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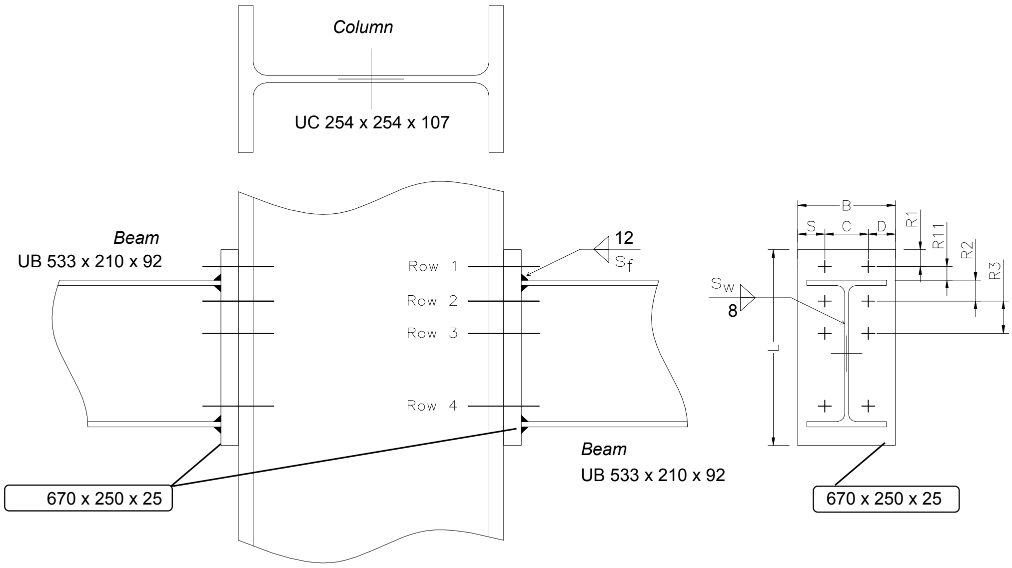
Reference clauses:
EN 1993-1-8

JOINT CONFIGURATION AND DIMENSIONS: bolted end plate connection (unstiffened)
Resistances for the extended end plate connnection.

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)	**RESISTANCE OF JOINT**			
Beams:	UB 533 x 210 x 92 in S 275 (EN 10025-2)	Bending moment:	$M_{j,Rd}$ =	**424 kNm**	☑
End plate:	670 x 250 x 25 in S 275 (EN 10025-2)	Vertical shear:	$V_{j,Rd}$ =	**499 kN**	☑

Bolts: M24 non preloaded class 8.8 bolts

ISO 4016
ISO 898-1
ISO 7089/7091

Welds: Fillet welds. Assumed weld sized:

s_f = 12mm (flange)

s_w = 8 mm (web)

Bolt spacings

S = D = 75 mm
C = w = 100 mm

R1 = e_x =	50 mm	R11 + R2 = p_{1-2} =	100 mm (spacing row 1-2)
R11 = x =	40 mm		
R2 =	60 mm	R3 = p_{2-3} =	90 mm (spacing row 2-3)
R3 =	90 mm		

Technical References: *Joints in Steel Construction: Moment-Resisting Joints to Eurocode 3*. Publication P398.
The Steel Construction Institute. www.steel-sci.com
The British Constructional Steelwork Association Limited. www.steelconstruction.org

Project: *Project No 1*

Client: *Client No 1*

Location: *Location No 1*

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mail@domain.com

Made by: *CS* **Date:** *25/05/2019* **Job No:** *1*

Checked: *CS* **Revision:** *CS*

Reference clauses:

DIMENSIONS AND SECTION PROPERTIES

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

Depth	h_c =	266,7 mm		
Width	b_c =	258,8 mm		
Web thickness	t_{wc} =	12,8 mm		
Flange thickness	t_{fc} =	20,5 mm		
Root radius	r_c =	12,7 mm		
Depth between fillets	d_c =	200,3 mm		
Area	A_c =	13640 mm²	=	136,4 cm²

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

Depth	h_b =	533,1 mm		
Width	b_b =	209,3 mm		
Web thickness	t_{wb} =	10,1 mm		
Flange thickness	t_{fb} =	15,6 mm		
Root radius	r_b =	12,7 mm		
Depth between fillets	d_b =	476,5 mm		
Area	A_b =	11740 mm²	=	117,4 cm²

End plates

Depth	h_p =	670 mm
Width	b_p =	250 mm
Thickness	t_p =	25 mm

Bolts

M24 non preloaded class 8.8 bolts

Diameter of bolt shank	d =	24 mm
Diameter of hole	d_0 =	26 mm
Shear area	A_s =	353 mm²
Diameter of washer	d_w =	44 mm (ISO 7089/7091)

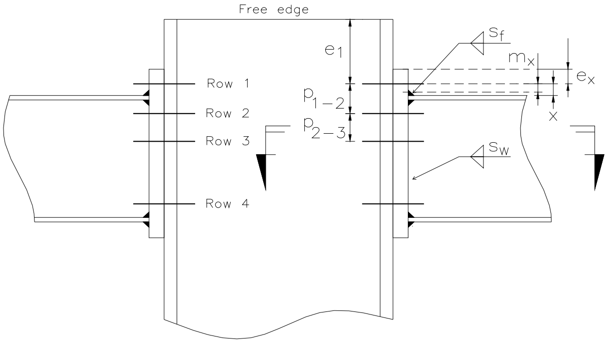
Bolts spacings

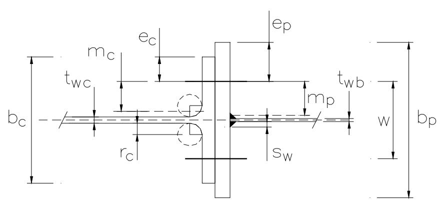
Column

End distance (row 1 from column edge)	e_1 =	1000 mm
Spacing (guage)	w =	100 mm
Edge distance	e_c =	79,4 mm
Spacing row 1-2	$p_{1\text{-}2}$ =	100 mm
Spacing row 2-3	$p_{2\text{-}3}$ =	90 mm

End plate

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





EN 1993-1-8
Figure 6.8
Figure 6.10
Figure 6.13

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Reference clauses:

MATERIAL STRENGTHS

Steel strength — **BS EN 1993-1-1 NA. 2.4**

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) — **EN 10025-2 Table 7**

For t ≤ 16 mm	$f_y = R_{eH}$ =	275 N/mm²
For 16 mm < t ≤ 40 mm	$f_y = R_{eH}$ =	265 N/mm²
For 3 mm ≤ t ≤ 100 mm	$f_u = R_{eH}$ =	410 N/mm²

Hence:

Beam yield strength	$f_{y,b}$ =	275 N/mm²
Beam ultimate strength	$f_{u,b}$ =	410 N/mm²
Column yield strength	$f_{y,c}$ =	265 N/mm²
Column ultimate strength	$f_{u,c}$ =	410 N/mm²
End plate yield strength	$f_{y,p}$ =	265 N/mm²
End plate ultimate strength	$f_{u,p}$ =	410 N/mm²

Bolt strength — **EN 1993-1-8 Table 3.1**

Nominal yield strength	f_{yb} =	640 N/mm²
Nominal ultimate strength	f_{ub} =	800 N/mm²

PARTIAL FACTORS FOR RESISTANCE — **EN 1993-1-1**

Structural steel — **Table 2.1** (see NA)

γ_{M0} = 1,00

γ_{M1} = 1,00

γ_{M2} = 1,10

Parts in connection — **Table 2.1** (see NA)

γ_{M2} = 1,25 (bolts, welds, plates in bearing).

