SCIA
BY ALLPLAN



\ SCIA ENGINEER

ADVANCED TRAINING

STEEL CONNECTIONS

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1. Introduction

This course will explain the calculation of steel connections in SCIA Engineer following the EN 1993-1-8: Design of steel structures – Part 1-8: Design of joints.

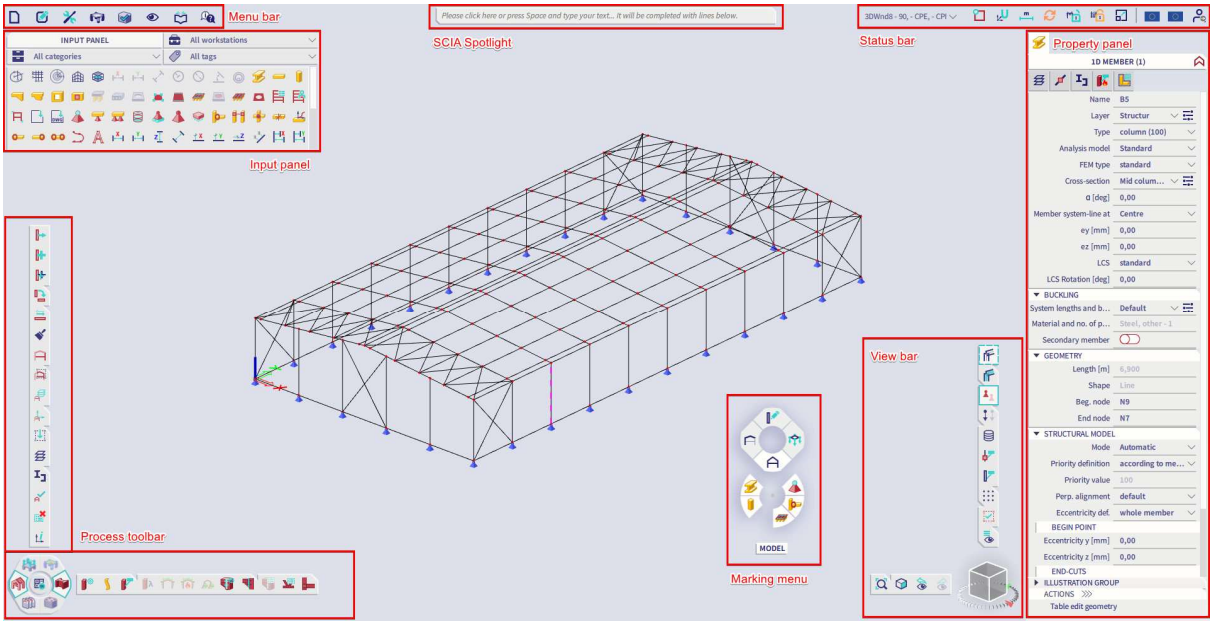
Most of the options in the course can be calculated/checked in SCIA Engineer with the **Steel or Professional edition**.

For some supplementary checks an extra module (or edition) is required, but this will always be indicated in those paragraphs.

The design methods for connection design are explained. More details and references to the applied articles can be found in (Ref.[2]).

The following chapters are valid for the bolted and welded column-beam joints. The design methods for the beam-column joints are principally for moment-resisting joints between I or H sections in which the beams are connected to the flanges of the column. In this document we will describe the total procedure for this type of connection. The other connection types can be found at the end of this document.

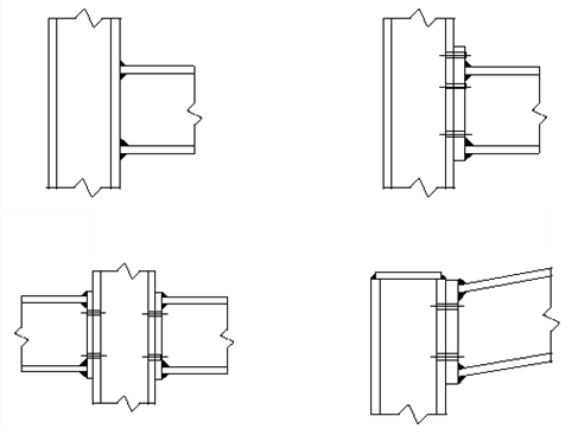
Below you can find an image of the workspace of SCIA Engineer and where to find the different menus.



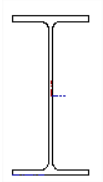
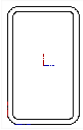
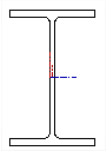
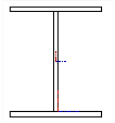
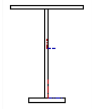
2. Possible connections in SCIA Engineer

The design methods for the column-beam joints are taken from EN 1993-1-8. More detailed information about the applied rules and specific implementations are found in Ref.[1].

The following column-beam and beam-beam connections are possible in SCIA Engineer:



Only the following cross-sections can be used for connections in SCIA Engineer:

Rolled I beam (I+H) 	Rolled hollow section (RHS) 	I section with a haunch (I var) 
Symmetrical welded I section (made of three flats - Iw) 	Asymmetrical welded I section (made of three flats - Iwn) 	

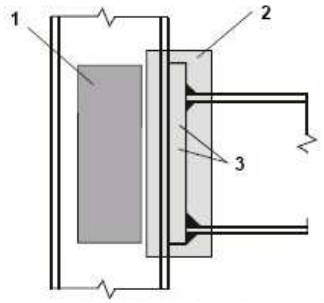
The possible combinations of supported cross-sections with relevance to a geometric connection type is indicated in the tables below. Column base connections support all cross-sections.

		Beam				
		Rolled I+H	Iw	Iwn	I section + haunch	RHS
Column	Rolled I+H	yes	yes	yes	yes	yes
	Iw	yes	yes	yes	yes	yes
	Iwn	yes	yes	yes	yes	yes
	I section + haunch	no	no	no	no	no
	RHS	no	no	no	no	no

		Beam				
		Rolled I+H	Iw	Iwn	I section + haunch	RHS
Beam	Rolled I+H	yes				
	Iw	yes	yes			
	Iwn	yes	yes	yes		
	I section + haunch	yes	yes	yes	yes	
	RHS	yes	yes	yes	yes	yes

It is important to mention that in this context, beam is the entity connected to a column. It is perfectly possible that a column may be horizontal and a beam vertical.

In the checks in SCIA Engineer not only the connection itself will be checked, but also the total joint. A joint is the connection and the web panel in shear, as shown in the picture below.



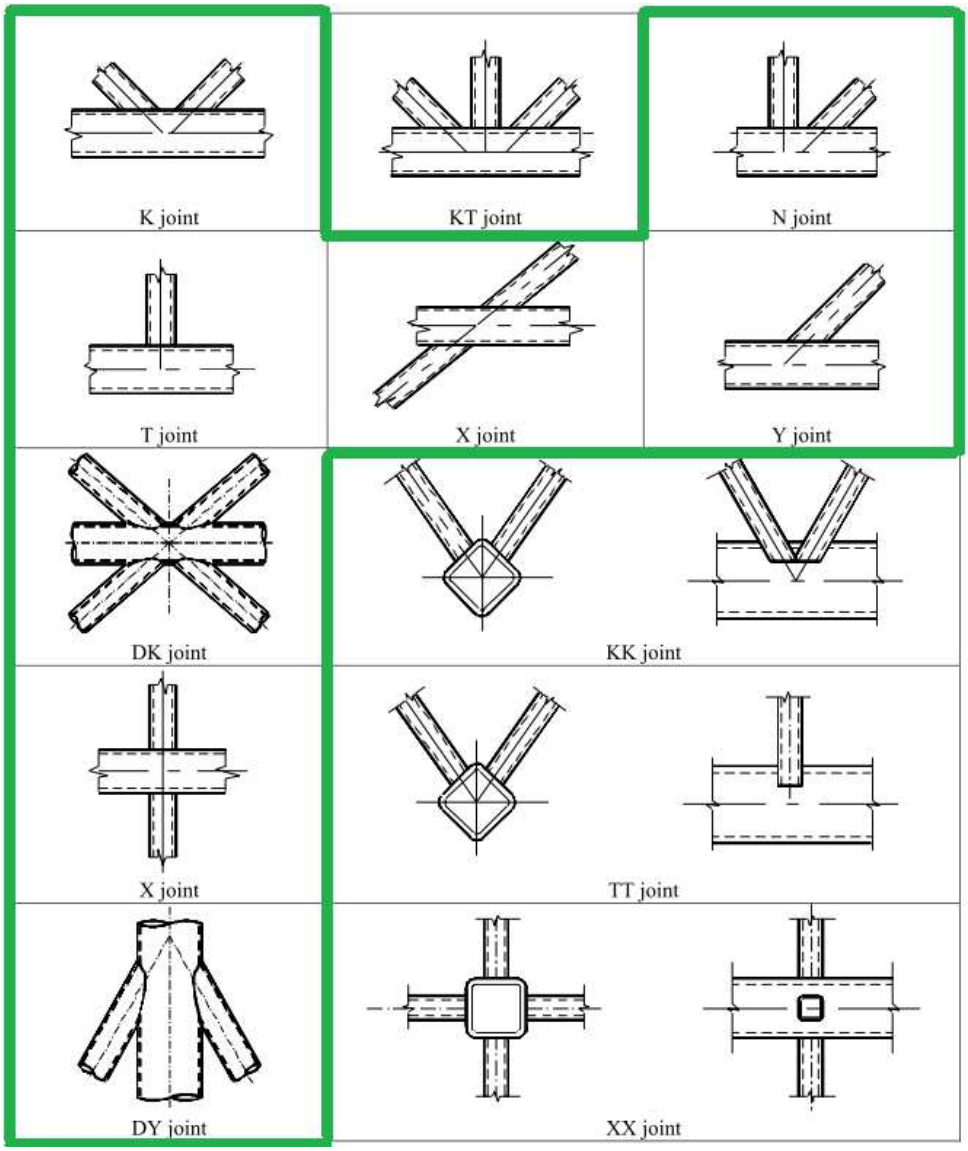
Joint = web panel in shear + connection

1 - web panel in shear

2 - connection

3 - components (e.g. bolts, endplate)

Since SCIA Engineer 17.0, it is also possible to design truss connections of circular hollow sections. The following image shows what is supported so far:

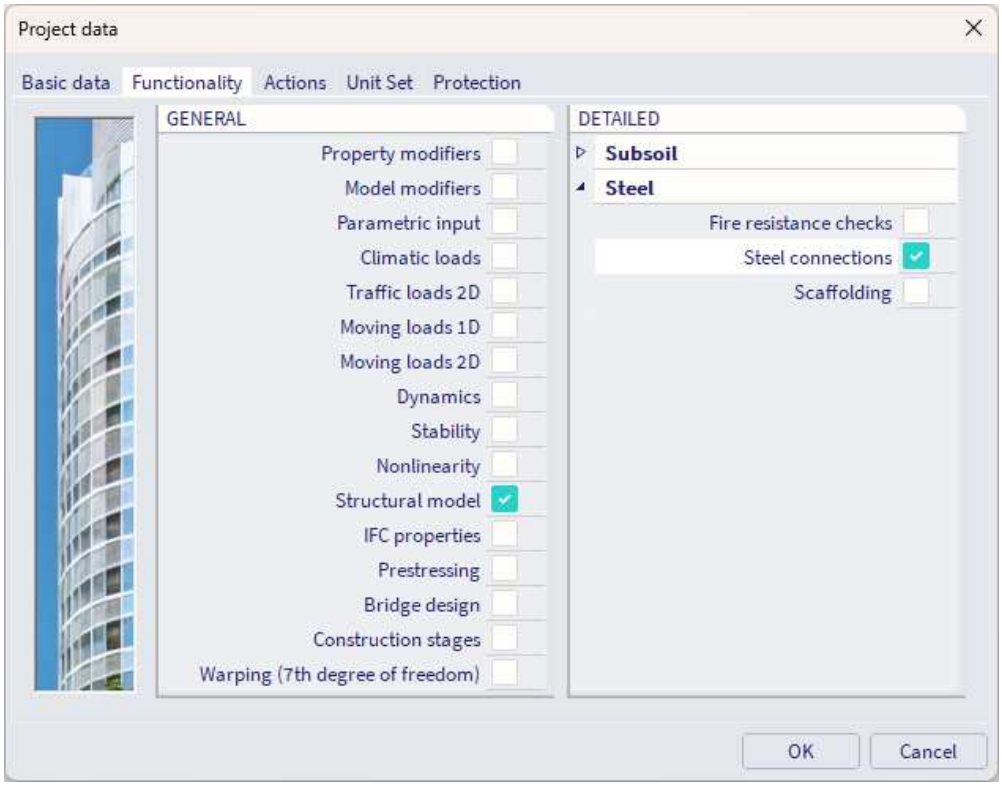


3. Creation of a small example in SCIA Engineer

3.1. Modelling the example

First in this chapter a small example in SCIA Engineer will be shown. Afterwards all principles and the theoretical background will be explained in the next chapter.

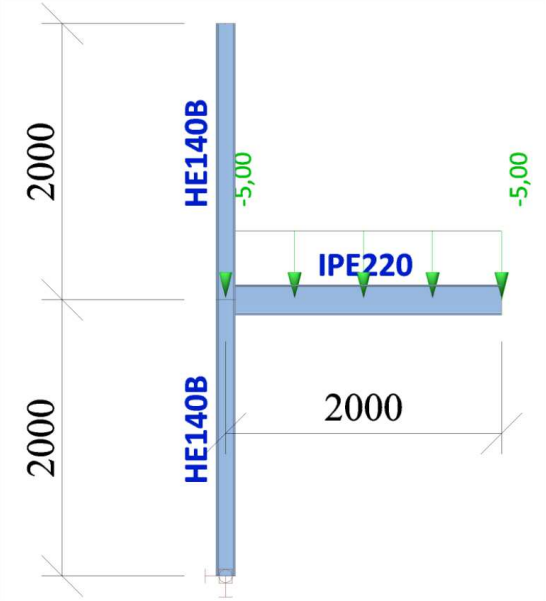
Create a new project in **Frame XYZ**, activate the material **Steel S 235** and activate the functionality **Steel connections:**



The following options are available for connections:

Frame strong-axis	Calculation of bolted and welded (rigid and semi-rigid) connections for the strong axis of the column
Frame weak-axis	Calculation of bolted and welded (rigid and semi-rigid) connections for the weak axis of the column
Pinned grid connections	Calculation of pinned connection in the horizontal plane
Hollow section joints	Calculation of welded tubes in trusses
Expert database	Use a library with default connections in SCIA Engineer or add your own connections to this library
Connection monodrawings	Make overview drawings of your connection(s)

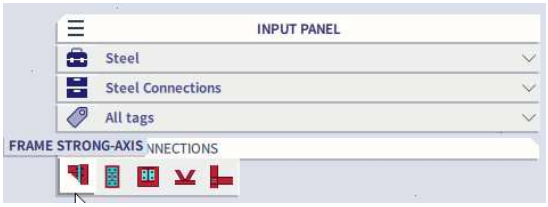
Choose for the column a **HE140B** profile and for the beam an **IPE220** with the following geometry and the only load is a **line load of 5 kN/m on the beam** (no self weight).



3.2. Input of the connection

Calculate the model and go to the **Steel** workstation of the process toolbar.

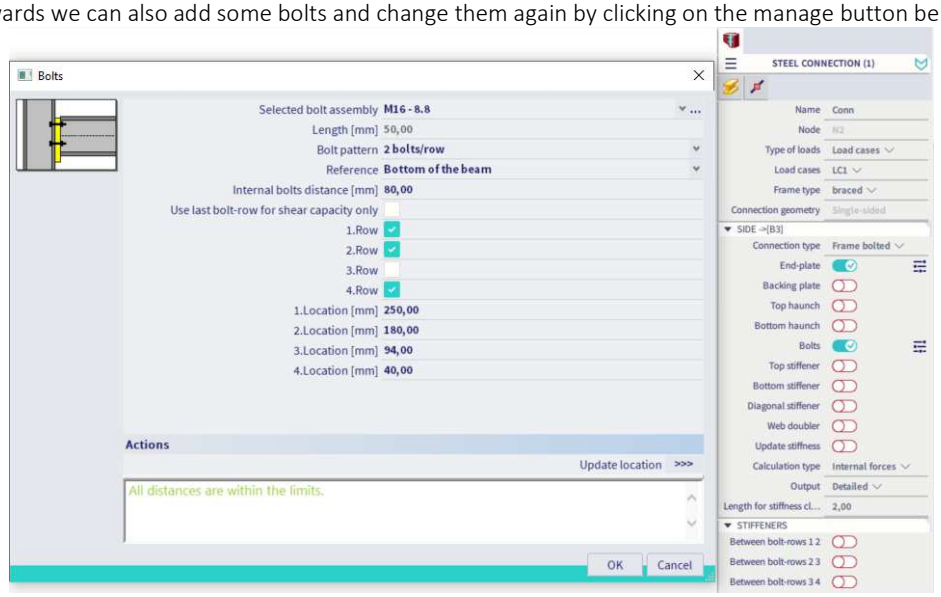
The beam is connected with the strong axis of the column, so we choose "Frame strong-axis" from the input panel by filtering to **Steel > Steel Connections**. Click on this option and select the node between the column and the beam to input the connection.



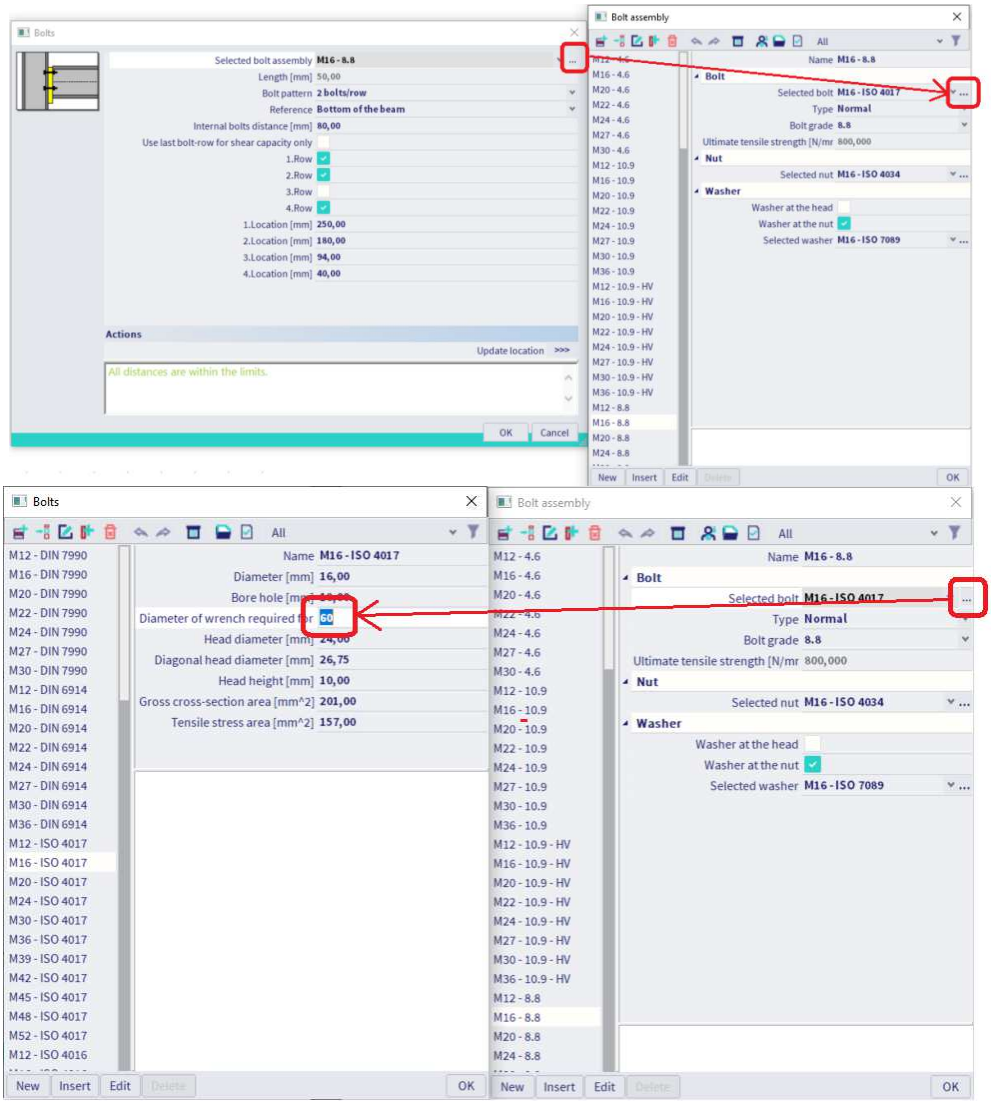
In the properties panel of the connection, you can activate what you want to add on the connection. We choose for a **Frame bolted** connection and we add an end-plate. By clicking on the manage button behind the end-plate option, you can adapt the endplate and we change it into:



Afterwards we can also add some bolts and change them again by clicking on the manage button behind it:



Remark: by default you will get warnings for the bolt locations, modify the wrench diameter from 80mm to 60mm to solve it. In reality you'll need to apply the correct wrench diameter in order to be able to fasten the bolts. See images below to change this parameter:



To check the connection, you have to click on Refresh.
With the option “Open Preview” you can have a summary output of the connection:

Steel connection

Name	Conn
Node	N2
Connection type	Frame bolted
Connection geometry	Single-sided
Calculation type	Internal forces
Lc/Combi	LC1

Connection analysis (summary): Side [B3]

Internal forces

NEd	0.00	kN
Vz,Ed	10.00	kN
My,Ed	-10.00	kNm
Vy,Ed	0.00	kN
Mz,Ed	0.00	kNm

...::STRONG-AXIS CALCULATION::..

Design resistances

Mj,y,Rd	36.65	kNm
Nj,Rd	245.40	kN
Vz,Rd	154.02	kN

Limiting parts

In tension	End plate in tension
In compression	Column web in shear

Stiffness

Sj,ini	10.9694	MNm/rad
Sj	10.9694	MNm/rad

Classification	System SEMI RIGID
Stiffness check	**Not ok**

...::WEAK-AXIS CALCULATION::...

Design resistances

Mj,z,Rd	1.82	kNm
Vy,Rd	144.47	kN

Stiffness

Sj,ini	0.0885	MNm/rad
Sj	0.0885	MNm/rad

...::RESULTS::...

Unity checks

My,Ed/Mj,y,Rd	0.27
Mz,Ed/Mj,z,Rd	0.00
NEd/Nj,Rd	0.00
Vz,Ed/Vz,Rd	0.06
Vy,Ed/Vy,Rd	0.00
Vz,Ed/Vz,Rd + Vy,Ed/Vy,Rd	0.06
My,Ed/Mj,y,Rd + Mz,Ed/Mj,z,Rd	0.27

The calculated unity checks and stiffness’s will be explained in detail in the following chapters.

4. Check of the connection (unity check)

The whole check of the chapters below will be discussed using the example made in the chapter “Creation of a small example in SCIA Engineer” or using example “CON_004.esa”.

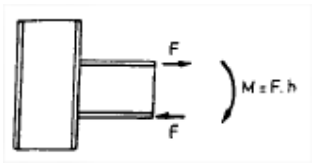
When looking in SCIA Engineer at the detailed output you will find the detailed calculation of SCIA Engineer.

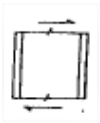
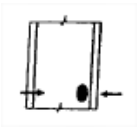
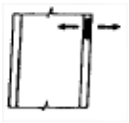


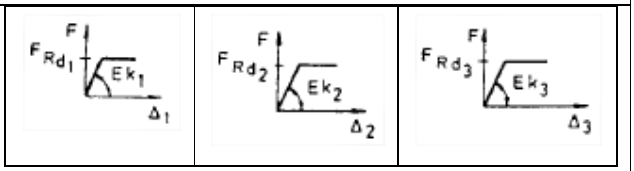
In this document we will describe all checks in SCIA Engineer step by step based on EN 1993-1-8. Ref.[1].

