Manual calculation

Design example of a joint with extended end plate

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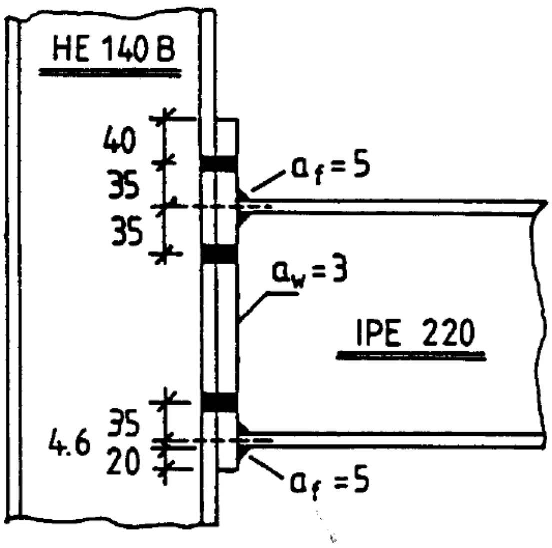
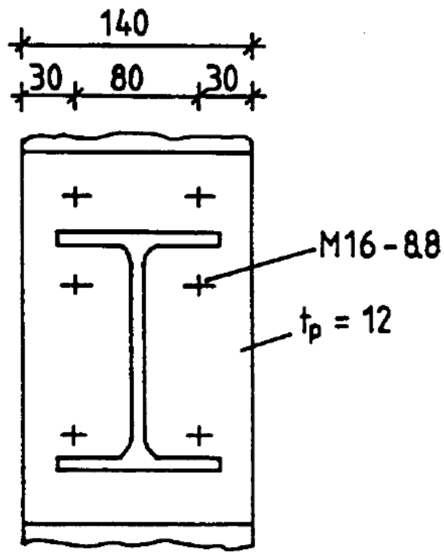
Design example of a joint with extended end plate

in a single-sided beam-to-column joint configuration





HEB140:

h = 140 mm
b = 140 mm
t_f = 12 mm
t_w = 7 mm
r = 12 mm
A = 4300 mm²
f_y = 235 N/mm²

IPE220:

h = 220 mm
b = 110 mm
t_f = 9,2 mm
t_w = 5,9 mm
r = 12 mm
A = 3340 mm²
W_{pl} = 285000 mm³
f_y = 235 N/mm²

End plate:

b_p = 140 mm
t_p = 12 mm
w = 80 mm
f_y = 235 N/mm²

Bolts:

6 M16 – 8.8
d_n = 16 mm
A_s = 157 mm²
f_u = 800 N/mm²

1. Partial Safety factors

In Scia Engineer the correct Partial Safety factors are given:

Partial safety factors	
Gamma M0	1.00
Gamma M1	1.00
Gamma M2	1.25
Gamma M3	1.25

And the check will be done in this example for the following internal forces:

1.Internal forces

LC1		
N	0.00	kN
Vz	10.00	kN
My	-10.00	kNm

Tension top

A negative moment results at tension for the top flange.

2. Design Moment resistance MRd

1.1. Design resistance of basic components

1.1.1. Column web panel in shear (EN 1993-1-8 art. 6.2.6.1)

$$V_{wp,Rd} = \frac{0{,}9 f_{y,w} A_v}{\sqrt{3}\gamma_{M0}}$$

Shear area of the column:

$$A_{vc} = A - 2 \cdot b \cdot t_f + (t_w + 2r) \cdot t_f$$

$$A_{vc} = 4300 - 2 \cdot 140 \cdot 12 + (7 + 2 \cdot 12) \cdot 12 = 1312 \text{ mm}^2$$

$$\boldsymbol{V_{wp,Rd}} = \frac{0{,}9 f_{y,w} A_v}{\sqrt{3}\gamma_{M0}} = \frac{0{,}9 \cdot 235 \cdot 1312}{\sqrt{3} \cdot 1} \cdot 10^{-3} = \mathbf{160{,}2}\ \boldsymbol{kN}$$

In Scia Engineer:

2.1. Design resistance of basic components

2.1.1. Column web panel in shear (EN 1993-1-8 art. 6.2.6.1)

Column web in shear (Vwp,Rd) data		
Column web in shear (Vwp,Rd)	160.21	kN
Beta	1.00	
Avc	1312.00	mm^2

1.1.2. Column web in compression (EN 1993-1-8 art. 6.2.6.2)

(6.9): $F_{c,wc,Rd} = \frac{\omega \cdot k_{wc} \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M0}}$ but $F_{c,wc,Rd} \leq \frac{\omega \cdot k_{wc} \cdot \rho \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M1}}$

(6.11): $b_{eff} = t_{fb} + 2\sqrt{2}a_p + 5\left(t_{fc} + s\right) + s_p$

$$s_p = 12 + \left(15 - \sqrt{2} \cdot 5\right) = 19{,}93$$

Above the bottom flange, there is sufficient room to allow 45° dispersion

Below the bottom flange, there is NOT sufficient room. Thus the dispersion is limited.

$$b_{eff} = 9{,}2 + 2\sqrt{2} \cdot 5 + 5(12 + 12) + 19{,}93 = 163{,}27mm$$

Table 5.4: $\beta = 1$ => Table 6.3: $\omega = \omega_1$

$$\omega = \omega_1 = \frac{1}{\sqrt{1+1{,}3(b_{eff,c,wc} \cdot \frac{t_{wc}}{A_{vc}})^2}} = \frac{1}{\sqrt{1+1{,}3(162{,}3 \cdot \frac{7}{1312})^2}} = 0{,}71$$

$k_{wc} = 1$

$$\boldsymbol{F_{c,wc,Rd}} = \frac{\omega \cdot k_{wc} \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M0}} = \frac{0{,}71 \cdot 1 \cdot 163{,}27 \cdot 7 \cdot 235 \cdot 10^{-3}}{1} = \mathbf{190,7\ kN}$$

In Scia Engineer:

2.1.2. Column web in compression (EN 1993-1-8 art. 6.2.6.2)

Column web in compression (Fc,wc,Rd) data		
Column web in compression (Fc,wc,Rd)	190.56	kN
beff,c,wc	163.27	mm
twc	7.00	mm
omega 1	0.71	
omega 2	0.45	
omega	0.71	
kwc	1.00	
lambda_rel	0.55	
reduction factor for plate buckling	1.00	
dwc	92.00	mm

1.1.3. Beam flange and web in compression (EN 1993-1-8 art. 6.2.6.7)

(6.21): $\boldsymbol{F_{c,fb,Rd}} = \frac{M_{c,Rd}}{(h-t_{fb})} = \frac{W_{pl} \cdot f_{yb}}{\gamma_{M0} \cdot (h-t_{fb})} = \frac{285 \cdot 10^3 \cdot 135 \cdot 10^{-3}}{1 \cdot (220-9{,}2)} = \boldsymbol{317{,}7\ kN}$

$$M_{c,Rd} = \frac{W_{pl} \cdot f_{yb}}{\gamma_{M0}} = \frac{285 \cdot 10^3 mm^3 \cdot 235 \cdot 10^{-3} kN/mm^2}{1} = 66975\ kNmm = 66{,}98\ kNm$$

$$h - t_{fb} = 220 - 9{,}2 = 210{,}80\ mm$$

$$\boldsymbol{F_{c,fb,Rd}} = \frac{M_{c,Rd}}{(h-t_{fb})} = \frac{66975\ kNmm}{210{,}80\ mm} = \boldsymbol{317{,}72\ kN}$$

In Scia Engineer:

2.1.3. Beam flange and web in compression (EN 1993-1-8 art. 6.2.6.7)

Beam flange in compression (Fc,fb,Rd) data		
Beam flange in compression (Fc,fb,Rd)	317.72	kN
section class	1	
Mc,Rd	66.98	kNm
hb-tfb	210.80	mm

1.1.4. Design tension resistance of bolt row

General data of the used bolts (M16 – 8.8)

$F_{t,Rd} = \frac{0{,}9 \cdot f_{ub} \cdot A_s}{\gamma_M} = \frac{0{,}9 \cdot 800\ MPa \cdot 157\ mm^2}{1{,}25}$ =90432 N = 90,43 kN

2.1.4. Design tension resistance of bolt-row

Ft,Rd data		
fub	800.00	MPa
As	157.00	mm^2
k2	0.90	-
Ft,Rd	90.43	kN
Lb	38.80	mm

Note: The bolt-rows are numbered starting from the bolt-row farthest from the centre of compression as specified in EN 1993-1-8 Article 6.2.7.2 (1).

1.1.4.1. Column flange

When looking at Table 6.4 of the EN 1993-1-8, we can make the following bolt-row locations:

Row 1 and row 3: End bolt-row

Row 2: Inner bolt-row

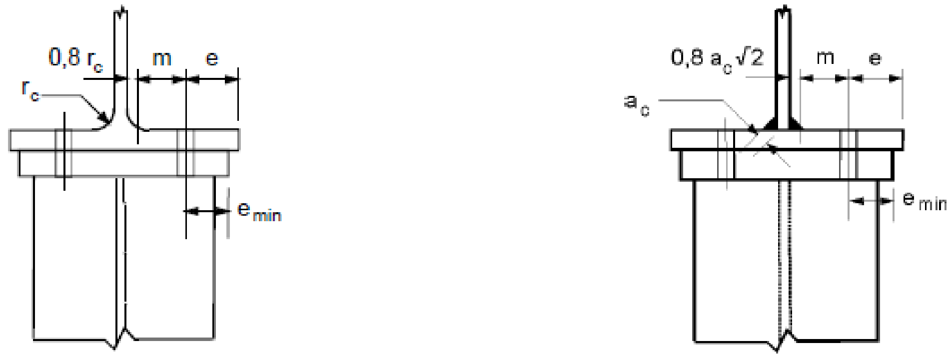
And the same bolt-row location will be shown in Scia Engineer:

2.1.4.1. Column flange

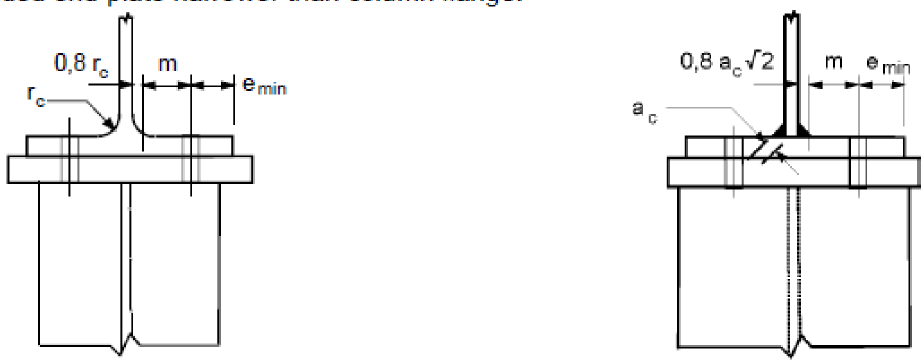
According to EN 1993-1-8 Article 6.2.6.3, 6.2.6.4
(effective lengths in mm, resistance in kN)

row	Bolt-row location
1	Other end bolt-row
2	Other inner bolt-row
3	Other end bolt-row

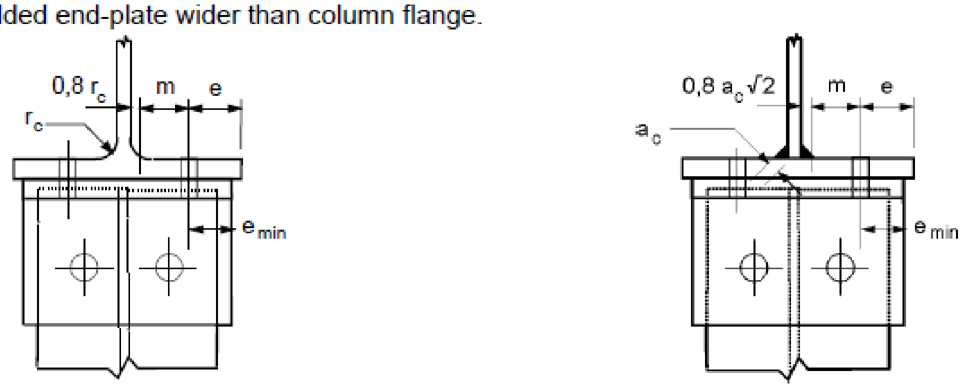
Definitions of some parameters:



a) Welded end-plate narrower than column flange.

