma_0} \]
\[ = \frac{(2 \times 9.27 \times 10^4 + 41.8 \times 813)/(33.4 + 41.8)} \]
\[ F_{t,2,\text{res}} = \left[ \frac{2M_{\text{eff},2} + n\sum F_{t,\text{res}}}{m + n} \right] = 698 \text{ kN} \]

Mode 3 resistance (bolt failure)
\[ F_{t,3,\text{res}} = \sum F_{t,\text{res}} = 4 \times 203 = 813 \text{ kN} \]

Resistance of column flange in bending
\[ F_{t,\text{flange}} = \min[F_{t,1,\text{res}}; F_{t,2,\text{res}}; F_{t,3,\text{res}}] = \min[1.472; 698; 813] = 698 \text{ kN} \]

Column web in transverse tension
The design resistance of an unstiffed column web in transverse tension is:
\[ F_{t,\text{web}} = \frac{b_{\text{eff}},t_{\text{web}} f_{\text{pl},t}}{\gamma_0} \]
where \( b_{\text{eff}},t_{\text{web}} \) is the equivalent T-stub representing the column flange from Sec. 6.2.6.4 (EN 1993-1-8). Conservatively use the lesser of the values of effective lengths for Mode 1 and Mode 2:
The equation to use to calculate $\omega$ depends on $\beta$. As before, $\beta = 0.0$ and therefore $\omega = 1.00$:

$$
F_{t,wc,Rd} = \frac{\pi d_{eff,t,wc} f_y c y_{eff,t,wc}}{T_{wc}} = (1.00 \times 333 \times 12.8 \times 265/1.00) / 10^3 = 1.130 \text{kN}
$$

**End plate in bending**

There is no group mode for the end plate.

**SUMMARY: RESISTANCE OF BOLT ROWS 1 AND 2 COMBINED**

Resistance of bolt row 1 and 2 combined, on the column side, is the smaller value of:

- Column flange in bending: $F_{t,fc,Rd} = 698 \text{kN}$  
- Column web in tension: $F_{t,wc,Rd} = 1.130 \text{kN}$

Therefore, the resistance of bolts 1 and 2 combined is: $F_{t,1-2,Rd} = 698 \text{kN}$

The resistance of bolt row 2 on the column side is therefore limited to:

$$
F_{t,2,Rd} = F_{t,1-2,Rd} - F_{t,1,Rd} = (698 - 377) = 321 \text{kN}
$$

**SUMMARY: RESISTANCE OF BOLT ROW 2**

Resistance of bolt row 2 is the smallest value of:

- Column flange in bending: $F_{t,fc,Rd} = 398 \text{kN}$  
- Column web in tension: $F_{t,wc,Rd} = 790 \text{kN}$  
- Beam web in tension: $F_{t,wb,Rd} = 673 \text{kN}$  
- End plate in bending: $F_{t,ep,Rd} = 407 \text{kN}$  
- Column side, as part of a group (see above): $F_{t,2,Rd} = 321 \text{kN}$

Therefore, the resistance of bolt row 2 is: $F_{t,2,Rd} = 321 \text{kN}$

**BOLT ROW 3**

First, consider row 3 alone.

**Column flange in bending**

The column flange in bending resistance is the same as bolt rows 1 and 2 therefore:  
$$F_{t,fc,Rd} = 398 \text{kN}$$

**Column web in transverse tension**

The column web resistance to transverse tension is as calculated for bolt rows 1 and 2. Therefore:  
$$F_{t,wc,Rd} = 790 \text{kN}$$

**End plate in bending**

Bolt row 3 is the second bolt row below the beam’s tension flange, considered as an “other end bolt-row” in Table 6.6. The key dimensions are as noted above for bolt row 2. Determine $m$, $e$ and $I_{eff}$:

$$
e = e_p = 75 \text{mm} \quad m = 38.6 \text{mm} \\
I_{eff,comp} = 2 \times 11 \times 38.6 = 242 \text{mm}$$
\[ l_{\text{eff},\text{nc}} = 4m + 1.25e = 4 \times 38.6 + 1.25 \times 75 = 248 \text{ mm} \]
\[ l_{\text{eff},1} = \text{min}\{l_{\text{eff},\text{nc}}, l_{\text{eff},2}\} = \text{min}\{242; 248\} = 242 \text{ mm} \]
\[ l_{\text{eff},2} = l_{\text{eff},\text{nc}} = 248 \text{ mm} \]