The general analytical procedure which is used for determining the resistance and stiffness properties of a joint, is the so-called component method. The component method considers any joint as a set of individual basic components. Each of these basic components possesses its own strength and stiffness. The application of the component method requires the following steps:

1. identification of the active components in the joint being considered
2. evaluation of the stiffness and/or resistance characteristics for each individual basic component
3. assembly of all the constituent components and evaluation of the stiffness and/or resistance characteristics of the whole joint

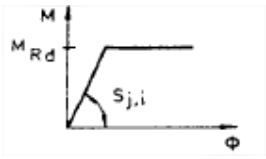
<table>
<tr><td>Three steps</td><td>
</td></tr>
<tr><td>First step:

Definition of the components</td><td>Column web in shear

Column web in compression

Column web in tension
</td></tr>
<tr><td>Second step:

Response of the components</td><td>

Stiffness coefficient k_i of each component

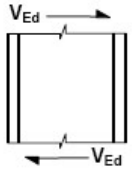
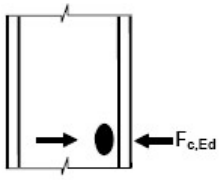
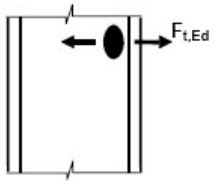
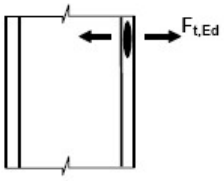
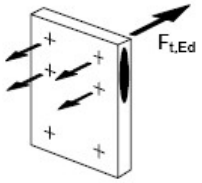
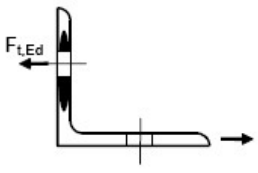
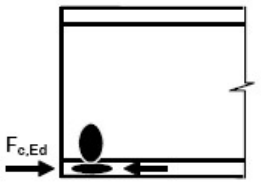
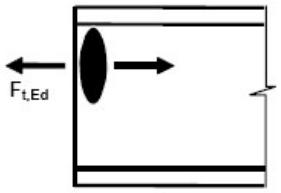
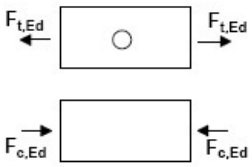
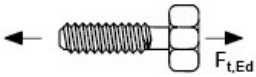
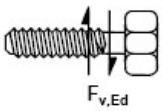
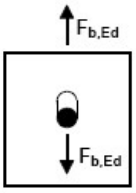
Resistance F_{Rd} of Each component</td></tr>
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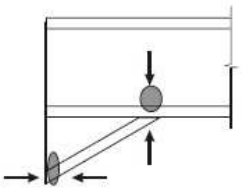
Assembling of the components</td><td>

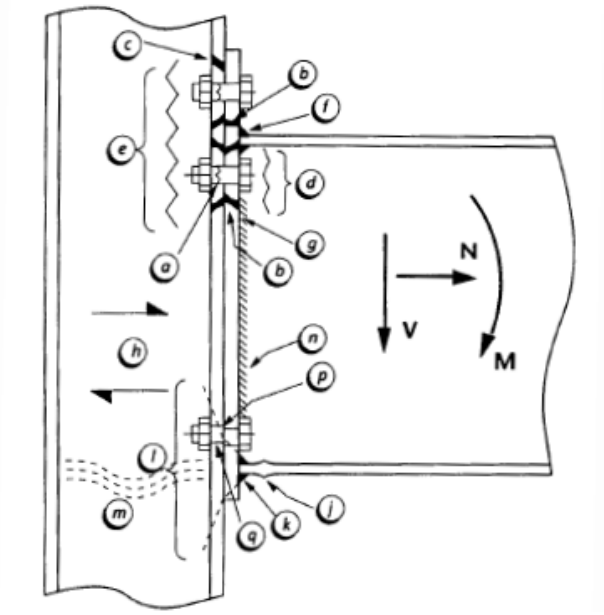
Stiffness of the joint $S_{j,i} = E\ h^2\ /\ \sum l/k_i$

Resistance of the joint $M_{Rd} = min(F_{Rd,i}) \cdot h$</td></tr>
</table>

In the following tables all different components are shown:

Component		
1	Column web panel in shear	
2	Column web In transverse compression	
3	Column web in transverse tension	
4	Column flange in bending	
5	End-plate in bending	
6	Flange cleat in bending	
7	Beam or column flange and web in compression	
8	Beam web in tension	
9	Plate in tension or compression	
10	Bolts in tension	
11	Bolts in shear	
12	Bolts in bearing (on beam flange, column flange, end-plate or cleat)	

13	Concrete in compression including grout	
14	Base plate in bending under compression	
15	Base plate in bending under tension	
16	Anchor bolts in tension	
17	Anchor bolts in shear	
18	Anchor bolts in bearing	
19	Welds	
20	Haunched beam	



Tension	a	bolts in tension
	b	end plate bending
	c	column flange bending
	d	beam web tension
	e	column web tension
	f	flange to end plate weld
	G	web to end plate weld
Horizontal shear	H	column web panel shear
Compression	J	beam flange compression
	K	beam flange weld
	l, m	column web in compression
Vertical shear	n	web to end plate weld
	p	bolt shear
	q	bolt bearing

4.1. General data

In the preview in SCIA Engineer, first general data is shown about the used sections, the used bolts, …

Steel connection

Name	Conn
Node	N2
Connection type	Frame bolted
Connection geometry	Single-sided
Calculation type	Internal forces
Lc/Combi	LC1

Connected beams

Name	Cross-section	Material	Length [m]	Beg. node	End node	Type
B1	CS1 - HE140B	S 235	2,000	N1	N2	column (100)
B2	CS1 - HE140B	S 235	2,000	N2	N3	column (100)
B3	CS2 - IPE220	S 235	2,000	N2	N4	beam (80)

Parts of connection: Side [B3]

Bolts

M16 - 8.8			
Name		Bolt pattern	2 bolts/row
Internal bolts distance [mm]	80,00	External bolts distance [mm]	
Length [mm]	50,00	Reference	Bottom of the beam
1.Location [mm]	250,00	2.Location [mm]	180,00
3.Location [mm]		4.Location [mm]	40,00

End-plate

End			
Material	S 235	Left extension [mm]	15,00
Thickness[mm]	12,00	Right extension [mm]	15,00
Input	Top/Bottom/Left/Right	Total width [mm]	140,00
Top extension [mm]	70,00	Total height [mm]	305,00
Bottom extension [mm]	15,00		

Afterwards the safety factors according EN 1993-1-8 are shown:

Connection analysis: Side [B3]

According to EN 1993-1-8
National annex: Standard EN

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

Those safety factors can be adapted in the National Annex Setup in SCIA Engineer.

And afterwards the internal forces are shown for the chosen load case or combination:

1. Internal forces

LC1		
NEd	0.00	kN
Vz,Ed	10.00	kN
My,Ed	-10.00	kNm
Vy,Ed	0.00	kN
Mz,Ed	0.00	kNm

Tension top
Note: NEd <= 0.05 * Npl,Rd,Beam.

The internal forces, shown here, will result in the biggest unity check or in a stiffness check which is not okay.

You can see in this example that we have a negative moment My, so we have tension in the top flange of the beam. If we have tension in the bottom flange of the beam, the whole calculation is the same, but the first bolt-row will be taken as the bottom one.

Next the calculation of the connection will be shown, both for strong-axis as the weak-axis side.

4.2. Column web panel in shear

As shown in SCIA Engineer, this will be calculated following EN 1993-1-8, art. 6.2.6.1:

$$V_{wp,Rd} = \frac{0.9 \cdot f_{y,w} \cdot A_v}{\sqrt{3} \cdot \gamma_{M0}}$$

Shear area of the column:

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

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

$$\mathbf{V_{wp,Rd}} = \frac{0.9 \cdot f_{y,w} \cdot A_v}{\sqrt{3} \cdot \gamma_{M0}} = \frac{0.9 \cdot 235 \cdot 1312}{\sqrt{3} \cdot 1} \cdot 10^{-3} = \mathbf{160.21\ kN}$$

In SCIA Engineer:

2. Design moment resistance Mj,y,Rd
2.1. Design resistance of basic components
2.1.1. Column web panel in shear
According to EN 1993-1-8 Article 6.2.6.1

Vwp,Rd	data	
Beta	1.00	
Avc	1312.00	mm^2
Vwp,Rd	160.21	kN

4.3. Column web in compression

As shown in SCIA Engineer, this will be calculated following 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): $b_{eff} = t_{fb} + 2\sqrt{2}a_p + 5(t_{fc} + s) + s_p$

$$s_p = 12 + \left(15 - \sqrt{2} \cdot 5\right) = 19.93$$

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.

$$b_{eff} = 9.2 + 2\sqrt{2} \cdot 5 + 5(12 + 12) + 19.93 = 163.27 \text{ mm}$$

Table 5.4: β = 1 → Table 6.3: ω = ω_1

$$\omega = \omega_1 = \frac{1}{\sqrt{1+1.3(b_{eff,c,wc}\frac{t_{wc}}{A_{vc}})^2}} = \frac{1}{\sqrt{1+1.3(163.27\frac{7}{1312})^2}} = 0.71$$

k_{wc} = 1

$$\mathbf{F_{c,wc,Rd}} = \frac{\omega \cdot k_{wc} \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M0}} = \frac{0.71 \cdot 1 \cdot 163.27 \cdot 7 \cdot 235 \cdot 10^{-3}}{1} = \mathbf{190.56\ kN}$$

In SCIA Engineer:

2.1.2. Column web in compression

According to EN 1993-1-8 Article 6.2.6.2

Fc,wc,Rd data		
beff,c,wc	163.27	mm
twc	7.00	mm
ω1	0.71	
ω2	0.45	
ω	0.71	
dwc	92.00	mm
λp,rel	0.55	
ρ	1.00	
kwc	1.00	
Fc,wc,Rd	190.56	kN

4.4. Beam flange and web in compression

As shown in SCIA Engineer, this will be calculated following EN 1993-1-8, art. 6.2.6.7:

(6.21): $\mathbf{F_{c,fb,Rd}} = \frac{M_{c,Rd}}{(h-t_{fb})} = \frac{W_{pl} \cdot f_{yb}}{\gamma_{M0} \cdot (h-t_{fb})}$

$$M_{c,Rd} = \frac{W_{pl} \cdot f_{yb}}{\gamma_{M0}} = \frac{285 \cdot 10^3 mm^3 \cdot 235 \cdot 10^{-3} kN/mm^2}{1} = 66975\ kNmm = 66.98\ kNm$$

$$h - t_{fb} = 220 - 9.2 = 210.80\ mm$$

$$\mathbf{F_{c,fb,Rd}} = \frac{M_{c,Rd}}{(h-t_{fb})} = \frac{66975\ kNmm}{210.80\ mm} = \mathbf{317.72\ kN}$$

In SCIA Engineer:

2.1.3. Beam flange and web in compression

According to EN 1993-1-8 Article 6.2.6.7

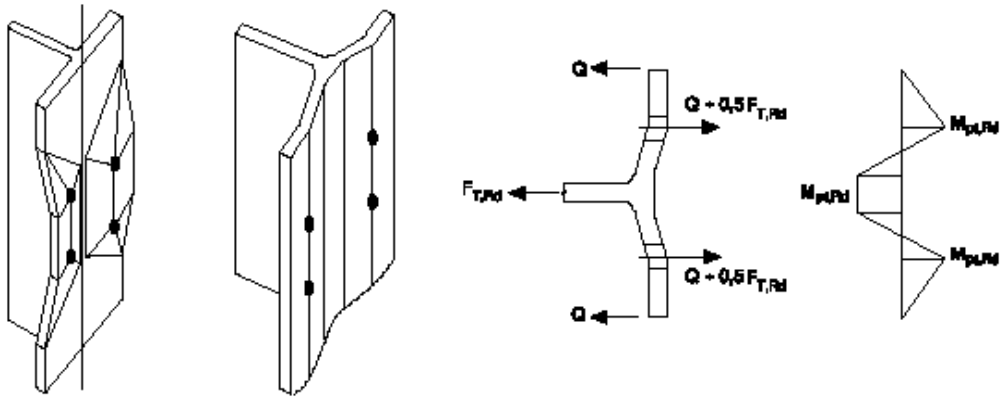
Fc,fb,Rd data		
Section class	1	
Mc,Rd	66.97	kNm
hb-tfb	210.80	mm
Fc,fb,Rd	317.72	kN

4.5. Resistance of the T-stub

4.5.1. Principle of a T-stub calculation

The end plate bending and the column flange bending or bolt yielding, are analysed, using an equivalent T-stub. The three possible modes of failure of the flange of the T stub and the resistance strength for each mode are:

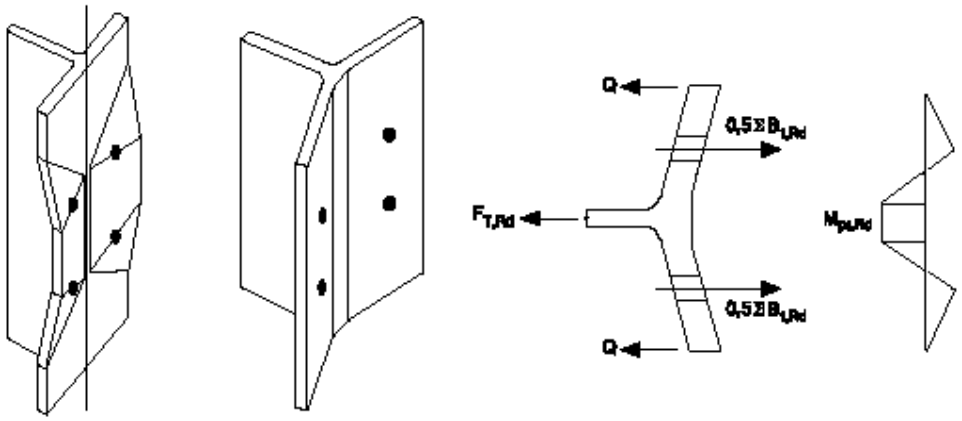
1. complete flange yielding



The bolts stay intact, only the column flange (or end plate) will yield.

$$F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m} \qquad \text{with } M_{pl,1,Rd} = 0{,}25 \sum l_{eff,1} t_f^2 f_y / \gamma_{M0}$$

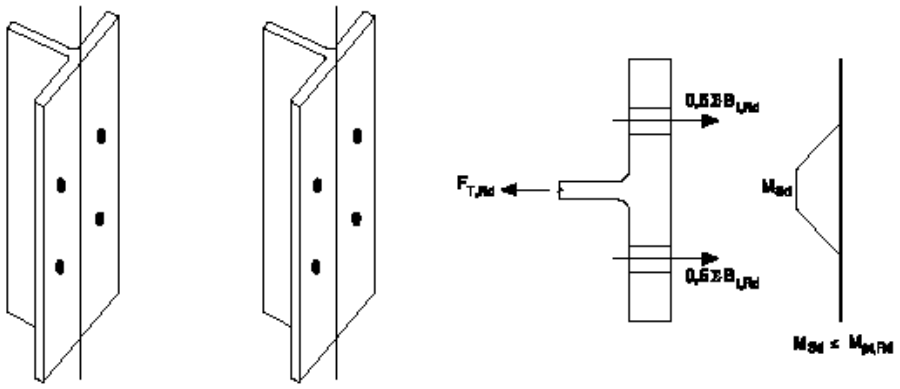
2. bolt failure with flange yielding



The bolts brake together with the yielding of the column flange (or end plate).

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m+n} \qquad \text{with } M_{pl,2,Rd} = 0{,}25 \sum l_{eff,2} t_f^2 f_y / \gamma_{M0}$$

3. bolt failure



The bolts brake. But there is no influence on the column flange (or end plate).

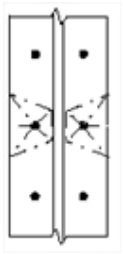
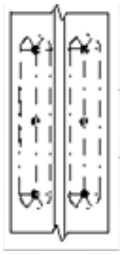
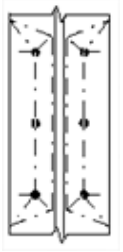
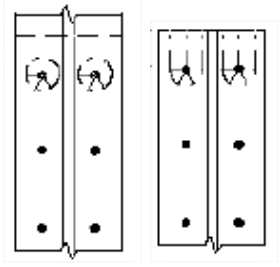
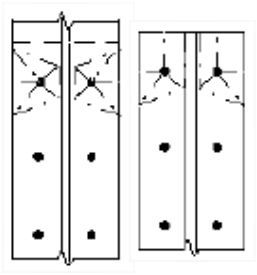
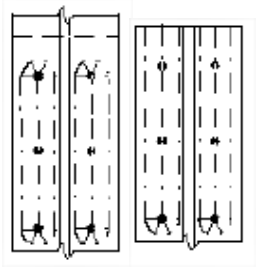
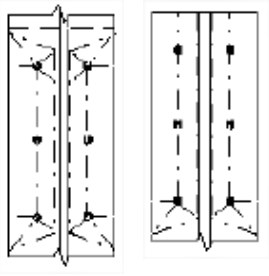
$$F_{T,3,Rd} = \sum F_{t,Rd}$$

And the minimum of $F_{T,1,Rd}$, $F_{T,2,Rd}$ and $F_{T,3,Rd}$ is the limiting tension strength value for the bolt row or bolt group:

$$F_{t,Rd} = \min (F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd})$$

For the failure of the end plate or column flange, an effective length for the different bolt locations will be calculated.

We will assume the effective length for a bolt row or a bolt group and the failure mode could be with a circular pattern or with a non-circular pattern. In the table below some examples are shown for the circular and the non circular patterns:

Circular pattern Bolt row	Non-circular pattern Bolt row	Circular pattern Bolt group	Non-circular pattern Bolt group
Inner bolt row 	Inner bolt row 	Inner bolt row 	Inner bolt row 
End bolt row 	End bolt row 	End bolt row 	End bolt row 

Remark: The formulas given for the calculation of $F_{T,Rd}$ for the different failure mode are only applicable if prying forces may develop. This criterion is given in EN 1993-1-8, Table 6.2:

Table 6.2: Design Resistance $F_{T,Rd}$ of a T-stub flange

	Prying forces may develop, i.e. $L_b \le L_b^*$		No prying forces
Mode 1	Method 1	Method 2 (alternative method)	$F_{T,1-2,Rd} = \frac{2M_{pl,1,Rd}}{m}$
without backing plates	$F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m}$	$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)}$	
with backing plates	$F_{T,1,Rd} = \frac{4M_{pl,1,Rd} + 2M_{bp,Rd}}{m}$	$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd} + 4nM_{bp,Rd}}{2mn - e_w(m+n)}$	
Mode 2	$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$		
Mode 3	$F_{T,3,Rd} = \Sigma F_{t,Rd}$		

If no prying forces may develop, Mode 1 and 2 will be calculated as follows:

$$F_{T,1-2,Rd} = \frac{2M_{pl,2,Rd}}{m}$$

4.5.2. Bolts info

From the general data of the used bolts (M16 – 8.8) the tension resistance of one bolt can be calculated as follows:

$$F_{t,Rd} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_M} = \frac{0.9 \cdot 800\ MPa \cdot 157\ mm^2}{1.25} = 90432\ N = 90.43\ kN$$

2.1.4. Design tension resistance of bolt-row
According to EN 1993-1-8 Article 3.6

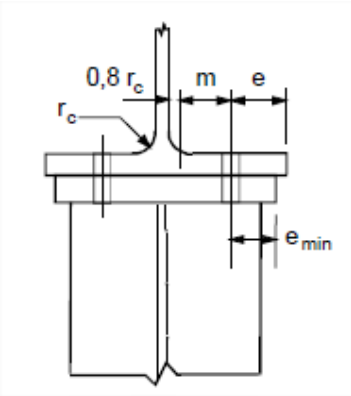
Ft,Rd data		
fub	800.00	N/mm²
As	157.00	mm²
k2	0.90	-
Ft,Rd	90.43	kN
Lb	38.80	mm

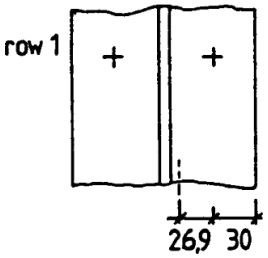
Note: The bolt-rows are numbered starting from the bolt-row farthest from the centre of compression as given by EN 1993-1-8 Article 6.2.7.2 (1).

4.5.3. Column flange

4.5.3.1. General parameters

First some definitions of the parameters, following EN 1993-1-8 (Ref[1]), Figure 6.8 a:





e = 30 mm

$m = \frac{b_c - t_{wc}}{2} - 0.8r - e$

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

$m = \frac{140 - 7}{2} - 0.8 \cdot 12 - 30 = 26.9 \text{ mm}$

e_{min} = 30 mm

$n = e_{min} \leq 1{,}25 \cdot m = 1.25 \cdot 26.9 = 33.6 \text{ mm}$
(see also EN1993-1-8 (Table 6.2))

n = 30 mm

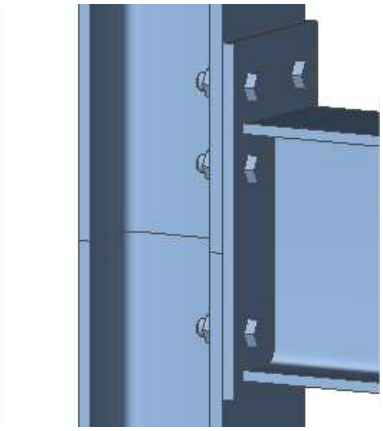
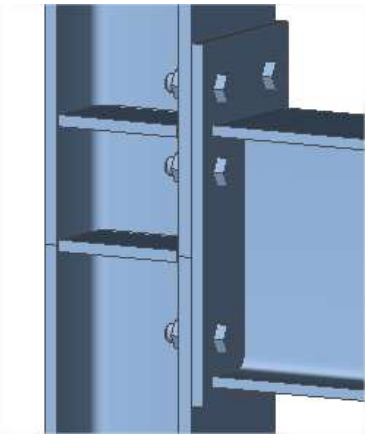
Row	p (p_1 + p_2)
1	0.0 + 35.0
2	35.0 + 70.0
3	70.0 + 0.0

And this is also shown in SCIA Engineer:

row	p (p1+p2)	e	e1	m	n
1	0.00+35.00	30.00	1860.00	26.90	30.00
2	35.00+70.00	30.00	-	26.90	30.00
3	70.00+0.00	30.00	1930.00	26.90	30.00

To calculate the column flange, we need to choose between the effective lengths of an unstiffened column flange (Table 6.4 En 1993-1-8 - Ref.[1]) or for the effective lengths of a stiffened column flange (Table 6.5 EN 1993-1-8 - Ref.[1]).

In this case the column flange is unstiffened. In the table below the difference is shown:

Unstiffened column flange	Stiffened column flange
	

So in this example the following table is used for the calculation of the effective lengths:

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 $\ell_{eff,1} \le \ell_{eff,cp}$		$\sum\ell_{eff,1} = \sum\ell_{eff,nc}$ but $\sum\ell_{eff,1} \le \sum\ell_{eff,cp}$	
Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$		$\sum\ell_{eff,2} = \sum\ell_{eff,nc}$	

This table of the EN1993-1-8 has been extended in SCIA Engineer based on the publications "Joints in Steel Construction – Moment resisting joints to Eurocode 3" and "HERON vol. 20 by P. Zoetemeijer". You can find the effective length formulas for column flange classifications in the table below.