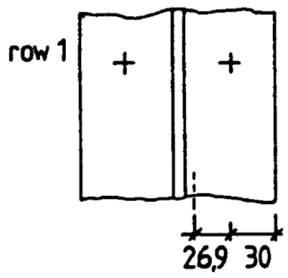


b) Welded end-plate wider than column flange.



c) Angle flange cleats.

Figure 6.8: Definitions of e, e_{min}, r_c and m



$$\boldsymbol{e = 30\ mm}$$

$$\boldsymbol{m} = \frac{b_c - t_{wc}}{2} - 0{,}8r - e$$

(see also EN1993-1-8 (Figure 6.8))

$$= (140 - 7)/2 - 0{,}8 \cdot 12 - 30$$

$$= \boldsymbol{26{,}9\ mm}$$

$$e_{min} = 30\ mm$$

$$n = e_{min}$$

$$\leq 1{,}25 \cdot m = 1{,}25 \cdot 26{,}9 = 33{,}6mm$$

(see also EN1993-1-8 (Table 6.2))

$$\boldsymbol{n = 30mm}$$

Row	p ($p_1 + p_2$)
1	0.0 + 35.0
2	35.0 + 70.0
3	70.0 + 0.0

In Scia Engineer:

row	p (p1+p2)	alpha	e	e1	m	n
1	0.0+35.0	-	30.00	1860.00	26.90	30.00
2	35.0+70.0	-	30.00	-	26.90	30.00
3	70.0+ 0.0	-	30.00	1930.00	26.90	30.00

l_{eff} will be calculated by following table for an unstiffened column flange:

Table 6.4: Effective lengths for an unstiffened column flange

Bolt-row Location	Bolt-row considered individually		Bolt-row considered as part of a group of bolt-rows	
	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$
Inner bolt-row	$2\pi m$	$4m + 1{,}25e$	$2p$	p
End bolt-row	The smaller of: $2\pi m$ $\pi m + 2e_1$	The smaller of: $4m + 1{,}25e$ $2m + 0{,}625e + e_1$	The smaller of: $\pi m + p$ $2e_1 + p$	The smaller of: $2m + 0{,}625e + 0{,}5p$ $e_1 + 0{,}5p$
Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$		$\sum\ell_{eff,1} = \sum\ell_{eff,nc}$ but $\sum\ell_{eff,1} \leq \sum\ell_{eff,cp}$	
Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$		$\sum\ell_{eff,2} = \sum\ell_{eff,nc}$	

Bolts rows considered individually

Row 1

l_{eff} circular patterns: the smaller of:

$2\pi m$ = 2*3.14*26,9 = **169,02**

$\pi m + e_1$ = 3.14*26,9 + 1860 = 1944,51

l_{eff} non-circular patterns: the smaller of:

$4m + 1{,}25e = 4*26{,}9 + 1{,}25*30 =$ **145,10**

$2m + 0{,}625e + e_1 = 2*26{,}9 + 0{,}625*30 + 1860 = 1932{,}55$

Row 2

l_{eff} circular patterns: $2\pi m = 2*3.14*26{,}9 =$ **169,02**

l_{eff} non-circular patterns: $4m + 1{,}25e = 4*26{,}9 + 1{,}25*30 =$ **145,10**

Row 3

l_{eff} circular patterns: the smaller of:

$2\pi m = 2*3.14*26{,}9 =$ **169,02**

$\pi m + e_1 = 3.14*26{,}9 + 1930 = 2014{,}51$

l_{eff} non-circular patterns: the smaller of:

$4m + 1{,}25e = 4*26{,}9 + 1{,}25*30 =$ **145,10**

$2m + 0{,}625e + e_1 = 2*26{,}9 + 0{,}625*30 + 1930 = 2002{,}55$

Row	**l_{eff} circular patterns**	**l_{eff} non-circular patterns**
1	169,02	145.10
2	169,02	145.10
3	169,02	145.10

In Scia Engineer:

row	leff,cp,i	leff,nc,i
1	169.02	145.10
2	169.02	145.10
3	169.02	145.10

Mode 1 : $l_{eff,1} = l_{eff,nc}$ but $l_{eff,1} \leq l_{eff,cp}$ => $l_{eff,1}$ = 145.10

Mode 2 : $l_{eff,2} = l_{eff,nc}$ => $l_{eff,2}$ = 145.10

Following Table 6.2 (EN 1993-1-8) Mode 1, Mode 2 and Mode 3 has to be calculated if the check for the prying forces is fulfilled.

L_b is the bolt elongation length, taken as equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut.

$L_b = t_f + t_p + t_{washer} + (h_{bolt\_head} + h_{nut})/2$

= 12 + 12 + 3,3 + (10 + 13)/2

= 38,8mm

Prying forces may develop if $L_b \leq L_b$*

A is the tensile stress area of the bolt A_s

$$L_b^* = \frac{8{,}8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8{,}8\,(26{,}9)^3 \cdot 157}{145{,}10 \cdot (12)^3} \cdot 1 = 107\ mm$$

(with n_b = number of bolt rows)

- ⇨ $L_b < L_b^*$
- ⇨ Prying forces may develop

This check is fulfilled, so mode 1; Mode 2 and Mode 3 will be calculated:

$$M_{pl,1,Rd} = M_{pl,2,Rd} = 0{,}25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0{,}25 \cdot 145{,}10 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 1227{,}5\ kNm$$

Mode 1:

$$F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m} = \frac{4 \cdot 1227{,}5}{26{,}9} = 182{,}5\ kN$$

Mode 2:

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m + n} = \frac{2 \cdot 1227{,}5 + 30 \cdot 2 \cdot 90{,}43}{26{,}9 + 30} = 138{,}5\ kN$$

Mode 3:

$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 90{,}43 = 180{,}9\ kN$

⇨ $\boldsymbol{F_{T,fc,Rd} = 138,5\ kN}$

And this is also shown in Scia Engineer:

For individual bolt-row :

row	leff,1	leff,2	Lb*	Prying forces	FT,1,Rd	FT,2,Rd	FT,3,Rd	Ft,fc,Rd,i
1	145.10	145.10	107.26	✓	182.53	138.51	180.86	138.51
2	145.10	145.10	107.26	✓	182.53	138.51	180.86	138.51
3	145.10	145.10	107.26	✓	182.53	138.51	180.86	138.51

COLUMN WEB IN TENSION:

The design resistance of an unstiffened column web subject to transverse tension should be determined from:

$F_{T,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$ (see also EN 1993-1-8 : 2005; formula (6.15))

With: $b_{eff,t,wc} = l_{eff}$ = 145,10

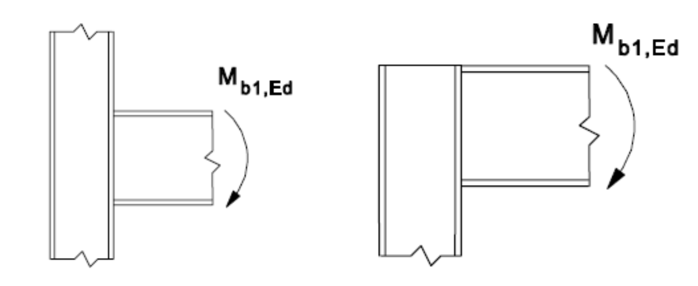
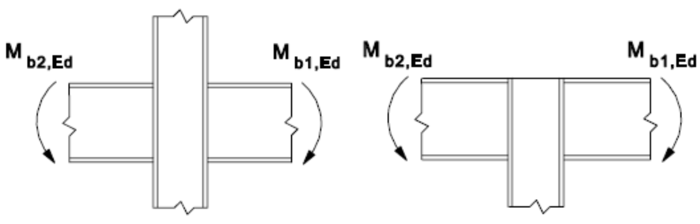
And ω, to allow for the possible effects of shear in the column web panel, should be determined from Table 6.3 (EN 1993-1-8):

Table 6.3: Reduction factor ω for interaction with shear

Transformation parameter β	Reduction factor ω
$0 \leq \beta \leq 0{,}5$	$\omega = 1$
$0{,}5 < \beta < 1$	$\omega = \omega_1 + 2(1-\beta)(1-\omega_1)$
$\beta = 1$	$\omega = \omega_1$
$1 < \beta < 2$	$\omega = \omega_1 + (\beta - 1)(\omega_2 - \omega_1)$
$\beta = 2$	$\omega = \omega_2$
$\omega_1 = \frac{1}{\sqrt{1+1{,}3(b_{eff,c,wc}\, t_{wc} / A_{vc})^2}}$	$\omega_2 = \frac{1}{\sqrt{1+5{,}2(b_{eff,c,wc}\, t_{wc} / A_{vc})^2}}$
A_{vc} is the shear area of the column, see 6.2.6.1;	
β is the transformation parameter, see 5.3(7).	

And:

Table 5.4: Approximate values for the transformation parameter β

Type of joint configuration	Action	Value of β
	$M_{b1,Ed}$	$\beta \approx 1$
	$M_{b1,Ed} = M_{b2,Ed}$	$\beta = 0$ *)
	$M_{b1,Ed} / M_{b2,Ed} > 0$	$\beta \approx 1$
	$M_{b1,Ed} / M_{b2,Ed} < 0$	$\beta \approx 2$
	$M_{b1,Ed} + M_{b2,Ed} = 0$	$\beta \approx 2$
*) In this case the value of β is the exact value rather than an approximation.		

In this example:

$\beta = 1$

$\omega = \omega_1$

$$\omega = \omega_1 = \frac{1}{\sqrt{1 + 1{,}3(b_{eff,c,wc} t_{wc}/A_{vc})^2}}$$

$$A_{vc} = A - 2 \cdot b_c \cdot t_{fc} + (\boldsymbol{t_{wc}} + \boldsymbol{2r_c}) \cdot t_{fc}$$

$$A_{vc} = 4300 - 2 \cdot 140 \cdot 12 + (\mathbf{7} + \mathbf{2} \cdot \mathbf{12}) \cdot 12 = 1312\ mm^2$$

$$\omega = \omega_1 = \frac{1}{\sqrt{1+1{,}3(b_{eff,c,wc} t_{wc}/A_{vc})^2}} = \frac{1}{\sqrt{1+1{,}3(145{,}10 \cdot 7/1312)^2}} = 0{,}75$$

⇨ $F_{T,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} = \frac{0{,}75 \cdot 145{,}10 \cdot 7 \cdot 235 \cdot 10^{-3}}{1}$

⇨ $\boldsymbol{F_{T,wc,Rd} = 179\ kN}$

In Scia Engineer:

row	beff,t,wc	omega 1	omega 2	omega	Ft,wc,Rd,i
1	145.10	0.75	0.49	0.75	178.95
2	145.10	0.75	0.49	0.75	178.95
3	145.10	0.75	0.49	0.75	178.95

__Bolts rows considered as part of a group of bolt-rows__

ROW 1

L_{eff} circular begin bolt-row = $\pi m + p_{end}$ = 3,14 * 26,9 + 70 = 154,51

L_{eff} non circular begin bolt-row = 2m + 0,625e + 0,5p = 2*26,9 + 0,625 * 30 + 0,5 * 70 = 107,55

ROW 2

L_{eff} circular inner bolt-row = 2p = 2 * (35.0 + 70.0) = 210

L_{eff} non circular inner bolt-row = p = 35.0 + 70.0 = 105

L_{eff} circular end bolt-row = $\pi m + p_{end}$ = 3,14 * 26,9 + 70 = 154,51

L_{eff} non circular end bolt-row = 2m + 0,625e + 0,5p = 2*26,9 + 0,625 * 30 + 0,5 * 70 = 107,55

ROW 3

L_{eff} circular end bolt-row = $\pi m + p_{end}$ = 3,14 * 26,9 + 140 = 224,51

L_{eff} non circular end bolt-row = 2m + 0,625e + 0,5p = 2*26,9 + 0,625 * 30 + 0,5 * 140 = 142,55

Summary:

Row	**l_{eff} circular inner bolt-row**	**l_{eff} non circular inner bolt-row**	**l_{eff} circular end bolt-row**	**l_{eff} non circular end bolt-row**	**l_{eff} circular begin bolt-row**	**l_{eff} non circular begin bolt-row**
1	-	-	-	-	154,51	107,55
2	210.00	105.00	154.51	107,55	224,51	142,55
3	-	-	224,51	142,55	-	-

In Scia Engineer:

row	leff,cp,g,inner	leff,nc,g,inner	leff,cp,g,end	leff,nc,g,end	leff,cp,g,start	leff,nc,g,start
1	-	-	-	-	154.51	107.55
2	210.00	105.00	154.51	107.55	-	-
3	-	-	224.51	142.55	-	-

Mode 1 : $\sum l_{eff,1} = \sum l_{eff,nc}$ but $\sum l_{eff,1} \leq \sum l_{eff,cp}$

Mode 2 : $\sum l_{eff,2} = \sum l_{eff,nc}$

Row 1-1 : not considered, same as the individual bolt row.