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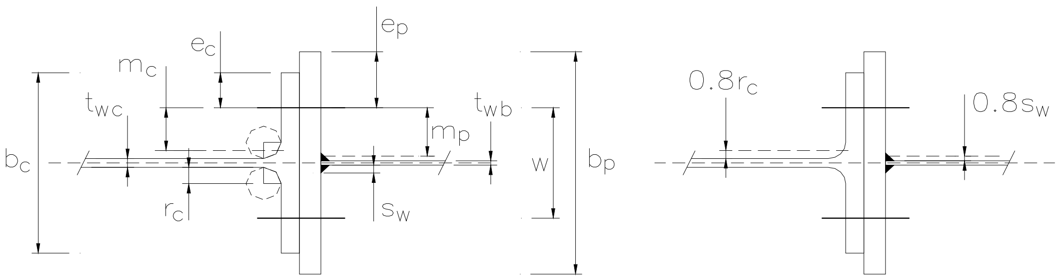
TENSION ZONE T-STUBS

When prying forces may develop, the design tension resistance ($F_{T,Rd}$) 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). **EN 1993-1-18 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_{min}, m and l_{eff} for the unstiffed column flange **EN 1993-1-18 6.2.4.1(2)**

$$m = m_c = \frac{w - t_{wc} - 2 \cdot 0.8 r_c}{2} = \quad 0{,}5 \times [100 - 12{,}8 - 2 \times 0{,}8 \times 12{,}7] = 33{,}4 \text{ mm}$$

$e_{min} = \min[e_p; e_c] =$ min(75; 79,4] = 75 mm **EN 1993-1-18 Figure 6.8**

For Mode 1, $l_{eff,1}$ is the lesser of $l_{eff,cn}$ and $l_{eff,cp}$ **Table 6.6**

$l_{eff,cp} = 2\pi m =$ 2 x 3,14 x 33,4 = 210 mm

$l_{eff,nc} = 4m + 1.25e_c =$ 4 x 33,4 + 1,25 x 79,4 = 233 mm

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

For failure Mode 2, $l_{eff,2} = l_{eff,nc}$ **STEP 1**

Therefore $l_{eff,2} = l_{eff,nc} =$ 233 mm

Mode 1 resistance

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

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)}$$ **EN 1993-1-18 Table 6.2**

where:

$m = m_c =$ 33,4 mm

$n = \min[e_{min}; 1.25m] =$ min[75; (1.25 x 33,4)] = min[75; 41,8] = 41,8 mm

(cont'd)

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$M_{pl,1,Rd} = \frac{0.25 \sum l_{eff,1} t_{fc}^2 f_{y,c}}{\gamma_{M0}} =$ [0,25 x 210 x (20,5)² x 265]/1,00 = 5847 x 10³ Nmm = 5,85 kNm

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

$d_w =$ 44 mm

$e_w = d_w/4 =$ (44)/4 = 11 mm

Therefore,

$F_{T,1,Rd} = \frac{(8n - 2e_w) M_{pl,1,Rd}}{2mn - e_w (m+n)} =$ [(8 x 41,8 − 2 x 11) x 5847 x 10³]/{10³ x [2 x 33,4 x 41,8 − 11 x (33,4 + 41,8)]} = 928,1 kN

Mode 2 resistance

For Mode 2

$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m+n} =$ [2 x (6487 x 10³) + 41,8 x 406,7 x 10³)/(33,4 + 41,8) = 398,4 kN — **EN 1993-1-18 Table 6.2**

where:

$M_{pl,2,Rd} = \frac{0,25 \sum l_{eff,2} t_{fc}^2 f_{y,c}}{\gamma_{M0}} =$ [0,25 x 233 x (20,5)² x 265]/1 = 6487 x 10³ Nmm = 6,49 kNm

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

$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 f_{ub} A_s}{\gamma_{M2}} =$ (0,9 x 800 x 353)/1,25 = 203,3 x 10³ N = 203,3 kN (for a single bolt) — **EN 1993-1-18 Table 3.4**

For 2 bolts in the row $\sum F_{t,Rd}$ = 2 x 203,3 x 10³ = 406,7 x 10³ N = 406,7 kN

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \sum F_{t,Rd} =$ 406,7 kN — **EN 1993-1-18 Table 6.2**

Resistance of column flange in bending

$F_{t,fc,Rd} = \min[F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}] =$ min[928,1; 398,4; 406,7] = 398,4 kN — **EN 1993-1-18 6.2.4.1(6)**

Column web in transverse tension

The design resistance of an unstiffened column web to transverse tension is determined from: — **EN 1993-1-18 6.2.6.3(1)**

$$F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$$ **Eq. (6.15)**

where ω is a reduction factor that allows for the interaction with shear in the column web panel.

Trasformation parameter β = 0,0 [-] ⇨ Reduction factor: $0 \leq \beta \leq 0.5 \rightarrow \omega = 1{,}00$

EN 1993-1-8
Table 5.4
Table 6.3

Here, as $\min[F_{T,1,Rd}; F_{T,2,Rd}] =$ [928,1; 398,4] = 398,4 kN

with:

$l_{eff,1} =$ 210 mm

$l_{eff,2} =$ 233 mm

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

$b_{eff,t,wc} =$ 233 mm.

$f_{y,wc} = f_{y,c} =$ 265 N/mm²

$\omega =$ 1,00 [-]

Thus,

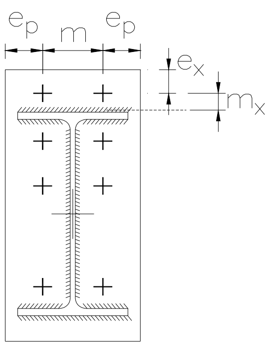
$F_{t,wc,Rd} =$ [(1,00 x 233 x 12,8 x 265)/1,00]/10³ = 790,4 kN

End plate in bending

STEP 1

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

EN 1993-1-8
Figure 6.10



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

$e = e_p =$ 75 mm

$e_x =$ 50 mm

$m_x = x - 0.8s_f =$ 40 − 0,8 x 12 = 30,4 mm

For Mode 1, $l_{eff,1}$ is the lesser of $l_{eff,nc}$ and $l_{eff,cp}$

EN 1993-1-8
Table 6.6

$l_{eff,cp} = \min[2\pi m_x; \pi m_x + w; \pi m_x + 2e] =$ min[(2π x 30,4); (π x 30,4 + 100); (π x 30,4 + 2 x 75)] = min[191; 196; 246] = 191 mm