**Mode 1 resistance**

For Mode 1, using Method 2:

\[ F_{T,\text{Mode 1}} = \frac{(8n - 2e)M_{\text{eff}}}{2n - e}(m + n) \]

where:

- \( n = 48.2 \text{ mm} \)
- \( m = 38.6 \) (as for row 2)
- \( e = 11.0 \text{ mm} \)

\[ M_{\text{eff}} = 0.25 \sum \left( I_{\text{eff}} \frac{t}{\gamma_{\text{eff}}} \right) = (0.25 \times 242 \times 25^2 \times 265) / 1 = 10.0 \times 10^4 \times 10^3 \text{ Nmm} = 10.0 \text{ kNm} \]

\[ F_{T,\text{Mode 1}} = \frac{(8n - 2e)M_{\text{eff}}}{2n - e}(m + n) = [(8 \times 48.2 - 2 \times 11.0) \times 10.0 \times 10^3] / [(2 \times 38.6 + 48.2 - 11.0) \times (38.6 + 48.2)] = 1,320 \text{ kN} \]

**Mode 2 resistance**

\[ F_{T,\text{Mode 2}} = \frac{2M_{\text{eff}} + n \sum F_{I,\text{eff}}}{m + n} \]

\[ M_{\text{eff}} = 0.25 \sum \left( I_{\text{eff}} \frac{t}{\gamma_{\text{eff}}} \right) = (0.25 \times 248 \times 25^2 \times 265) / 1 = 10.3 \times 10^4 \times 10^3 \text{ Nmm} = 10.3 \text{ kNm} \]

\[ F_{T,\text{Mode 2}} = \frac{2M_{\text{eff}} + n \sum F_{I,\text{eff}}}{m + n} = [(2 \times 10.3 \times 10^4 + 48.2 \times 407)(38.6 + 48.2)] / (38.6 + 48.2) = 463 \text{ kN} \]

**Mode 3 resistance (bolt failure)**

\[ F_{T,\text{Mode 3}} = \sum F_{I,\text{eff}} = 407 \text{ kN} \]

**Resistance of end plate in bending**

\[ F_{\text{ Resist, ep}} = \text{min}\{F_{T,\text{Mode 1}}; F_{T,\text{Mode 2}}; F_{T,\text{Mode 3}}\} = \text{min}\{1,320; 463; 407\} = 407 \text{ kN} \]

**Beam web in tension**

\[ F_{T,\text{web}} = \frac{b_{\text{eff},\text{web}} f_{\text{ub}}}{\gamma_{\text{t}}} \]

where:

- \( b_{\text{eff},\text{web}} \)
- \( f_{\text{ub}} \)
Conservatively, consider minimum $l_{\text{eff}}$. Therefore:

$$b_{\text{eff,wb}} = l_{\text{eff}} = \min\{l_{\text{eff,1}}, l_{\text{eff,2}}\} = \min\{242; 248\} = 242 \text{ mm}.$$  

Therefore, 

$$F_{T,\text{wb,Rd}} = \frac{b_{\text{eff,wb}} l_{\text{eff,wb}}}{A_{\text{wb}}} = \frac{242 \times 10.1 \times 275}{10^3} / 1.0 = 673 \text{ kN}$$

The above resistances for row 3 all consider the resistance of the row acting alone. However, on the column side the resistance may be limited by the resistance of the group of rows 1, 2, and 3 or by the group of rows 2 and 3. On the beam side, the resistance may be limited by groups of rows 2 and 3. Those group resistances are now considered.

**ROWS 1, 2 AND 3 COMBINED**

*Column flange in bending*

Circular and non-circular yield line patterns are:

![Circular and non-circular yield line patterns](image)

The effective length for bolt row 1, as part of a group, is the same as that determined as part of the group of rows 1 an 2. Thus:

- **Row 1:** $l_{\text{eff,1}} = 167 \text{ mm}$
- $l_{\text{eff,2}} = 206 \text{ mm}$

Row 3 is also an "end bolt row", similar to row 1, but the value of bolt spacing $p$ is different.