Bolt-row classification	Bolt-row considered individually		Bolt-row considered as part of a group of bolt-rows	
	circular	non-circular	circular	non-circular
Bolt-row adjacent to stiffener	2πm	αm	πm + p	0,5p + αm – (2m + 0,625e)
Other inner bolt-row	2πm	4m + 1,25e	2p	p
Other end bolt-row at end of column	2πm	4m + 1,25e	πm + p	2m + 0,625e + 0,5p
	πm + 2e1	2m + 0,625e + e1	2e1 + p	e1 + 0,5p
End bolt-row adjacent to stiffener	2πm	αm	-	-
	πm + 2e1	e1 + αm – (2m + 0,625e)		
Other end bolt-row	2πm	4m + 1,25e	πm + p	2m + 0,625e + 0,5p
Bolt-row between stiffeners	2πm	αm + α'm - (4m + 1,25e)	-	-

Within SCIA Engineer, a bolt row may be classified on a column side as:

- Bolt-row adjacent to stiffener - if the bolt row lies next to a stiffener and is within limit distance
- Other inner bolt-row - if the bolt-row lies between other bolt-rows
- Other end bolt-row - if the bolt-row lies next to a stiffener, which is farther away to the axis of a connected beam, and is outside the limit distance
- End bolt-row adjacent to stiffener - if the bolt-row is the first or the last bolt-row, lies next to a stiffener, which is closer to the axis of a connected beam
- Other end bolt-row at end of column - if the bolt-row is the first or the last bolt-row, lies next to a stiffener, which is closer to the axis of a connected beam, and is outside the limit distance or if the bolt-row is the first or the last bolt-row and does not lie next to a stiffener
- Bolt-row between stiffeners - if the bolt row is the only bolt-row between stiffeners and lies within the limit distance of both stiffeners

First we choose for each bolt row the classification/location. In this example:

Row 1 and Row 3: Other end bolt-row at end of column
Row 2: Other inner bolt-row

And the same is 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	m2U	m2L	limit	near stiff. U	near stiff. L	Classification
1	-	-	-	-	-	Other end bolt-row at end of column
2	-	-	-	-	-	Other inner bolt-row
3	-	-	-	-	-	Other end bolt-row at end of column

4.5.3.2. Ft,fc,Rd of bolt rows considered individually

The calculation of l_{eff} can be done using Table 6.4. of the EN 1993-1-8 (Ref.[1]).

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

And now from the bottom of Table 6.4:

Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$
Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$

So this results in:

Mode 1 : $l_{eff,1} = l_{eff,nc}$ but $l_{eff,1} \leq l_{eff,cp}$ => $l_{eff,1} = 145.10$
Mode 2 : $l_{eff,2} = l_{eff,nc}$ => $l_{eff,2} = 145.10$

Now we can calculate $M_{pl,1,Rd}$ and $M_{pl,2,Rd}$ for the two modes, with the formula given at the bottom of Table 6.2 of the EN 1993-1-8 (Ref.[1])

$$M_{pl,1,Rd} = M_{pl,2,Rd} = 0.25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0.25 \cdot 145.10 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 1227.5 \text{ kNm}$$

To decide which formula we are using for the calculation of $F_{T,1,Rd}$ and $F_{T,2,Rd}$ we have to check if **prying forces** may develop.

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

= 38.8 mm

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

$L_b^* = \frac{8.8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8.8\,(26{,}9)^3 \cdot 157}{145.10 \cdot (12)^3} \cdot 1 = 107 \text{ mm}$ (see formula in Table 6.2 of EN 1993-1-8 (Ref.[1]))

(with A = tensile stress area of the bolt A_s and n_b = number of bolt rows)

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

So now we can use the formulas given in Table 6.2 En 1993-1-8 (Ref.[1]) to calculate the different mode. The effective lengths for all bolt-rows are the same so:

Mode 1:
By default the alternative method is used for determining $F_{T,1,Rd}$, this can be changed in the connections setup > structural joints : 'Use alternative method for Ft,1,Rd'

$$F_{T,1,Rd} = \frac{(8n - 2e_w) M_{pl,1,Rd}}{2mn - e_w(m+n)} = \frac{(8 \cdot 30 - 2 \cdot 6.75) \cdot 1227.5}{2 \cdot 26.9 \cdot 30 - 6.75 \cdot (26.9 + 30)} = 225.5 \text{ kN}$$

Mode 2:

$$F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m+n} = \frac{2 \cdot 1227.5 + 30 \cdot 2 \cdot 90.43}{26.9 + 30} = 138.5 \text{ kN}$$

Mode 3:

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 90.43 = 180.9 \text{ kN}$$

- ⇨ $\mathbf{F_{T,fc,Rd} = 138.5\ kN}$ (smallest of the three modes)

All those results are shown in SCIA Engineer:

Note: The Alternative method for FT,1,Rd according to EN 1993-1-8 Article 6.2.4, Table 6.2 is used.

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	✓	225.53	138.51	180.86	138.51
2	145.10	145.10	107.26	✓	225.53	138.51	180.86	138.51
3	145.10	145.10	107.26	✓	225.53	138.51	180.86	138.51

The value for L_b was given already in the data of the bolts itself:

2.1.4. Design tension resistance of bolt-row
According to EN 1993-1-8 Article 3.6

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

4.5.3.3. Column web in tension for the individual bolt rows

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) – Ref.[1])

With: $b_{eff,t,wc} = l_{eff}$ = 145.10

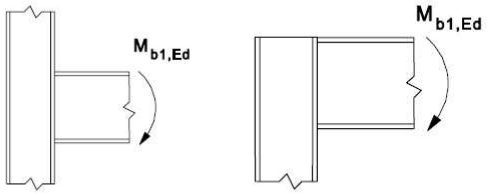
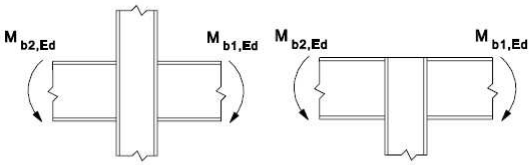
And ω, to allow for the possible effects of shear in the column web panel, should be determined from Table 6.3 (EN 1993-1-8):

Table 6.3: Reduction factor ω for interaction with shear

Transformation parameter β	Reduction factor ω
$0 \le \beta \le 0{,}5$	$\omega = 1$
$0{,}5 < \beta < 1$	$\omega = \omega_1 + 2(1-\beta)(1-\omega_1)$
$\beta = 1$	$\omega = \omega_1$
$1 < \beta < 2$	$\omega = \omega_1 + (\beta - 1)(\omega_2 - \omega_1)$
$\beta = 2$	$\omega = \omega_2$
$\omega_1 = \frac{1}{\sqrt{1 + 1{,}3(b_{eff,c,wc} t_{wc} / A_{vc})^2}}$	$\omega_2 = \frac{1}{\sqrt{1 + 5{,}2(b_{eff,c,wc} t_{wc} / A_{vc})^2}}$
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:

β = 1
ω = ω_1

$$\omega = \omega_1 = \frac{1}{\sqrt{1 + 1.3(b_{eff,c,wc} t_{wc} / A_{vc})^2}}$$

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

$$A_{vc} = 4296 - 2 \cdot 140 \cdot 12 + (\mathbf{7 + 2 \cdot 12}) \cdot 12 = 1308 \text{ mm}^2$$

$$\omega = \omega_1 = \frac{1}{\sqrt{1+1.3(b_{eff,c,wc} t_{wc}/A_{vc})^2}} = \frac{1}{\sqrt{1+1.3(145.10 \cdot 7/1308)^2}} = 0.75$$

$$\Rightarrow \quad F_{T,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} = \frac{0.75 \cdot 145.10 \cdot 7 \cdot 235 \cdot 10^{-3}}{1} = \mathbf{179 \text{ kN}}$$

In SCIA Engineer:

row	beff,t,wc	twc	ω1	ω2	ω	Ft,wc,Rd,i
1	145.10	7.00	0.75	0.49	0.75	178.95
2	145.10	7.00	0.75	0.49	0.75	178.95
3	145.10	7.00	0.75	0.49	0.75	178.95

4.5.3.4. Ft,fc,Rd of bolt rows considered as part of a group

Row 1

l_{eff} circular begin bolt-row = πm + 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 = πm + 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 = πm + 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	224.51	142.55
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

$$M_{pl,1,Rd} = M_{pl,2,Rd} = 0.25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0.25 \cdot 215.1 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 1819.8 \text{ kNm}$$

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

L_b = 38.8 mm

$$L_b^* = \frac{8.8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8.8\,(26.9)^3 \cdot 157}{215.10 \cdot (12)^3} \cdot 2 = 145 \text{ mm}$$ (with n_b = number of bolt rows)

⇨ $L_b < L_b^*$

⇨ Prying forces may develop

Mode 1: $F_{T,1,Rd} = \frac{(8n - 2e_w) M_{pl,1,Rd}}{2mn - e_w(m+n)} = \frac{(8 \cdot 30 - 2 \cdot 6.75) \cdot 1819.8}{2 \cdot 26.9 \cdot 30 - 6.75 \cdot (26.9 + 30)} = 335.13 \text{ kN}$

Mode 2: $F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m+n} = \frac{2 \cdot 1819.8 + 30 \cdot 4 \cdot 90.43}{26.9 + 30} = 254.7 \text{ kN}$

Mode 3: $F_{T,3,Rd} = \sum F_{t,Rd} = 4 \cdot 90.43 = 361.7 \text{ kN}$

⇨ $\mathbf{F_{T,Rd} = 254.7\ kN}$

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

$$M_{pl,1,Rd} = M_{pl,2,Rd} = 0.25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0.25 \cdot 355.1 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 3004.1 \text{ kNm}$$

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

L_b = 38.8 mm

$$L_b^* = \frac{8.8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8.8\,(26.9)^3 \cdot 157}{355.10 \cdot (12)^3} \cdot 3 = 131 \text{ mm}$$ (with n_b = number of bolt rows)

⇨ $L_b < L_b^*$

⇨ Prying forces may develop

Mode 1: $F_{T,1,Rd} = \frac{(8n - 2e_w) M_{pl,1,Rd}}{2mn - e_w(m+n)} = \frac{(8 \cdot 30 - 2 \cdot 6.75) \cdot 3004.1}{2 \cdot 26.9 \cdot 30 - 6.75 \cdot (26.9 + 30)} = 553.2 \text{ kN}$

Mode 2: $F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m+n} = \frac{2 \cdot 3004.1 + 30 \cdot 6 \cdot 90.43}{26.9 + 30} = 391.7 \text{ kN}$

Mode 3: $F_{T,3,Rd} = \sum F_{t,Rd} = 6 \cdot 90.43 = 542.6 \text{ kN}$

⇨ $\mathbf{F_{T,Rd} = 391.7\ kN}$

Row 2-3:

$$\sum l_{eff,cp} = 224.51 + 224.51 = 449.02$$

$$\sum l_{eff,nc} = 142.55 + 142.55 = 285.10$$

Mode 1 = Mode 2: l_{eff} = 285.10

$$M_{pl,1,Rd} = M_{pl,2,Rd} = 0.25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0.25 \cdot 285.1 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 2411.9 \text{ kNm}$$

Prying forces may develop if $L_b \leq L_b^*$
L_b = 38.8 mm

$$L_b^* = \frac{8.8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8.8\,(26.9)^3 \cdot 157}{285.10 \cdot (12)^3} \cdot 2 = 109.2 \text{ mm}$$ (with n_b = number of bolt rows)

⇨ $L_b < L_b^*$

⇨ Prying forces may develop

Mode 1: $F_{T,1,Rd} = \frac{(8n - 2e_w) M_{pl,1,Rd}}{2mn - e_w(m+n)} = \frac{(8 \cdot 30 - 2 \cdot 6.75) \cdot 2411.9}{2 \cdot 26.9 \cdot 30 - 6.75 \cdot (26.9+30)} = 444.2 \text{ kN}$

Mode 2: $F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m+n} = \frac{2 \cdot 2411.9 + 30 \cdot 6 \cdot 90.43}{26.9+30} = 275.5 \text{ kN}$

Mode 3: $F_{T,3,Rd} = \sum F_{t,Rd} = 4 \cdot 90.43 = 361.72 \text{ kN}$

⇨ $\mathbf{F_{T,Rd} = 275.5 \text{ kN}}$

In SCIA Engineer:

For group of bolt-rows:

group	leff,1	leff,2	Lb*	Prying forces	FT,1,Rd	FT,2,Rd	FT,3,Rd	Ft,fc,Rd,g
1- 2	215.10	215.10	144.71	✓	334.34	254.68	361.73	254.68
1- 3	355.10	355.10	131.48	✓	551.95	391.67	542.59	391.67
2- 3	285.10	285.10	109.18	✓	443.14	275.50	361.73	275.50

4.5.3.5. Column web in tension for bolt rows considered as part of a group

Row 1-2:

$$\omega = \omega_1 = \frac{1}{\sqrt{1 + 1.3 (b_{eff,c,wc} t_{wc} / A_{vc})^2}} = \frac{1}{\sqrt{1 + 1.3 (215.10 \cdot 7 / 1312)^2}} = 0.61$$

⇨ $F_{T,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} = \frac{0.61 \cdot 215.10 \cdot 7 \cdot 235 \cdot 10^{-3}}{1}$

⇨ $\mathbf{F_{T,wc,Rd} = 214,86 \text{ kN}}$

Row 1-3:

$$\omega = \omega_1 = \frac{1}{\sqrt{1+1.3(b_{eff,c,wc}t_{wc}/A_{vc})^2}} = \frac{1}{\sqrt{1+1.3(355.10\cdot7/1312)^2}} = 0.42$$

$$\Rightarrow F_{T,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} = \frac{0.42\cdot355.10\cdot7\cdot235\cdot10^{-3}}{1}$$

$$\Rightarrow \mathbf{F_{T,wc,Rd} = 245.40\ kN}$$

Row 2-3:

$$\omega = \omega_1 = \frac{1}{\sqrt{1+1.3(b_{eff,c,wc}t_{wc}/A_{vc})^2}} = \frac{1}{\sqrt{1+1.3(285.1\cdot7/1312)^2}} = 0.50$$

$$\Rightarrow F_{T,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} = \frac{0.50\cdot285.10\cdot7\cdot235\cdot10^{-3}}{1}$$

$$\Rightarrow \mathbf{F_{T,wc,Rd} = 234.26\ kN}$$

In SCIA Engineer:

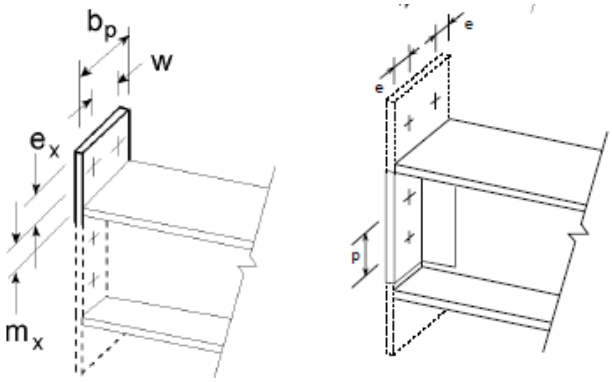
group	beff,t,wc	ω1	ω2	ω	Ft,wc,Rd,g
1- 2	215.10	0.61	0.36	0.61	214.86
1- 3	355.10	0.42	0.23	0.42	245.40
2- 3	285.10	0.50	0.28	0.50	234.26

4.5.4. End plate

We can repeat the whole principle of the column flange calculation on the end plate. In this case we are using Table 6.6 of the EN 1993-1-8 (Ref.[1]).

4.5.4.1. General parameters

First some definitions of the parameters, following EN 1993-1-8 (Ref[1]), Figure 6.8 a:



Some picture 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

$e_x = h_{endplate} - h_{row1} - distance_{Endplate_under - IPE220_under}$

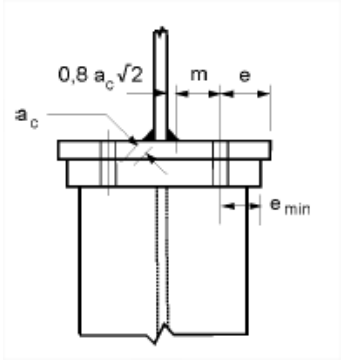
$e_x = 305 - 250 - 15 = 40$

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

$a_f = 0.5 \cdot t_{fb} = 0.5 \cdot 9.2 = 4.6 \rightarrow a_f = 5\ mm$

$m_x = Top - e_x - 0.8 \cdot a \cdot \sqrt{2}$ (see also EN1993-1-8 (Figure 6.10))

$m_x = (305 - 220 - 15) - 40 - 0.8 \cdot 5 \cdot \sqrt{2} = 24.34$



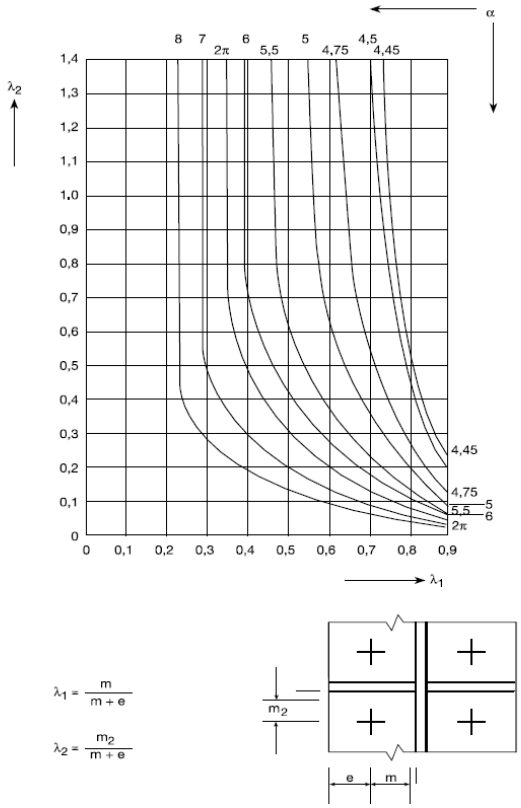
$n = e_{min} = 40\ mm \leq 1.25 \cdot m = 1.25 \cdot 24.34 = 30{,}42\ mm$

n = 30.42 mm

w = 80 mm

Row 2 and Row 3

Using Figure 6.11 of the EN 1993-1-8



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	$a_f \geq 0.7\ t_{fb}$ $a_w \geq 0.7\ t_{wb}$

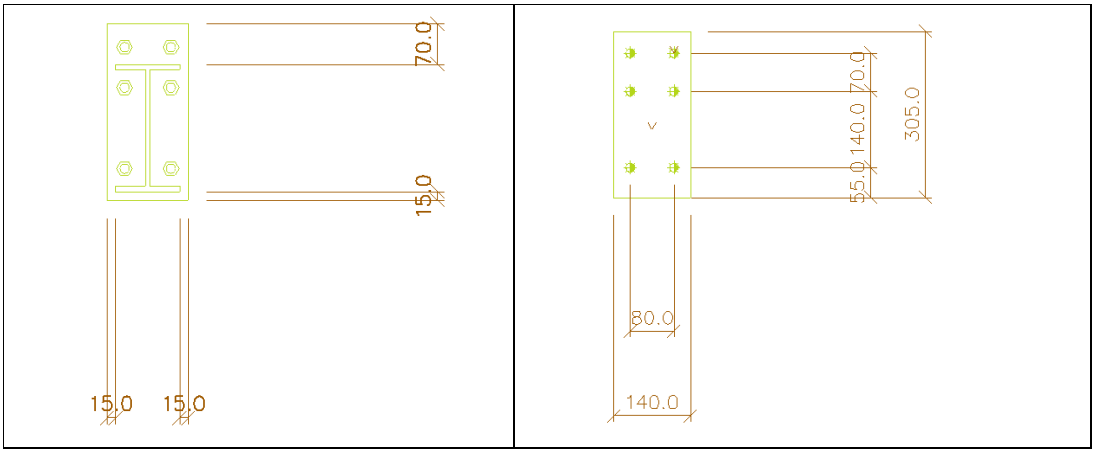
$a_w = 0.5 \cdot t_{wb} = 0.5 \cdot 5.9 = 3.0$

$m = \frac{b_{endplate} - t_{wc}}{2} - e - 0.8 \cdot a \cdot \sqrt{2}$ (see also EN1993-1-8 (Figure 6.10))

$m = \frac{140 - 5.9}{2} - 30 - 0.8 \cdot 3 \cdot \sqrt{2} = 33.66\ mm$

$n = e_{min} = 30\ mm \leq 1.25 \cdot m = 1.25 \cdot 33.66 = 42.01\ mm$

n = 30 mm



$m_{2,row2} = e_x - t_f - 0.8 \cdot a_f \cdot \sqrt{2}$

$m_{2,row2} = \left(35 + \frac{9.2}{2}\right) - 9.2 - 0.8 \cdot 5 \cdot \sqrt{2} = 24.74\ mm$

$m_{2,row3} = h_{row3} - t_f - 0.8 \cdot a_f \cdot \sqrt{2}$

$m_{2,row3} = \left(35 + \frac{9.2}{2}\right) - 9.2 - 0.8 \cdot 5 \cdot \sqrt{2} = 24.74\ mm$

$\lambda_1 = \frac{m}{m + e} = \frac{33.66}{33.66 + 30} = 0.53$

$\lambda_{2,row2} = \lambda_{2,row3} = \frac{m_{2,row2}}{m + e} = \frac{24.74}{33.66 + 30} = 0.39$

⇨ α = 5.77 (Figure 6.6; EN 1993-1-8)

Row	p (p_1 + p_2)	e	m	n	λ_1	λ_2	α
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	5.77
3	70.0 + 0.0	30	33.66	30	0.53	0.39	5.77

In SCIA Engineer:

row	p (p1+p2)	e	ex	m	mx	n	λ1U	λ2U	αU	λ1L	λ2L	αL
1	0.00+0.00	30.00	40.00	-	24.34	30.43	-	-	-	-	-	-
2	0.00+70.00	30.00	-	33.66	-	30.00	0.53	0.39	5.77	-	-	-
3	70.00+0.00	30.00	-	33.66	-	30.00	-	-	-	0.53	0.39	5.77

To calculate the end plate Table 6.6 of the EN 1993-1-8 - Ref.[1] is used.

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.				

This table of the EN1993-1-8 has been extended in SCIA Engineer based on the publications “Joints in Steel Construction – Moment resisting joints to Eurocode 3” and “HERON vol. 20 by P. Zoetemeijer”. You can find the effective length formulas for end plates in the table below.

Bolt-row classification	Bolt-row considered individually		Bolt-row considered as part of a group of bolt-rows	
	circular	non-circular	circular	non-circular
Bolt-row outside of beam	$2\pi m_x$	$4m_x + 1{,}25e_x$	-	-
	$\pi m_x + w$	$e + 2m_x + 0{,}625e_x$		
	$\pi m_x + 2e_x$	$0{,}5b_p$		
	-	$0{,}5w + 2m_x + 0{,}625e_x$		
Bolt-row adjacent to beam flange	$2\pi m$	αm	$\pi m + p$	$0{,}5p + \alpha m - (2m + 0{,}625e)$
Other inner bolt-row	$2\pi m$	$4m + 1{,}25e$	$2p$	p
Other end bolt-row	$2\pi m$	$4m + 1{,}25e$	$\pi m + p$	$2m + 0{,}625e + 0{,}5p$
Bolt-row at the end of stiffened extension adjacent to beam flange	$2\pi m$	αm	-	-
	$\pi m + 2e_x$	$\alpha m - (2m + 0{,}625e) + e_x$		
Bolt-row at the end of stiffened extension away from beam flange	$2\pi m$	$4m + 1{,}25e$	$2e_x + p$	$e_x + 0{,}5p$
	$\pi m + 2e_x$	$2m + 0{,}625e + e_x$	$\pi m + p$	$2m + 0{,}625e + 0{,}5p$
Bolt-row between flanges	$2\pi m$	$\alpha m + \alpha' m - (4m + 1{,}25e)$	-	-

Within SCIA Engineer, a bolt row may be classified on an end-plate side as:

- **Bolt-row outside of beam** - if the bolt-row lies outside of the connected beam on an unstiffened end-plate extension (no plate haunch is present)
- **Bolt-row adjacent to beam flange** - if the bolt-row lies next to a beam flange and is within limit distance
- **Other inner bolt-row** - if the bolt-row lies between other bolt-rows
- **Other end bolt-row** - if the bolt-row lies next to a beam flange, which is farther away to the axis of a connected beam, and is outside the limit distance or lies on a stiffened end-plate extension, lies next to a beam flange and is outside limit distance
- **Bolt-row at the end of stiffened extension adjacent to beam flange** - if the bolt-row is the first or the last bolt-row, lies on a stiffened end-plate extension, lies next to a beam flange and is within limit distance
- **Bolt-row at the end of stiffened extension away from beam flange** - if the bolt-row is the first or the last bolt-row, lies on a stiffened end-plate extension, lies next to a beam flange and is outside limit distance or if the

bolt-row is the first or the last bolt-row, lies on a stiffened end-plate extension and does not lie next to a beam flange

When looking at the previous table we can make the following bolt-row locations:

- Row 1: Bolt-row outside of beam
- Row 2: Bolt-row adjacent to beam flange
- Row 3: Bolt-row adjacent to beam flange

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

2.1.4.2. End-plate

According to EN 1993-1-8 Article 6.2.6.5, 6.2.6.8
(effective lengths in mm, resistance in kN)

row	m2U	m2L	limit	near flange U	near flange L	Classification
1	-	-	-	-	-	Bolt-row outside of beam
2	25.14	-	104.88	✓	-	Bolt-row adjacent to beam flange
3	-	25.14	104.88	-	✓	Bolt-row adjacent to beam flange

4.5.4.2. Bolt rows considered individually

Row 1 - Bolt-row outside of beam:

l_{eff} circular patterns = smallest of:

$2\pi m_x = 2*3.14*24.34 =$ **152.93**
$\pi m_x + w = 3.14*24.34 + 80 = 156.47$
$\pi m_x + 2e = 3.14*24.34 + 2*40 = 156.47$

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 - Bolt-row adjacent to beam flange:

l_{eff} circular patterns = $2\pi m = 2*3.14*33.66 = 211.49$

l_{eff} non-circular patterns: $\alpha m = 5.77*33.66 = 194.22$

Row 3 - Bolt-row adjacent to beam flange:

l_{eff} circular patterns = $2\pi m = 2*3.14*33.66 = 211.49$

l_{eff} non-circular patterns: $\alpha m = 5.77*33.66 = 194.22$

Row	l_{eff} circular patterns	l_{eff} non-circular patterns
1	152.93	70.00
2	211.49	194.22
3	211.49	194.22

In SCIA Engineer:

row	leff,cp,i	leff,nc,i
1	152.95	70.00
2	211.47	194.10
3	211.47	194.10

And now from the bottom of Table 6.6:

Mode 1:	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$
Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$

So this results in:

Bolt-row 1:

Mode 1 : $l_{eff,1} = l_{eff,nc}$ but $l_{eff,1} \leq l_{eff,cp}$ → $l_{eff,1}$ = 70.00
Mode 2 : $l_{eff,2} = l_{eff,nc}$ → $l_{eff,2}$ = 70.00

Bolt-row 2; Bolt-row 3:

Mode 1 : $l_{eff,1} = l_{eff,nc}$ but $l_{eff,1} \leq l_{eff,cp}$ → $l_{eff,1}$ = 194.10
Mode 2 : $l_{eff,2} = l_{eff,nc}$ → $l_{eff,2}$ = 194.10

Now the same check for prying forces can be executed and the same formulas for the different mode.