$l_{eff,nc} = \min[4m_x + 1.25e_x; e + 2m_x + 0.625e_x; 0.5b_p; 0.5w + 2m_x + 0.625e_x] =$

EN 1993-1-8
Table 6.6

min[(4 x 30,4 + 1.25 x 50); (75 + 2 x 30,4 + 0.625 x 50); (0.5 x 250); (0.5 x 100 + 2 x 30,4 + 0.625 x 50)] =

$l_{eff,nc} =$ min[184; 167; 125; 142] = 125 mm

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$l_{eff,1} = \min[l_{eff,nc}; l_{eff,cp}] =$ min[125; 191] = 125 mm

For Mode 2, $l_{eff,2} = l_{eff,nc} =$ 125 mm

Mode 1 resistance

For Mode 1 failure, using Method 2: **EN 1993-1-8 Table 6.2**

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

where:

$n = e_{min} = \min[e_x; 1.25m_x] =$ min[50; 1.25 x 30,4] = 38,3 mm

$m = m_x =$ 30,4 mm **Sheet 6**

$e_w =$ 11,0 mm **Sheet 5**

$f_y = f_{y,p} =$ 265 N/mm²

$t = t_p =$ 25 mm

$$M_{pl,1,Rd} = \frac{0.25\sum l_{eff,1} t_p^2 f_{y,p}}{\gamma_{M0}} =$$ (0.25 x 125 x 25² x 265)/1 = 5.176 x 10³ = Nmm = 5,2 kNm

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)} =$$ (8 x 38,3 − 2 x 11,0) x 5.176 x 10³/{10³ x [2 x 30,4 x 38,3 − 11,0 x (30,4 + 38,3]} = 936 kN

Mode 2 resistance

EN 1993-1-8 Table 6.2

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n}$$

where:

$$M_{pl,2,Rd} = \frac{0.25\sum l_{eff,2} t_p^2 f_{y,p}}{\gamma_{M0}} =$$ (0.25 x 125 x 25² x 265)/1 = 5.176 x 10³ = Nmm = 5,2 kNm

$l_{eff,2} =$ 125 mm

$\Sigma F_{t,Rd} =$ 407 x 10³ N = 407 kN **Sheet 5**

Therefore, $F_{T,2,Rd} =$ (2 x 5.176 x 10³ + 38,3 x 407 x 10³)/{10³ x (30,4 + 38,3)} = 377 kN

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \Sigma F_{t,Rd} =$ 407 kN

Resistance of end plate in bending

$F_{t,ep,Rd} = \min[F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}] =$ min[936; 377; 407] = 377 kN **EN 1993-1-8 6.2.4.1(6)**

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 tothis 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 kN

Column web in tension: $F_{t,wc,Rd}$ = 790 kN

End plate in bending: $F_{t,ep,Rd}$ = 377 kN

Therefore, the resistance of bolt row 1 is:

$F_{t1,Rd} = \min[F_{t,fc,Rd}; F_{t,wc,Rd}; F_{t,ep,Rd}]$ = min[398; 790; 377] = 377 kN

BOLT ROW 2

Firstly, consider row 2 alone.

Column flange in bending **STEP 1**

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

$F_{t,fc,Rd}$ = 398 kN **Sheet 5**

Column web in transverse tension **STEP 1B**

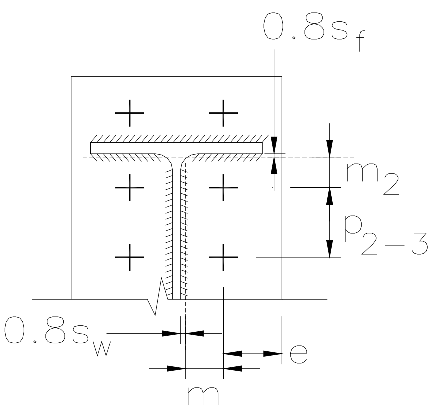
The column web resistance to transverse tension will also be as calculated for bolt row 1.

Therefore:

$F_{t,wc,Rd}$ = 790 kN **Sheet 6**

End plate in bending **STEP 1**

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. **Sheet 4**



$$m = m_p = \frac{w - t_{wb} - 2 \cdot 0.8 s_w}{2} =$$ (100 − 10,1 − 2 x 0.8 x 8)/2 = 38,6 mm

$e = e_p$ = 75 mm

$m_2 = R2 - t_{fb} - 0.8 s_f$ = 60 − 15,6 − (0.8 x 12) = 34,8 mm

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

Parameters required to determine α: $\lambda_1 = \frac{m}{m+e}$ and $\lambda_2 = \frac{m_2}{m+e}$

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λ_1 = 38,6/(38,6 + 75) = 0,34

λ_2 = 34,8/(38,6 + 75) = 0,31

Thus, by interpolation (see EN 1993-1-8, Figure 6.11), α = 7,3 [-]

EN 1993-1-8 Figure 6.11

Table 6.6

$l_{eff,cp}$ = $2\pi m$ = 2 x π x 38,6 = 242 mm

$l_{eff,nc}$ = αm = 7,3 x 38,6 = 283 mm

$l_{eff,2}$ = $l_{eff,nc}$ = 283 mm

$l_{eff,1}$ = min[$l_{eff,cp}$; $l_{eff,nc}$] = min[242; 283] = 242 mm

EN 1993-1-8 Table 6.2

Mode 1 resistance

For Mode 1, using Method 2:

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

where:

SHEET 4

n = min[e_{min}; 1.25m] = min[75; (1.25 x 38,6)] = 48,2 mm

SHEETS 5/2/3

e_w = 11,0 mm

t_p = 25 mm

$f_{y,p}$ = 265 N/mm²

$$M_{pl,1,Rd} = \frac{0.25\sum l_{eff,1} t_p^2 f_{y,p}}{\gamma_{M0}} =$$ (0,25 x 242 x 25² x 265)/1,00 = 10,0 x 10³ x 10³ Nmm

EN 1993-1-8 Table 6.2

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)} =$$ [(8 x 48,2 − 2 x 11,0) x 10³]/[(2 x 38,6 x 48,2) − 11,0 x (38,6 + 48,2)] = 1.320 kN

SHEET 5

Mode 2 resistance

$$F_{T,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n}$$

$F_{t,Rd}$ = 203 kN

$$M_{pl,2,Rd} = \frac{0.25\sum l_{eff,2} t_p^2 f_{y,p}}{\gamma_{M0}} =$$ (0,25 x 283 x 25² x 265)/1,00 = 11,7 x 10³ x 10³ Nmm

$\sum F_{t,Rd}$ = 2 x 203 = 407 kN

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n} =$$ (2 x 11,7 x 10³ + 48,2 x 407)/(38,6 + 48,2) = 496 kN

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \sum F_{t,Rd}$ = 407 kN

Resistance of end plate in bending

$F_{t,ep,Rd} = \min[F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}] =$ [1.320; 496; 407] = 407 kN

EN 1993-1-8 6.2.4.1(6)

Beam web in tension

STEP 1B

The design tension resistance of the web is determined from:

EN 1993-1-8 6.2.6.8(1)

$$F_{t,wb,Rd} = \frac{\omega b_{eff,t,wb} t_{wb} f_{y,b}}{\gamma_{M0}}$$

Eq. (6.22)

where:

EN 1993-1-8 6.2.6.8(2)

$b_{eff,t,wb} = \min[l_{eff,1}; l_{eff,2}] =$ 242 mm (conservatively, consider the smallest l_{eff} from earlier calculations).