- **Row 3:**
  - $p = p_{3,3} = 90 \text{ mm}$

Thus:

$$l_{\text{eff,nc}} = 2m + 0.625e + 0.5p = (2 \times 33.4) + (0.625 \times 79.4) + (0.5 \times 90) = 162 \text{ mm}$$

$$l_{\text{eff,cp}} = \pi m + p = (\pi \times 33.4) + 90 = 195 \text{ mm}$$

For this group, bolt row 2 is an "other inner bolt row". Therefore:

- $l_{\text{eff,cp}} = 2p$
- $l_{\text{eff,nc}} = p$

Here, the vertical spacing between bolts above and below row 2 is different, therefore use:

- **Row 2**:
  - $p = p_{2,3} + p_{3,3} = (100/2) + (90/2) = 95 \text{ mm}$
  - $l_{\text{eff,cp}} = 2p = 2 \times 95 = 190 \text{ mm}$
  - $l_{\text{eff,nc}} = p = 95 \text{ mm}$

Therefore, the total effective lengths for this group of rows are:

$$\sum l_{\text{eff,nc}} = 167 + 162 + 195 = 423 \text{ mm}$$
**Mode 1 resistance**

For Mode 1, using Method 2:

\[
F_{T,1,Rd} = \frac{(8n - 2e_\alpha)M_{pl,1}}{2mn + e_\alpha(m + n)}
\]

\[
M_{pl,1} = \frac{0.25 \sum_i \ell_{i,1} t_i f_{\mu,i}}{w_{pl,1}} = (0.25 \times 423 \times 20.5 \times 265)/1.0 = 11.8 \times 10^3 \times 10^3 \text{ Nmm} = 11.8 \text{ kNmm}
\]

\[
F_{T,1,Rd} = \frac{(8n - 2e_\alpha)M_{pl,1}}{2mn + e_\alpha(m + n)} = \frac{[(8 \times 41.8 - 2 \times 11.0) \times 11.8 \times 10^3](23.34 \times 41.8) - 11.0 \times (33.4 + 41.8)]}{1.870 \text{ kN}}
\]

\[
M_{pl,2} = \frac{0.25 \sum_i \ell_{i,2} t_i f_{\mu,i}}{w_{pl,2}}
\]

**Mode 2 resistance**

\[
F_{T,2,Rd} = \frac{2M_{pl,2} + n \sum_i F_{i,Rd}}{m + n}
\]

where:

\[
F_{i,Rd} = 203 \text{ kN}
\]

\[
\sum F_{i,Rd} = 6 \times 203 = 1.220 \text{ kN}
\]

**Mode 3 resistance (bolt failure)**

\[
F_{T,3,Rd} = \sum F_{i,Rd} = 6 \times 203 = 1.220 \text{ kN}
\]

### Resistance of column flange in bending

\[
F_{pl,1,Rd} = \min[F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}] = \min\{1.870; 991; 1.220\} = 991 \text{ kN}
\]

### Column web in transverse tension

The design resistance of an unstiffened column web in transverse tension is:

\[
F_{wc,Rd} = \frac{\sigma_{wc,Rd} w c f\c}{w_{wc,Rd}}
\]

where:

\[
b_{eff,t} = \sum \ell_{i,2} = 423 \text{ mm}
\]
The equation to use to calculate $\omega$ depends on $\beta$. As before, with $\beta = 0.00$ [-] therefore:

$$\omega = 1.00 [-]$$

**Summary: resistance of bolts rows 1, 2 and 3 combined**

Resistance of bolt rows 1, 2 and 3 combined, on the column side, is the smaller value of:

- Column flange in bending: $F_{t,1-3,Rd} = 991$ kN
- Column web in tension: $F_{w,1-3,Rd} = 1.435$ kN

Therefore, the resistance of bolt row 1, 2 and 3 combined is: $F_{t,1-3,Rd} = 991$ kN

The resistance of bolt row 3 on the column side is therefore limited to:

$$F_{t,3,Rd} = (F_{t,1-3,Rd} - F_{t,1-2,Rd}) = (991 - 698) = 293$$ kN

**ROWS 2 AND 3 COMBINED**

**Column side-flange in bending**

Following the same process as for row 1, 2 and 3 combined,

$$\Sigma_{w,c} = 2m + 2p = 2 \times \pi \times 33.4 + 2 \times 90 = 390$$ mm

$$\Sigma_{w,q} = 4m + 1.25e + p = 4 \times 33.4 + 1.25 \times 79.4 + 90 = 323$$ mm