Afterwards also the beam web in tension can be calculated again using the same formulas.

The manual calculation of this can be found in our calculation Steel design example of a joint with extended end plate".

This will result in the following tables for the individual bolt-rows:

Note: The Alternative method for FT,1,Rd according to EN 1993-1-8 Article 6.2.4, Table 6.2 is used.

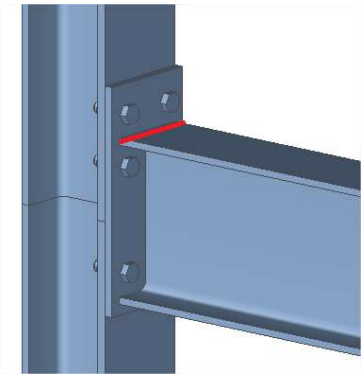
For individual bolt-row:

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	✓	122.17	122.10	180.86	122.10
2	194.10	194.10	157.04	✓	233.51	136.83	180.86	136.83
3	194.10	194.10	157.04	✓	233.51	136.83	180.86	136.83

row	beff,t,wb	Ft,wb,Rd,i
1	-	-
2	194.10	269.11
3	194.10	269.11

4.5.4.3. Bolt rows considered as part of a group

Again for the bolt rows considered as part of a group, we can follow the same principle. For the end plate, only the group 2-3 is a possible group. Between row 1 and row 2 we have the flange of the beam, which will be seen as a stiffener.



So Row 1 and Row 2 are separate.

The only group is thus Row 2-3 and l_{eff} is calculated again using Table 6.6 of the EN 1993-1-8 (Ref.[1]).

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 + 5.77*33.66 – (2*33.66 + 0.625*30) = 178.15

Row 3:

l_{eff} circular end bolt-row = πm + p = 3.14*33.66 + 140 = 245.73

l_{eff} non-circular end bolt-row = 0.5p + αm – (2m + 0.625e) = 0.5*140 + 5.77*33.66 – (2*33.66 + 0.625*30) = 178.15

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	178.15
3	-	-	245.73	178.15	-	-

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	-	-	-	-	-	-
2	-	-	-	-	245.73	178.03
3	-	-	245.73	178.03	-	-

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:

$$\sum l_{eff,cp} = 245{,}73 + 245{,}73 = 491.46$$

$$\sum l_{eff,nc} = 178{,}15 + 178{,}15 = 356{,}29$$

Mode 1 = Mode 2: l_{eff} = 356.29

In SCIA Engineer:

group	leff,cp,g	leff,nc,g
2- 3	491.47	356.07

$$M_{pl,1,Rd} = M_{pl,2,Rd} = 0.25 \sum l_{eff} t_f^2 f_y / \gamma_{M0} = \frac{0.25 \cdot 356.29 \cdot 12^2 \cdot 235 \cdot 10^{-3}}{1} = 3014.27 \text{ kNm}$$

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

L_b = 38.8 mm

$$L_b^* = \frac{8.8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8.8\,(33.66)^3 \cdot 157}{356.29 \cdot (12)^3} \cdot 2 = 171.16 \text{ mm}$$ (with n_b = number of bolt rows)

⇨ $L_b < L_b^*$

⇨ Prying forces may develop

Mode 1: $F_{T,1,Rd} = \frac{(8n - 2e_w) M_{pl,1,Rd}}{2mn - e_w(m+n)} = \frac{(8 \cdot 30 - 2 \cdot 6.75) \cdot 3014.27}{2 \cdot 33.66 \cdot 30 - 6.75 \cdot (33.66 + 30)} = 429.42 \text{ kN}$

Mode 2: $F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n \sum F_{t,Rd}}{m+n} = \frac{2 \cdot 3014.27 + 30 \cdot 4 \cdot 90.43}{33.66 + 30} = 265.16 \text{ kN}$

Mode 3: $F_{T,3,Rd} = \sum F_{t,Rd} = 4 \cdot 90.43 = 361.7 \text{ kN}$

⇨ $\mathbf{F_{T,Rd} = 265.12\ kN}$

In SCIA Engineer:

group	leff,1	leff,2	Lb*	Prying forces	FT,1,Rd	FT,2,Rd	FT,3,Rd	Ft,ep,Rd,g
2- 3	356.07	356.07	171.21	✓	428.37	265.12	361.73	265.12

4.5.4.4. Beam web in tension for bolt rows considered as part of a group

Row 2-3:

⇨ $F_{T,wb,Rd} = \frac{b_{eff,t,wb} t_{wb} f_{y,wb}}{\gamma_{M0}} = 356.29 \cdot 5.9 \cdot 235 \cdot 10^{-3} / 1$

⇨ $\mathbf{F_{T,wc,Rd} = 493.9 kN}$

In SCIA Engineer:

group	beff,t,wb	Ft,wb,Rd,g
2- 3	356.07	493.69

4.5.5. Potential tension resistance for each bolt row

In SCIA Engineer all results for the column flange and end plate are summarized in one table:

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	-	178.95	-	122.10	-	-	-	122.10
2	138.51	132.58	178.95	92.75	136.83	-	269.11	-	92.75
3	138.51	176.82	178.95	30.54	136.83	172.37	269.11	400.94	30.54

Σ Ft,Rd = 245.40 kN

The minimum value of all those calculated value is the limited value for the tension resistance of one bolt row:

- Row 1: 122.10 kN (End plate failure)
- Row 2: 92.75 kN (Column flange failure)
- Row 3: 30.54 kN (Column flange failure)

This will be used in the calculation of M_{Rd} in the next chapter.

4.6. Calculation of M_{Rd}

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

$$M_{j,Rd} = \sum_r h_r F_{tr,Rd} \qquad \text{(EN 1993-1-8; §6.2.7.2 – Ref.[1])}$$

$F_{t,min}$ for each bolt row:

- Row 1: 122.10 kN (End plate failure)
- Row 2: 92.75 kN (Column flange failure)
- Row 3: 30.54 kN (Column flange failure)

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: 159,72kN

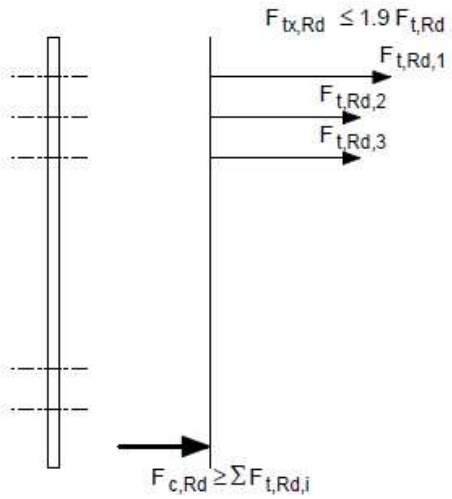
This is also shown in SCIA Engineer:

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

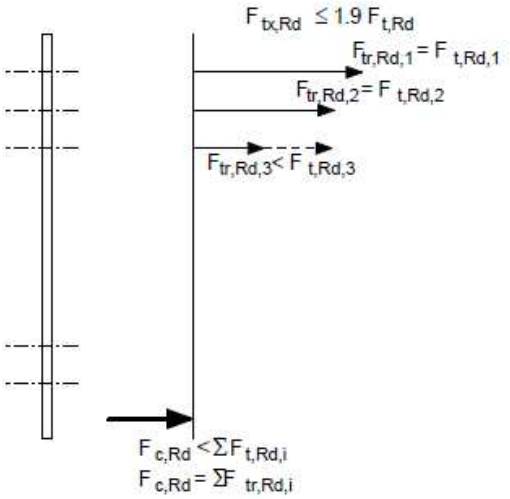
Limiting resistance = 160.21 kN

This limit and the triangular limit (see further) are shown on the next page.



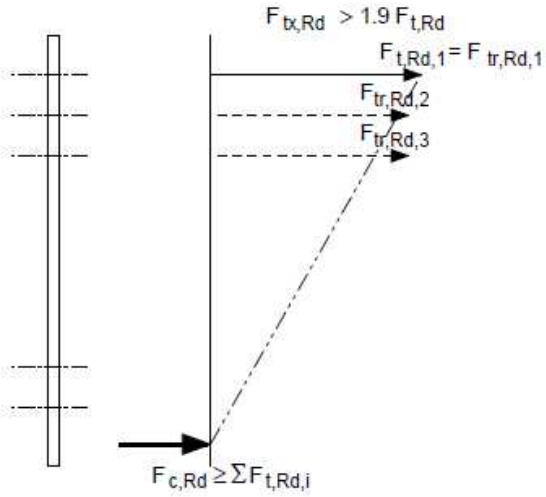
(a) Plastic distribution

- 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

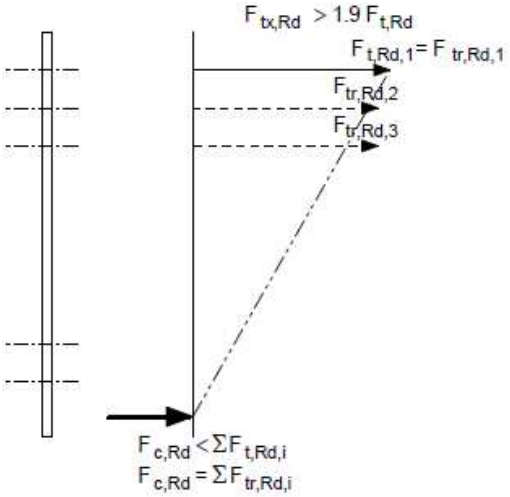
- Because $F_{c,Rd}$ and/or $V_{wp,Rd} < F_{t,Rd,i}$ therefore the effective tension resistances ($F_{tr,Rd}$) have to be reduced starting from the closest bolt to the compression centre:



(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

For the first bolt row $F_{t,Rd,1}$ = 122.10 kN.

The maximum value for bolt row 2 is: $F_{t,Rd,2}$ = Limiting resistance - $F_{t,Rd,1}$ = 160.21 – 122.10 = 38.11kN.

And row 3 will not take any resistance because $F_{t,Rd,1} + F_{t,Rd,2}$ = Limiting resistance = 160.21 kN

This principle is shown on the next page.

- Row 1: 122.10 kN (End plate failure)
- Row 2: 38.10 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	122.10	0.00	122.10
2	92.75	54.65	38.10
3	30.54	30.54	0.00

Following EN 1993-1-8 §6.2.7.2 (9) (Ref.[1]) 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 $F_{tx,Rd} \leq 1.9 \cdot F_{t,Rd}$ 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	122.10	X	-	122.10
2	38.10	X	-	38.10
3	0.00	X	-	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. In SCIA Engineer the values are given as the distance to the bottom of the beam, so we have to subtract the half of the thickness of the flange (=9.2 mm / 2) of this distance.

Row	h [mm]	F_t [kN]
1	245.4	122.10
2	175.4	38.10
3	35.4	0

$M_{j,Rd}$ = 245.4 * 122.10 + 175.4 * 38.10 = 36646 kNmm =36.64 kNm

In SCIA Engineer:

2.3. Determination of Mj,y,Rd

According to EN 1993-1-8 Article 6.2.7.2 (1)

row	hr[mm]	Ft,r,Rd[kN]
1	245.40	122.10
2	175.40	38.10
3	35.40	0.00

Mj,y,Rd = 36.65 kNm

4.7. Calculation of NRd

The value for $N_{j,Rd}$ is calculated as follows:

If $N_{j,Ed}$ is a tensile force, the $N_{j,Rd}$ is determined by critical value for the following components:

- For bolted connection, as a combination for all bolt rows:
  - column web in transverse tension
  - column flange in bending
  - end plate in bending
  - beam web in tension
  - bolts in tension
- For welded connection:
  - Column web in transverse tension, where the value for t_{fb} in formulas (6.10) and (6.11) is replaced by the beam height.
  - Column flange in bending, by considering the sum of formula (6.20) at the top and bottom flange of the beam.
  - If $N_{j,Ed}$ is a compressive force, the $N_{j,Rd}$ is determined by the following components:
    - Column web in transverse compression, where the value for t_{fb} in formulas (6.16) is replaced by the beam height.
    - Column flange in bending, by considering the sum of formula (6.20) at the top and bottom flange of the beam.

In all cases, $N_{j,R} \leq N_{pl,Rd}$.

In our example the normal force 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	beff,t,wc	ω1	ω2	ω	Ft,wc,Rd,g
1- 2	215.10	0.61	0.36	0.61	214.86
1- 3	355.10	0.42	0.23	0.42	245.40
2- 3	285.10	0.50	0.28	0.50	234.26

⇨ **245.40 kN**

Beam Web in tension:

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

group	beff,t,wb	Ft,wb,Rd,g
2- 3	356.07	493.69

⇨ **493.69 kN**

Endplate in bending:

Here the most limiting value of the endplate (individual rows and groups) will be calculated.

In this case the limiting value is

- Bolt row 1
- Group of bolt row 2+3

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	✓	122.17	122.10	180.86	122.10
2	194.10	194.10	157.04	✓	233.51	136.83	180.86	136.83
3	194.10	194.10	157.04	✓	233.51	136.83	180.86	136.83

row	beff,t,wb	Ft,wb,Rd,i
1	-	-
2	194.10	269.11
3	194.10	269.11

For group of bolt-rows:

group	leff,1	leff,2	Lb*	Prying forces	FT,1,Rd	FT,2,Rd	FT,3,Rd	Ft,ep,Rd,g
2- 3	356.07	356.07	171.21	✓	428.37	265.12	361.73	265.12

⇨ And this results in: 122.10 kN + 265.12 kN = **387.22 kN**

Column Flange in tension:

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

For group of bolt-rows:

group	leff,1	leff,2	Lb*	Prying forces	FT,1,Rd	FT,2,Rd	FT,3,Rd	Ft,fc,Rd,g
1- 2	215.10	215.10	144.71	✓	334.34	254.68	361.73	254.68
1- 3	355.10	355.10	131.48	✓	551.95	391.67	542.59	391.67
2- 3	285.10	285.10	109.18	✓	443.14	275.50	361.73	275.50

⇨ **391.67 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:

2.4. 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)	493.69	kN
Endplate in bending (Ft,ep,Rd)	387.22	kN
Column Flange in bending (Ft,fc,Rd)	391.67	kN

Nj,Rd = 245.40 kN

4.8. Calculation of VRd

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: $\alpha_v = 0.6$

$F_{ub} = 800$ MPa

A is the tensile stress area of the bolt A_s

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A_s}{\gamma_{M2}} = \frac{0.6 \cdot 800 \cdot 157 \cdot 10^{-3}}{1.25} = \mathbf{60.29\ kN}$$

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 * 0.4/1.4 + 2) * 60.29 kN = 189.48 kN

In SCIA Engineer:

3. Design shear resistance Vz,Rd

Vz,Rd data		
Fv,Rd	60.29	kN
nb,full	2	
nb,reduced	4	
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
Avz	1591.08	mm^2
l1 (tension)	0.57	
l2 (compression)	0.43	
Vz,Rd beam	154.02	kN
Vz,Rd	154.02	kN

4.9. Weak axis resistances

The calculation of out-of-plane moment $M_{j,z,Rd}$ resistance is based on the publications "DESIGN OF STRUCTURAL JOINTS CONNECTING H OR I SECTIONS subjected to in-plane and out-of-plane bending" by Neumann N, Nuhic F:, EUROSTEEL, 2011 and publication "Single-sided structural beam-to-column joint of H- or I-profiles with bolted endplate exposed to in-plane and out-of-plane bending" by Kristensen SO, Stavanger, 2010.

The study addresses the strong axis beam-to-column joints between H or I section members. The focus is on bolted end-plate joints, with two lines and two or more rows of bolts symmetrical about both the beam's major and its minor axis.

In SCIA Engineer this theory adopted when needed, will be used for all strong-axis frame bolted beam-column and splice connections with I or H sections. The method will be used only for 2 bolts / row configuration.

If needed, the complete weak-axis calculation (out-of-plane moment resistance Mj,z,Rd, shear force resistance Vy,Rd and stiffness for weak bending) may be skipped by activating "Neglect weak-axis calculation" check box in the connection setup. If so, the weak-axis calculation is not performed and a message is displayed on the output in the part dedicated to the weak-axis calculation. The check box is deactivated by default.



If you want to perform weak-axis calculation (the check box mentioned above is deactivated), but the connection does not fulfill the required conditions, weak-axis calculation is not performed and you are informed about that. It is still possible to calculate shear force resistance $V_{y,Rd}$ even if the conditions are not fulfilled, but only in case design bending moment $M_{z,Ed}$ is zero.

The additional weak-axis bending unity check is displayed by the check and also linear interpolations for strong and weak-axis bending moment components are performed and unity checks calculated if weak-axis bending moment resistance $M_{j,z,Rd}$ is calculated:

$$\frac{M_{z,Ed}}{M_{j,z,Rd}} \leq 1.0$$

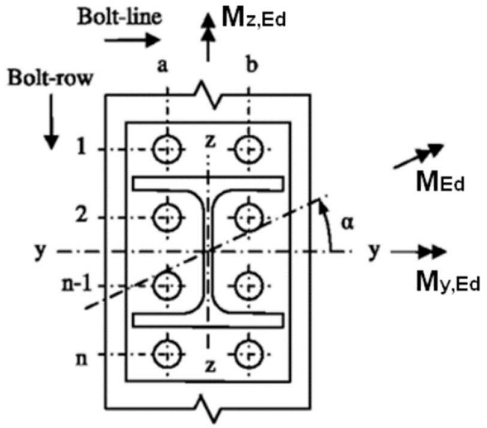
$$\frac{M_{y,Ed}}{M_{j,y,Rd}} + \frac{M_{z,Ed}}{M_{j,z,Rd}} \leq 1.0$$

And if the design normal force $N_{Ed} > 0.05 * N_{pl,Rd}$ also the following check is performed:

$$\frac{M_{y,Ed}}{M_{j,y,Rd}} + \frac{M_{z,Ed}}{M_{j,z,Rd}} + \frac{N_{Ed}}{N_{j,Rd}} \leq 1.0$$

4.9.1. Bending moment resistance

The connection geometry and the design bending moment may be seen on the picture below:



The strong-axis moment resistance $M_{j,y,Rd}$ of the joint is determined based on EN 1993-1-8, assuming no weak-axis bending influence. Similarly to this, it is assumed that the strong-axis moment bending will not influence calculation of weak-axis moment resistance $M_{j,z,Rd}$ of the joint.

The weak-axis moment resistance $M_{j,z,Rd}$ of the joint may be determined by:

$$M_{j,z,Rd} = F_{ta,Rd} * y$$

y is the design distance from the bolt-line in tension to the center of compression for weak-axis bending. The leverage arm is dependent on the stiffness of the components. Within SCIA Engineer it is assumed, that joint components are infinitely stiff. The additional split is based on the type of the bolts

Case	Stiffness		Preload	Distance from the bolt-line in tension to the centre of compression *y*
	Column flange	Bolts		
1	Infinite	Infinite	No	$\frac{b_b + p_2}{3}$
2	Infinite	Infinite	Yes	$\frac{b_b + 3p_2}{6}$

With:

b_b width of the beam (in case of non-symmetric beam or splice connection the minimum is used)

p_2 horizontal spacing between the two lines of bolts

$F_{ta,Rd}$ effective design tension resistance of a bolt-line for the weak-axis moment calculation taken as:

$$F_{ta,Rd} = \min [F_{ta,fc,Rd}, F_{ta,ep,Rd}, F_{cb,fb,Rd}, (F_{ta,fct,Rd} + F_{ta,wbc,Rd})]$$

With:

$F_{ta,fc,Rd}$	design tension resistance for bolt-line a of the column flange in transverse bending
$F_{ta,ep,Rd}$	design tension resistance for bolt-line a of the end-plate in bending
$F_{cb,fb,Rd}$	design compression resistance for bolt-line b of the beam flange in compression
$F_{ta,fct,Rd}$	design tension resistance for bolt-line a of the column flange in twisting
$F_{ta,wbc,Rd}$	design tension resistance for bolt-line a of the column web in bending

For splice connections only $F_{ta,ep,Rd}$ and $F_{cb,fb,Rd}$ component resistances are calculated for each side and minimum resistance from the four components is taken as $F_{ta,Rd}$.

In SCIA Engineer:

6.2. Determination of Mj,z,Rd

Mj,z,Rd data		
Fta,Rd	28.79	kN
y	63.33	mm
Mj,z,Rd	1.82	kNm

Calculation of the weak-axis components may be seen in the following subchapters.

4.9.1.1. Column flange in bending

The component resistance $F_{ta,fc,Rd}$ for each bolt row is already calculated according to EN1993-1-8 Art. 6.2.6.4 in strong axis moment resistance $F_{t,fc,Rd}$. Since tension of weak-axis bending only concerns one side of the connected member, the final weak-axis component resistance for each bolt $F_{ta,fc,Rd}$ is calculated as $F_{t,fc,Rd}$ divided by two.

In SCIA Engineer:

...::WEAK-AXIS CALCULATION::...

6. Design moment resistance Mj,z,Rd
6.1. Design resistance of basic components
6.1.1. Column flange in bending

row	Ft,fc,Rd,i	Ft,fc,Rd,g	Ft,fc,Rd	Fta,fc,Rd
1	138.51	-	138.51	69.25
2	138.51	132.58	132.58	66.29
3	138.51	176.82	138.51	69.25

Fta,fc,Rd = 204.79 kN

4.9.1.2. End plate in bending

The component resistance $F_{ta,ep,Rd}$ for each bolt-row is already calculated according to EN 1993-1-8 Art.6.2.6.5 in strong-axis moment resistance calculation as resistance $F_{t,ep,Rd}$. The final bolt-row resistances for that component are shown in the appropriate columns in the table of potential tension resistances, where the individual and group approaches are accounted for.