$t_{wb} =$ 10,1 mm **SHEET 2**

ω = 1,00 [-] **SHEET 6**

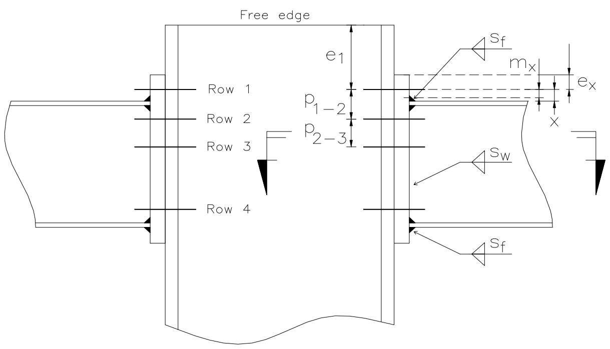
Therefore:

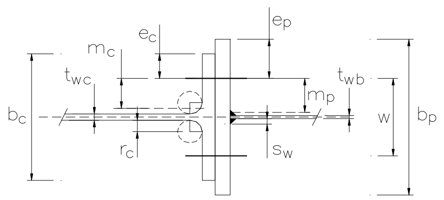
$$F_{t,wb,Rd} = \frac{\omega b_{eff,t,wb} t_{wb} f_{y,b}}{\gamma_{M0}} =$$ [(1,00 x 242 x 10,1 x 275)/1,00]/10³ = 673 kN

The above resitances 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





$e_c =$ 79,4 mm **SHEET 2**
$m_c =$ 33,4 mm **SHEET 4**
$p_{2\text{-}3} =$ 100 mm **SHEET 2**
$e_1 =$ 1000 mm **SHEET 3**

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:

$l_{eff,nc} = \min[(2m + 0.625e + 0.5p); (e_1 + 0.5p)] =$ min[(2 x 33,4 + 0.625 x 79,4 + 0.5 x 100); (1000 + 0.5 x 100)] = 167 mm

$l_{eff,cp} = \min[(\pi m + p); (2e_1 + p)] =$ min[(π x 33,4 + 100); (2 x 1000 + 100)] = 205 mm

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

$\Sigma l_{eff,nc} =$ 2 x 167 = 333 mm

$\Sigma l_{eff,cp} =$ 2 x 205 = 410 mm

The effective lengths for the group of bolts is:

Mode 1 — EN 1993-1-8 Table 6.4

$\sum l_{eff,1} = \min[\sum l_{eff,nc}; \sum l_{eff,cp}] =$ min[333; 410] = 333 mm

Mode 2: — Table 6.4

$\sum l_{eff,2} = \sum l_{eff,nc} =$ 333 mm

Mode 1 resistance

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)}$$ Table 6.2

where:

$m =$ 33,4 mm — SHEET 4

$n =$ 41,8 mm — SHEET 4

$e_w =$ 11,0 mm — SHEET 5

$$M_{pl,1,Rd} = \frac{0.25\sum l_{eff,1} t_{fc}^2 f_{y,c}}{\gamma_{M0}} =$$ (0.25 x 333 x 20,5² x 265)/1,00 = 9,27 x 10³ x 10³ Nmm

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)} =$$ [(8 x 41,8 − 2 x 11,0) x 9,27 x 10³ x 10³)/{10³ x [2 x 33,4 x 41,8 − 11,0 x (33,4 + 41,8)]} = 1.472 kN

Mode 2 resistance

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n}$$ EN 1993-1-8 Table 6.2

where:

$F_{t,Rd} =$ 203 kN — SHEET 5

$\sum F_{t,Rd} =$ 4 x 203 = 813 kN

$$M_{pl,2,Rd} = \frac{0.25\sum l_{eff,2} t_{fc}^2 f_{y,c}}{\gamma_{M0}} =$$ (0.25 x 333 x 20,5² x 265)/1,00 = 9,27 x 10³ x 10³ Nmm

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n} =$$ (2 x 9,27 x 10³ + 41,8 x 813)/(33,4 + 41,8) = 698 kN

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \sum F_{t,Rd} =$ 4 x 203 = 813 kN — EN 1993-1-8 Table 6.2

Resistance of column flange in bending

$F_{t,fc,Rd} = \min[F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}] =$ min[1.472; 698; 813] = 698 kN — EN 1993-1-8 6.2.4.1(6)

Column web in transverse tension — STEP 1B

The design resistance of an unstiffed column web in transverse tension is: $F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,c}}{\gamma_{M0}}$ — EN 1993-1-8 6.2.6.3(3)

where $b_{eff,t,wc}$ is the equivalente 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: — SHEET 10

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$b_{eff,t,wc} = \sum l_{eff,2} =$ 333 mm

with $t_{wc} =$ 12,8 mm; $f_{y,c} =$ 265 N/mm²

The equation to use to calculate ω depends on β. As before, β = 0,0 and therefore ω = 1,00: **SHEET 6**

$$F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,c}}{\gamma_{M0}} =$$ (1,00 x 333 x 12,8 x 265/1,00)/10³ = 1.130 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 kN
Column web in tension:	$F_{t,wc,Rd} =$	1.130 kN
Therefore, the resistance of bolts 1 and 2 combined is:	$F_{t,1\text{-}2,Rd} =$	698 kN

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

$F_{t2,c,Rd} = F_{t,1\text{-}2,Rd} - F_{t1,Rd} =$ (698 − 377) = 321 kN **SHEET 8**

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 kN	**SHEET 8**
Column web in tension:	$F_{t,wc,Rd} =$	790 kN	**SHEETS 6/8**
Beam web in tesnion:	$F_{t,wb,Rd} =$	673 kN	**SHEET 10**
End plate in bending:	$F_{t,ep,Rd} =$	407 kN	**SHEET 10**
Column side, as part of a group (see above):	$F_{t2,c,Rd} =$	321 kN	
Therefore, the resistance of bolt row 2 is:	$F_{t,2,Rd} =$	321 kN	

BOLT ROW 3
First, consider row 3 alone.

Column flange in bending **STEP 1**

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

Column web in transverse tension **STEP 1B**

The column web resistance to transverse tension is as calculated for bolt rows 1 and 2.