Row 1-2:

$$\sum l_{eff,cp} = 154.10 + 154.50 = 309.02$$

$$\sum l_{eff,nc} = 107.55 + 107.55 = 215.10$$

Mode 1 = Mode 2 : l_{eff} = 215.10

$$M_{pl,1,Rd} = M_{pl,2,Rd} = 0{,}25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0{,}25 \cdot 215{,}1 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 1819{,}8\ kNm$$

Row 1-3:

$$\sum l_{eff,cp} = 154.51 + 210.00 + 224.51 = 589.02$$

$$\sum l_{eff,nc} = 107.55 + 105.00 + 142.55 = 355.10$$

Mode 1 = Mode 2 : l_{eff} = 355.10

$$M_{pl,1,Rd} = M_{pl,2,Rd} = 0{,}25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0{,}25 \cdot 355{,}1 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 3004{,}1\ kNm$$

Prying forces may develop if $L_b \leq L_b^*$

L_b = 38,8mm

Row 1-2:

$$L_b^* = \frac{8{,}8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8{,}8\,(26{,}9)^3 \cdot 157}{215{,}10 \cdot (12)^3} \cdot 2 = 145\ mm$$

(with n_b = number of bolt rows)

- ⇨ $L_b < L_b^*$
- ⇨ Prying forces may develop

Row 1-3:

$$L_b^* = \frac{8{,}8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8{,}8\,(26{,}9)^3 \cdot 157}{355{,}10 \cdot (12)^3} \cdot 3 = 131\ mm$$

(with n_b = number of bolt rows)

⇨ $L_b < L_b^*$
⇨ Prying forces may develop

Row 1-2:

Mode 1: $F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m} = \frac{4 \cdot 1819{,}8}{26{,}9} = 270{,}6\ kN$

Mode 2: $F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m+n} = \frac{2 \cdot 1819{,}8 + 30 \cdot 4 \cdot 90{,}43}{26{,}9+30} = 254{,}7\ kN$

Mode 3: $F_{T,3,Rd} = \sum F_{t,Rd} = 4 \cdot 90{,}43 = 361{,}7\ kN$

⇨ $\boldsymbol{F_{T,Rd} = 254{,}7\ kN}$

Row 1-3:

Mode 1: $F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m} = \frac{4 \cdot 3004{,}1}{26{,}9} = 446{,}7\ kN$

Mode 2: $F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m+n} = \frac{2 \cdot 3004{,}1 + 30 \cdot 6 \cdot 90{,}43}{26{,}9+30} = 391{,}7\ kN$

Mode 3: $F_{T,3,Rd} = \sum F_{t,Rd} = 6 \cdot 90{,}43 = 542{,}6\ kN$

⇨ $\boldsymbol{F_{T,Rd} = 391{,}7\ kN}$

In Scia Engineer:

For bolt group :

group	leff,1	leff,2	Lb*	Prying forces	FT,1,Rd	FT,2,Rd	FT,3,Rd	Ft,fc,Rd,g
1- 1	145.10	145.10	107.26	✓	182.53	138.51	180.86	138.51
1- 2	215.10	215.10	144.71	✓	270.59	254.68	361.73	254.68
1- 3	355.10	355.10	131.48	✓	446.71	391.67	542.59	391.67

COLUMN WEB IN TENSION for row 1-2:

$$\omega = \omega_1 = \frac{1}{\sqrt{1+1{,}3(b_{eff,c,wc} t_{wc}/A_{vc})^2}} = \frac{1}{\sqrt{1+1{,}3(215{,}10 \cdot 7/1312)^2}} = 0{,}61$$

$$\Rightarrow \quad F_{T,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} = \frac{0{,}61 \cdot 215{,}10 \cdot 7 \cdot 235 \cdot 10^{-3}}{1}$$

$$\Rightarrow \quad \boldsymbol{F_{T,wc,Rd} = 215\ kN}$$

COLUMN WEB IN TENSION for row 1-3:

$$\omega = \omega_1 = \frac{1}{\sqrt{1+1{,}3(b_{eff,c,wc} t_{wc}/A_{vc})^2}} = \frac{1}{\sqrt{1+1{,}3(355{,}10 \cdot 7/1312)^2}} = 0{,}42$$

$$\Rightarrow \quad F_{T,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} = \frac{0{,}42 \cdot 355{,}10 \cdot 7 \cdot 235 \cdot 10^{-3}}{1}$$

$$\Rightarrow \quad \boldsymbol{F_{T,wc,Rd} = 245\ kN}$$

In Scia Engineer:

group	beff,t,wc	omega 1	omega 2	omega	Ft,wc,Rd,g
1- 1	145.10	0.75	0.49	0.75	178.95
1- 2	215.10	0.61	0.36	0.61	214.86
1- 3	355.10	0.42	0.23	0.42	245.40

1.1.4.1. End plate

When looking at Table 6.6 of the EN 1993-1-8, we can make the following bolt-row locations:

Row 1: Bolt-row outside tension flange of beam

Row 2: First bolt-row below tension flange of beam

Row 3: Other end bolt-row

And the same bolt-row location will be shown in Scia Engineer:

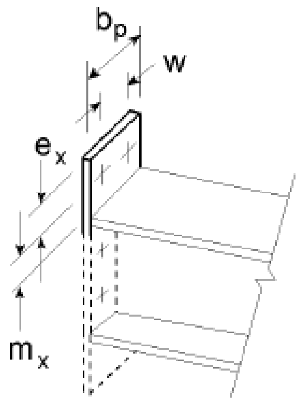
2.1.4.2. Endplate

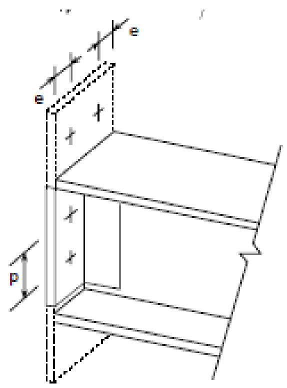
According to EN 1993-1-8 Article 6.2.6.5, 6.2.6.8

(effective lengths in mm, resistance in kN)

row	Bolt-row location
1	Bolt-row outside of beam
2	First bolt-row below tension flange of beam
3	Other end bolt-row

Definitions of some parameters:





Some pictures from Figure 6.10 of EN 1993-1-8.
For the end-plate extension, use e_x and m_x in place of e and m when determining the design resistance of the equivalent T-stub flange.

Row 1

e_x = $h_{endplate}$ - h_{row1} – $distance_{Endplate\_under – IPE220\_under}$

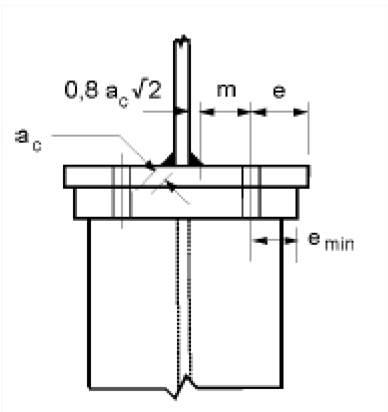
e_x = 305 – 250 – 15 = 40

fyd	Weld size
≤ 240 N/mm²	$a_f \geq 0.5\ t_{fb}$ $a_w \geq 0.5\ t_{wb}$
> 240 N/mm²	$a_f \geq 0.7\ t_{fb}$ $a_w \geq 0.7\ t_{wb}$

$a_f\ =\ 0{,}5 \cdot t_{fb}\ =\ 0{,}5 \cdot 9{,}2 = 4{,}6 \quad => \ a_f\ =\ 5\,mm$

$m_x\ =\ Top - e_x - 0{,}8 \cdot a \cdot \sqrt{2}$ (see also EN1993-1-8 (Figure 6.10))

$m_x\ = (305 - 220 - 15) - 40 - 0{,}8 \cdot 5 \cdot \sqrt{2}$ = 24,34



$n = e_{min}$ = 40mm

$\leq 1{,}25 \cdot m = 1{,}25 \cdot 24{,}34 = 30{,}42mm$

$n = 30{,}42mm$

w = 80 mm

<u>**Row 2 and Row 3**</u>

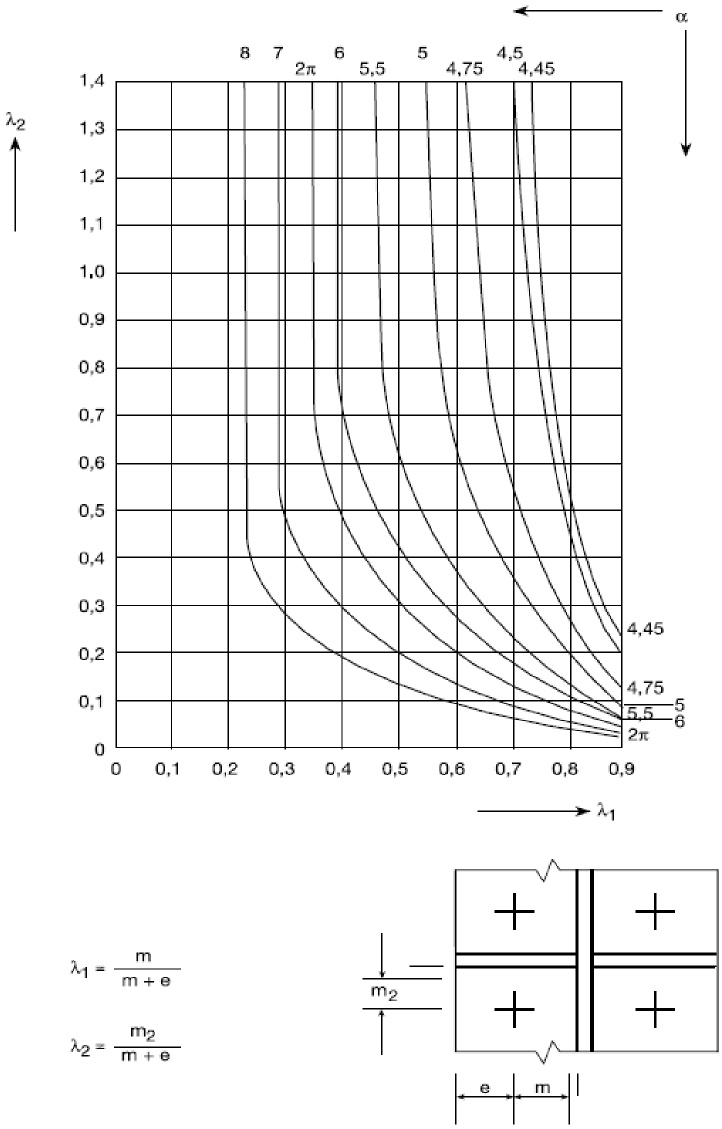


Figure 6.11: Values of α for stiffened column flanges and end-plates

e = 30 mm

fyd	Weld size
≤ 240 N/mm²	$a_f \geq 0.5\ t_{fb}$ $a_w \geq 0.5\ t_{wb}$
> 240 N/mm²	$a_f \geq 0.7\ t_{fb}$ $a_w \geq 0.7\ t_{wb}$