Since tension of weak-axis bending concerns only one side of the connected member, the final weak-axis component resistance for each bolt $F_{ta,ep,Rd}$ is calculated as $F_{t,ep,Rd}$, divided by two.

Minor modification in calculation of effective length for individual approach is done for bolt-rows classified as "Bolt-row outside of beam". Compared to the strong-axis calculation, the patterns breaching the z-axis of the connected beam were removed. Weak-axis calculation formulas for the given classification may be seen below:

Bolt-row classification	Bolt-row considered individually	
	circular	non-circular
Bolt-row outside of beam	$2\pi m_x$	$4m_x + 1{,}25e_x$
	$\pi m_x + 2e_x$	$e + 2m_x + 0{,}625e_x$

In SCIA Engineer:

6.1.2. End-plate in bending

row	leff,cp,i	leff,nc,i
1	152.95	103.69

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	103.69	103.69	111.24	✓	180.96	132.51	180.86	132.51

row	Ft,ep,Rd,i	Ft,ep,Rd,g	Ft,ep,Rd	Fta,ep,Rd,r
1	132.51	-	132.51	66.26
2	136.83	-	136.83	68.41
3	136.83	172.37	136.83	68.41

Fta,ep,Rd = 203.08 kN

4.9.1.3. Beam flange in compression

The design component resistance $F_{cb,fb,Rd}$ may be taken as the design compression resistance of one beam flange. Final resistance of the component is given as:

$$F_{t,fb,cp,Rd} = \frac{2f_y \cdot b_{c,fb,eff} \cdot t_{fb}}{\gamma_{M0}}$$

With:

- f_y yield strength of the beam
- t_{fb} thickness of the beam flange
- γ_{M0} partial safety factor for resistance of cross-sections
- $b_{c,fb,eff}$ effective width of the component given as:

$$b_{c,fb,eff} = 0.5 \cdot \left(t_{wc} + 2r_c + 7k\left(t_{fc} + t_{ep}\right)\right) = 0.5t_{wc} + r_c + 3.5k(t_{fc} + t_{ep})$$

But is not greater than:

$$b_{c,fb,eff} = \frac{b_p}{2}$$

For splice connections, this value is only calculated as:

$$b_{c,fb,eff} = \frac{b_p}{2}$$

Where:

- t_{wc} thickness of the column web
- t_{fc} thickness of the column flange
- t_{ep} thickness of the end plate
- r_c rounding r1 of the column
- b_p width of the beam
- k thickness reduction coefficient given as:

$$k = \frac{t_{fc}}{t_{fb}} \text{ and } k \leq 1$$

In SCIA Engineer:

6.1.3. Beam flange in compression

Fcb,fb,Rd data		
bc,fb,eff	70.00	mm
k	1.00	
Fcb,fb,Rd	302.68	kN

4.9.1.4. Column flange in twisting

The design resistance of the bolt-line in tension for the column flange in twisting is taken as:

$$f_{ta,fct,Rd} = \frac{0.3 \cdot b_c \cdot t_{fc}^2 \left(\frac{f_{yc}}{\sqrt{3}}\right)}{y \cdot \gamma_{M0}}$$

With:

- b_c width of the column
- t_{fc} thickness of the column flange
- f_{yc} yield strength of the column
- γ_{M0} partial safety factor for resistance of cross-sections
- y design distance from the bolt-line in tension to the center of compression for weak-axis bending

In SCIA Engineer:

6.1.4. Column flange in twisting

Fta,fct,Rd data		
bc	140.00	mm
y	63.33	mm
Fta,fct,Rd	12.96	kN

4.9.1.5. Column web in bending

For determination of the design resistance of the bolt-line in tension for the column web in bending $F_{ta,wbc,Rd}$ the effective length of the web in bending $b_{eff,wbc}$ is proposed to be calculated assuming a maximum spread of 60° from the other bolts.

$$F_{ta,wbc,Rd} = \frac{0.25 \cdot b_{eff,wbc} \cdot t_{wc}^2 \cdot f_{yc}}{y \cdot \gamma_{M0}}$$

With:

- t_{wc} web thickness of the column web
- $b_{eff,wbc}$ effective length of the web in bending given as:

$$b_{eff,wbc} = \sum p_1 + 1.73 \cdot p_2$$

With:

$\sum p_1$ vertical spacing between the first and last bolt-row
p_2 horizontal spacing between the two bolt-lines

In SCIA Engineer:

6.1.5. Column web in bending

Fta,wbc,Rd data		
p1	210.00	mm
p2	80.00	mm
y	63.33	mm
beff,wbc	348.40	mm
Fta,wbc,Rd	15.84	kN

4.9.2. Shear force resistance

The shear resistance calculation is similar to the strong-axis shear resistance calculation, however several modifications are needed in calculation of:

- The design shear force resistance of a joint $V_{y,Rd}$ for normal bolt:
  For weak-axis shear force resistance $V_{y,Rd}$ the number of bolts n_t,which are also required to resist tension, and number of bolts n_n,not required to resist tension, is set to half of the total number of all bolts each. The above applies to the case when a design bending moment $M_{z,Ed}$ is present. If the design bending moment $M_{z,Ed}$ is zero, the number of bolts n_t is set to zero and number of bolts n_n is set to number of all bolts.
- The design bearing resistance of a joint $F_{b,Rd}$
  The endplate $F_{b,ep,Rd}$ and column flange $F_{b,cf,Rd}$ component resistances are re-used. No resistance recalculation is done for weak-axis.
- The reduction of shear force resistance $V_{pl,Rd}$ of the connected beam:
  The shear force resistance $V_{y,Rd}$ of a beam the formula given by EN 1993-1-1 Article 6.2.6 (2) is used, but the final resistance is multiplied by coefficient 0,5.

The additional weak-axis shear unity check is displayed by the check and also linear interpolations for strong and weak-axis shear force components is performed and unity checks calculated if weak-axis shear force resistance $V_{y,Rd}$ is calculated:

$$\frac{V_{y,Ed}}{V_{y,Rd}} \leq 1.0$$

$$\frac{V_{z,Ed}}{V_{z,Rd}} + \frac{V_{y,Ed}}{V_{y,Rd}} \leq 1.0$$

In SCIA Engineer:

7. Design shear resistance Vy,Rd

Vy,Rd data		
Fv,Rd	60.29	kN
nb,full	3	
nb,reduced	3	
Fb,ep,Rd	102.40	kN
Fb,cf,Rd	102.40	kN
Avy	2129.61	mm²
Vy,Rd beam	144.47	kN
Vy,Rd	144.47	kN

Note: Bolt shear resistances are taken from the strong-axis calculation.

4.10. Unity checks

4.10.1. Influence of the normal force

If the axial force N_{Ed} in the connected beam exceeds 5% of the design resistance, $N_{pl,Rd}$ the following unity check is added:

$$\frac{M_{j,Ed}}{M_{j,Rd}} + \frac{N_{j,Ed}}{N_{j,Rd}} \leq 1.0$$

- $M_{j.Rd}$ design moment resistance of the joint, assuming no axial force
- $N_{j.Rd}$ axial design resistance of the joint, assuming no applied moment
- $N_{j,Ed}$ actual normal force in the connection
- $M_{j,Ed}$ actual bending moment in connection

4.10.2. General unity checks

Assume following internal forces in this connection:

N_{Ed} = 0 kN

$V_{z,Ed}$ = 10 kN

$M_{y,Ed}$ = 10 kNm

Check M: M/M_{Rd} = 10/36.65 = 0.27 < 1 => ok!

Check N: N/N_{Rd} = 0/245.40 = 0 < 1 => ok!

Check V: V/V_{Rd} = 10/154.02 = 0.06 < 1 => ok!

In SCIA Engineer:

...::RESULTS::...

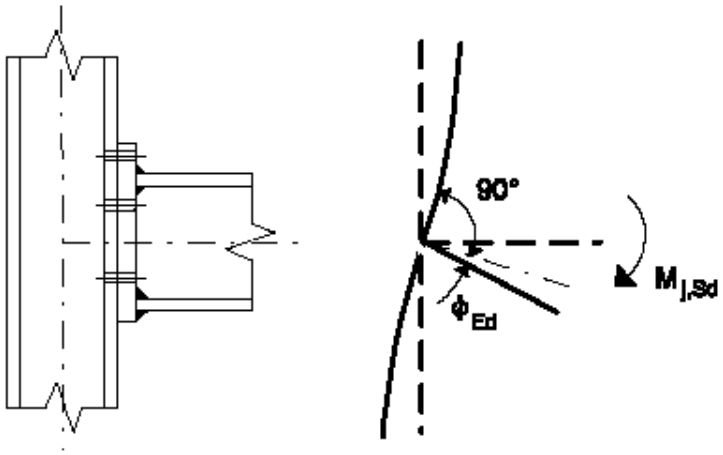
Unity checks

My,Ed/Mj,y,Rd	0.27
Mz,Ed/Mj,z,Rd	0.00
NEd/Nj,Rd	0.00
Vz,Ed/Vz,Rd	0.06
Vy,Ed/Vy,Rd	0.00
Vz,Ed/Vz,Rd + Vy,Ed/Vy,Rd	0.06
My,Ed/Mj,y,Rd + Mz,Ed/Mj,z,Rd	0.27

5. Stiffness of the connection

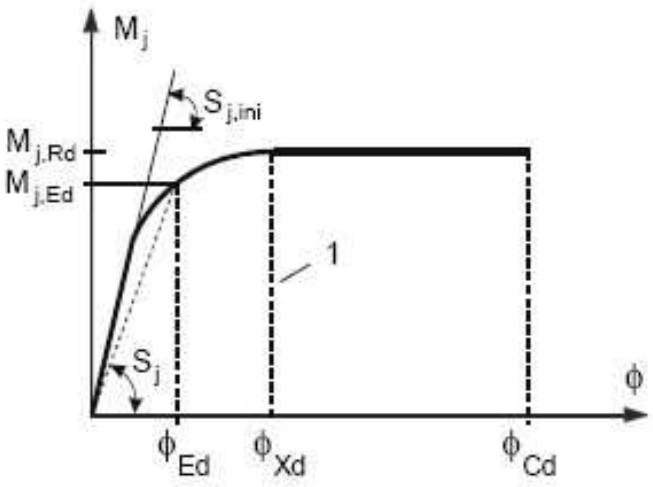
5.1. The Moment-Rotation characteristic

A joint is defined by the moment rotation characteristic that describes the relationship between the bending moment $M_{j,Sd}$ applied to a joint by the connected beam and the corresponding rotation ϕ_{Ed} between the connected members.



This moment-rotation characteristic defines three main properties:

- moment resistance $M_{j,Rd}$
- rotational stiffness S_j
- rotation capacity ϕ_{Cd}



5.2. Calculation of the stiffness

5.2.1. General formulas

In EN 1993-1-8 Table 6.11 (Ref. [1]) the stiffness coefficients for basic joint components are given:

Coefficient	Basic component	Formula	
k_1	Column web panel in shear	Unstiffened: $k_1 = \frac{0.38 \cdot A_{VC}}{\beta \cdot z}$	Stiffened: $k_1 = \infty$
k_2	Column web in compression	Unstiffened: $k_2 = \frac{0.7 \cdot b_{eff,c,wc} t_{wc}}{d_c}$	Stiffened: $k_2 = \infty$
k_3	Column web in tension	Unstiffened: $k_3 = \frac{0.7 \cdot b_{eff,t,wc} t_{wc}}{d_c}$	Stiffened: $k_3 = \infty$
k_4	Column flange in bending (for a single bolt-row in tension)	$k_4 = \frac{0.9 \cdot l_{eff} t_{fc}^3}{m^3}$	
k_5	End-plate in bending (for a single bolt-row in tension)	$k_5 = \frac{0.9 \cdot l_{eff} t_p^3}{m^3}$	
k_6	Flange cleat in bending	$k_6 = \frac{0.9 \cdot l_{eff} t_a^3}{m^3}$	
k_{10}	Bolts in tension (for a single bolt-row)	$k_{10} = 1.6 \cdot A_s / L_b$	
k_{11} (or k_{17})	Bolts in shear	$k_{11}(\text{or } k_{17}) = \frac{16 \cdot n_b \cdot d^2 \cdot f_{ub}}{E \cdot d_{M16}}$	
k_{12} (or k_{18})	Bolts in bearing (for each component j on which the bolts bear)	$k_{12}(\text{or } k_{18}) = \frac{24 \cdot n_b \cdot k_b \cdot k_t \cdot d \cdot f_{ub}}{E}$	
k_{13}	Concrete in compression (including grout)	$k_{13} = \frac{E_c \sqrt{b_{eff} l_{eff}}}{1.275\ E}$	
k_{14}	Plate in bending under compression	$k_{14} = \infty$	
k_{15}	Base plate in bending under tension (for a single bolt row in tension)	With prying forces $k_{15} = \frac{0.85 \cdot l_{eff} t_p^3}{m^3}$	Without prying forces $k_{15} = \frac{0.425 \cdot l_{eff} t_p^3}{m^3}$
k_{16}	Flange cleat in bending	With prying forces $k_{16} = 1.6\ A_s / L_b$	Without prying forces $k_{16} = 2.0\ A_s / L_b$

With

- A_{vc} shear area of the column
- z lever arm
- β transformation parameter
- b_{eff} effective width of the column web
- d_c clear depth of the column web
- l_{eff} smallest effective length for the bolt
- m distance bolt to beam/column web
- A_s tensile stress area of the bolt
- L_b elongation length of the bolt

5.2.2. Calculation of the stiffness in detail

In Table 6.10 of the EN 1993-1-8 (Ref.[1]) the stiffness coefficients which has to be taken into account, are given.

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.

5.2.2.1. Column web in tension: k_3

$$k_3 = \frac{0.7 \cdot b_{eff,t,wc}\, t_{wc}}{d_c}$$

⇨ $b_{eff,t,wc}$ 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.

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	224.51	142.55
3	-	-	224.51	142.55	-	-

⇨ $b_{eff,t,wc,row1} = 107.55$

⇨ $b_{eff,t,wc,row2} = 105$

⇨ $b_{eff,t,wc,row3} = 142.55$

$$\mathbf{k_{3,row1} = \frac{0.7 \cdot 107.55 \cdot 7}{92} = 5.73\ mm}$$

$$\mathbf{k_{3,row2} = \frac{0.7 \cdot 105 \cdot 7}{92} = 5.59\ mm}$$

$$\mathbf{k_{3,row3} = \frac{0.7 \cdot 142{,}55 \cdot 7}{92} = 7.59\ mm}$$

In SCIA Engineer:

4.1. Design rotational stiffness

According to EN 1993-1-8 Article 6.3

row	k3[mm]	k4[mm]	k5[mm]	k10[mm]	keff[mm]
1	5.73	8.59	7.55	6.47	1.73
2	5.59	8.39	7.26	6.47	1.69
3	7.59	11.39	7.26	6.47	1.95

5.2.2.2. Column flange in bending: k_4

$$k_4 = \frac{0.9 \cdot l_{eff} t_p^3}{m^3}$$

⇨ l_{eff} is the smallest of the effective lengths given for this bolt-row given in Table 6.4 or Table 6.5.

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	224.51	142.55
3	-	-	224.51	142.55	-	-

⇨ $l_{eff,t,wc,row1} = 107.55$

⇨ $l_{eff,t,wc,row2} = 105$

⇨ $l_{eff,t,wc,row3} = 142.55$

$$\mathbf{k_{4,row1} = \frac{0.9 \cdot 107.55 \cdot 12^3}{26.9^3} = 8.59\ mm}$$

$$\mathbf{k_{4,row2} = \frac{0.9 \cdot 105 \cdot 12^3}{26.9^3} = 8.39\ mm}$$

$$\mathbf{k_{4,row3} = \frac{0.9 \cdot 142.55 \cdot 12^3}{26.9^3} = 11.39\ mm}$$

In SCIA Engineer:

4.1. Design rotational stiffness

According to EN 1993-1-8 Article 6.3

row	k3[mm]	k4[mm]	k5[mm]	k10[mm]	keff[mm]
1	5.73	8.59	7.55	6.47	1.73
2	5.59	8.39	7.26	6.47	1.69
3	7.59	11.39	7.26	6.47	1.95

5.2.2.3. End-plate in bending: k_5

$$k_5 = \frac{0.9 \cdot l_{eff} t_p^3}{m^3}$$

⇨ l_{eff} is the smallest of the effective lengths given for this bolt-row given in Table 6.6.

row	p (p1+p2)	e	ex	m	mx	n	λ1U	λ2U	αU	λ1L	λ2L	αL
1	0.00+0.00	30.00	40.00	-	24.34	30.00	-	-	-	-	-	-
2	0.00+70.00	30.00	-	33.66	-	30.00	0.53	0.39	5.77	-	-	-
3	70.00+0.00	30.00	-	33.66	-	30.00	-	-	-	0.53	0.39	5.77

row	leff,cp,i	leff,nc,i
1	152.95	70.00
2	211.47	194.10
3	211.47	194.10

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	-	-	-	-	-	-
2	-	-	-	-	245.73	178.03
3	-	-	245.73	178.03	-	-

⇨ $l_{eff,\ row1} = 70$

⇨ $l_{eff,\ row2} = 178.03$

⇨ $l_{eff,\ row3} = 178.03$

$$\mathbf{k_{5,row\ 1}} = \frac{0.9 \cdot 70 \cdot 12^3}{(24.34)^3} = \mathbf{7.55\ mm}$$

$$\mathbf{k_{5,row\ 2}} = \frac{0.9 \cdot 178.09 \cdot 12^3}{(33.66)^3} = \mathbf{7.26\ mm}$$

$$\mathbf{k_{5,row\ 3}} = \frac{0.9 \cdot 178.09 \cdot 12^3}{(33.66)^3} = \mathbf{7.26\ mm}$$

In SCIA Engineer:

4.1. Design rotational stiffness
According to EN 1993-1-8 Article 6.3

row	k3[mm]	k4[mm]	k5[mm]	k10[mm]	keff[mm]
1	5.73	8.59	7.55	6.47	1.73
2	5.59	8.39	7.26	6.47	1.69
3	7.59	11.39	7.26	6.47	1.95

5.2.2.4. Bolts in tension: k_{10}

$$k_{10} = 1.6 \cdot \frac{A_s}{L_b}$$

- ⇨ A_s is the tensile stress area of the bolt A_s = 157 mm²
- ⇨ 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.
  $L_b = t_f + t_p + t_{washer} + (h_{bolt_head} + h_{nut})/2 = 12 + 12 + 3.3 + (10 + 13)/2 = 38.8$ mm

$$\mathbf{k_{10}} = 1.6 \cdot \frac{157}{38.8} = \mathbf{6.47\ mm}$$

In SCIA Engineer:

4.1. Design rotational stiffness
According to EN 1993-1-8 Article 6.3

row	k3[mm]	k4[mm]	k5[mm]	k10[mm]	keff[mm]
1	5.73	8.59	7.55	6.47	1.73
2	5.59	8.39	7.26	6.47	1.69
3	7.59	11.39	7.26	6.47	1.95

5.2.2.5. Equivalent stiffness

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

$$k_{eff,r} = 1/\sum_i \left(\frac{1}{k_{i,r}}\right)$$ (see also formula (6.30) of EN 1993-1-8 – Ref.[1])

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 for k_3, k_4, k_5 and k_{10}.

- $k_{eff,row1} = \frac{1}{\frac{1}{5.73}+\frac{1}{8.59}+\frac{1}{7.55}+\frac{1}{6.47}} = 1.73$
- $k_{eff,row2} = \frac{1}{\frac{1}{5.59}+\frac{1}{8.39}+\frac{1}{7.26}+\frac{1}{6.47}} = 1.69$
- $k_{eff,row3} = \frac{1}{\frac{1}{7.59}+\frac{1}{11.39}+\frac{1}{7.26}+\frac{1}{6.47}} = 1.95$

In SCIA Engineer:

4.1. Design rotational stiffness
According to EN 1993-1-8 Article 6.3

row	k3[mm]	k4[mm]	k5[mm]	k10[mm]	keff[mm]
1	5.73	8.59	7.55	6.47	1.73
2	5.59	8.39	7.26	6.47	1.69
3	7.59	11.39	7.26	6.47	1.95

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

$$z_{eq} = \frac{\sum_r k_{eff,r} h_r^2}{\sum_r k_{eff,r} h_r} = \frac{k_{eff,row1} h_{row1}^2 + k_{eff,row2} h_{row2}^2 + k_{eff,row3} h_{row3}^2}{k_{eff,row1} h_{row1} + k_{eff,row2} h_{row2} + k_{eff,row3} h_{row3}}$$

$$z = \frac{1.73 \cdot (245.4)^2 + 1.69 \cdot (175.4)^2 + 1.95 \cdot (35.4)^2}{1.73 \cdot 245.4 + 1.69 \cdot 175.4 + 1.95 \cdot 35.4} = \frac{158619.4}{790} = 200.73 \text{ mm}$$

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 (Ref.[1]))

$$k_{eq} = \frac{1.73 \cdot 245.4 + 1.69 \cdot 175.4 + 1.95 \cdot 35.4}{200.73} = 3.94 \text{ mm}$$

And those values are also given in SCIA Engineer:

Sj data		
Sj	10.97	MNm/rad
Sj,ini	10.97	MNm/rad
z	200.73	mm
mu	1.00	
k1	2.48	mm
k2	8.70	mm
keq	3.94	mm

5.2.2.6. Column web panel in shear: k_1

$$k_1 = \frac{0.38 \cdot A_{vc}}{\beta \cdot z}$$

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.

⇨ $z = z_{eq} = 200.73$ mm

β is the transformation parameter from 5.3 (7)

⇨ $\beta = 1$

$$\mathbf{k_1 = \frac{0.38 \cdot 1312}{1 \cdot 200.73} = 2.48 \text{ mm}}$$

In SCIA Engineer:

Sj data		
Sj	10.97	MNm/rad
Sj,ini	10.97	MNm/rad
z	200.73	mm
mu	1.00	
k1	2.48	mm
k2	8.70	mm
keq	3.94	mm

5.2.2.7. Column web in compression: k_2

$$k_2 = \frac{0.7 \cdot b_{eff,c,wc}\, t_{wc}}{d_c}$$

⇨ d = h_c -2 (t_f + r_c) = 140 – 2 (12 + 12) = 92 mm

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

$$s_p = 12 + \left(15 - \sqrt{2} \cdot 5\right) = 19.93$$

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.

⇨ $b_{eff} = 9.2 + 2\sqrt{2} \cdot 5 + 5(12 + 12) + 19.93 = 163.27$ mm

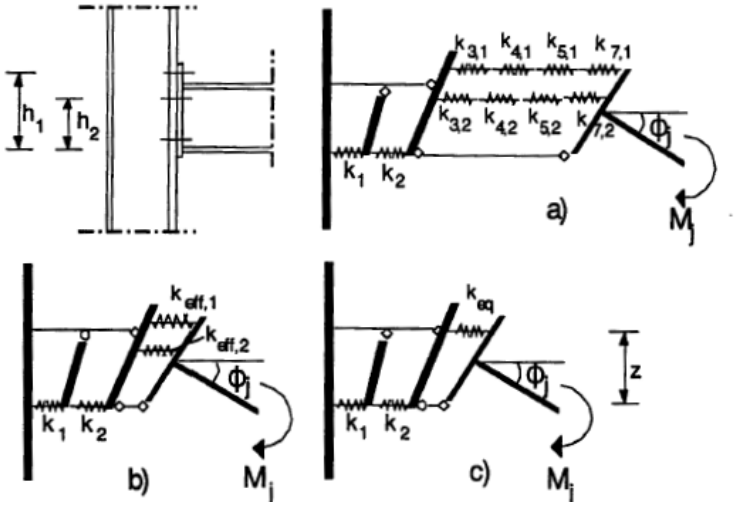
$$\mathbf{k_2 = \frac{0.7 \cdot 163.3 \cdot 7}{92} = 8.70\ mm}$$

In SCIA Engineer:

Sj data		
Sj	10.97	MNm/rad
Sj,ini	10.97	MNm/rad
z	200.73	mm
mu	1.00	
k1	2.48	mm
k2	8.70	mm
keq	3.94	mm

5.2.2.8. Design rotational stiffness

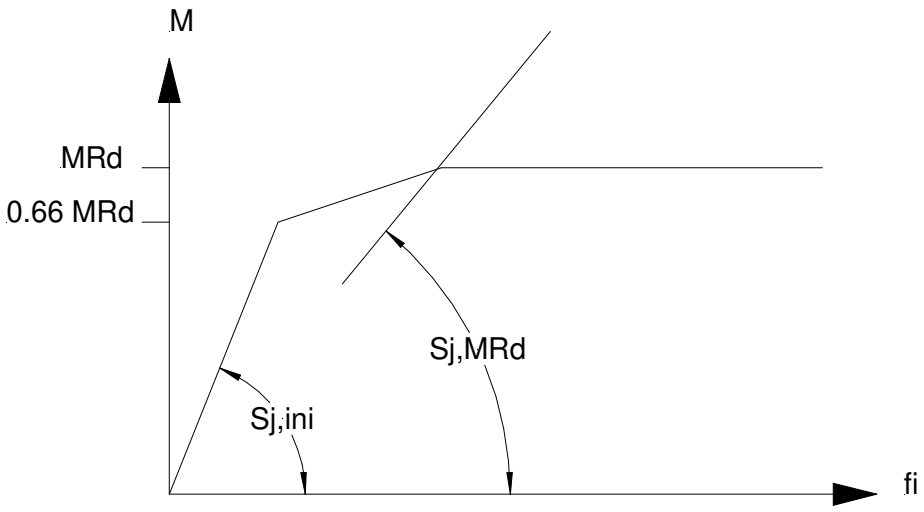
When all different stiffness of all components are known, we can assembly this to one stiffness for the joint.