Therefore: $F_{t,wc,Rd} =$ 790 kN **SHEET 6**

End plate in bending **STEP 1**

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 l_{eff}:

$e = e_p =$ 75 mm $m =$ 38,6 mm **SHEETS 2/8**

$l_{eff,cp} = 2\pi m =$ 2 x π x 38,6 = 242 mm

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$l_{eff,nc}$ = 4m + 1.25e = 4 x 38,6 + 1.25 x 75 = 248 mm

$l_{eff,1}$ = min[$l_{eff,cp}$; $l_{eff,nc}$] = min[242; 248] = 242 mm

$l_{eff,2}$ = $l_{eff,nc}$ = 248 mm

Mode 1 resistance

For Mode 1, using Method 2:

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

EN 1993-1-8 Table 6.2

where:

n = 48,2 mm and m = 38,6 (as for row 2) **SHEETS 8/9**

n_w = 11,0 mm **SHEET 5**

$$M_{pl,1,Rd} = \frac{0.25\sum l_{eff,1} t_p^2 f_{y,p}}{\gamma_{M0}} =$$ (0.25 x 242 x 25² x 265)/1,00 = 10,0 x 10³ x 10³ Nmm = 10,0 kNm

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)} =$$ [(8 x 48,2 − 2 x 11,0) x 10,0 x 10³]/{[2 x 38,6 x 48,2 − 11,0 x (38,6 + 48,2)]} = 1.320 kN

Mode 2 resistance

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n}$$

$$M_{pl,2,Rd} = \frac{0.25\sum l_{eff,2} t_p^2 f_{y,p}}{\gamma_{M0}}$$

t_p = 25 mm $F_{t,Rd}$ = 203 kN $\Sigma F_{t,Rd}$ = 2 x 203 = 407 kN

$$M_{pl,2,Rd} = \frac{0.25\sum l_{eff,2} t_p^2 f_{y,p}}{\gamma_{M0}} =$$ (0.25 x 248 x 25² x 265)/1,00 = 10,3 x 10³ x 10³ Nmm = 10,3 kNm

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n} =$$ (2 x 10,3 x 10³ + 48,2 x 407)/(38,6 + 48,2) = 463 kN

Mode 3 resistance (bolt failure)

$F_{t,3,Rd}$ = Σ $F_{t,Rd}$ = 407 kN

Resistance of end plate in bending **EN 1993-1-8 6.2.4.1(6)**

$F_{t,ep,Rd}$ = min[$F_{T,1,Rd}$; $F_{T,2,Rd}$; $F_{T,3,Rd}$] = min[1.320; 463; 407] = 407 kN

Beam web in tension **STEP 1B EN 1993-1-8 6.2.6.8(1) Eq. (6.22)**

$$F_{T,wb,Rd} = \frac{b_{eff,t,wb}\, t_{wb}\, f_{y,b}}{\gamma_{M0}}$$

where:

$b_{eff,t,wb}$ = l_{eff}

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Conservatively, consider minimum l_{eff}. Therefore:

$b_{eff,t,wb} = l_{eff} = \min[l_{eff,1}; l_{eff,2}] =$ min[242; 248] = 242 mm. **SHEET 13**

$t_{wb} =$ 10,1 mm.

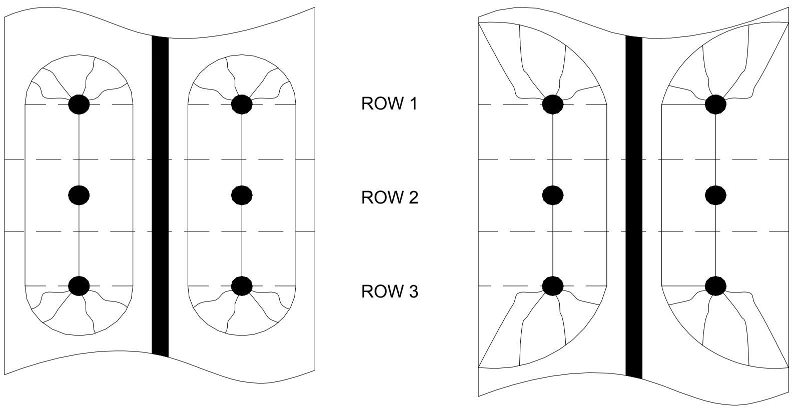
Therefore, $F_{T,wb,Rd} = \dfrac{b_{eff,t,wb}\, t_{wb}\, f_{y,b}}{\gamma_{M0}} =$ [(242 x 10,1 x 275)/10³]/1,0 = 673 kN

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 **STEP 1**

Circular and non-circular yield line patterns are:



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: $l_{eff,nc} =$ 167 mm **SHEET 10**

$l_{eff,cp} =$ 205 mm

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

$p = p_{2\text{-}3} =$ 90 mm

Thus:

$l_{eff,nc} = 2m + 0.625e + 0.5p =$ (2 x 33,4) + (0.625 x 79,4) + (0.5 x 90) = 162 mm

$l_{eff,cp} = \pi m + p =$ (π x 33,4) + 90 = 195 mm

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

$l_{eff,cp} = 2p$
$l_{eff,nc} = p$

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

$p = \dfrac{p_{1-2}}{2} + \dfrac{p_{2-3}}{2} =$ (100/2) + (90/2) = 95 mm $l_{eff,cp} = 2p =$ 2 x 95 = 190 mm $l_{eff,nc} = p =$ 95 mm

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

$\Sigma l_{eff,nc} =$ 167 + 95 + 162 = 423 mm

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$\sum l_{eff,cp}$ = 205 + 190 + 195 = 590 mm

Therefore, assume:

$\sum l_{eff,2} = \sum l_{eff,1} =$ 423 mm ($\sum l_{eff,2} = \sum l_{eff,nc} = \sum l_{eff,1} \leq \sum l_{eff,cp}$) — **EN 1993-1-8 Table 6.4**

Mode 1 resistance

For Mode 1, using Method 2:

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

where:

m = 33,4 mm — **SHEET 4**

n = 41,8 mm — **SHEET 4**

e_w = 11,0 mm — **SHEET 5**

$$M_{pl,1,Rd} = \frac{0.25\sum l_{eff,1} t_f^2 f_{y,c}}{\gamma_{M0}} =$$ (0,25 x 423 x 20,5² x 265)/1,0 = 11,8 x 10³ x 10³ Nmm = 11,8 kNm

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn + e_w(m+n)} =$$ [(8 x 41,8 − 2 x 11,0) x 11,8 x 10³]/[(2 x 33,4 x 41,8) − 11,0 x (33,4 + 41,8)] = 1.870 kN

Mode 2 resistance

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n}$$

where:

$F_{t,Rd}$ = 203 kN — **SHEET 5**

$\sum F_{t,Rd}$ = 6 x 203 = 1.220 kN

$$M_{pl,2,Rd} = \frac{0,25\sum l_{eff,2,Rd} t_f^2 f_{y,c}}{\gamma_{M0}}$$

Here, as $l_{eff,2} = l_{eff,1}$, $M_{pl,2,Rd} = M_{pl,1,Rd}$:

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n} =$$ (2 x 11,8 x 10³ + 41,8 x 1.220)/(33,4 + 41,8) = 991 kN