Therefore,

$$\Sigma_{w,2} = \Sigma_{w,1} = 323$$ mm

**Mode 1 resistance**

$$M_{0,1,Rd} = \frac{0.25 \sum_{i} l_{i} x_{i} f_{i}}{7m} = \frac{(0.25 \times 323 \times 20.5^2 \times 265) \times 1.0}{10^3} = 9.0 \times 10^5 \text{ Nmm} = 9.0$$ kNm

$$F_{t,1,Rd} = \frac{(8n - 2e_{n})M_{0,1,Rd}}{2mn + e_{n}(m - n)} = \frac{[(8 \times 41.8 - 2 \times 11.0) \times 9.0 \times 10^5](2 \times 33.4 \times 41.8) - 11.0 \times (33.4 + 41.8)]}{(2 \times 33.4 \times 41.8)} = 1.428$$ kN

**Mode 2 resistance**

Here, as $l_{w,2} = l_{w,1} = M_{0,1,Rd} = M_{0,2,Rd}$:

$$F_{t,2,Rd} = \frac{2M_{0,2,Rd} + \sum_{i} F_{i,Rd}}{m + n} = \frac{(2 \times 9.0 \times 10^5 + 41.8 \times 4 \times 203)(33.4 + 41.8)}{323} = 691$$ kN

**Mode 3 resistance (bolt failure)**

$$F_{t,3,Rd} = \sum_{i} F_{i,Rd} = (6 \times 203) = 1.220$$ kN

**Column web in transverse tension**

$b_{w,t,wc} = 323$ mm

The equation to use to calculate $\omega$ depends on $\beta$. As before, with $\beta = 0.00$ [-] therefore:

$$\omega = 1.00 [-]$$

$$F_{w,1-3,Rd} = \frac{c_{b}b_{w,t,wc} l_{w} f_{w}}{7m} = \frac{[(1.0 \times 323 \times 12.8 \times 265) \times 10^6] \times 1.0}{10^3} = 1.096$$ kN
Beam side-end plate in bending

On the beam side, row 1 is not part of a group but the resistance of row 3 may be limited by the resistance of rows 2 and 3 as a group. Determine the effective lengths for rows 2 and 3 combined:

Row 2 is a "first bolt-row below tension flange of beam" in Table 6.6.

\[ l_{eff,2} = mm + p \]

Here: \( p = p_{2,3} = 90 \text{ mm} \) \( n = 48.2 \) \( m = 38.6 \) (as for row 2 alone)

\[ l_{eff,2} = mm + p = (n \times 38.6) + 90 = 211 \text{ mm} \]

Obtain \( \alpha \) from Figure 6.11 - EN 1993-1-8 (or Annex G) using (see sheet 9):

\[ \lambda_1 = 0.3395 \quad \lambda_2 = 0.30647 \]

From Figure 6.11 \( \alpha = 7.3 \) [\]

\[ l_{eff,2} = 0.5p + \alpha m - (2m + 0.625e) = 0.5 \times 90 + 7.3 \times 38.6 - [2 \times 38.6 + (0.625 \times 75)] = 204 \text{ mm} \]

Row 3 is an "other end bolt-row" in Table 6.6

\[ l_{eff,3} = mm + p = (n \times 38.6) + 90 = 211 \text{ mm} \]

\[ l_{eff,3} = 2m + 0.625e + 0.5p = (2 \times 38.6) + (0.625 \times 75) + (0.5 \times 90) = 169 \text{ mm} \]

Therefore, the total effective lengths for this group of rows are:

\[ \Sigma l_{eff,2} = 204 + 169 = 373 \text{ mm} \]

\[ \Sigma l_{eff,3} = 211 + 211 = 422 \text{ mm} \]

Hence, \[ \Sigma l_{eff} = \Sigma l_{eff,2} = 373 \text{ mm} \]

Mode 1 resistance (rows 2 + 3)

For Mode 1 failure, using Method 2:

\[ F_{T,1,UM} = \frac{(8n - 2e_n)M_{U,1}}{2mn + e_n (m + n)} \]

where:

\( n = 48.2 \text{ mm} \)