The program will calculate 3 stiffnesses :

- $S_{j,ini}$ initial rotational stiffness
- S_j rotational stiffness, related to the actual moment $M_{j,Sd}$
- $S_{j,MRd}$ rotational stiffness, related to $M_{j,Rd}$ (without the influence of the normal force)

The moment-rotation diagram is based on the values of $S_{j,ini}$ and $S_{j,MRd}$.



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

- ⇨ z = 200.73 mm
- ⇨ μ is the stiffness ration $S_{j,ini}$ / S_j
  - If $M_{j,Ed} \leq M_{j,Rd}$ => $\mu = 1$
  - If $2/3\ M_{j,Rd} < M_{j,Ed} \leq M_{j,Rd}$ => $\mu = (1.5\ M_{j,Ed} / M_{j,Rd})^{\Psi}$

  $M_{j,Ed}$ = 10 kNm

  $M_{j,Rd}$ = 36.65 kNm => 2/3 $M_{j,Rd}$ = 24.43 kNm

- ⇨ μ = 1
- ⇨ $S_j = \frac{E \cdot z^2}{\mu \cdot \sum_i \frac{1}{k_i}} = \frac{210000 \cdot (200.73)^2}{1 \cdot \left(\frac{1}{2.48} + \frac{1}{8.70} + \frac{1}{3.94}\right)} \cdot 10^{-6} = 10966\ \text{kNm/rad}$

In SCIA Engineer:

Sj data		
Sj	10.97	MNm/rad
Sj,ini	10.97	MNm/rad
z	200.73	mm
mu	1.00	
k1	2.48	mm
k2	8.70	mm
keq	3.94	mm

5.3. The classification on stiffness

The joint is classified as rigid, pinned or semi-rigid according to its stiffness by using the initial rotational stiffness $S_{j,ini}$ and comparing this with classification boundaries given in EN 1993-1-8 (Ref. [1]).

- If $S_{j,ini} >= S_{j,rigid}$, the joint is rigid.
- If $S_{j,ini} <= S_{j,pinned}$, the joint is classified as pinned.
- If $S_{j,ini} < S_{j,rigid}$ and $S_{j,ini} > S_{j,pinned}$, the joint is classified as semi-rigid.

For braced frames:

$$S_{j,rigid} = 8 \cdot \frac{E \cdot I_b}{L_b}$$

$$S_{j,pinned} = 0.5 \cdot \frac{E \cdot I_b}{L_b}$$

For unbraced frames:

$$S_{j,rigid} = 25 \cdot \frac{E \cdot I_b}{L_b}$$

$$S_{j,pinned} = 0.5 \cdot \frac{E \cdot I_b}{L_b}$$

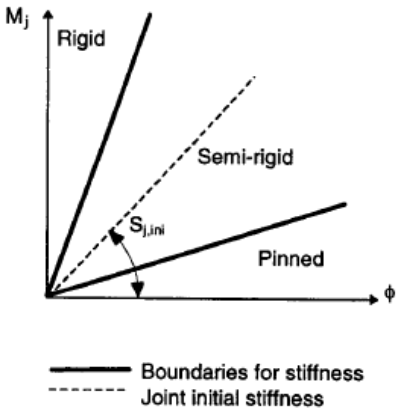
For column base joints:

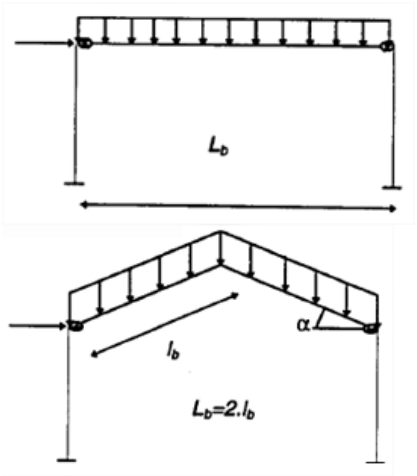
$$S_{j,rigid} = 15 \cdot \frac{E \cdot I_c}{L_c}$$

$$S_{j,pinned} = 0.5 \cdot \frac{E \cdot I_c}{L_c}$$

With:

- I_b second moment of area of the beam
- L_b span of the beam
- I_c second moment of area of the column
- L_c storey height of the column
- E Young modulus





In our example we have chosen for a braced frame.

SCIA Engineer will take the length of the beam in SCIA Engineer as the length for L_b. But you can change this value manually:



$$S_{j,rigid} = 8 \cdot \frac{E \cdot I_b}{L_b} = 8 \cdot \frac{\left(210000 \frac{N}{mm^2}\right) \cdot (2.772 \cdot 10^7\ mm^4)}{2000\ mm} = 23.28\ MNm/rad$$

$$S_{j,pinned} = 0.5 \cdot \frac{E \cdot I_b}{L_b} = 0.5 \cdot \frac{\left(210000 \frac{N}{mm^2}\right) \cdot (2.772 \cdot 10^7\ mm^4)}{2000\ mm} = 1.46\ MNm/rad$$

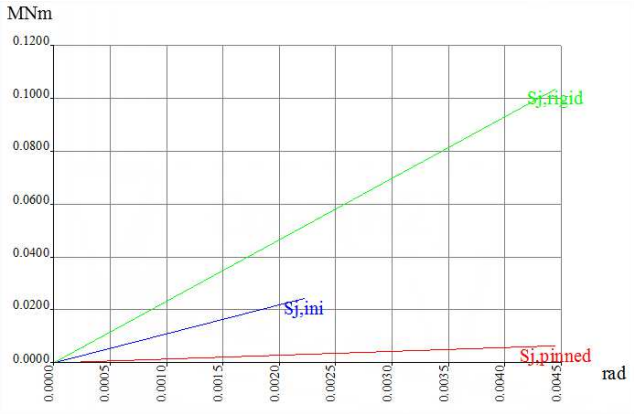
In SCIA Engineer:

4.2. Stiffness classification

Stiffness data		
E	210000.00	N/mm²
Ib	27720000.00	mm⁴
Lb	2000.00	mm
frame type	braced	
Sj,rigid	23.28	MNm/rad
Sj,pinned	1.46	MNm/rad

System SEMI RIGID

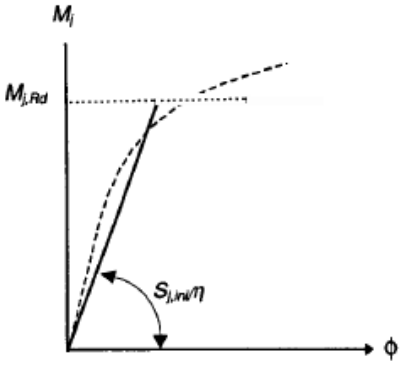
And this is also given in SCIA Engineer in a picture:



5.4. Transferring the joint stiffness to the analysis model

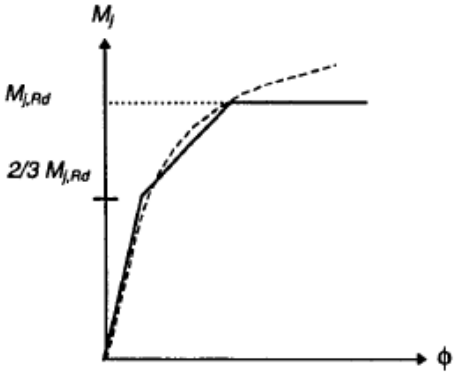
When requested, the actual stiffness of the joint can be transferred to the analysis model. The linear spring value for **<fi y>** (in the hinge dialog) is taken as $S_{j,ini}$ divided by the stiffness modification coefficient η.

For asymmetric joint s which are loaded in both directions (i.e. tension on top and tension in bottom), the linear spring value for **<fi y>** (in the hinge dialog) is taken as the smallest $S_{j,ini}$ (from both directions) divided by the stiffness modification coefficient η:



Type of joint	η
bolted beam-to-column	2
welded beam-to-column	2
welded plate-to-plate	3
column base	3

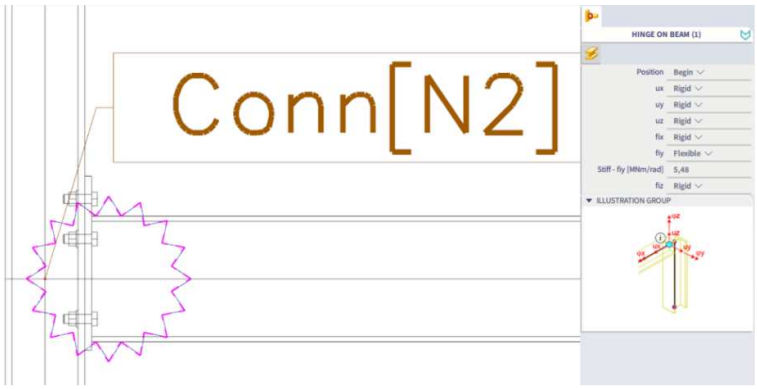
At the same time, a non-linear function is generated, representing the moment-rotation diagram.



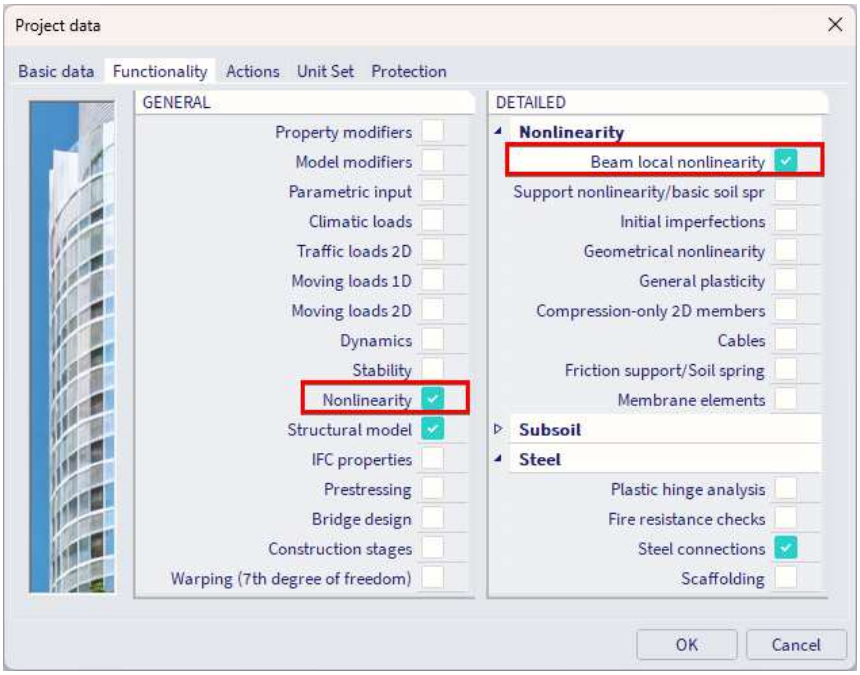
You can add this stiffness to the connection by activating the option “Update Stiffness”:



After recalculating the project, a hinge will be added to this connection with this stiffness $S_{j,ini}/\eta$.

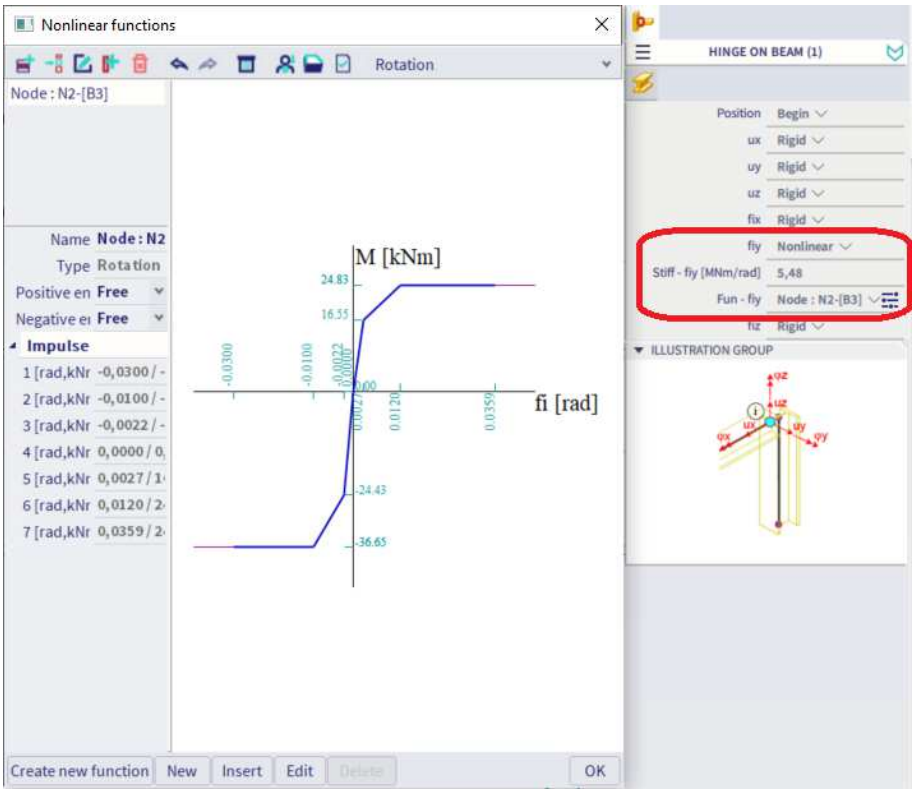


If you want to take into account the nonlinear stiffness of the connection, you have to activate the following functionality in the Project settings dialog:



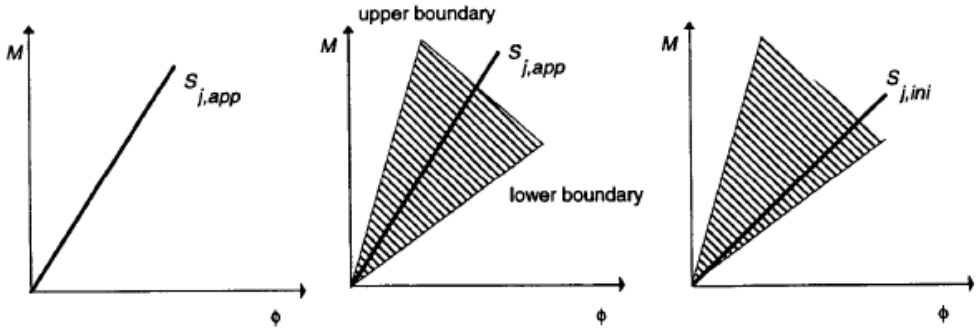
After the calculation, you can select the input hinge and choose here for a nonlinear stiffness.

The stiffness function will be registered automatically for each node in SCIA Engineer. So you only have to choose the corresponding node for each connection.



5.5. The required stiffness

The actual stiffness of the joints is compared with the required stiffness, based on the approximate joint stiffness used in the analysis model. A lower boundary and an upper boundary define the required stiffness.



When a linear spring is used in the analysis model, we check the following:

When $S_{j,ini} >= S_{j,low}$ and $S_{j,ini} <= S_{j,upper}$, the actual joint stiffness is conform with the applied $S_{j,app}$ in the analysis model.

When a nonlinear function is used during the analysis model, we check the following:

When $S_j >= S_{j,low}$ and $S_j <= S_{j,upper}$, the actual joint stiffness is conform with the applied $S_{j,app}$ in the analysis model.

The boundaries are calculated with 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	$\frac{24 \cdot Sj,app \cdot E \cdot I_b}{30 \cdot E \cdot I_b + Sj,app \cdot L_b}$	$Sj,app \le \frac{24 \cdot E \cdot I_b}{L_b}$	$\frac{30 \cdot Sj,app \cdot E \cdot I_b}{24 \cdot E \cdot I_b - Sj,app \cdot L_b}$
		$Sj,app > \frac{24 \cdot E \cdot I_b}{L_b}$	∞

And for a column base connection:

Lower boundary		Upper boundary
$\frac{16 \cdot Sj,app \cdot E \cdot I_c}{20 \cdot E \cdot I_c + Sj,app \cdot L_c}$	$Sj,app \le \frac{16 \cdot E \cdot I_c}{L_c}$	$\frac{20 \cdot Sj,app \cdot E \cdot I_c}{16 \cdot E \cdot I_c - Sj,app \cdot L_c}$
	$Sj,app > \frac{16 \cdot E \cdot I_c}{L_c}$	∞

With

- I_b second moment of area of the beam
- L_b span of the beam
- I_c second moment of area of the column
- L_c storey height of the column
- E Young modulus
- $S_{j,app}$ approximate joint stiffness
- $S_{j,ini}$ actual initial joint stiffness
- $S_{j,low}$ lower boundary stiffness
- $S_{j,upper}$ upper boundary stiffness
- S_j actual joint stiffness

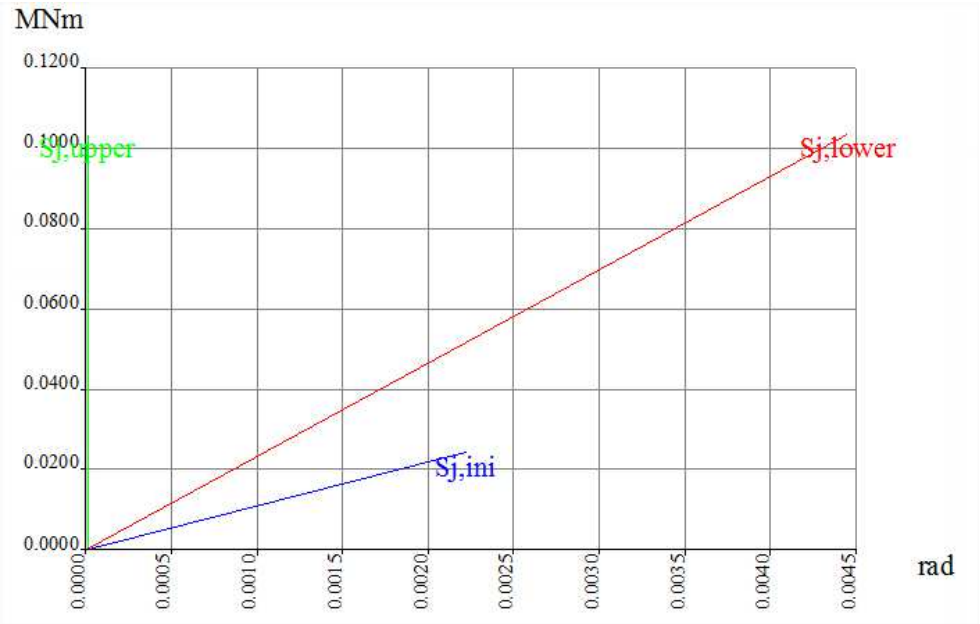
In this case we have a braced system and we did not take into account any stiffness, so the upper boundary equals infinity and the lower boundary is the boundary for a rigid connection:

4.3. Stiffness check

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 also in the graph you can see that $S_{j,ini}$ is not between the boundaries:



When activating the option "Update stiffness" and recalculating the project, the value for $S_{j,app}$ equals $S_{j,ini}$. The stiffness taken into account in the calculation equals $S_{j,ini}/2$ because η = 2 for a beam-column connection.

In SCIA Engineer we have:

fi y	The stiffness taken into account in the calculation, thus: $S_{j,ini}/\eta = S_{j,ini}/2$ = 10.95 MNm/rad /2 = 5.475 MNm/rad
Stiffness modification coef.	Factor η and η = 2 for a beam-column connection
$S_{j,app}$	In this case $S_{j,ini}$ = 10.97 MNm
$S_{j,lower}$ boundary	For a braced system: $= \frac{8 \cdot S_{j,app} \cdot E \cdot I_b}{10 \cdot E \cdot I_b + S_{j,app} \cdot L_b}$ $= \frac{8 \cdot \frac{10.95\text{MNm}}{\text{rad}} \cdot 210000\text{MPa} \cdot 2.77 \cdot 10^{-5}\ \text{m}^4}{10 \cdot 210000\text{MPa} \cdot 2.77 \cdot 10^{-5}\ \text{m}^4 + 10.95\frac{\text{MNm}}{\text{rad}} \cdot 2\text{m}} = 6.37\ \text{MNm/rad}$

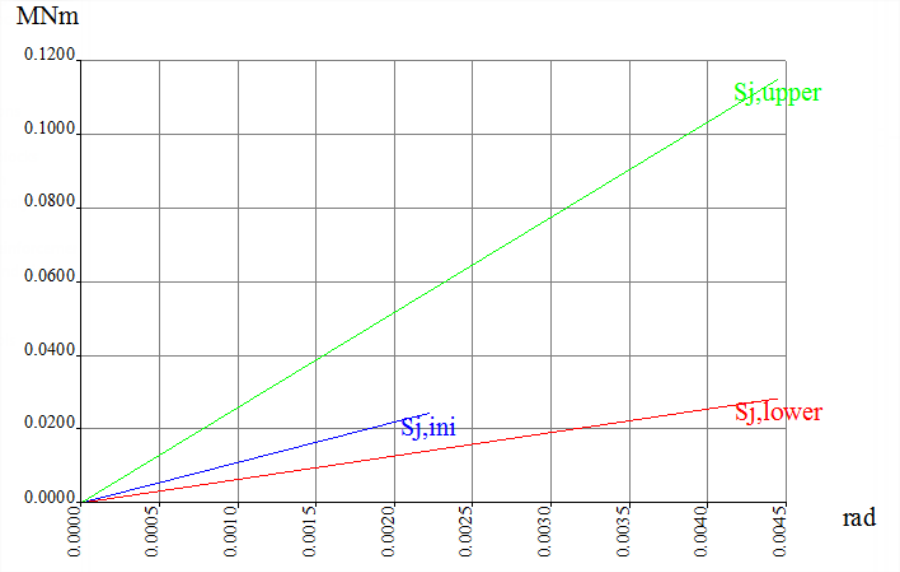
$S_{j,upper}$ boundary	For a braced system: First we have to check if $S_{j,app}$ is bigger or smaller than $\frac{8 \cdot E \cdot I_b}{L_b} = \frac{8 \cdot 210000\ \text{MPa} \cdot 2.77 \cdot 10^{-5}\ \text{m}^4}{2\ \text{m}} = 23.3\ \text{MPa}$ Thus $S_{j,app} = 10.95 \leq \frac{8 \cdot E \cdot I_b}{L_b}$ And now the upper boundary can be calculated with the following formula: $= \frac{10 \cdot S_{j,app} \cdot E \cdot I_b}{8 \cdot E \cdot I_b - S_{j,app} \cdot L_b}$ $= \frac{10 \cdot \frac{10.95\text{MNm}}{\text{rad}} \cdot 210000\text{MPa} \cdot 2.77 \cdot 10^{-5}\ \text{m}^4}{(8 \cdot 210000\text{MPa} \cdot 2.77 \cdot 10^{-5}) - \left(10.95 \frac{\text{MNm}}{\text{rad}} \cdot 2\text{m}\right)}$ = 25.92 MNm/rad

4.3. Stiffness check

Stiffness data		
Fi y	5.48	MNm/rad
Stiffness modification coef.	2.00	
Sj,app	10.97	MNm/rad
Sj,lower boundary	6.37	MNm/rad
Sj,upper boundary	25.92	MNm/rad

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

And now $S_{j,ini}$ will be in between the two boundaries on the graph also:



6. Calculation of welds

6.1. Calculation of a_f

Within SCIA Engineer there is a possibility to choose from three methods for determination of the flange weld sizes by modifying the **Weld size determination** parameter in the connection setup.