Mode 3 resistance (bolt failure)

$F_{T,3,Rd} = \sum F_{t,Rd} =$ 6 x 203 = 1.220 kN — **EN 1993-1-8 Table 6.2**

Resistance of column flange in bending

$F_{t,fc,Rd} = \min[F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}] =$ min[1.870; 991; 1.220] = 991 kN — **EN 1993-1-8 6.2.4.1(6)**

Column web in transverse tension — **STEP 1B**

The design resistance of an unstiffened column web in tranverse tension is: — **EN 1993-1-8 6.2.6.3(1) Eq. (6.15)**

$$F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,c}}{\gamma_{M0}}$$

where:

beff,t,wc is the effective length of the equivalent T-stub representing the column flange from 6.2.6.4. As the failure is Mode 2 (see sheet 15) take: — **EN 1993-1-8 6.2.6.3(3)**

$b_{eff,t,wc} = \sum l_{eff,2} =$ 423 mm — **SHEET 15**

The equation to use to calculate ω depends on β. As before, with β = 0,00 [-] therefore: ω = 1,00 [-] — **SHEET 6**

$$F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,c}}{\gamma_{M0}} =$$ [(1,0 x 423 x 12,8 x 265)/10³]/1,0 = 1.435 kN

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: $F_{t,fc,Rd}$ = 991 kN

column web in tension: $F_{t,wc,Rd}$ = 1.435 kN

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

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

$F_{t3,c,Rd} = (F_{t,1\text{-}3,Rd} - F_{t,1\text{-}2,Rd})$ = (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,

$\sum l_{eff,cp} = 2\pi m + 2p$ = 2 x π x 33,4 + 2 x 90 = 390 mm

$\sum l_{eff,nc} = 4m + 1.25e + p$ = 4 x 33,4 + 1,25 x 79,4 + 90 = 323 mm

Therefore,

$\sum l_{eff,2} = \sum l_{eff,1}$ = 323 mm

<u>*Mode 1 resistance*</u>

EN 1993-1-8 Table 6.2

$$M_{pl,1,Rd} = \frac{0{,}25 \sum l_{eff,1} t_f^2 f_{y,c}}{\gamma_{M0}} =$$ (0,25 x 323 x 20,5² x 265)/1,0 = 9,0 x 10³ x 10³ Nmm = 9,0 kNm

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn + e_w(m+n)} =$$ [(8 x 41,8 − 2 x 11,0) x 9,0 x 10³]/[(2 x 33,4 x 41,8) − 11,0 x (33,4 + 41,8)] = 1.428 kN

<u>*Mode 2 resistance*</u>

Here, as $l_{eff,2} = l_{eff,1}$, $M_{pl,2,Rd} = M_{pl,1,Rd}$:

EN 1993-1-8 Table 6.2

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n} =$$ (2 x 9,0 x 10³ + 41,8 x 4 x 203)/(33,4 + 41,8) = 691 kN

<u>*Mode 3 resistance (bolt failure)*</u>

EN 1993-1-8 Table 6.2

$F_{T,3,Rd} = \sum F_{t,Rd}$ = (6 x 203) = 1.220 kN

Column web in transverse tension

EN 1993-1-8 6.2.6.3(1) Eq. (6.15)

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

The equation to use to calculate ω depends on β. As before, with β = 0,00 [-] therefore: ω = 1,00 [-] — **SHEET 6**

$$F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,c}}{\gamma_{M0}} =$$ [(1,0 x 323 x 12,8 x 265)/10³]/1,0 = 1.096 kN

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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.

$l_{eff,cp} = \pi m + p$

Here: $p = p_{2\text{-}3} =$ 90 mm $n = 48{,}2$ $m = 38{,}6$ (as for row 2 alone)

$l_{eff,cp} = \pi m + p =$ (π x 38,6) + 90 = 211 mm **STEP 1 A**

Obtain a from Figure 6.11 - EN 1993-1-8 (or Annex G) using (see sheet 9): $\lambda_1 =$ 0,3395 $\lambda_2 =$ 0,30647

From Figure 6.11 $\alpha =$ 7,3 [-]

$l_{eff,nc} = 0.5p + \alpha m - (2m + 0.625e) =$ 0,5 x 90 + 7,3 x 38,6 − [2 x 38,6 + (0.625 x 75)] = 204 mm

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

$l_{eff,cp} = \pi m + p =$ (π x 38,6) + 90 = 211 mm **STEP 1 A**

$l_{eff,nc} = 2m + 0.625e + 0.5p =$ (2 x 38,6) + (0.625 x 75) + (0.5 x 90) = 169 mm

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

$\sum l_{eff,nc} =$ 204 + 169 = 373 mm

$\sum l_{eff,cp} =$ 211 + 211 = 422 mm

Hence,

$\sum l_{eff,2} = \sum l_{eff,1} =$ 373 mm

Mode 1 resistance (rows 2 + 3)

For Mode 1 failure, using Method 2: **EN 1993-1-8 Table 6.2**

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

where:

$n = 48{,}2$ mm **SHEET 9**

$e_w =$ 11,0 mm **SHEET 5**

$$M_{pl,1,Rd} = \frac{0.25\sum l_{eff,1} t_p^2 f_{y,p}}{\gamma_{M0}} =$$ 0.25 x 373 x 25² x 265/1,0 = 15,4 x 10³ x 10³ Nmm = 15,4 kNm

$m = 38{,}6$ mm **SHEET 8**

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn + e_w(m+n)} =$$ [(8 x 48,2 − 2 x 11,0) x 15,4 x 10³]/[2 x 38,6 x 48,2 − 11,0 x (38,6 + 48,2)] = 2.034 kN

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Mode 2 resistance (rows 2 + 3)

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n}$$

where:

$F_{t,Rd}$ = 203 kN

$\Sigma F_{t,Rd}$ = 4 x 203 = 813 kN

$$M_{pl,2,Rd} = \frac{0.25\sum l_{eff,2}\, t_f^2\, f_{y,p}}{\gamma_{M0}}$$

Here, as $l_{eff,2} = l_{eff,1}$:

$M_{pl,2,Rd} = M_{pl,1,Rd}$ = 15,4 kNm

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n} =$$ (2 x 15,4 x 10³ + 48,2 x 813)/(38,6 + 48,2) = 808 kN

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

$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 4 x 203 = 813 kN

Resistance of end plate in bending

$F_{t,ep,Rd} = \min[F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}]$ = min[2.034; 808; 813] = 808 kN — **EN 1993-1-8 6.2.4.1(6)**

Beam web in tension

This verification is not applicable as the beam flange (stiffner) 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: $F_{t,ep,Rd}$ = 808 kN
Therefore(*), on the beam side: $F_{t,2\text{-}3,Rd} = F_{t,ep,Rd}$ = 808 kN

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

$F_{t3,b,Rd} = F_{t2\text{-}3,Rd} - F_{t2,Rd}$ = (808 − 321) = 487 kN — **SHEET 12**