$a_w = 0{,}5 \cdot t_{wb} = 0{,}5 \cdot 5{,}9 = 3{,}0$

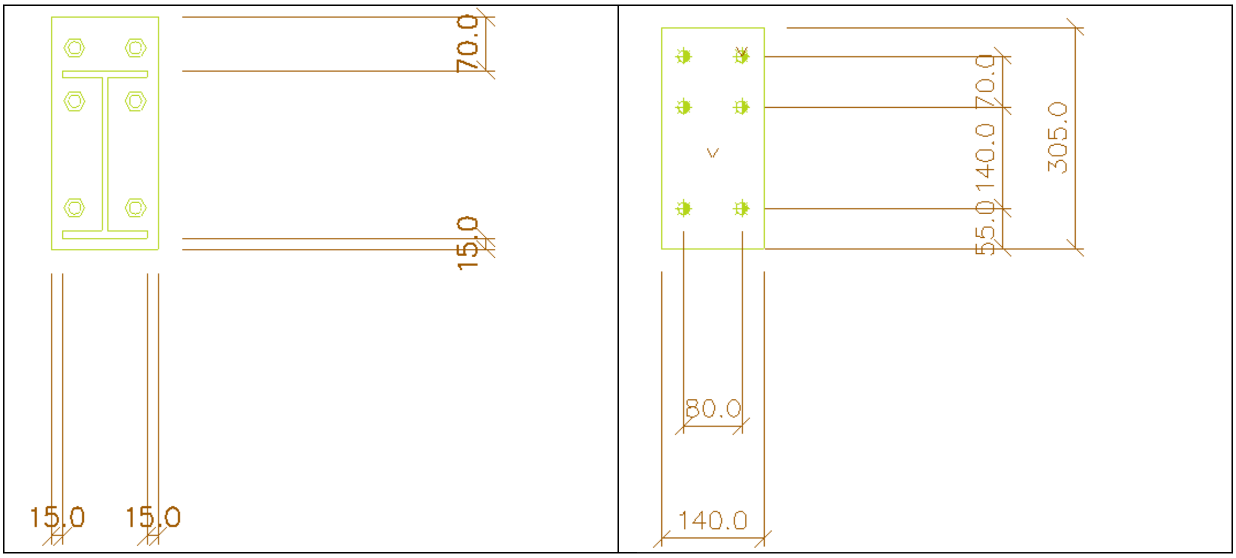
$$\boldsymbol{m} = \frac{b_{endplate} - t_{wc}}{2} - e - 0{,}8 \cdot a \cdot \sqrt{2}$$ (see also EN1993-1-8 (Figure 6.10))

$$\boldsymbol{m} = \frac{140 - 5{,}9}{2} - 30 - 0{,}8 \cdot 3 \cdot \sqrt{2}$$ = **33,66 mm**

$n = e_{min}$ = 30mm

$\leq 1{,}25 \cdot m = 1{,}25 \cdot 33{,}66 = 42{,}01 mm$

$\boldsymbol{n = 30\ mm}$



$m_{2,row2} = e_x - t_f - 0{,}8 \cdot a_f \cdot \sqrt{2}$

$m_{2,row2} = (35 + \frac{9{,}2}{2}) - 9{,}2 - 0{,}8 \cdot 5 \cdot \sqrt{2}$ = 24,74 mm

$m_{2,row3} = h_{row3} - t_f - 0{,}8 \cdot a_f \cdot \sqrt{2}$

$m_{2,row3} = 35 + \frac{9{,}2}{2}) - 9{,}2 - 0{,}8 \cdot 5 \cdot \sqrt{2}$ = 24,74mm

$$\lambda_1 = \frac{m}{m + e} = \frac{33{,}66}{33{,}66 + 30} = 0{,}53$$

$$\lambda_{2,row2} = \lambda_{2,row3} = \frac{m_{2,row2}}{m + e} = \frac{24{,}74}{33{,}66 + 30} = 0{,}39$$

⇨ Alpha = 5,9 (Figure 6.6; EN 1993-1-8)

Row	p ($p_1 + p_2$)	e	m	n	Lambda_1	Lamba_2	alpha
1	0.0 + 35.0	40 (= e_x)	24,34	30,42	-	-	-
2	35.0 + 70.0	30	33,66	30	0,53	0,39	5,99
3	70.0 + 0.0	30	33,66	30	0,53	0,39	5,99

In Scia Engineer:

row	p (p1+p2)	alpha	e	ex	m	mx	n
1	0.0+35.0	-	30.00	40.00	-	24.34	30.43
2	35.0+70.0	5.99	30.00	-	33.66	-	30.00
3	70.0+ 0.0	-	30.00	-	33.66	-	30.00

l_{eff} will be calculated by following table for an extended end-plate:

Table 6.6: Effective lengths for an end-plate

Bolt-row location	Bolt-row considered individually		Bolt-row considered as part of a group of bolt-rows	
	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$	Circular patterns $\ell_{eff,cp}$	Non-circular patterns $\ell_{eff,nc}$
Bolt-row outside tension flange of beam	Smallest of: $2\pi m_x$; $\pi m_x + w$; $\pi m_x + 2e$	Smallest of: $4m_x + 1{,}25e_x$; $e+2m_x+0{,}625e_x$; $0{,}5b_p$; $0{,}5w+2m_x+0{,}625e_x$	—	—
First bolt-row below tension flange of beam	$2\pi m$	αm	$\pi m + p$	$0{,}5p + \alpha m - (2m + 0{,}625e)$
Other inner bolt-row	$2\pi m$	$4m + 1{,}25\,e$	$2p$	p
Other end bolt-row	$2\pi m$	$4m + 1{,}25\,e$	$\pi m + p$	$2m+0{,}625e+0{,}5p$
Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \le \ell_{eff,cp}$		$\sum\ell_{eff,1} = \sum\ell_{eff,nc}$ but $\sum\ell_{eff,1} \le \sum\ell_{eff,cp}$	
Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$		$\sum\ell_{eff,2} = \sum\ell_{eff,nc}$	
α should be obtained from Figure 6.11.				

Bolts rows considered individually

Row 1:

l_{eff} circular patterns = smallest of:

- $2\pi m_x$ = 2*3,14*24,34 = **152,93**
- πm_x +w = 3,14*24,34 + 80 = 156,47
- πm_x +2e = 3,14*24,34 + 2*40 = 156,47

l_{eff} non circular patterns = smallest of:

- $4m_x$ +1,25 e_x = 4*24,34 +1,25 * 40 = 147,36
- e + $2m_x$ + 0,625e_x = 30 + 2*24,34 + 0,625*40 =103,68
- 0,5 b_p =0,5 * 140 = **70**
- 0,5 w + $2m_x$ + 0,625 e_x = 0,5 *80 + 2*24,34 + 0,625*40 = 113,68

Row 2:

l_{eff} circular patterns = $2\pi m$ = 2*3.14*33,66 = 211,49

l_{eff} non circular patterns: αm = 5,99 * 33,66 = 201,62

Row 3:

l_{eff} circular patterns = $2\pi m$ = 2*3.14*33,66 = 211,49

l_{eff} non circular patterns: 4m + 1,25e = 4 * 33,66 + 1.25 * 30 = 172,14mm

Row	**l_{eff} circular patterns**	**l_{eff} non-circular patterns**
1	152,93	70,00
2	211,49	201,62
3	211,49	172,14

In Scia Engineer:

row	leff,cp,i	leff,nc,i
1	152.95	70.00
2	211.47	201.57
3	211.47	172.12

ROW 1:

Mode 1 : $l_{eff,1} = l_{eff,nc}$ but $l_{eff,1} \leq l_{eff,\,cp}$ => $l_{eff,1}$ = 70

Mode 2 : $l_{eff,2} = l_{eff,nc}$ => $l_{eff,2}$ = 70

Prying forces may develop if $L_b \leq L_b$*

L_b = 38,8mm

$$L_b^* = \frac{8{,}8\, m^3\, A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8{,}8\,(24{,}34)^3 \cdot 157}{70 \cdot (12)^3} \cdot 1 = 165\, mm$$

(with n_b = number of bolt rows)

- ⇨ $L_b < L_b^*$
- ⇨ Prying forces may develop

This check is fulfilled, so mode 1; Mode 2 and Mode 3 will be calculated:

Following Table 6.2 (EN 1993-1-8) Mode 1, Mode 2 and Mode 3 has to be calculated:

$$M_{pl,1,Rd} = 0{,}25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0{,}25 \cdot 70 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 592\ kNm$$

Mode 1:

$$F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m} = \frac{4 \cdot 592}{24{,}34} = 97{,}32\ kN$$

Mode 2:

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m + n} = \frac{2 \cdot 592 + 30{,}43 \cdot 2 \cdot 90{,}43}{24{,}34 + 30{,}43} = 122\ kN$$

Mode 3:

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 90{,}43 = 180{,}9\ kN$$

⇨ $\boldsymbol{F_{T,fc,Rd} = 97{,}32\ kN}$

ROW 2:

Mode 1 : $l_{eff,1} = l_{eff,nc}$ but $l_{eff,1} \leq l_{eff,\ cp}$ => $l_{eff,1}$ = 201,57

Mode 2 : $l_{eff,2} = l_{eff,nc}$ => $l_{eff,2}$ = 201,57

Prying forces may develop if $L_b \leq L_b^*$

L_b = 38,8mm

$$L_b^* = \frac{8{,}8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8{,}8\,(33{,}66)^3 \cdot 157}{201{,}57 \cdot (12)^3} \cdot 1 = 151\ mm$$

(with n_b = number of bolt rows)

- ⇨ $L_b < L_b^*$
- ⇨ Prying forces may develop

This check is fulfilled, so mode 1; Mode 2 and Mode 3 will be calculated (Following Table 6.2 (EN 1993-1-8):

$$M_{pl,1,Rd} = 0{,}25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0{,}25 \cdot 201{,}57 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 1705{,}3\ kNm$$

Mode 1:

$$F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m} = \frac{4 \cdot 1705{,}3}{33{,}66} = 202{,}65\ kN$$

Mode 2:

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n} = \frac{2 \cdot 1705{,}3 + 30 \cdot 2 \cdot 90{,}43}{33{,}66 + 30} = 138{,}81\ kN$$

Mode 3:

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 90{,}43 = 180{,}9\ kN$$

⇨ $\boldsymbol{F_{T,fc,Rd} = 138,81}$

BEAM WEB IN TENSION:

In a bolted end plate connection, the design tension resistance of the beam web should be obtained from:

$F_{T,wb,Rd} = b_{eff,t,wb} t_{wb} f_{y,wb} / \gamma_{M0}$ (see also EN 1993-1-8 : 2005; formula (6.22))

$b_{eff,t,wb} = l_{eff,nc} = 201{,}57$

⇨ $F_{T,wb,Rd} = \frac{b_{eff,t,wb} t_{wb} f_{y,wb}}{\gamma_{M0}} = 201{,}57 \cdot 5{,}9 \cdot 235 \cdot 10^{-3}/1$