\( e_n = 11.0 \text{ mm} \)

\[ M_{U,1} = \frac{0.25 \sum_{i=1}^{n} l_{eff,i} t_i f_{w,i}}{y_{G1}} = 0.25 \times 373 \times 25^2 \times 265/1.0 = 15.4 \times 10^3 \times 10^3 \text{ Nmm} = 15.4 \text{ kNm} \]

\( m = 38.6 \text{ mm} \)

\[ F_{T,1,UM} = \frac{(8n - 2e_n)M_{U,1}}{2mn + e_n (m + n)} = \frac{(8 \times 48.2 - 2 \times 11.0) \times 15.4 \times 10^3}{(2 \times 38.6 \times 48.2 - 11.0 \times (38.6 + 48.2))} = 2.034 \text{ kN} \]
Mode 2 resistance (rows 2 + 3)

\[
F_{T,2,3,Rd} = \frac{2M_{d,2,3,Rd} + n \sum F_{d,Rd}}{m + n}
\]

where:

\[
F_{d,Rd} = 203 \text{ kN}
\]

\[
\sum F_{d,Rd} = 4 \times 203 = 813 \text{ kN}
\]

\[
M_{d,2,3,Rd} = \frac{0.25 \sum I_{w,2,3} f_{y,p}}{f_{y,p}}
\]

Here, as \(I_{w,2} = I_{w,1}\):

\[
M_{d,2,3,Rd} = M_{d,1,2,Rd} = 15.4 \text{ kNm}
\]

\[
F_{T,2,3,Rd} = \frac{2M_{d,2,3,Rd} + n \sum F_{d,Rd}}{m + n} = \frac{(2 \times 15.4 \times 10^3 + 48.2 \times 813)/(38.6 + 48.2)}{808 \text{ kN}}
\]

Mode 3 resistance (bolt failure) (rows 2 + 3)

\[
F_{T,3,Rd} = \sum F_{d,Rd} = 4 \times 203 = 813 \text{ kN}
\]

Resistance of end plate in bending

\[
F_{T,3,Rd} = \min(F_{T,1,Rd}; F_{T,2,3,Rd}; F_{T,3,Rd}) = \min(2.034; 808; 813) = 808 \text{ kN}
\]

Beam web in tension

This verification is not applicable as the beam flange (stiffener) is within the tension length. (*)

Summary: resistance of bolt rows 2 and 3 combined

Resistance of bolt rows 2 and 3 combined, on the beam side, is:

End plate in bending:

\[
F_{T,2,3,Rd} = 808 \text{ kN}
\]

Therefore(*), on the beam side:

\[
F_{T,2,3,Rd} = F_{T,3,Rd} = 808 \text{ kN}
\]

The resistance of bolt row 3 on the beam side is therefore limited to:

\[
F_{O,3,Rd} = F_{O,3,Rd} = (808 - 321) = 487 \text{ kN}
\]

Resistance of bolts rows 2 and 3 combined, on the column side, is:

Column flange in bending:

\[
F_{T,3,Rd} = 691 \text{ kN}
\]

Column web in tension:

\[
F_{T,3,Rd} = 1.096 \text{ kN}
\]

Therefore, on the column side:

\[
F_{O,3,Rd} = 691 \text{ kN}
\]

The resistance of bolt row 3 on the column side is therefore limited to:

\[
F_{O,3,Rd} = F_{O,3,Rd} = (691 - 321) = 370 \text{ kN}
\]

Summary: resistance of bolt row 3

Resistance of bolt row 3 is the smallest value of:

Column flange in bending:

\[
F_{W,Rd} = 398 \text{ kN}
\]

Column web in tension:

\[
F_{W,Rd} = 790 \text{ kN}
\]

Beam web in tension:

\[
F_{W,Rd} = 673 \text{ kN}
\]

End plate in bending:

\[
F_{W,Rd} = 407 \text{ kN}
\]

(cont'd)
SUMMARY OF TENSION RESISTANCES

The above derivation of effective resistances of the tension rows may be summarized in tabular form, as shown below.