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

Column flange in bending: $F_{t,fc,Rd}$ = 691 kN — **SHEET 16**
Column web in tension: $F_{t,wc,Rd}$ = 1.096 kN — **SHEET 16**
Therefore, on the column side: $F_{t2\text{-}3,Rd}$ = 691 kN

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

$F_{t3,b,Rd} = F_{t2\text{-}3,Rd} - F_{t2,Rd}$ = (691 − 321) = 370 kN

Summary: resistance of bolt row 3

Resistance of bolt row 3 is the smallest value of:

Column flange in bending:	$F_{t,fc,Rd}$ = 398 kN	**SHEET 5/12**
Column web in tension:	$F_{t,wc,Rd}$ = 790 kN	**SHEET 6/12**
Beam web in tension:	$F_{t,wb,Rd}$ = 673 kN	**SHEET 14**
End plate in bending:	$F_{t,ep,Rd}$ = 407 kN	**SHEET 10**

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Column side, as part of a group with 2 & 1:	$F_{t3,c,Rd}$ = 293 kN	**SHEET 16**
Column side, as part of a group with 2:	$F_{t3,c,Rd}$ = 370 kN	**SHEET 18**
Beam side, as part of a group with 2:	$F_{t3,b,Rd}$ = 487 kN	**SHEET 18**

Therefore, the resistance of bolt row 3 is: $F_{t3,Rd}$ = 293 kN.

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]

	Column flange	**Column web**	**End plate**	**Beam web**	**MIN**	**Effect. resist.**	
ROW 1, alone	398	790	377	N/A	377	**377** (min)	**ROW 1**
ROW 2, alone	398	790	407	673	398		
ROW 2 + 1	698	1130	N/A	N/A	698		
ROW2 (effect.)					(698 − 398)	**300** (min)	**ROW 2**
ROW 3, alone	398	790	407	673	398		
ROW 3 + 2 + 1	991	1435	N/A	N/A	991		
ROW 3 (effect. a)					(991 − 698)	293	
ROW 3 + 2	691	1096	808	N/A	691		
ROW 3 (effect. b)					(691 − 321)	370	
						293 (min)	**ROW 3**

COMPRESSION ZONE T-STUBS

Column web in transverse compression **STEP 2**

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

$$F_{c,wc,Rd} = \frac{\omega k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}} \qquad \text{(crushing resistance)}$$

EN 1993-1-8 6.2.6,2(1) Eq. (6.9)

but:

$$F_{c,wc,Rd} \le \left(\frac{\omega k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}}\right)\left(\frac{\gamma_{M0}}{\gamma_{M1}}\right)\rho \qquad \text{(buckling resistance)}$$

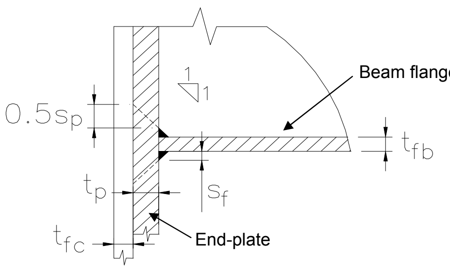
For a bolted end plate:

$$b_{eff,c,wc} = t_{fb} + 2a_p\sqrt{2} + 5\,(t_{fc} + s) + s_p = t_{fb} + 2s_f + 5\,(t_{fc} + s) + s_p$$

EN 1993-1-8 Eq. (6.11)

For rolled **I** and **H** column section $s = r_c$. Thus: $s = r_c$ = 12,7 mm **SHEET 2**

s_p is the length obtained by dispersion at 45° through the end plate:



$0.5s_p$ = 25 mm		**SHEET2**
t_p = 25 mm		"
h_p = 670 mm	(End-plate depth)	
s_f = 12 mm		"
t_{fc} = 20,5 mm		"
t_{fb} = 15,6 mm		"

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$s_p = 2t_p =$ 2 x 25 = 50 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:

$h_p \geq e_x + x + h_b + s_f + t_p =$ (50 + 40 + 533,1 + 12 + 25) = 660,1 mm

$h_p =$ 670 mm — **SHEET 2**

As 670 mm ≥ 660,1, the depth of the end plate is sufficient. — [Satisfactory]

Therefore:

$b_{eff,c,wc} = t_{fb} + 2a_p\sqrt{2} + 5(t_{fc} + s) + s_p = t_{fb} + 2s_f + 5(t_{fc} + s) + s_p$ = 15,6 + 2 x 12 + 5 x (20,5 + 12,7) + 50 = 255,6 mm.

ρ is the reduction factor for plate buckling

If $\overline{\lambda}_p \leq 0.72 \quad \Rightarrow \quad \rho = 1.0$ — **EN 1993-1-8 Eq. (6.13a)**

If $\overline{\lambda}_p > 0.72 \quad \Rightarrow \quad \rho = \dfrac{\overline{\lambda}_p - 0.2}{\left(\overline{\lambda}_p\right)^2}$ — **EN 1993-1-8 Eq. (6.13b)**

$\overline{\lambda}_p$ is the plate slenderness:

$$\overline{\lambda}_p = 0.932\sqrt{\frac{b_{eff,c,wc}\, d_{wc}\, f_{y,wc}}{E t_{wc}^2}} =$$ 0.932 x √ {(255,6 x 200,3 x 265)/(210 x 10³ + 12,8²)} = 0,59 [-] — **EN 1993-1-8 Eq. (6.13c)**

As 0,59 mm ≤ 0.72 Eq. (6.13a) gives:

$\rho =$ 1,00 [-]

ω is determined from Table 6.3 based on β (see sheet).
The equation to use to calculate ω depends on β. As before, with β = 0,00 [-] therefore: ω = 1,00 [-] — **SHEET 6**

k_{wc} is a reduction factor that takes account of compression in the column web.
Here, it is assumed that $k_{wc} =$ 1,00 [-] — **EN 1993-1-8 Note to 6.2.6.2(2)**

$$F_{c,wc,Rd} = \frac{\omega k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}} =$$ (1,00 x 1,00 x 255,6 x 12,8 x 265/1,0)/10³ = 867 kN

$$\left(\frac{\omega k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}}\right)\left(\frac{\gamma_{M0}}{\gamma_{M1}}\right)\rho =$$ 867 x (1,00/1,00) x 1,00 = 867 kN

Therefore:

$F_{c,wc,Rd} =$ 867 kN <u>(crushing resistance governs).</u>

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: — **EN 1993-1-8 6.2.6.7(1)**

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{h - t_{fb}}$$ — **Eq. (6.21)**

where:

$M_{c,Rd}$ is the design resistance of the beam

At this stage, assume that the design shear force in the beam does not reduce $M_{c,Rd}$.