Minimum for full strength

The default calculation of flange weld size is based on the Ref. [35] - ECCS N° 126. The final formula is derived as:

$$a \geq \frac{f_y \cdot \beta_w \cdot \gamma_{M2} \cdot t}{\sqrt{2} \cdot f_u \cdot \gamma_{M0}} = \frac{235 \cdot 0.8 \cdot 1.25 \cdot 9.2}{\sqrt{2} \cdot 360 \cdot 1.00} = 4.25 \text{ mm}$$

With:

- f_y yield strength of the weaker part
- f_u ultimate tensile strength of the weaker part
- β_w correlation factor
- γ_{M0} partial safety factor for material
- γ_{M2} partial safety factor for welds
- t thickness of the beam flange

In SCIA Engineer:

5.1. Flange welds

data		
Mj,y,Rd	36.65	kNm
α	1.40	
h	210.80	mm
FRd	243.39	kN
Nt,Rd	237.82	kN
fu	360.00	N/mm²
β_w	0.80	
minimum af	4.25	mm
af	5.00	mm

Calculated from connection resistance

The weld size a_f is designed according to the resistance of the joint. The design force in the beam flange can be estimated as:

$$\gamma \cdot F_{Rd} = \frac{M_{Rd} \cdot \gamma}{h} = \frac{35.75 \cdot 1.4}{0.220 - 0.0092} = \frac{50.05}{0.2108} = 237.43 \text{ kN}$$

With:

- F_{Rd} design force in the beam flange
- M_{Rd} design moment resistance of the connection
- h lever arm of the connection
- γ factor: 1.7 for unbraced frames or 1.4 for braced frames

However, in no case shall the weld design resistance be required to exceed the design plastic resistance of the beam flange $N_{t.Rd}$:

$$N_{t,Rd} = \frac{b_f \cdot t_{fb} \cdot f_{yb}}{\gamma_{M0}} = \frac{110 \cdot 9.2 \cdot 235}{1.00} = 237820 \text{ N} = 237.82 \text{ kN}$$

With:

- b_f beam flange width
- t_{fb} beam flange thickness
- f_{yb} yield strength of the beam

So, we have
F_w = min ($N_{t.Rd}$, γ*F_{Rd}) = min (237.82 kN; 237.43 kN) = 237.43 kN
The weld size design for a_f:

$$a_f \geq \frac{F_w \cdot \gamma_{M2} \cdot \beta_w}{f_u \cdot b_f \cdot \sqrt{2}} = \frac{237430 \cdot 1.25 \cdot 0.8}{360 \cdot 110 \cdot \sqrt{2}} = 4.24 \text{ mm}$$

With:

- F_w design resistance of the weld
- b_f beam flange width
- f_u ultimate tensile strength of the weaker part
- β_w correlation factor
- γ_{M2} partial safety factor for welds

In SCIA Engineer:

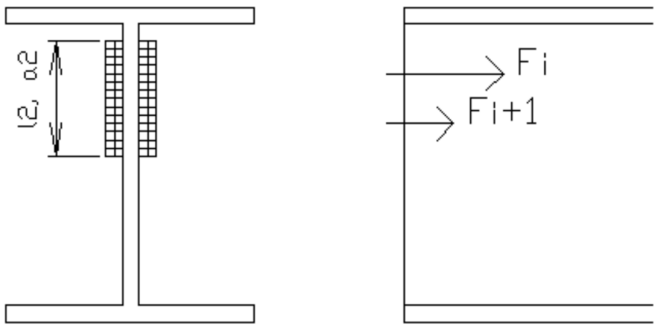
5.1. Flange welds

data		
Mj,y,Rd	35.75	kNm
α	1.40	
h	210.80	mm
FRd	237.43	kN
Nt,Rd	237.82	kN
fu	360.00	N/mm²
βw	0.80	
minimum af	4.24	mm
af	5.00	mm

Calculated using Internal forces

The weld size **a_f** is then designed similarly as in the previous, with the only difference that in the **F_{Rd}** does not take into account the connection moment resistance **$M_{j,Rd}$**, but direct design moment **M_{Ed}**.

6.2. Calculation of a_w



For all possible bolt groups, the maximum tension pro unit length is calculated.
The tension pro unit length is (F_i + F_{i+1})/l_2.
l2 is taken as the effective length of non-circular pattern for the considered bolt group.

On the weld 2 x l_2 x a_2, the normal force N (=F_i + F_{i+1}) and the shear force D is acting. The 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.
To determine the weld size a_2 in a connection, we use a iterative process with a_2 as parameter until the Von Mises rules is respected:

$$\sqrt{\sigma_1^2 + 3 \cdot \left(\tau_1^2 + \tau_2^2\right)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M_w}} \quad \text{and} \quad \sigma_1 \leq \frac{f_u}{\gamma_{M_w}}$$

$$\sigma_1 = \tau_2 = \left(\frac{N}{A}\right)\frac{1}{\sqrt{2}}$$

$$\tau_1 = \frac{D}{2 \cdot a_2 \cdot l_2}$$

With:

- f_u ultimate tensile strength of the weaker part
- β_w correlation factor
- γ_{Mw} partial safety factor for welds
- A 2 a_2 l_2

In SCIA Engineer:

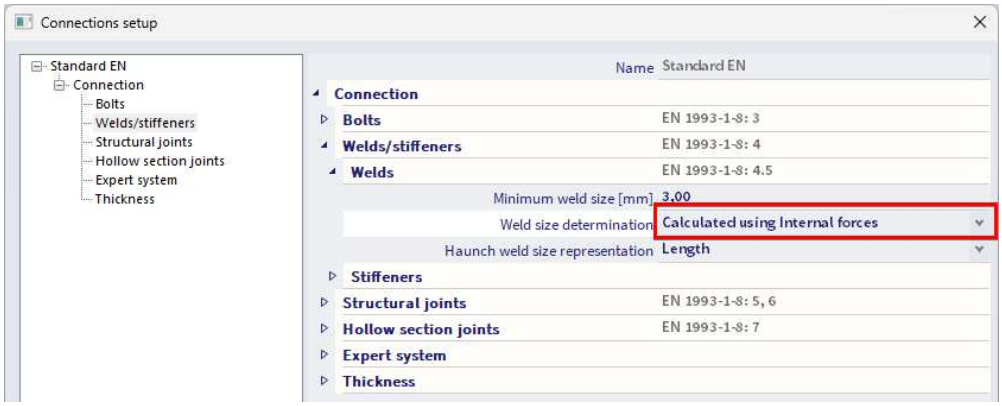
5.2. Web welds

data		
M	10.00	kNm
N	0.00	kN
V	10.00	kN
fu	360.00	N/mm²
β_w	0.80	
a1	3.00	mm
a3	3.00	mm
l1	110.00	mm
l2	168.40	mm
l3	40.05	mm
A	1477.40	mm²
I	13665133.83	mm⁴
minimum aw (a2)	1.00	mm
aw	3.00	mm

6.3. Calculation with the internal forces

In the previous chapters the calculation of a_f and a_w are given, using the design resistance values. In SCIA Engineer it is also possible to calculate a_f and a_w using the internal forces of the chosen combination or load case. This will result in a lower value for the welds than with the previous calculation, since the internal forces are lower than the design forces, if the connection is satisfying all checks.

You can activate this in SCIA Engineer via “Main menu > Design > Steel connections > Settings” and here with the option “Weld size determination = Calculated using internal forces”.



7. Ductility class

7.1. Ductility classes

The following classification is valid for joints:

- Class 1 joint: $M_{j,Rd}$ is reached by full plastic redistribution of the internal forces within the joints and a sufficiently good rotation capacity is available to allow a plastic frame analysis and design.
- Class 2 joint: $M_{j,Rd}$ is reached by full plastic redistribution of the internal forces within the joints but the rotational capacity is limited. An elastic frame analysis possibly combined with a plastic verification of the joints has to be performed. A plastic frame analysis is also allowed as long as it does not result in a too high required rotation capacity of the joints where the plastic hinges are likely to occur.
- Class 3 joint: brittle failure (or instability) limits the moment resistance and does not allow a full redistribution of the internal forces in the joints. It is compulsory to perform an elastic verification of the joints unless it is shown that no hinge occurs in the joint locations.

From this description it is clear that it is better to modell a joint as a ductile joint. In this case, when failure appears, the load can be transferred to other parts of the joint and you can see that it is going to brake slowly: you can see that the column web is yieling for example. If you have a brittle failure mode (non-ductile) the connection will brake immediately when reaching the failure mode.

7.2. Ductility classification for bolted joints

If the failure mode of the joint is the situated in the shear zone of the column web, the joint is classified as a ductile, i.e. a class 1 joint .

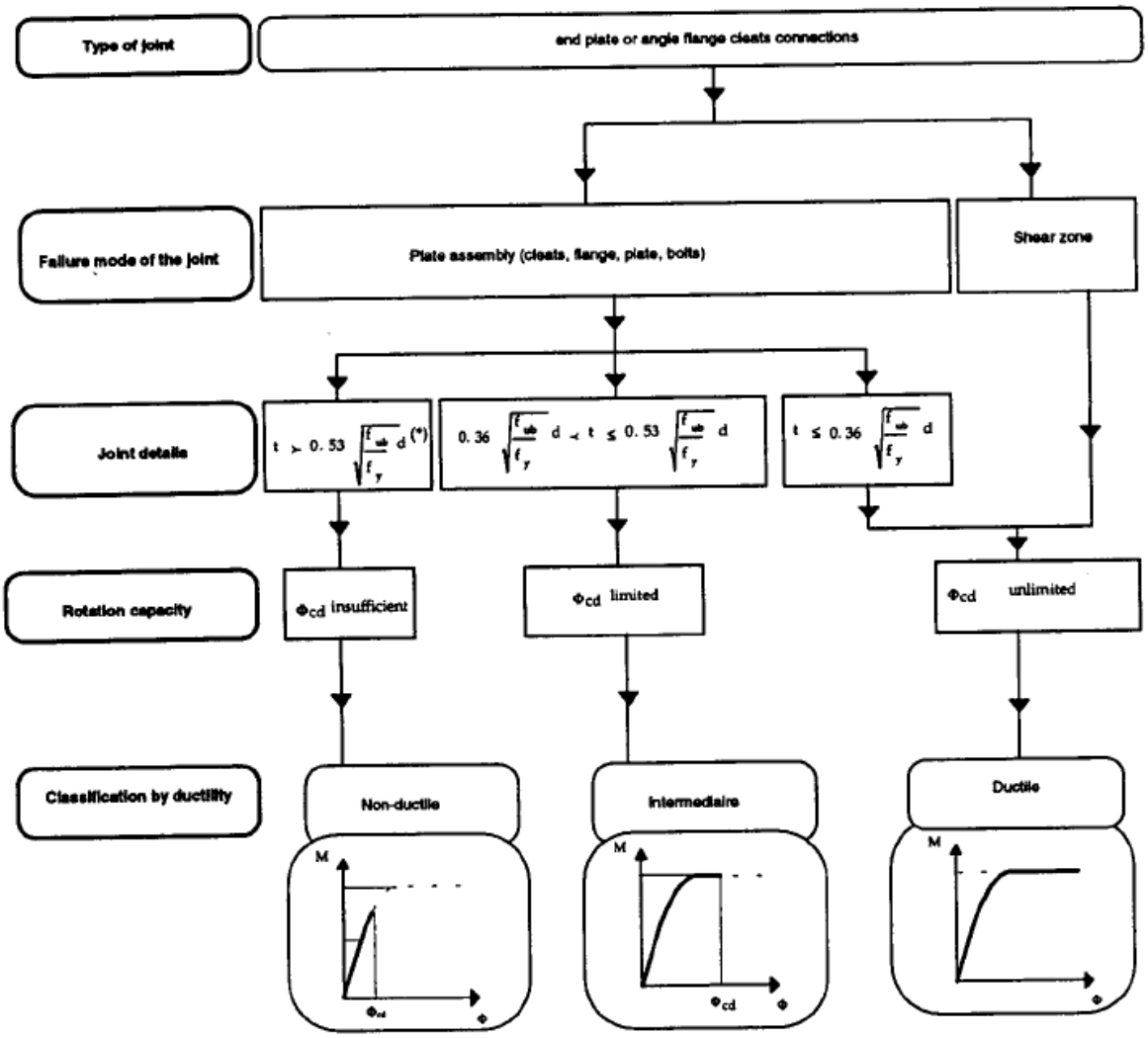
If the failure mode is not in the shear zone, the classification is based on the following:

	Classification by ductility	Class
$t \le 0.36\sqrt{\frac{f_{ub}}{f_y}d}$	Ductile	1
$0.36\sqrt{\frac{f_{ub}}{f_y}d} < t \le 0.53\sqrt{\frac{f_{ub}}{f_y}d}$	Intermediary	2
$t > 0.53\sqrt{\frac{f_{ub}}{f_y}d}$	Non-ductile	3

With:

- t thickness of either the column flange or the endplate
- d nominal diameter of the bolts
- f_{ub} ultimate tensile strength of the bolt
- f_y yield strength of the proper basic component

This principle is also shown in the graph below:



Calculation of this example following the schema above:

Type of joint: We have an end plate, so we can follow the schema

Failure mode of the joint: We know the tension in the bolts is limited by the column web in shear (see also chapter "Calculation of M_{Rd}"). So the failure mode is in the shear zone. This will lead directly to a ductile joint.

And this is also shown in SCIA Engineer:

4.4. Ductility classification

The failure mode is situated in the column shear zone.
This results in a ductile classification for ductility : class 1.

7.3. Ductility classification for welded joints

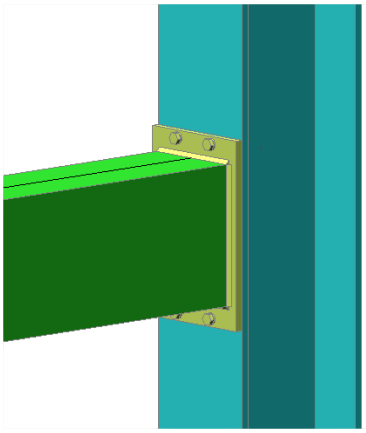
If the failure mode of the joint is the situated in the shear zone of the column web, the joint is classified as a ductile, i.e. a class 1 joint. If the failure mode is not in the shear zone, the joint is classified as intermediary for ductility, i.e. a class 2 joint.

8. Extra options in SCIA Engineer

8.1. RHS beam

In SCIA Engineer it is possible to use an RHS beam and make a between this beam and a I or H column. For more info about this topic, we refer to Ref.[2].

A connection with an RHS beam can be found in **example CON_005.esa, node N7**.

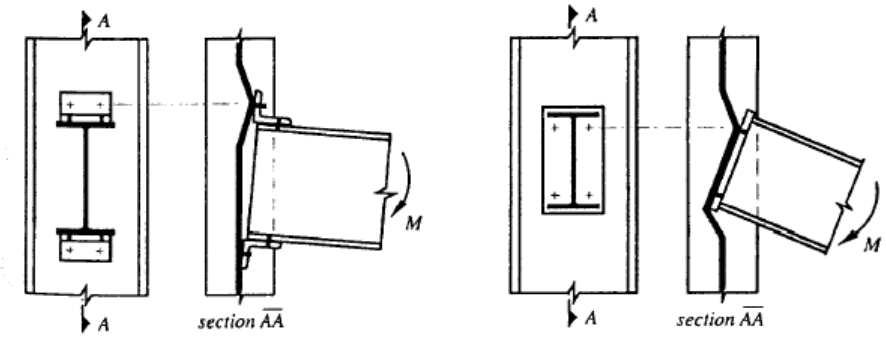


8.2. Column in minor axis configuration

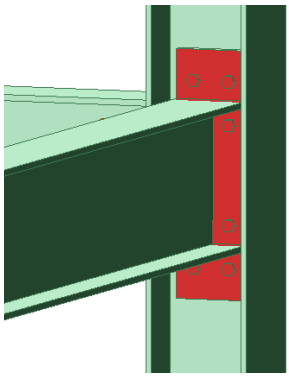
In beam-to-column minor-axis joints, the beam is directly connected to the web of an I-section column, causing bending about the minor-axis of the column section. In order to determine the strength of a column web in bending and punching, the following failure mechanisms are considered:

- Local mechanism : the yield pattern is localised in the compression zone or in the tension zone
- Global mechanism: the yield line pattern involves both compression and tension zone.

For more info about this topic, we refer to Ref.[2].



An example of a minor axis connection is given in **Example CON_007.esa, node N4**.

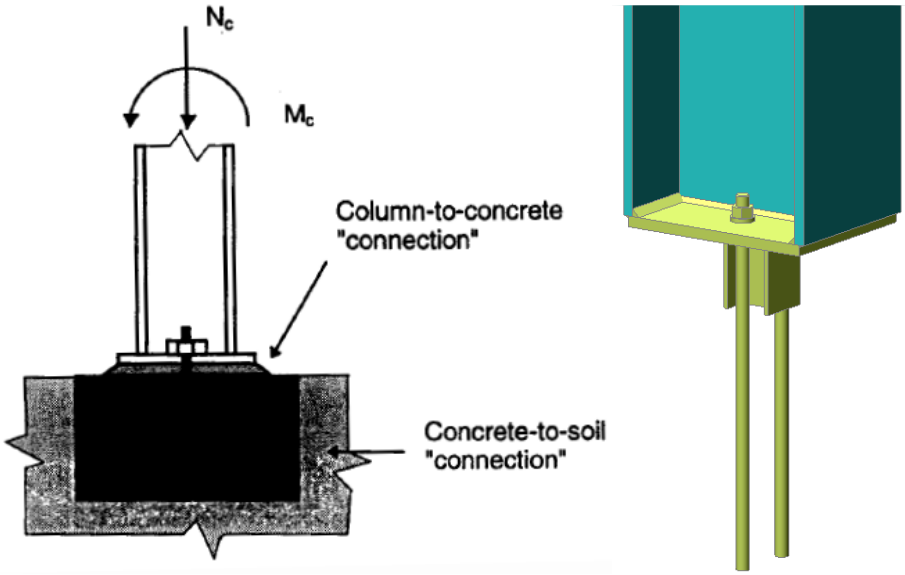


8.3. Base plate connections: shear iron, flange wideners

In a column base, 2 connection deformability's need to be distinguished:

1. the deformability of the connection between the column and the concrete foundation
2. the deformability of the connection between the concrete foundation and the soil.

In the Frame Connect base plate design, the column-to-concrete "connection' is considered.



For more info about base plate design (shear irons, etc…), we refer to Ref.[2]. An example of a base plate connection in SCIA Engineer is given in **Example CON_005.esa, Node N9**

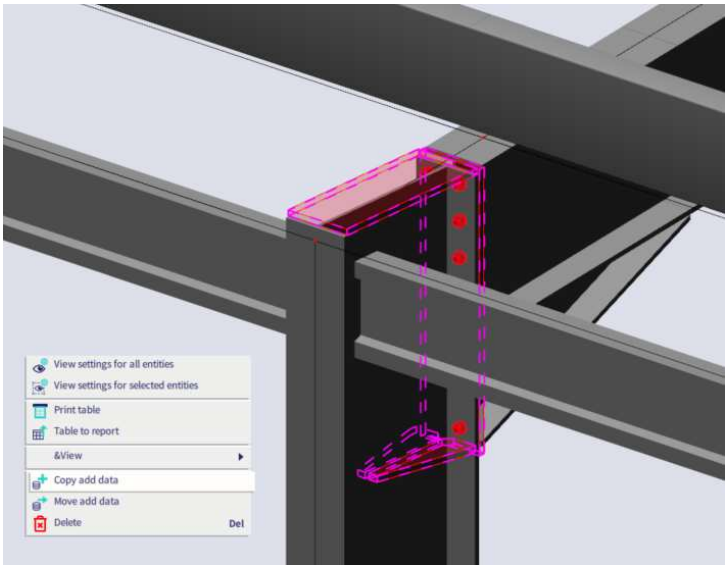
8.4. Extra options for the calculation of connections

In SCIA Engineer it is possible to perform an overall check for multiple connections at the same time. For example we can have a look at **example CON_008.esa**. In this project several connections have been input.

8.4.1. Copy of connections

It is possible to select a connection and to copy this connection to another node. So first you have to select the connection and afterwards you can right click on the screen and choose for "Copy add data Conn".

Afterwards you have to select the nodes to which you want to copy your connection and click on Escape to end this "copy function".



8.4.2. Multiple check of connections

With the option “Overall steel connections check” in the menu “Design > Steel Connections” you can do an overall check for all connections in a project. In the preview window, you will find a list of all connections, with all checks next to it.

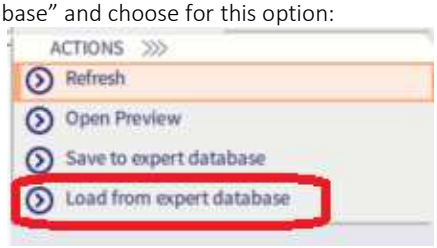
Check of connection

Name	Node	Case	Beam	uc M [-]	uc V [-]	uc N [-]	Stiffness
Conn	N2	LC1	B3	0,29	0,05	0,00	Not ok
Conn1	N5	LC1	B6	0,34	0,05	0,00	Not ok
Conn2	N9	LC1	B9	0,38	0,15	0,00	Not ok

8.4.3. Expert system

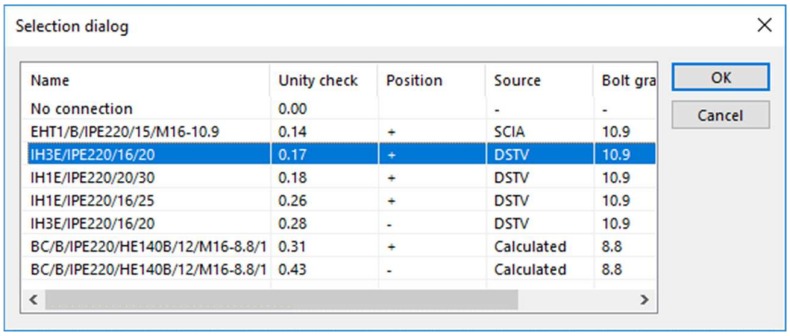
Open example CON_004.esa.

Now delete the connection “Conn” in this example and add a new connection to this node. In the Actions menu, you will the option “Load from expert database” and choose for this option:

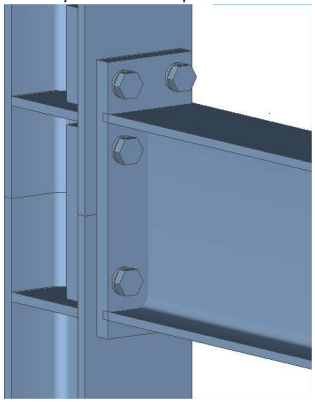


This option is only available if you did not add anything yet to the connection.

In this expert database, you will see some registered connections in SCIA Engineer and the unity check of this connection. Choose the third connection with a unity check of 0.17.



Now this connection will be input on the node and you can adapt this default connection afterwards.