Resistances of rows $F_{tr,Rd}$ [kN]

<table>
<thead>
<tr>
<th></th>
<th>Column flange</th>
<th>Column web</th>
<th>End plate</th>
<th>Beam web</th>
<th>MIN</th>
<th>Effect. resist.</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROW 1, alone</td>
<td>398</td>
<td>790</td>
<td>377</td>
<td>N/A</td>
<td>377</td>
<td>(min)</td>
</tr>
<tr>
<td>ROW 2, alone</td>
<td>398</td>
<td>790</td>
<td>407</td>
<td>673</td>
<td>398</td>
<td>ROW 2</td>
</tr>
<tr>
<td>ROW 2 + 1</td>
<td>698</td>
<td>1130</td>
<td>N/A</td>
<td>N/A</td>
<td>698</td>
<td>ROW 2 (effect.)</td>
</tr>
<tr>
<td>ROW 2 (effect.)</td>
<td>(698 – 398)</td>
<td>(300)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ROW 3, alone</td>
<td>398</td>
<td>790</td>
<td>407</td>
<td>673</td>
<td>398</td>
<td>ROW 3</td>
</tr>
<tr>
<td>ROW 3 + 2 + 1</td>
<td>991</td>
<td>1435</td>
<td>N/A</td>
<td>N/A</td>
<td>991</td>
<td>ROW 3 (effect. a)</td>
</tr>
<tr>
<td>ROW 3 (effect. a)</td>
<td>(991 – 698)</td>
<td>(293)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ROW 3 + 2</td>
<td>691</td>
<td>1096</td>
<td>808</td>
<td>N/A</td>
<td>691</td>
<td>ROW 3 (effect. b)</td>
</tr>
<tr>
<td>ROW 3 (effect. b)</td>
<td>(691 – 321)</td>
<td>(370)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

COMPRESSION ZONE T-STUBS

Column web in transverse compression

The design resistance of an unstiffed column web in transverse compression is determined from:

$$F_{w,c,Rd} = \frac{\sigma_k \cdot b_{stiff,c} \cdot t_{w,c} \cdot f_{w,c}}{T_{w,c}}$$

(crusising resistance)

but:

$$F_{w,c,Rd} \leq \left( \frac{\sigma_k \cdot b_{stiff,c} \cdot t_{w,c} \cdot f_{w,c}}{T_{w,c}} \right) \rho$$

(buckling resistance)

For a bolted end plate:

$$b_{stiff,c} = t_b + 2s/2 + 5 \left( t_b + s \right) + s_p = t_b + 2s + 5 \left( t_b + s \right) + s_p$$

For rolled I and H column section $s = t_{c,c}$. Thus: $s = t_{c,c} = 12.7$ mm

$s_p$ is the length obtained by dispersion at $45^\circ$ through the end plate:

$$0.5s_p = 25 \text{ mm}$$

$t_b = 25 \text{ mm}$

$b_p = 670 \text{ mm}$ (End-plate depth)

$s = 12 \text{ mm}$

$t_b = 20.5 \text{ mm}$

$t_b = 15.6 \text{ mm}$
\[ s_p = 2 b_p = 2 \times 25 = 50 \text{ mm} \]

Verify that the depth of the end plate \((h_p)\) is sufficient to allow the dispersion of the force. Minimum \(h_p\) required is:

\[ h_p \geq a_v + x + h_v + s_v + t_b = (50 + 40 + 533.1 + 12 + 25) = 660.1 \text{ mm} \]

\[ h_p = 670 \text{ mm} \]

As 670 mm ≥ 660.1, the depth of the end plate is sufficient. 

[Satisfactory]

Therefore:

\[ b_{eff,c,wc} = t_b + 2a_v \sqrt{2} + 5 (b_w + s) + s_v = t_b + 2a_v + 5 (b_w + s) + s_v = 15.6 + 2 \times 12 + 5 \times (20.5 + 12.7) + 50 = 255.6 \text{ mm}. \]

\( \rho \) is the reduction factor for plate buckling

\[ \text{If } \bar{x}_p \leq 0.72 \implies \rho = 1.0 \]

\[ \text{If } \bar{x}_p > 0.72 \implies \rho = \frac{1}{1 \bar{x}_p} \]

\( \bar{x}_p \) is the plate slenderness:

\[ \bar{x}_p = 0.932 \sqrt{\frac{b_{eff,c,wc} d_{eff,c,wc}}{E I_{wc}}} = 0.932 \sqrt{\frac{(255.6 \times 200.3 \times 265)}{(210 \times 10^8 + 12.8^2)}} = 0.59 [-] \]

As 0.59 mm ≤ 0.72 Eq. (6.13a) gives:

\( \rho = 1.00 [-] \)

\( \varpi \) is determined from Table 6.3 based on \( \beta \) (see sheet ).