Therefore:

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{h_b - t_{fb}} \approx 1.3\,\frac{b_b t_{fb}\left(h_b - t_{fb}\right) f_{y,b} / \gamma_{M0}}{h_b - t_{fb}} = 1.3\,\frac{f_{y,b}}{\gamma_{M0}} b_b t_{fb}$$ (approximated, conservative)

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$F_{c,fb,Rd} \square 1.3 \dfrac{f_{y,b}}{\gamma_{M0}} b_b t_{fb} =$ 1,3 x (275/1,0) x 209,3 x 15,6 = 1.167 x 10³ N = 1.167 kN

Summary: resistance of compression zone

Column web in transverse compression: $F_{c,wc,Rd}$ = 867 kN

Beam flange and web in compression: $F_{c,fb,Rd}$ = 1.167 kN

Compression resistance:

$F_{c,Rd} = \min[F_{c,wc,Rd}; F_{c,fb,Rd}] =$ min[867; 1.167] = 867 kN

Resistance of column web panel in shear — **STEP 3**

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

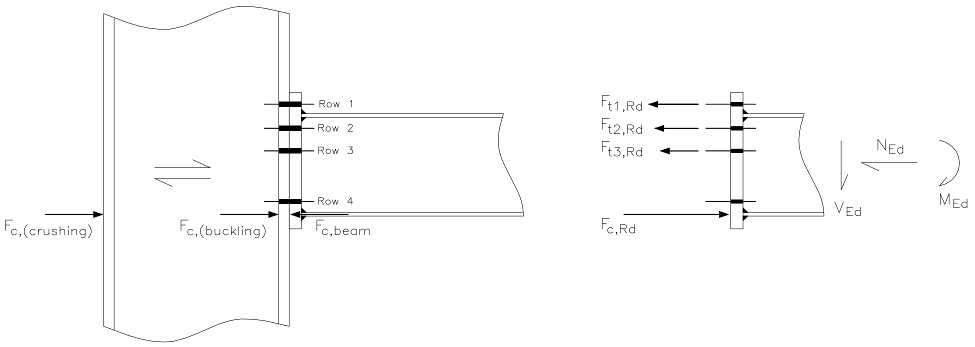
EN 1993-1-8
6.2.6.7(1)
Eq. (6.7)

$$V_{wp,Rd} = \frac{0,9 f_{y,wc} A_{vc}}{\gamma_{M0}\sqrt{3}}$$

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 — **STEP 4**



The resistances of each of the three bolt rows in the tension zone are: **SHEET 19**

$F_{t1,Rd}$ = 377 kN (for ROW 1)

$F_{t2,Rd}$ = 300 kN (for ROW 2)

$F_{t3,Rd}$ = 293 kN (for ROW 3)

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

(here: $1.9\ F_{t,Rd}$ = 1,9 x 203 = 386 kN ≥ 377 mm) (no reduction is necessary according to UK NA). — **SHEET 5**

or (no reduction is also necessary): — **Note**

$$t_p > \frac{d}{1.9}\sqrt{\frac{f_{ub}}{f_{y,p}}} \quad \text{or} \quad t_{fc} > \frac{d}{1.9}\sqrt{\frac{f_{ub}}{f_{y,c}}}$$

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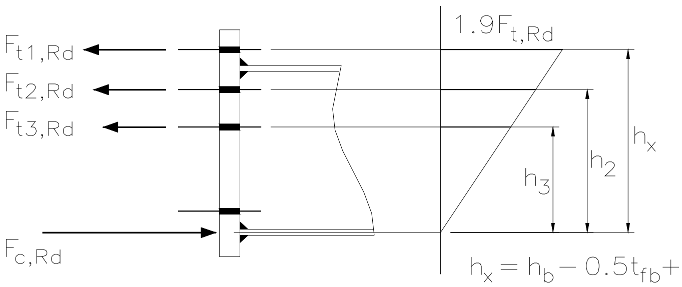
UK NA

In this case, the limiting thickness are respectively:

$t_p \leq$	21,9 mm	with t_p = 25 mm	(reduction is necessary).
$t_{fc} \leq$	21,9 mm	with t_{fc} = 20,5 mm	(no reduction is necessary).

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



Reduction factors:

h_2/h_x = 0,823 [-] (for ROW 2)

h_3/h_x = 0,664 [-] (for ROW 3)

$h_x = h_b - 0.5t_{fb} + R11$ = 565,3 mm — **SHEET 1**

$h_2 = h_x - (R11 + R2)$ = 465,3 mm — "

$h_3 = h_2 - R3$ = 375,3 mm — "

NOTE - UK NA statements: no reduction applied.

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

$F_{t1,Rd}$ = 377 kN (for ROW 1)

$F_{t2,Rd}$ = 300 kN (for ROW 2)

$F_{t3,Rd}$ = 293 kN (for ROW 3)

EQUILIBRIUM OF FORCES

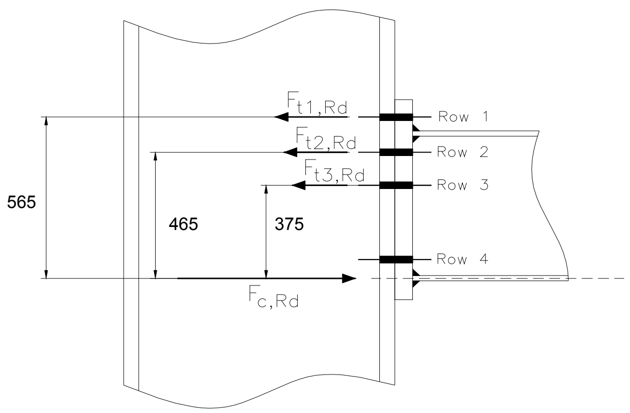
The sum of the tensile forces, together with any axial compression in the beam, cannot exceed the resistance of the compression zone. Similarly, the design shear cannot exceed the shear resistance of the column web panel; thisi is not relevant in thi case as the moments in the identical beams are equal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

$\sum F_{tr,Rd} = F_{c,Rd}$



Effective resistances to achieve equilibrium:

$F_{t1,Rd}$ = 377 kN +

$F_{t2,Rd}$ = 300 kN +

$F_{t3,Rd}$ = 190 kN =

$F_{c,Rd}$ = 867 kN

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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 bot 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:

EN 1993-1-8 6.2.7.2(1) Eq. (6.25)

$$M_{j,Rd} = \sum_r h_r F_{tr,Rd}$$

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

h_{r1} = 565 mm h_{r2} = 465 mm h_{r3} = 375 mm

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

$$M_{j,Rd} = \sum_r h_r F_{tr,Rd} = h_{r1} F_{t1,Rd} + h_{r2} F_{t2,Rd} + h_{r3} F_{t3,Rd} =$$

= (565 x 377 + 465 x 300 + 375 x 190)/10³ = 424 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 kN.

$F_{b,Rd}$ = 259 kN (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 kN

STEP 5

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 x 0.28) x 136 = 3.68 x 136 = 499 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: $t_{fb}/2$ = 15,6/2 = 7,8 mm.
Weld throat provided: a_f = 12/√2 = 8,5 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: $t_{fw}/2$ = 10,1/2 = 5,1 mm.
Weld throat provided: a_p = 8/√2 = 5,7 mm, which is adequate.