⇨ $\boldsymbol{F_{T,wc,Rd} = 279,48\ kN}$

ROW 3:

Mode 1 : $l_{eff,1} = l_{eff,nc}$ but $l_{eff,1} \leq l_{eff,\,cp}$ => $l_{eff,1}$ = 172,14

Mode 2 : $l_{eff,2} = l_{eff,nc}$ => $l_{eff,2}$ = 172,14

Prying forces may develop if $L_b \leq L_b*$

L_b = 38,8mm

$$L_b^* = \frac{8{,}8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8{,}8\, (33{,}66)^3 \cdot 157}{172{,}14 \cdot (12)^3} \cdot 1 = 177{,}14\, mm$$

(with n_b = number of bolt rows)

- ⇨ $L_b < L_b^*$
- ⇨ Prying forces may develop

This check is fulfilled, so mode 1; Mode 2 and Mode 3 will be calculated (Following Table 6.2 (EN 1993-1-8):

$$M_{pl,1,Rd} = 0{,}25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0{,}25 \cdot 172{,}14 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 1456{,}30\, kNm$$

Mode 1:

$$F_{T,1,Rd} = \frac{4 M_{pl,1,Rd}}{m} = \frac{4 \cdot 1456{,}3}{33{,}66} = 173{,}06\, kN$$

Mode 2:

$$F_{T,2,Rd} = \frac{2 M_{pl,2,Rd} + n \sum F_{t,Rd}}{m + n} = \frac{2 \cdot 1456.3 + 30 \cdot 2 \cdot 90{,}43}{33{,}66 + 30} = 130{,}98\, kN$$

Mode 3:

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 90{,}43 = 180{,}9\, kN$$

- ⇨ $\boldsymbol{F_{T,fc,Rd} = 130,98}$

BEAM WEB IN TENSION:

In a bolted end plate connection, the design tension resistance of the beam web should be obtained from:

$F_{T,wb,Rd} = b_{eff,t,wb} t_{wb} f_{y,wb} / \gamma_{M0}$ (see also EN 1993-1-8 : 2005; formula (6.22))

$b_{eff,t,wb} = l_{eff,nc}$ = 201,57

⇨ $F_{T,wb,Rd} = \frac{b_{eff,t,wb} t_{wb} f_{y,wb}}{\gamma_{M0}} = 172{,}14 \cdot 5{,}9 \cdot 235 \cdot 10^{-3}/1$

⇨ $\boldsymbol{F_{T,wc,Rd} = 238,67\ kN}$

In Scia Engineer:

For individual bolt-row :

row	leff,1	leff,2	Lb*	Prying forces	FT,1,Rd	FT,2,Rd	FT,3,Rd	Ft,ep,Rd,i
1	70.00	70.00	164.77	✓	97.31	122.10	180.86	97.31
2	201.57	201.57	151.22	✓	202.67	138.82	180.86	138.82
3	172.12	172.12	177.08	✓	173.07	130.99	180.86	130.99

row	beff,t,wb	Ft,wb,Rd,i
1	-	-
2	201.57	279.47
3	172.12	238.65

<u>Bolts rows considered as part of a group of bolt-rows</u>

l_{eff} will be calculated by following table for an extended end-plate:

ROW 1

Same as individual bolt row

ROW 2

L_{eff} circular begin bolt-row = πm + p = 3,14 * 33,66 + 140 = 245,73

L_{eff} non circular begin bolt-row = 0,5p + αm – (2m + 0,625e) = 0,5*140 + 5,99*33,66 – (2*33,66 + 0,625*30) = 185,55

ROW 3

L_{eff} circular end bolt-row = πm + p = 3,14 * 33,66 + 140 = 245,73

L_{eff} non circular end bolt-row = 2m + 0,625e + 0,5p = 2*33,66 + 0,625*30 + 0,5*140 = 156,07

Summary of values:

Row	**l_{eff} circular inner bolt-row**	**l_{eff} non circular inner bolt-row**	**l_{eff} circular end bolt-row**	**l_{eff} non circular end bolt-row**	**l_{eff} circular begin bolt-row**	**l_{eff} non circular begin bolt-row**
1	-	-	-	-	-	-
2	-	-	-	-	245,73	185,51
3	-	-	245,73	175,51	-	-

In Scia Engineer:

row	leff,cp,g,inner	leff,nc,g,inner	leff,cp,g,end	leff,nc,g,end	leff,cp,g,start	leff,nc,g,start
1	152.95	70.00	-	-	-	-
2	-	-	-	-	245.73	185.51
3	-	-	245.73	156.06	-	-

Mode 1 : $\sum l_{eff,1} = \sum l_{eff,nc}$ but $\sum l_{eff,1} \leq \sum l_{eff,cp}$

Mode 2 : $\sum l_{eff,2} = \sum l_{eff,nc}$

Row 2-3:

$\sum l_{eff,cp} = 245{,}73 + 245{,}73 = 491{,}46$

$\sum l_{eff,nc} = 185{,}51 + 185{,}51 = 341{,}57$

Mode 1 = Mode 2 : l_{eff} = 341,57

In Scia Engineer:

group	leff,cp,g	leff,nc,g
1- 1	152.95	70.00
2- 2	211.47	201.57
2- 3	491.47	341.57

Prying forces may develop if $L_b \leq L_b^*$

L_b = 38,8mm

$$L_b^* = \frac{8{,}8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8{,}8\,(33{,}66)^3 \cdot 157}{341{,}57 \cdot (12)^3} \cdot 2 = 179\, mm$$

(with n_b = number of bolt rows)

- ⇨ $L_b < L_b^*$
- ⇨ Prying forces may develop

This check is fulfilled, so mode 1; Mode 2 and Mode 3 will be calculated (Following Table 6.2 (EN 1993-1-8):

$$M_{pl,1,Rd} = M_{pl,2,Rd} = 0{,}25 \sum l_{eff} t_f^2 f_y/\gamma_{M0} = \frac{0{,}25 \cdot 341{,}57 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 2889{,}7\, kNm$$

Mode 1:

$$F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m} = \frac{4 \cdot 2889{,}7}{33{,}66} = 343\ kN$$

Mode 2:

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n} = \frac{2 \cdot 2889{,}7 + 30 \cdot 4 \cdot 90{,}43}{33{,}66 + 30} = 261{,}2\ kN$$

Mode 3:

$$F_{T,3,Rd} = \sum F_{t,Rd} = 4 \cdot 90{,}43 = 361{,}7\ kN$$

⇨ $\boldsymbol{F_{T,Rd} = 261{,}2\ kN}$

In Scia Engineer:

For bolt group :

group	leff.1	leff.2	Lb*	Prying forces	FT.1.Rd	FT.2.Rd	FT.3.Rd	Ft.ep.Rd.g
1- 1	70.00	70.00	164.77	✓	97.31	122.10	180.86	97.31
2- 2	201.57	201.57	151.22	✓	202.67	138.82	180.86	138.82
2- 3	341.57	341.57	178.47	✓	343.44	261.27	361.73	261.27

BEAM WEB IN TENSION:

⇨ $F_{T,wb,Rd} = \frac{b_{eff,t,wb} t_{wb} f_{y,wb}}{\gamma_{M0}} = 341{,}57 \cdot 5{,}9 \cdot 235 \cdot 10^{-3}/1$

⇨ $\boldsymbol{F_{T,wc,Rd} = 473{,}6\ kN}$

In Scia Engineer:

group	beff.t.wb	Ft.wb.Rd.g
1- 1	-	-
2- 2	201.57	279.47
2- 3	341.57	473.58

1.2. Determination of $M_{j,Rd}$

The design moment resistance $M_{j,Rd}$ of a beam-to-column joint with a bolted end-plate connection may be determined from:

$M_{j,Rd} = \sum_r h_r F_{tr,Rd}$ (EN 1993-1-8; §6.2.7.2)

Ft,min for each boltrow:

Row 1: 97,31 kN (End plate failure)

Row 2: 117,55 kN (Column flange failure)

Row 3: 30,54 kN (Column flange failure)

In Scia Engineer:

2.2. Force distribution in bolt-rows
2.2.1. Potential tension resistance
According to EN 1993-1-8 Article 6.2.7.2 (6),(8)

row	Ft,fc,Rd,i	Ft,fc,Rd,g	Ft,wc,Rd,i	Ft,wc,Rd,g	Ft,ep,Rd,i	Ft,ep,Rd,g	Ft,wb,Rd,i	Ft,wb,Rd,g	Ft,r,Rd
1	138.51	138.51	178.95	178.95	97.31	97.31	-	-	97.31
2	138.51	157.37	178.95	117.55	138.82	138.82	279.47	279.47	117.55
3	138.51	176.82	178.95	30.54	130.99	143.72	238.65	356.04	30.54

Following §6.2.7.2 (6) and (8)

The lowest value for the column web in tension, the column flange in bending, the end-plate in bending and the beam web in tension has to be checked. All these values are higher than column web in shear, which also have to be checked following §6.2.7.2 (7).

The column web in shear has the lowest resistance : 160,2kN

This is also shown in Scia Engineer:

2.2.2. Assessment of the shear and compression zone
According to EN 1993-1-8 Article 6.2.7.2 (7)

Column web in shear (Vwp,Rd/Beta)	160.21	kN
Column web in compression (Fc,wc,Rd)	190.56	kN
Beam flange and web in compression (Fc,fb,Rd)	317.72	kN

Limiting resistance = 160.21 kN

For the first boltrow $F_{t,Rd,1}$ = 97,31kN.

The maximum value for bolt row 2 is: $F_{t,Rd,2}$ = 160,2 - $F_{t,Rd,2}$ = 62,9kN.

And row 3 will not take any resistance.

This principle is shown on the next page.

- ⇨ Row 1: 97,31 kN (End plate failure)
- ⇨ Row 2: 62,9 kN (Reduced by column web in shear)
- ⇨ Row 3: 0 kN (Reduced by column web in shear)

This is also shown in Scia Engineer:

row	Ft.r.Rd	Decrease	Ft.r.Rd
1	97.31	0.00	97.31
2	117.55	54.65	62.90
3	30.54	30.54	0.00

Following §6.2.7.2 (9) the value 1,9 $F_{t,Rd}$ has to be checked also:

1,9 $F_{t,Rd}$ = 1,9 * 90,43 kN = 171,82 kN

The formula $F_{tx,Rd} \leq 1{,}9F_{t,Rd}$ is fulfilled for all the rows.

So also no reduction in Scia Engineer for the triangular limit:

2.2.3. Triangular limit
According to EN 1993-1-8 Article 6.2.7.2 (9)
Limit: 1.9*Ft,Rd = 171.82 kN

row	Ft.r.Rd	> Limit	Decrease	Ft.r.Rd
1	97.31	no	-	97.31
2	62.90	no	-	62.90
3	0.00	no	-	0.00

So $M_{j,Rd}$ can be calculated with the following values:

$h_{row\ 1}$ = 250 – 9,2/2 = 245.4 mm

$h_{row\ 2}$ = 180 – 9,2/2 = 175.4 mm

$h_{row\ 3}$ = 40 – 9,2/2 = 35,4 mm

Those values are calculated as the distance from the bolt to the middle of the bottom flange.