The equation to use to calculate \( \varpi \) depends on \( \beta \). As before, with \( \beta = 0.00 [-] \) therefore: \( \varpi = 1.00 [-] \)

\( k_{wc} \) is a reduction factor that takes account of compression in the column web.

Here, it is assumed that \( k_{wc} = 1.00 [-] \)

\[ F_{Lc,wc,Rd} = 867 \text{ kN} \]

\( F_{Lc,wc,Rd} \) is the compressive resistance of the column web.

Therefore:

\[ F_{Lc,wc,Rd} = 867 \text{ kN} \] (crushing resistance governs)

**Beam flange and web in compression**

The resultant of the design resistance of a beam flange and adjacent compression zone of the web is determined using:

\[ F_{b,fl,Rd} = \frac{M_{b,fl}}{h - t_b} \]

where:

\( M_{b,fl} \) is the design resistance of the beam

At this stage, assume that the design shear force in the beam does not reduce \( M_{b,fl} \)

Therefore:

\[ F_{b,fl,Rd} = \frac{M_{b,fl}}{h - t_b} = 1.3 \frac{b_{fl} (h_b - t_b) f_{bc,fl}}{t_b} = 1.3 \frac{f_{bc,fl} b_{fl}}{t_b} \] (approximated, conservative)
Summary: resistance of compression zone

Column web in transverse compression: \( F_{c,wc,Rd} = 867 \text{ kN} \)
Beam flange and web in compression: \( F_{c,b,Rd} = 1.167 \text{ kN} \)

Compression resistance:
\[
F_{c,Rd} = \min\{F_{c,wc,Rd}; F_{c,b,Rd}\} = \min\{867; 1.167\} = 867 \text{ kN}
\]

Resistance of column web panel in shear

The plastic shear resistance of an unstiffened web is given by:

\[
V_{w,Rd} = 0.9 f_{w,y} A_w \frac{t_w}{V^3}
\]

The resistance is not evaluated here, since there is no design shear in the web because the moments for the beams are equal and opposite.

**MOMENT RESISTANCE**

**EFFECTIVE RESISTANCE OF BOLTS ROWS**

The resistances of each of the three bolt rows in the tension zone are:

\[
\begin{align*}
F_{t,1,Rd} &= 377 \text{ kN} \quad \text{(for ROW 1)} \\
F_{t,2,Rd} &= 300 \text{ kN} \quad \text{(for ROW 2)} \\
F_{t,3,Rd} &= 293 \text{ kN} \quad \text{(for ROW 3)}
\end{align*}
\]

The UK NA states that the effective resistances should be reduced when either the resistance of one of the higher rows exceeds 1.9\(F_{t,Rd}\)

\[
\begin{align*}
\text{(here: } 1.9 F_{t,1,Rd} &= 1.9 \times 203 = 386 \text{ kN} \quad \geq 377 \text{ mm} & \text{(no reduction is necessary according to UK NA)}
\end{align*}
\]

or (no reduction is also necessary):

\[
\begin{align*}
\ell_h &> \frac{d}{1.9 \sqrt{f_y}} \\
\ell_b &> \frac{d}{1.9 \sqrt{f_y}}
\end{align*}
\]
In this case, the limiting thickness are respectively:
\[ t_p \leq 21.9 \text{ mm} \quad \text{with} \quad t_p = 25 \text{ mm} \quad \text{(reduction is necessary).} \]
\[ t_c \leq 21.9 \text{ mm} \quad \text{with} \quad t_c = 20.5 \text{ mm} \quad \text{(no reduction is necessary).} \]

(No reduction is applied according to UK NA: a plastic distribution can be assumed).

\[
F_{t1,Rd} = 1.9 \times F_{t2,Rd} \quad \text{Reduction factors:}
\]

\[
h_2/h_1 = 0.823 \quad \text{(for ROW 2)}
\]
\[
h_2/h_1 = 0.664 \quad \text{(for ROW 3)}
\]

The effective resistances of each of the three bolt rows in the tension zone are:

\[
F_{t1,Rd} = 377 \text{ kN (for ROW 1)}
\]
\[
F_{t2,Rd} = 300 \text{ kN (for ROW 2)}
\]
\[
F_{t3,Rd} = 293 \text{ kN (for ROW 3)}
\]

**EQUILIBRIUM OF FORCES**

The sum of the tensile forces, together with any axial compression in the beam, cannot exceed the resistance of the compression zone. Similarly, the design shear cannot exceed the shear resistance of the column web panel; this is not relevant in this case as the moments in the identical beams are equal and opposite.

For horizontal equilibrium:

\[
\sum F_{tr,Rd} + N_{Ed} = F_{c,Rd}
\]

In this case there is no axial compression/tension in the beam \( N_{Ed} = 0 \)

Therefore, for equilibrium of forces:

\[
\sum F_{tr,Rd} = F_{t1,Rd}
\]

**Effective resistances to achieve equilibrium:**

\[
F_{t1,Rd} = 377 \text{ kN}
\]
\[
F_{t2,Rd} = 300 \text{ kN}
\]
\[
F_{t3,Rd} = 293 \text{ kN}
\]
\[
F_{t4,Rd} = 867 \text{ kN}
\]
Reduction do Tension Row Force

The tension forces in the bolts rows and the compression force at bottom flange level must be in equilibrium with any axial force in the beam. The forces cannot exceed the compression resistance of the joint, nor, where applicable, the shear resistance of the web panel. When the sum of the effective design tension resistances \( \sum F_{ti,Rd} \) exceeds \( (F_{c,Rd} - N_{Ed}) \), an allocation of reduced bolt forces must be determined that satisfies equilibrium. To achieve a set of bolt row forces that is in equilibrium, the effective tension resistances should be reduced from the values calculated in step 4, starting with the bottom row and working up progressively, until equilibrium is achieved.

This allocation achieves the maximum value of moment resistance that can be realised.

MOMENT RESISTANCE OF JOINT

The moment resistance of the beam to column joint \( (M_{j,Rd}) \) may be determined using:

\[
M_{j,Rd} = \sum h_i F_{ti,Rd}
\]

Taking the centre of compression to be at the mid-thickness of the compression flange of the beam:

\[
h_1 = 565 \text{ mm} \quad h_2 = 465 \text{ mm} \quad h_3 = 375 \text{ mm}
\]

Thus, the moment resistance of the beam to column joint is:

\[
M_{j,Rd} = \sum h_i F_{ti,Rd} = h_1 F_{1,Rd} + h_2 F_{2,Rd} + h_3 F_{3,Rd} =
\]

\[
= (565 \times 377 + 465 \times 300 + 375 \times 190)/10^3 = 424 \text{ kNm}.
\]

VERTICAL SHEAR RESISTANCE

Resistance of bolt group

The shear resistance of a non-preloaded M24 class 8.8 bolt in single shear is:

\[
F_{v,Rd} = 136 \text{ kN}.
\]

\[
F_{b,Rd} = 259 \text{ kN} \quad (\text{end bolts in 20,5 mm ply, bearing resistance})
\]

Hence 135,6 kN governs: \( F_{Rd} = \min\{F_{v,Rd}, F_{b,Rd}\} = \min\{136; 259\} = 136 \text{ kN} \)

The shear resistance of the upper rows may be taken conservatively as 28% of the shear resistance without tension (this assumes that these bolts are fully utilized in tension) and thus the shear resistance of all 4 rows is:

\[
V_{j,Rd} = (2 + 6 \times 0.28) \times 136 = 3.68 \times 136 = 499 \text{ kN}.
\]

WELD DESIGN

The simple approach requires that the welds to the tension flange and the web should be full strength and the weld to the compression flange is of nominal size only, assuming that it has been prepared with a sawn cut end.

BEAM TENSION FLANGE WELDS

A full strength weld is provided by symmetrical fillet welds with a total throat thickness at least equal to the flange thickness.

Required throat size: \( t_w/2 = 15,6/2 = 7,8 \text{ mm} \).

Weld throat provided: \( a_w = 12/2 = 8,5 \text{ mm} \), which is adequate.

BEAM COMPRESSION FLANGE WELDS

Provide a nominal fillet weld either side of the beam flange. An 8 mm leg length fillet weld will be satisfactory.

BEAM TENSION FLANGE WELDS

For convenience, a full strength weld is provided to the web:

Required throat size: \( t_w/2 = 10,1/2 = 5,1 \text{ mm} \).

Weld throat provided: \( a_w = 8/2 = 5,7 \text{ mm} \), which is adequate.