Row	h [mm]	F_t [kN]
1	245,4	97,3
2	175,4	62,9
3	35,4	0

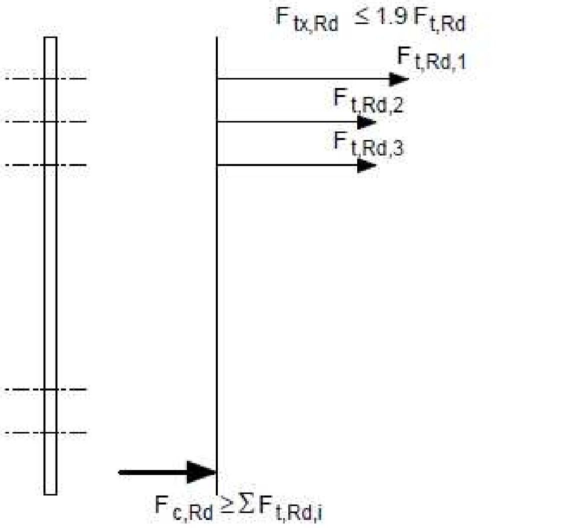
$M_{j,Rd}$ = 245,4 * 97,3 + 175,4 * 62,9 = 34910 kNmm =34,91 kNm

In Scia Engineer:

2.3. Determination of Mj,Rd
According to EN 1993-1-8 Article 6.2.7.2 (1)

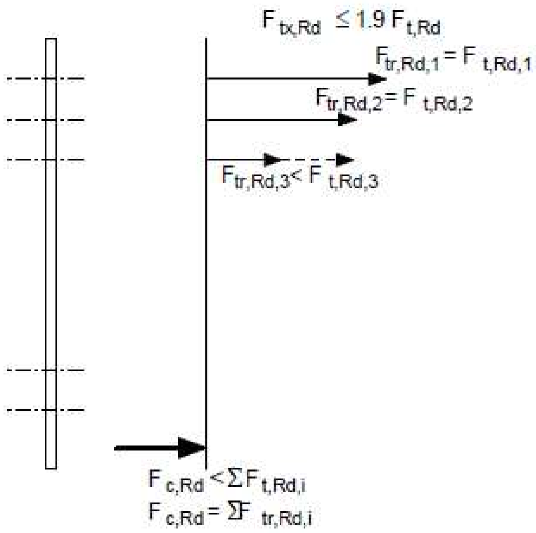
row	hr[mm]	Ft,r,Rd[kN]
1	245.40	97.31
2	175.40	62.90
3	35.40	0.00

Mj,Rd = 34.91 kNm



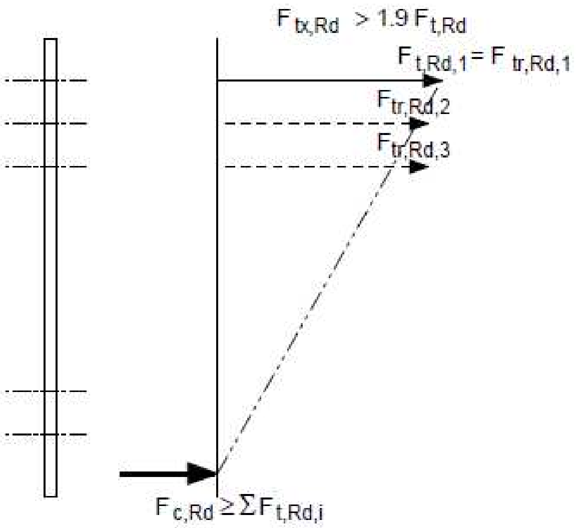
(a) Plastic distribution

- Because $F_{c,Rd}$ and $V_{wp,Rd} \geq F_{t,Rd,i}$ therefore the effective tension resistance ($F_{tr,Rd}$) is equal to the potential design resistance ($F_{t,Rd,i}$)



(b) Modified plastic distribution

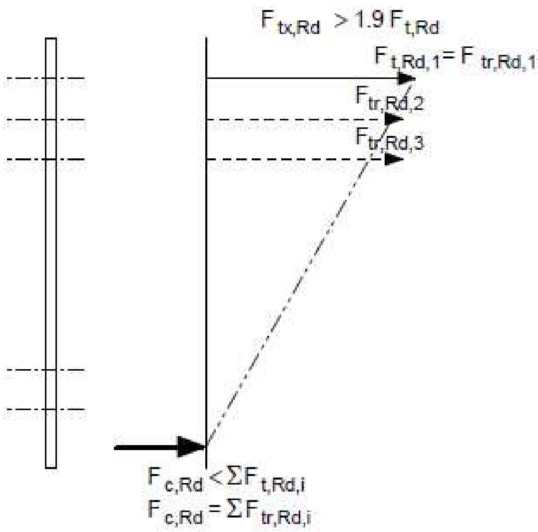
- Because $F_{c,Rd}$ and/or $V_{wp,Rd} < F_{t,Rd,i}$ therefore the effective tension resistances ($F_{tr,Rd}$) have to be reduced starting from the closest bolt to the compression centre:



(c) Triangular limit

- Because $F_{tx,Rd} > 1{,}9\ F_{t,Rd}$ the effective tension resistance has to be reduced:

$$F_{tr,Rd} = F_{tx,Rd} \frac{h_r}{h_x}$$



(d) Triangular limit

- Because $F_{tx,Rd} > 1{,}9\ F_{t,Rd}$ the effective tension resistance has to be reduced:

$$F_{tr,Rd} = F_{tx,Rd} \frac{h_r}{h_x}$$

- Because $F_{c,Rd}$ and/or $V_{wp,Rd} < F_{t,Rd,i}$ the effective tension resistances ($F_{tr,Rd}$) have to be reduced, starting from the closest bolt to the compression centre

2. Design shear resistance NRd

The design shear resistance N_{Rd} will be calculated as the minimum of the following 5 values:

Column web in tension:

This is calculated for the bolt group 1-3 for the column flange:

group	befft,wc	omega 1	omega 2	omega	Ft,wc,Rd,g
1- 1	145.10	0.75	0.49	0.75	178.95
1- 2	215.10	0.61	0.36	0.61	214.86
1- 3	355.10	0.42	0.23	0.42	245.40

⇨ **245, 40 kN**

Beam Web in tension:

This is calculated for the bolt group 2-3 for the endplate:

group	befft,wb	Ft,wb,Rd,g
1- 1	-	-
2- 2	201.57	279.47
2- 3	341.57	473.58

⇨ **473,58 kN**

Endplate in bending:

⇨ In this case the limiting value is
  - Bolt row 1
  - Group of bolt row 2+3

For bolt group :

group	leff,1	leff,2	Lb*	Prying forces	FT,1,Rd	FT,2,Rd	FT,3,Rd	Ft,ep,Rd,g
1- 1	70.00	70.00	164.77	✓	97.31	122.10	180.86	97.31
2- 2	201.57	201.57	151.22	✓	202.67	138.82	180.86	138.82
2- 3	341.57	341.57	178.47	✓	343.44	261.27	361.73	261.27

And this results in: 97,31 kN + 261,27 kN = **358,58 kN**

Column Flange in tension:

This is calculated for the bolt group 1-3 for the Column flange:

For bolt group :

group	leff,1	leff,2	Lb*	Prying forces	FT,1,Rd	FT,2,Rd	FT,3,Rd	Ft,fc,Rd,g
1- 1	145.10	145.10	107.26	✓	182.53	138.51	180.86	138.51
1- 2	215.10	215.10	144.71	✓	270.59	254.68	361.73	254.68
1- 3	355.10	355.10	131.48	✓	446.71	391.67	542.59	391.67

⇨ **391,68 kN**

Bolts in Tension:

6 bolts and $F_{T,Rd}$ for one bolt = 90,43 kN

⇨ 6 x 90,43 kN = **542,58 kN**

$N_{j,Rd}$

⇨ **Minimum of all previous values**

⇨ **245,40 kN**

In Scia Engineer:

2.5. Determination of Nj,Rd
According to EN 1993-1-8 Article 6.2.7.1 (3)

data		
Column Web in tension (Ft,wc,Rd)	245.40	kN
Beam Web in tension (Ft,wb,Rd)	473.58	kN
Endplate in bending (Ft,ep,Rd)	358.58	kN
Column Flange in tension (Ft,fc,Rd)	391.67	kN
Bolts in Tension (Ft,Rd)	542.59	kN

Nj,Rd = 245.40 kN

3. Design shear resistance V_{Rd}

Table 3.4 (En 1993-1-8):

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

For classes 4.6, 5.6 and 8.8: α_v = 0,6

F_{ub} = 800MPa

A is the tensile stress area of the bolt A_s

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A_s}{\gamma_{M2}} = \frac{0{,}6 \cdot 800 \cdot 157 \cdot 10^{-3}}{1{,}25}$$

⇨ **$F_{v,Rd}$ = 60,29 kN**

Following the NOTE of §6.2.2 (2) (EN 1993-1-8):

As a simplification, bolts required to resist in tension may be assumed to provide their full design resistance in tension when it can be shown that the design shear force does not exceed the sum of

a) The total design resistance of those bolts that are required to resist tension
b) (0,4 / 1,4) times the total design shear resistance of those bolts that are also required to resist tension

4 bolts (row 1 and 2) are required to resist tension, 2 bolts (of row 3) are not required to resist tension.

The value 0,4/1,4 will be simplified in Scia Engineer by the value 0,28:

⇨ V_{Rd} = (4 * 0,28 + 2) * 60,29kN = 188,10 kN

In Scia Engineer:

3. Design shear resistance VRd

VRd data		
VRd	188.10	kN
Fv,Rd	60.29	kN
e1,ep	40.00	mm
p1	70.00	mm
k1 plate	2.50	
k1 beam	2.50	
Alfa_b plate	0.74	
Alfa_b column	0.74	
Alfa_d plate	0.74	
Alfa_d column	0.74	
Fb,ep,Rd	102.40	kN
Fb,cf,Rd	102.40	kN
VRd beam	215.47	kN

4. Unity check

Assume following internal forces in this connection:

N_{Sd} = 0 kN

V_{Sd} = 10 kN

$M_{y,Sd}$ = 10 kNm

Check M: M/M_{Rd} = 10/34,9 = 0,29 < 1 => ok!

Check V: V/V_{Rd} = 10/189,48 = 0,05 < 1 => ok!

Check MN: $M/M_{Rd} + N/N_{Rd}$ = 10/34,9 + 0 = 0,29 < 1 => ok!

In Scia Engineer:

5. Unity checks

Unity checks	
MEd/MjRd	0.29
VEd/VRd	0.05
Unity check M/MRd + N/NRd	0.29

5. Stiffness calculation

5.1. Stiffness coefficients for basic joint components

Table 6.10: Joints with bolted end-plate connections and base plate connections

Beam-to-column joint with bolted end-plate connections	Number of bolt-rows in tension	Stiffness coefficients k_i to be taken into account
Single-sided	One	k_1; k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_1; k_2; k_{eq}
Double sided – Moments equal and opposite	One	k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_2; k_{eq}
Double sided – Moments unequal	One	k_1; k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_1; k_2; k_{eq}
Beam splice with bolted end-plates	Number of bolt-rows in tension	Stiffness coefficients k_i to be taken into account
Double sided - Moments equal and opposite	One	k_5[left]; k_5[right]; k_{10}
	Two or more	k_{eq}
Base plate connections	Number of bolt-rows in tension	Stiffness coefficients k_i to be taken into account
Base plate connections	One	k_{13}; k_{15}; k_{16}
	Two or more	k_{13}; k_{15} and k_{16} for each bolt row

For this connection (Single – sided), k_1, k_2, k_3, k_4 and k_{10} has to be calculated, using the formulas of Table 6.11 of EN 1993-1-8.

5.1.1. Column web in tension: k_3

$$k_3 = \frac{0{,}7\ b_{eff,t,wc}\ t_{wc}}{d_c}$$

- ⇨ $b_{eff,t,wc}$ is the effective width of the column web in tension from 6.2.6.3. For a joint with a single bolt-row in tension, $b_{eff,t,wc}$ should be taken as equal to the smallest of the effective lengths l_{eff} given for this bolt-row in Table 6.4 or Table 6.5.
- ⇨ $b_{eff,t,wc,row1} = 107{,}55$
- ⇨ $b_{eff,t,wc,row2} = 105$

$$\boldsymbol{k_{3,row1}} = \frac{\mathbf{0{,}7 \cdot 107{,}55 \cdot 7}}{\mathbf{92}} = \mathbf{5{,}73\ mm}$$

$$\boldsymbol{k_{3,row2}} = \frac{\mathbf{0{,}7 \cdot 105 \cdot 7}}{\mathbf{92}} = \mathbf{5{,}59\ mm}$$

In Scia Engineer:

4.1. Design rotational stiffness

row	k4[mm]	k3[mm]	k5[mm]	k10[mm]	keff[mm]
1	8.59	5.73	7.55	6.47	1.73
2	8.39	5.59	7.57	6.47	1.71

5.1.2. Column flange in bending: k_4

$$k_4 = \frac{0{,}9\ l_{eff} t_p^3}{m^3}$$

- ⇨ l_{eff} is the smallest of the effective lengths given for this bolt-row given in Table 6.4 or Table 6.5.
- ⇨ l_{eff} = 107,55
- ⇨ $b_{eff,t,wc,row1} = 107{,}55$
- ⇨ $b_{eff,t,wc,row2} = 105$

$$\boldsymbol{k_{4,row1}} = \frac{\mathbf{0{,}9 \cdot 107{,}55 \cdot 12^3}}{\mathbf{26{,}9^3}} = \mathbf{8{,}59\ mm}$$

$$\boldsymbol{k_{4,row2}} = \frac{\mathbf{0{,}9 \cdot 105 \cdot 12^3}}{\mathbf{26{,}9^3}} = \mathbf{8{,}39\ mm}$$

In Scia Engineer:

4.1. Design rotational stiffness

row	k4[mm]	k3[mm]	k5[mm]	k10[mm]	keff[mm]
1	8.59	5.73	7.55	6.47	1.73
2	8.39	5.59	7.57	6.47	1.71

5.1.3. End-plate in bending: k_5

$$k_5 = \frac{0{,}9\, l_{eff} t_p^3}{m^3}$$

- l_{eff} is the smallest of the effective lengths given for this bolt-row given in Table 6.6.
- $l_{eff,\ row1}$ = 70
- $l_{eff,\ row2}$ = 185,51

$$\boldsymbol{k_{5,row\,1}} = \frac{0{,}9 \cdot 70 \cdot 12^3}{(24{,}34)^3} = \mathbf{7{,}55\ mm}$$

$$\boldsymbol{k_{5,row\,2}} = \frac{0{,}9 \cdot 185{,}51 \cdot 12^3}{(33{,}66)^3} = \mathbf{7{,}57\ mm}$$

In Scia Engineer:

4.1. Design rotational stiffness

row	k4[mm]	k3[mm]	k5[mm]	k10[mm]	keff[mm]
1	8.59	5.73	7.55	6.47	1.73
2	8.39	5.59	7.57	6.47	1.71

5.1.4. Bolts in tension: k_{10}

$$k_{10} = 1{,}6 \frac{A_s}{L_b}$$

- A is the tensile stress area of the bolt A_s = 157mm²
- L_b is the bolt elongation length, taken as equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut.
- $L_b = t_f + t_p + t_{washer} + (h_{bolt\_head} + h_{nut})/2$
  = 12 + 12 + 3,3 + (10 + 13)/2
  = 38,8mm

$$\boldsymbol{k_{10}} = \mathbf{1{,}6 \cdot \frac{157}{38{,}8} = 6{,}47\ mm}$$

In Scia Engineer:

4.1. Design rotational stiffness

row	k4[mm]	k3[mm]	k5[mm]	k10[mm]	keff[mm]
1	8.59	5.73	7.55	6.47	1.73
2	8.39	5.59	7.57	6.47	1.71

5.2. Equivalent stiffness

The effective stiffness $k_{eff,r}$ for bolt-row r should be determined from

$k_{eff,r} = 1/\sum_i(\frac{1}{k_{i,r}})$ (see also formula (6.30) of EN 1993-1-8)

In the case of a beam-to-column joint with an end-plate connection, k_{eq} should be based upon (and replace) the stiffness coefficients k_i fork_3, k_4, k_5 and k_{10}.

- $k_{\text{eff,row1}} = \frac{1}{\frac{1}{5,73}+\frac{1}{8,59}+\frac{1}{7,55}+\frac{1}{6,47}} = 1,73$

- $k_{\text{eff,row2}} = \frac{1}{\frac{1}{5,59}+\frac{1}{8,39}+\frac{1}{7,57}+\frac{1}{6,47}} = 1,71$

The equivalent lever arm z_{eq} should be determined from:

$$z_{eq} = \frac{\sum_r k_{eff,r}h_r^2}{\sum_r k_{eff,r}h_r} = \frac{k_{eff,row1}h_{row1}^2 + k_{eff,row2}h_{row2}^2}{k_{eff,row1}h_{row1} + k_{eff,row2}h_{row2}}$$

$$= \frac{1,73 \cdot (245,4)^2 + 1,71 \cdot (175,4)^2}{1,73 \cdot 245,4 + 1,71 \cdot 175,4}$$

$$z_{eq} = \frac{156791}{724,48} = 216,42\ mm$$

The equivalent stiffness k_{eq} can now be determined from:

$k_{eq} = \frac{\sum_r(k_{eff,r}h_r)}{z_{eq}}$ (see also formula (6.29) from En 1993-1-8)

$$k_{eq} = \frac{1,73 \cdot 245,4 + 1,71 \cdot 175,4}{216,42} = 3,35\ mm$$

And those values are also given in Scia Engineer:

Sj data		
Sj	11.60	MNm/rad
Sj,ini	11.60	MNm/rad
z	216.42	mm
mu	1.00	
k1	2.30	mm
k2	8.70	mm
keq	3.35	mm

5.2.1. Column web panel in shear: k_1

$$k_1 = \frac{0{,}38\, A_{vc}}{\beta\, z}$$

z is the lever arm from Figure 6.15

Following option e) A more accurate value may be determined by taking the lever arm *z* as equal to *z*eq obtained using the method given in 6.3.3.1.

⇨ $z = z_{eq} = 216{,}8 = 216{,}8$mm

β is the transformation parameter from 5.3 (7)

⇨ β = 1

$$\boldsymbol{k_1 = \frac{0{,}38 \cdot 1312}{1 \cdot 216{,}8} = 2{,}30\ \text{mm}}$$

In Scia Engineer:

Sj data		
Sj	11.60	MNm/rad
Sj,ini	11.60	MNm/rad
z	216.42	mm
mu	1.00	
k1	2.30	mm
k2	8.70	mm
keq	3.35	mm

5.2.2. Column web in compression: k_2

$$k_2 = \frac{0{,}7\, b_{eff,c,wc}\, t_{wc}}{d_c}$$

⇨ $d = h_c - 2\,(t_f + r_c) = 140 - 2\,(12 + 12) = 92$ mm

⇨ $b_{eff} = t_{fb} + 2\sqrt{2}a_p + 5(t_{fc} + s) + s_p$

$s_p = 12 + (15 - \sqrt{2} \cdot 5) = 19{,}93$

Above the bottom flange, there is sufficient room to allow 45° dispersion

Below the bottom flange, there is NOT sufficient room. Thus the dispersion is limited.

- ⇨ $b_{eff} = 9{,}2 + 2\sqrt{2} \cdot 5 + 5(12 + 12) + 19{,}93 = 163{,}27mm$

$$\boldsymbol{k_2} = \frac{0{,}7 \cdot 163{,}3 \cdot 7}{92} = \mathbf{8{,}70\ mm}$$

In Scia Engineer:

Sj data		
Sj	11.60	MNm/rad
Sj,ini	11.60	MNm/rad
z	216.42	mm
mu	1.00	
k1	2.30	mm
k2	8.70	mm
keq	3.35	mm

5.3. Design rotational stiffness

$$S_j = \frac{E\,z^2}{\mu\ \sum_i \frac{1}{k_i}} = \frac{E\,z^2}{\mu \cdot \left(\frac{1}{k_1} + \frac{1}{k_2} + \frac{1}{k_{eq}}\right)}$$

- ⇨ z = 216,4 mm
- ⇨ µ is the stiffness ration $S_{j,\,ini}$ / S_j
  - o If $M_{j,Ed} \leq M_{j,Rd}$ => µ = 1
  - o If 2/3 $M_{j,Rd} < M_{j,Ed} \leq M_{j,Rd}$ => µ = $(1{,}5\ M_{j,Ed} / M_{j,\,Rd})^{\psi}$

$M_{j,Ed}$ = 10 kNm

$M_{j,Rd}$ = 34,9 kNm => 2/3 $M_{j,Rd}$ = 23,3 kNm

- ⇨ µ = 1
- ⇨ $S_j = \frac{E\,z^2}{\mu\ \sum_i \frac{1}{k_i}}$

$$S_j = \frac{210000 \cdot (216{,}42)^2}{1 \cdot \left(\frac{1}{2{,}30} + \frac{1}{8{,}70} + \frac{1}{3{,}35}\right)} \cdot 10^{-6} = 11596 \text{ kNm/rad}$$

In Scia Engineer:

Sj data		
Sj	11.60	MNm/rad
Sj,ini	11.60	MNm/rad
z	216.42	mm
mu	1.00	
k1	2.30	mm
k2	8.70	mm
keq	3.35	mm

5.4. Stiffness classification

The connection has been input for a braced frame, so the limits are:

$$S_{j,rigid} = 8\frac{E \cdot I_b}{L_b} = 8\frac{(210000\frac{N}{mm^2}) \cdot (2{,}772 \cdot 10^7\ mm^4)}{2000\ mm} = 23{,}28 \text{ MNm/rad}$$

$$S_{j,pinned} = 0{,}5\frac{E \cdot I_b}{L_b} = 0{,}5\frac{(210000\frac{N}{mm^2}) \cdot (2{,}772 \cdot 10^7\ mm^4)}{2000\ mm} = 1{,}46 \text{ MNm/rad}$$

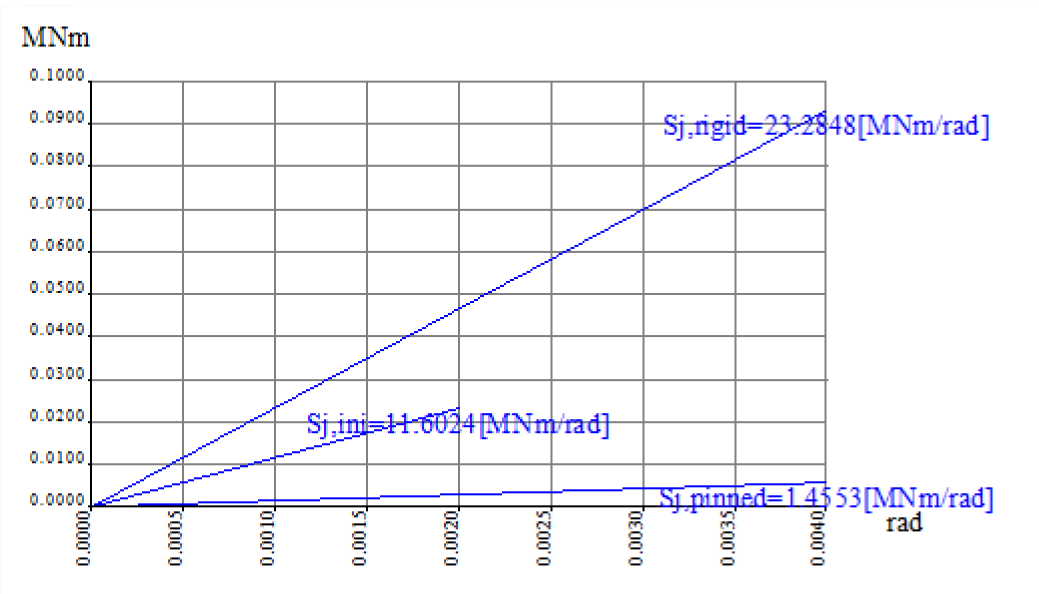
In Scia Engineer:

4.2. Stiffness classification

Stiffness data		
E	210000.00	N/mm^2
Ib	27720000.00	mm^4
Lb	2000.00	mm
frame type	braced	
S1	23.28	MNm/rad
S2	1.46	MNm/rad

System SEMI RIGID

And this is also given in Scia Engineer in a picture:



5.5. Check of stiffness requirement

The boundaries for the stiffness requirements are calculated using the following formulas:

Frame	Lower boundary Sj,low		Upper boundary Sj,upper
Braced	$\frac{8 \cdot Sj,app \cdot E \cdot I_b}{10 \cdot E \cdot I_b + Sj,app \cdot L_b}$	$Sj,app \le \frac{8 \cdot E \cdot I_b}{L_b}$	$\frac{10 \cdot Sj,app \cdot E \cdot I_b}{8 \cdot E \cdot I_b - Sj,app \cdot L_b}$
		$Sj,app > \frac{8 \cdot E \cdot I_b}{L_b}$	∞
Unbraced	$\frac{24 \cdot Sj,app \cdot E \cdot I_b}{30 \cdot E \cdot I_b + Sj,app \cdot L_b}$	$Sj,app \le \frac{24 \cdot E \cdot I_b}{L_b}$	$\frac{30 \cdot Sj,app \cdot E \cdot I_b}{24 \cdot E \cdot I_b - Sj,app \cdot L_b}$
		$Sj,app > \frac{24 \cdot E \cdot I_b}{L_b}$	∞

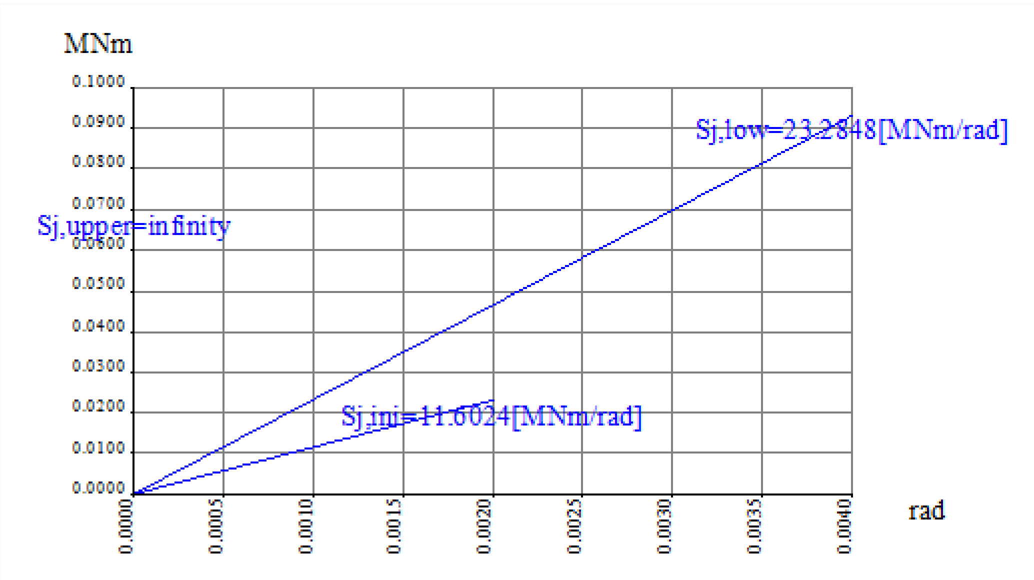
In a general calculation, Sj,app equals infinity and we have the following results:

4.3 Check of stiffness requirement

Stiffness data		
Fi y	infinity	MNm/rad
Stiffness modification coef.	2.00	
Sj,app	infinity	MNm/rad
Sj,lower boundary	23.28	MNm/rad
Sj,upper boundary	infinity	MNm/rad

Sj,ini is not inside the boundaries.
The actual joint stiffness does not conform with the joint stiffness of the analysis model.

And this is also shown in a graph:



When calculating with an Sj,app of 11,60 MNm/rad (equals Sj,ini), the lower and upper boundary can be calculated:

Lower boundary

$$= \frac{8 \cdot S_{j,app} \cdot E \cdot I_b}{10 \cdot E \cdot I_b + S_{j,app} \cdot L_b}$$

$$= \frac{8 \cdot \frac{11{,}60MNm}{rad} \cdot 210000MPa \cdot 2{,}77E - 05\ m^4}{10 \cdot 210000MPa \cdot 2{,}77E - 05\ m^4 + 11{,}60MNm/rad \cdot 2m}$$

= 6,64 MNm/rad

Upper boundary

First we have to check if Sj,app is bigger or smaller than

$$\frac{8 \cdot E \cdot I_b}{L_b} = \frac{8 \cdot 210000\ MPa \cdot 2{,}77E - 05\ m^4}{2m} = 23{,}3\ MPa$$

Thus

$$S_{j,app} = 11{,}60 \leq \frac{8 \cdot E \cdot I_b}{L_b}$$

And now the upper boundary can be calculated with the following formula:

$$= \frac{10 \cdot S_{j,app} \cdot E \cdot I_b}{8 \cdot E \cdot I_b - S_{j,app} \cdot L_b}$$

$$= \frac{10 \cdot \frac{11{,}60MNm}{rad} \cdot 210000MPa \cdot 2{,}77E - 05\ m^4}{(8 \cdot 210000MPa \cdot 2{,}77E - 05) - (11{,}60MNm/rad \cdot 2m)}$$

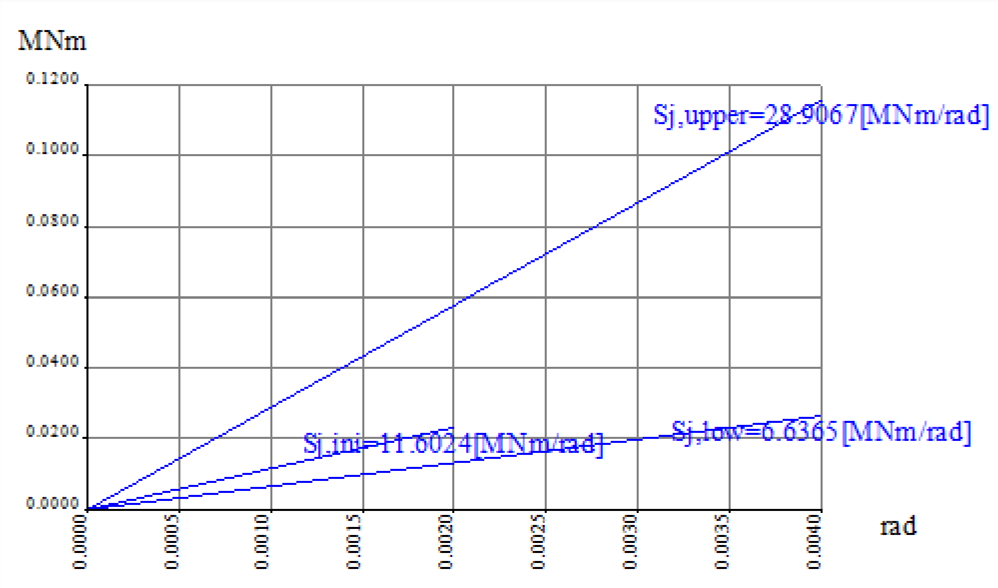
= 28,90 MNm/rad

And in Scia Engineer:

4.3 Check of stiffness requirement

Stiffness data		
Fi y	5.80	MNm/rad
Stiffness modification coef.	2.00	
Sj,app	11.60	MNm/rad
Sj,lower boundary	6.64	MNm/rad
Sj,upper boundary	28.91	MNm/rad

Sj,ini is inside the boundaries.
The actual joint stiffness conforms with the joint stiffness of the analysis model.



6. Calculation of weld sizes

6.1. Calculation of a_f

The weld size design for a_f, using Annex M of EC3:

$$a_f \geq \frac{F_w \cdot \gamma_{Mw} \cdot \beta_W}{f_u b_f \sqrt{2}}$$

$F_w = \min(N_{t,Rd}, \gamma F_{Rd})$

$$N_{t,Rd} = \frac{b_f \cdot t_{fb} \cdot f_{yb}}{\gamma_{M0}} = \frac{110 \cdot 9{,}2 \cdot 235 \cdot 10^{-3}}{1} = 237{,}8\ kN$$

$$F_{Rd} = \frac{M_{Rd}}{h}$$

⇨ h is the lever arm of the connection

⇨ M_{Rd} is the design moment resistance of the connection

$$F_{Rd} = \frac{M_{Rd}}{h} = \frac{34{,}9\ kNm}{216{,}4mm} = 161{,}3\ \text{kN}$$

γ = 1,7 for sway frames

γ = 1,4 for non sway frames

F_w = min ($N_{t,Rd}$, $\gamma\, F_{Rd}$) = min (237,8 kN; 1,4 ∗ 161,3 kN)

F_w = 226 kN

$$a_f \geq \frac{F_w \cdot \gamma_{Mw} \cdot \beta_W}{f_u b_f \sqrt{2}} = \frac{226\ kN \cdot 1{,}25 \cdot 0{,}8}{360 \cdot 10^{-3} \cdot 110 \cdot \sqrt{2}} = 4{,}03\ mm$$

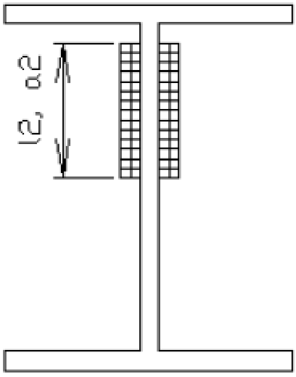
⇨ **a_f = 5mm**

In Scia Engineer :

6.1. Calculation weldsize af / Minimum thickness th for stiffener in column

data		
MRd	34.83	kNm
Gamma	1.40	
h	210.80	mm
FRd	231.29	kN
NT,Rd	237.82	kN
N	231.29	kN
Fu	360.00	MPa
BetaW	0.80	
minimum af	4.13	mm
af	5.00	mm
Minimum th	8.95	mm

6.2. Calculation of a_w



l_2 is taken as the effective length of non-circular pattern for the considered bolt group.

l_2 = 201,57 mm (non circular pattern for bolt row 2, bolt row 1 is above the flange)

Normal Force N = F_i = 62,9 kN (tensile force in bolt row 2)

Shear force D is taken as that part of the maximum internal shear force on the node that is acting on the bolt-rows i and i+1.

D = 10kN / 3 = 3,33 kN

To determine the weld size a_2 in a connection, we use an iterative process with a_2 as parameter until the Von Mises rules is respected:

Conditions:

$$\sqrt{\sigma_1^2 + 3 \cdot (\tau_1^2 + \tau_2^2)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M_w}} = \frac{360\,MPa}{0{,}8 \cdot 1{,}25} = 360\,MPa$$

And:

$$\sigma_1 \leq \frac{f_u}{\gamma_{M_w}} = \frac{360\,MPa}{1{,}25} = 288\,MPa$$

With:

$$\sigma_1 = \tau_2 = \left(\frac{N}{2 \cdot a_2 \cdot l_2}\right)\frac{1}{\sqrt{2}} = \frac{62{,}9\ \cdot 10^3 N}{2 \cdot a_2 \cdot 201{,}57mm}\frac{1}{\sqrt{2}} = \frac{110{,}33\,N/mm}{a_2}$$

$$\tau_1 = \frac{D}{2 \cdot a_2 \cdot l_2} = \frac{3{,}33\ \cdot 10^3 N}{2 \cdot a_2 \cdot 201{,}57mm} = \frac{8{,}26\,N/mm}{a_2}$$

- ⇨ After iteration: $a_2 = a_w$ = 0,7 mm
- ⇨ **a_w = 1 mm**

In Scia Engineer:

6.2. Calculation aw

data		
Ft	62.41	kN
Fv	3.33	kN
lw	201.57	mm
Fu	360.00	MPa
BetaW	0.80	
minimum aw (a2)	1.00	mm
aw	3.00	mm