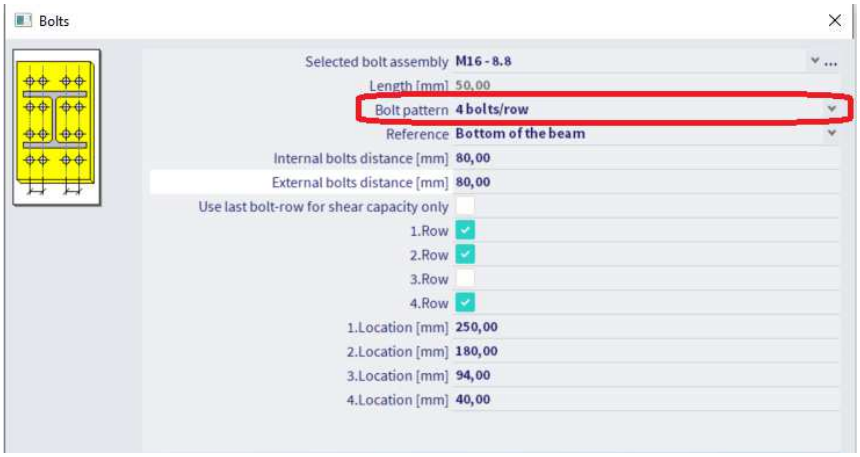


When selecting a connection, you can choose for the option “Save to expert database” and save your connection in this database and use it again in another project.

8.4.4. The use of 4 bolts / row

When 4 bolts per row are used, additional capacity F_{add} is added to the bolt row/group capacity of the column flange and/or the endplate. F_{add} is defined for the following conditions:

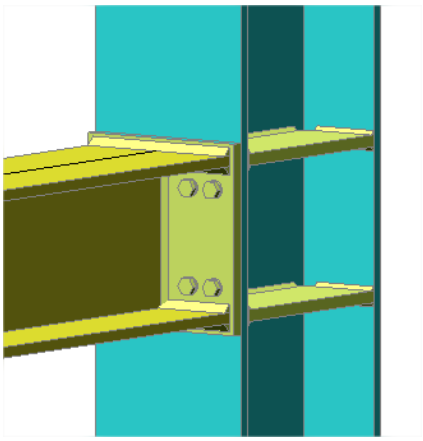
- the capacity of the inner two bolts is equal to the bolt tension resistance (failure mode 3) or is defined by a circular pattern
- the bolt row / group is stiffened
- the bolt group contains only 1 bolt row



For more info about this topic, we refer to Ref.[2].

This option to use 4 bolts in a row can be activated with the manage button behind the bolts.

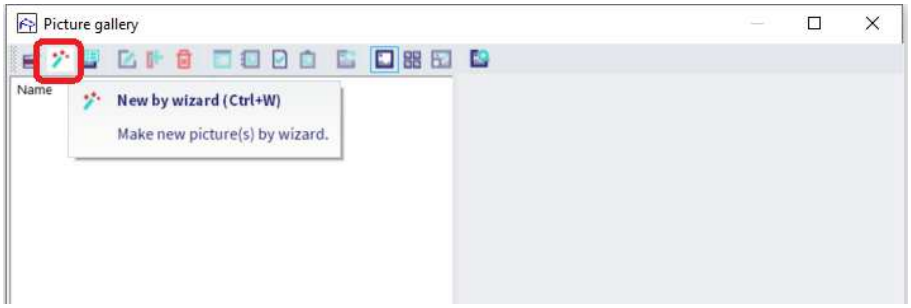
An example of this connection can be found in **example CON_005.esa, node N8**.



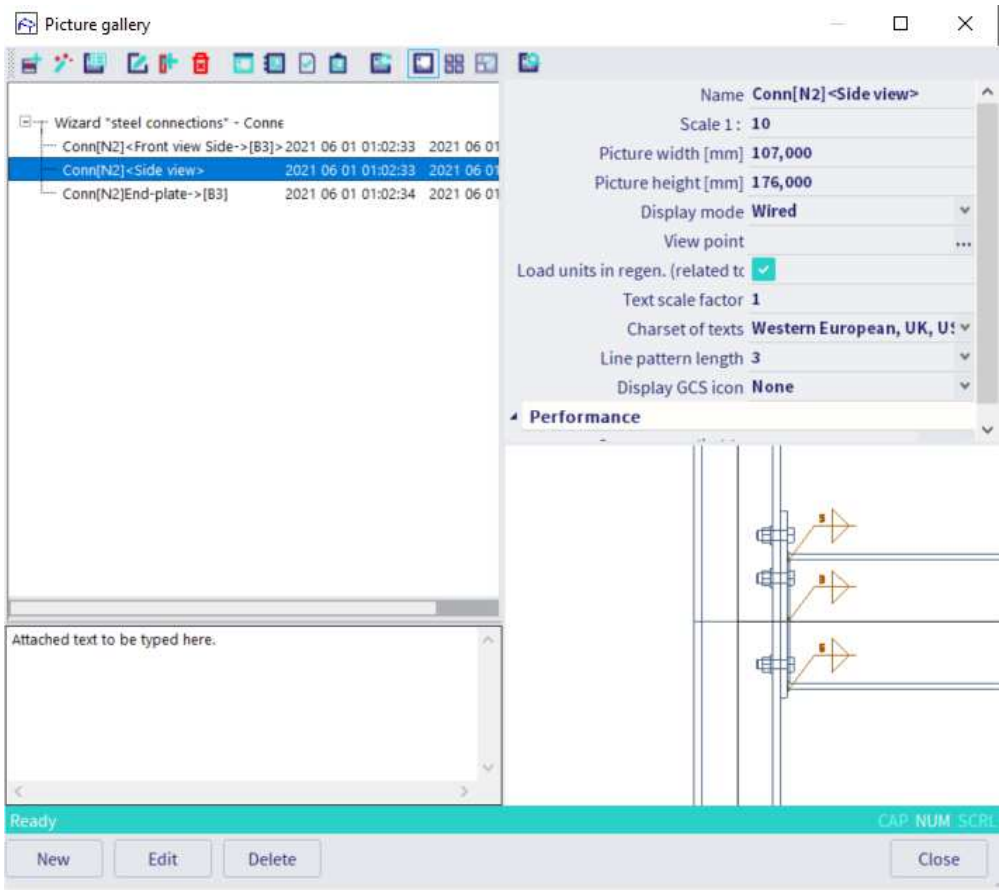
8.4.5. Monodrawings

It is possible to make automatic connection drawings in SCIA Engineer. To use this option, it is enough to select the functionality “Steel connections” in the project data and define at least one steel connection.

After this you can open the picture gallery from “Menu bar” > “Libraries” and choose to generate steel connections monodrawings from the wizard:



When the drawings are finished, they can be found again in the Picture gallery:



9. Welded connections

In this chapter we will show the calculation of a welded connection using example **CON_005.esa, node N3**.

The calculation is done with the Safety factors according the EN 1993-1-8 (Ref.[1]) and the following internal forces:

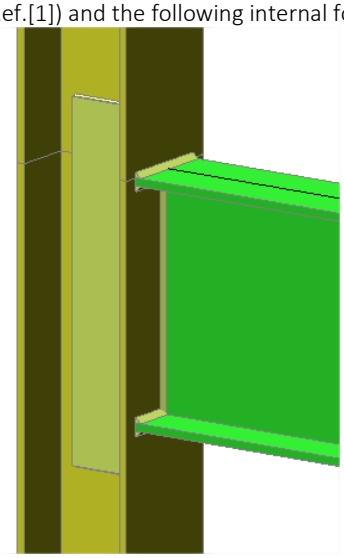
According to EN 1993-1-8
National annex: Standard EN

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

1. Internal forces

tert		
NEd	148.84	kN
Vz,Ed	85.18	kN
My,Ed	-123.96	kNm

Tension top
Note: NEd <= 0.05 * Npl,Rd,Beam.



A negative moment will result in tension at the top flange of the beam.

Calculation $V_{wp,Rd}$ (column web panel in shear)

$$V_{wp,Rd} = \frac{0.9 \cdot f_y \cdot A_{vc}'}{\sqrt{3} \cdot \gamma_{M0}}$$

When a web doubler is used:

$A_{vc}' = A_{vc} + b_s t_s$

$A_{vc} = A - 2bt_f + (t_w + 2r)t_f$

$A_{vc} = 13140 - 2*280*18 + (10.5 + 2*24)*18 = 4113$ mm²

$A_{vc}' = 4113 + 172*10.5 = 5919$ mm²

$$V_{wp,Rd} = \frac{0.9 \cdot f_y \cdot A_{vc}'}{\sqrt{3} \cdot \gamma_{M0}} = \frac{0.9 \cdot 235 \cdot 5919}{\sqrt{3} \cdot 1.0} = 722.77 \text{ kN}$$

And the same is shown in SCIA Engineer:

2.1.1. Column web panel in shear
According to EN 1993-1-8 Article 6.2.6.1

Vwp,Rd data		
Column web in shear (Vwp,Rd/Beta)	722.77	kN
Beta	1.00	
Avc	5919.00	mm²

Note: The shear area Avc has been increased due to the presence of a web doubler.

Calculation $F_{c,wc,Rd}$ (column web in compression)

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

In SCIA Engineer:

2.1.2. Column web in compression

According to EN 1993-1-8 Article 6.2.6.2

Fc,wc,Rd data		
beff,c,wc	249.83	mm
twc	15.75	mm
ω1	0.80	
ω2	0.55	
ω	0.80	
dwc	196.00	mm
λp,rel	0.44	
ρ	1.00	
kwc	1.00	
Fc,wc,Rd	736.92	kN

Note: The thickness twc has been increased due to the presence of a web doubler.

Calculation $F_{c,fb,Rd}$: Beam flange in compression

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{h_b - t_{fb}}$$

$$M_{c,Rd} = \frac{M_{pl,Rd}}{\gamma_{M0}} = \frac{655}{1.0} = 655 \text{ kNm}$$

$$F_{c,fb,Rd} = \frac{655000 \text{ kNmm}}{(550-17.2)\text{mm}} = 1229 \text{ kN}$$

In SCIA Engineer:

2.1.3. Beam flange and web in compression

According to EN 1993-1-8 Article 6.2.6.7

Fc,fb,Rd data		
Section class	1	
Mc,Rd	654.95	kNm
hb-tfb	532.80	mm
Fc,fb,Rd	1229.25	kN

Calculation $F_{t,fc,Rd}$ (column flange in bending)

$$F_{c,fc,Rd} = (t_{wc} + 2 \cdot r_c + 7 \cdot k \cdot t_{fc}) \cdot \frac{t_{wc} \cdot f_y}{\gamma_{M0}} = (10.5 + 2 \cdot 24 + 7 \cdot 1 \cdot 18) \cdot \frac{17.2 \cdot 235}{1.0} = 745 \text{ kN}$$

In SCIA Engineer:

2.1.4. Column flange in bending (EN 1993-1-8 art. 6.2.6.4)

(Ft,fc,Rd) data		
(Ft,fc,Rd)	745.75	kN
k	1.00	

Calculation $F_{t,wc,Rd}$ (column web in tension)

$$F_{t,wc,Rd} = \frac{\rho \cdot b_{eff} \cdot t_{wc} \cdot f_y}{\gamma_{M0}}$$

$t_{wc} = 1.4 \cdot 10.5 = 14.7 \text{ mm}$

$b_{eff} = t_{fb} + 2\sqrt{2} \cdot a + 5 \cdot (t_{fc} + r) = 17.2 + 2\sqrt{2} \cdot 9 + 5 \cdot (18 + 24) = 252 \text{ mm}$

$\rho = \rho_1$

$\rho_1 = \frac{1}{\sqrt{1+1.3 \cdot \left(\frac{b_{eff} \cdot t_{wc}}{A_{vc}'}\right)^2}} = 0.81$

$F_{t,wc,Rd} = \frac{0.81 \cdot 252 \cdot 14.7 \cdot 235}{1.0} = 705 \text{ kN}$

In SCIA Engineer:

2.1.5. Column web in tension (EN 1993-1-8 art. 6.2.6.3)

(Ft,wc,Rd) data		
(Ft,wc,Rd)	704.55	kN
beff	249.83	mm
twc	14.70	mm
omega_1	0.82	
omega_2	0.58	
omega	0.82	

Calculation MRd (design moment resistance)

705 kN x 0.532 m = 375 kNm

In SCIA Engineer:

2.2. Determination of Mj,y,Rd

According to EN 1993-1-8 Article 6.2.7.1 (4)

Mj,Rd data		
F	704.55	kN
h	532.80	mm
Mj,Rd	375.39	kNm

Calculation af

The weld size a_f is designed according to the resistance of the joint. The design force in the beam flange can be estimated as:

$$F_{Rd} = \frac{M_{Rd}}{h} = \frac{375}{0.532} = 705 \text{ kN}$$

The design resistance of the weld F_w shall be greater than the flange force F_{Rd}, multiplied by a factor γ. The value of the factor γ is:

- $\gamma = 1.7$ for sway frames
- $\gamma = 1.4$ for non sway frames

However, in no case shall the weld design resistance be required to exceed the design plastic resistance of the beam flange $N_{t.Rd}$:

$$N_{t,Rd} = \frac{b_f \cdot t_{fb} \cdot f_{yb}}{\gamma_{M0}} = \frac{210 \cdot 17.2 \cdot 235}{1.0} = 849 \text{ kN}$$

$F_w = \min (N_{t.Rd}; \gamma F_{Rd}) = \min (849; 1.4*705) = 849$ kN

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

$$a_f \geq \frac{F_w \cdot \gamma_{Mw} \cdot \beta_w}{f_u \cdot b_f \cdot \sqrt{2}} = \frac{849000 \cdot 1.25 \cdot 0.8}{360 \cdot 210 \cdot \sqrt{2}} = 7.94 \text{ mm}$$

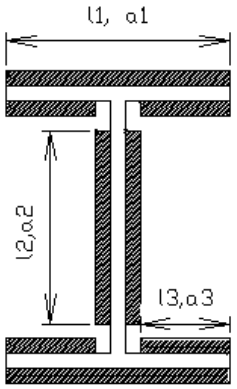
We take a_f = 8 mm.

In SCIA Engineer

6. Design calculations
6.1. Flange welds

data		
Mj,y,Rd	375.39	kNm
α	1.40	
h	532.80	mm
FRd	986.38	kN
Nt,Rd	771.65	kN
fu	360.00	MPa
β_w	0.80	
minimum af	6.35	mm
af	7.00	mm

Calculation of a_w



The section is sollicitated by the moment M, the normal force N and the shear force D.

The moment M is defined by the critical design moment resistance of the connection. The normal force N is taken as the maximum internal normal force on the node, the shear force D is taken as the maximum internal shear force on the node.

M = 375 kNm
N = 148 kN
D = 85 kN

(see calculation of M_{Rd} and the internal forces, given in the beginning of this chapter)

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:

$$\sqrt{\sigma_1^2 + 3 \cdot \left(\tau_1^2 + \tau_2^2\right)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M_w}} \quad \text{and} \quad \sigma_1 \leq \frac{f_u}{\gamma_{M_w}}$$

$$\sigma_1 = \tau_2 = \left(\frac{N}{A} + \frac{M \cdot l_2}{2 \cdot I}\right) \frac{1}{\sqrt{2}}$$

$$\tau_1 = \frac{D}{2 \cdot a_2 \cdot l_2}$$

With:

- f_u ultimate tensile strength of the weaker part
- β_w correlation factor
- γ_{Mw} partial safety factor for welds

In SCIA Engineer:

6.2. Web welds

data		
M	375.39	kNm
N	148.84	kN
V	85.18	kN
fu	360.00	MPa
β_w	0.80	
a1	7.00	mm
a3	7.00	mm
l1	210.00	mm
l2	450.40	mm
l3	75.45	mm
A	6854.20	mm^2
I	393198742.61	mm^4
minimum aw (a2)	5.12	mm
aw	6.00	mm

10.Pinned joints

In SCIA Engineer four types of joints are supported:

- Type 1: welded plate in beam, welded to column
- Type 2: bolted plate in beam, welded to column
- Type 3: bolted angle in beam and column
- Type 4: short endplate welded to beam, bolted in column

For each type, the design shear resistance V_{Rd} (taking into account the present normal force N) and the design compression/tension resistance N_{Rd} are calculated.

The design shear resistance is calculated for the following failure modes:

- design shear resistance for the connection element
- design shear resistance of the beam
- design block shear resistance
- design shear resistance due to the bolt distribution in the beam web
- design shear resistance due to the bolt distribution in the column

The design compression/tension resistance is calculated for the following failure modes:

- design compression/tension resistance for the connection element
- design compression/tension resistance of the beam
- design tension resistance due to the bolt distribution in the column

In Ref.[2], more info on the used formulas is given.

10.1. Welded fin plate connection

In this chapter we will show the calculation of a welded fin plate connection using example CON_009.esa, node N2.

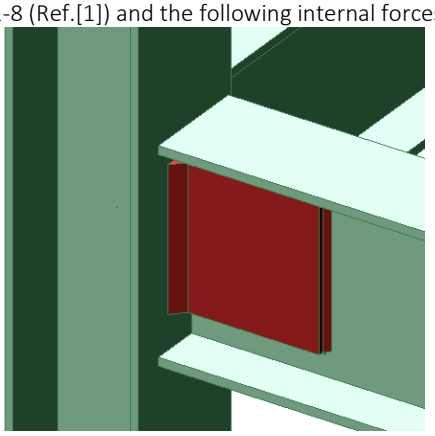
The calculation is done with the Safety factors according the EN 1993-1-8 (Ref.[1]) and the following internal forces:

The limit capacities are according to EN 1993-1-8

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

1. Internal forces

C1		
N	-1.43	kN
Vz	7.97	kN
My	0.00	kNm



10.1.1. Design shear resistance V_{Rd} for connection element

Transversal section of the plate: $A_{pl} = 2 \cdot h_{pi} \cdot t_{pi} = 2 \cdot 163 \cdot 12 = 3912\ mm^2$ (2 plates)

Normal stress: $\sigma_N = \frac{N}{A_{pl}} = \frac{-1440}{3912} = -0.3681\ N/mm^2$

Flexion module: $W_{pl} = 2 \cdot \frac{t_{pl} \cdot h_{pl}^2}{6} = 106276\ mm^3$

Design shear resistance: a = 163/2 = 81.5 mm is the centre

$$V_{Rd1} = \frac{A \cdot f_y}{\gamma_{M0} \cdot \sqrt{3}} = \frac{3912 \cdot 235}{1.00 \cdot \sqrt{3}} = 530\ 769\ N = 530.77\ kN$$

$$V_{Rd2} = \left[\frac{f_y}{\gamma_{M0}} - \sigma_N\right] \cdot \frac{W}{a} = \left[\frac{235}{1.00} - (-0.3681)\right] \cdot \frac{106276}{81.5} = 306\ 920N = 306.92\ kN$$

$$V_{Rd} = \min(V_{Rd1}; V_{Rd2}) = 306.92\ kN$$

In SCIA Engineer:

2. Design shear resistance

2.1. Design shear resistance VRd for connection element

data		
sigmaN	-0.37	MPa
A	3912.00	mm^2
Av,net	3912.00	mm^2
W	106276.00	mm^3
a	82.00	mm
VRd - bending	304.10	kN
VRd - shear	530.77	kN

10.1.2. Design shear resistance V_{Rd} for beam

Shear area: $A_v = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f = 3910 - 2 \cdot 120 \cdot 9.8 + (6.2 + 2 \cdot 15) \cdot 9.8 = 1912.76\ mm^2$

Shear resistance : $V_{Rd} = \frac{A_v \cdot f_y}{\gamma_{M0} \cdot \sqrt{3}} = 259518.1\ N = 259.52\ kN$

In SCIA Engineer:

2.2. Design shear resistance VRd for beam

data		
Av	1912.76	mm^2
VRd	259.52	kN

10.1.3. Design compression/tension resistance N_{Rd} for connection element

Area of the element: $A_{pl} = 2 \cdot h_{pi} \cdot t_{pi} = 3912\ mm^2$

Compression/tension resistance : $N_{Rd} = \frac{A_{pl} \cdot f_y}{\gamma_{M0}} = 919320\ N = 919.32\ kN$

In SCIA Engineer:

3.1. Design compression/tension resistance NRd for connection element

data		
A	3912.00	mm^2
NRd	919.32	kN

10.1.4. Design compression/tension resistance N_{Rd} for beam

Area of the beam: A = 0.003910 m^2

Compression/tension resistance : $N_{Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{3910 \cdot 235}{1.0} = 918850\ N = 918.85\ kN$

In SCIA Engineer:

3.2. Design compression/tension resistance NRd for beam

data		
A	3910.00	mm^2
NRd	918.85	kN

10.1.5. Weld size calculation for Plate, Beam and Column

To determine the weld size *a* for the plate on the beam and on the column, we must use a iterative process with *a* as parameter until the Von Mises rules is respected (Annex M/EC3):

$$\sqrt{\sigma_1^2 + 3 \cdot \left(\tau_1^2 + \tau_2^2\right)} \le \frac{f_u}{\beta_w \cdot \gamma_{M_w}} \quad \text{and} \quad \sigma_1 \le \frac{f_u}{\gamma_{M_w}}$$

We'll only check the weld size for the final value of *a*. For the weld between plate and beam we find *a = 4 mm* and for weld between plate and column, the weld size is *a = 11 mm*.

Weld size Plate/Beam

We define the play as the effective distance between the end of the beam and the flange of the column. In this case, the play is 10mm. By using EC3 and the Chapter 11 of the manual, we compute the following parameters:

Weld size: a = 0.004 m

Weld Length: $l_1 = h_{pl} - 2 \cdot t_{pl} = 0.163 - 2 \cdot 0.012 = 0.139 m$

$$l_2 = b_{pl} - Play - 2 \cdot t_{pl} = 0.13 m$$

$$l = l_{pl} - Play = 0.164 - 0.01 = 0.154 m$$

By EC3: f_{uw} = 360000000 N/m^2 and β_w = 0.8. The parameters are:

$$g = \frac{(0.707 \cdot a \cdot l_1 + 0.577 \cdot a \cdot l_1) \cdot l}{0.577 \cdot a \cdot l_1 + 1.414 \cdot a \cdot l_2} = 104.10$$

$$\delta = \frac{0.577 \cdot a \cdot l_1}{0.577 \cdot a \cdot l_1 + 1.414 \cdot a \cdot l_2} = 0.30377$$

$$\mu = \frac{0.117 \cdot a \cdot l_1^2}{0.117 \cdot a \cdot l_1^2 + 0.577 \cdot a \cdot l_2 \cdot h_{pl}} = 0.15603$$

$$\Gamma = \frac{0.707 \cdot a \cdot l_1}{0.707 \cdot a \cdot l_1 + 1.14 \cdot a \cdot l_2} = 0.3987$$

$$L = 10 + g = 10 + 104{,}10 = 114{,}10 mm$$

Shear force on one plate: $D = \frac{V_{Rd}}{2} = \frac{259.52}{2} = 129.76$ kN (for one plate)

Normal force on one plate: $N = \frac{N_{Ed}}{2} = \frac{-1.44}{2} = -0.72$ kN

Moment on the plate: $M = D \cdot L = 129.76 \cdot 0.1141 = 14.81$ kNm

Weld Check 1:

$$\tau_1 = \sigma_1 = \frac{6 \cdot \mu \cdot M}{\sqrt{2} \cdot a \cdot l_1^2} + \frac{\Gamma \cdot N}{\sqrt{2} \cdot a \cdot l_1} == 127{,}7 \, N/mm^2$$

$$\tau_2 = \frac{\delta \cdot D}{a \cdot l_1} = 70{,}97 \, N/mm^2$$

Unity Check: $\frac{\sqrt{\sigma_1^2 + 3 \cdot (\tau_1^2 + \tau_2^2)}}{f_u / \beta_w \cdot \gamma_{M_w}} = 0.63 \le 1$ and $\frac{\sigma_1}{f_u / \gamma_{M_w}} = 0{,}35 \le 1$

Weld Check 2:

$$\sigma_1 = \tau_1 = \frac{(1-\delta) \cdot D}{2 \cdot \sqrt{2} \cdot a \cdot l_2} = 61{,}67 \, N/mm^2$$

$$\tau_2 = \left(\frac{(1-\mu) \cdot M}{h \cdot a \cdot l_2} + \frac{(1-\Gamma) \cdot N}{2 \cdot a \cdot l_2} \right) = 147{,}99 \, N/mm^2$$

Unity Check: $\frac{\sqrt{\sigma_1^2 + 3 \cdot (\tau_1^2 + \tau_2^2)}}{f_u / \beta_w \cdot \gamma_{M_w}} = 0.63 \le 1$ and $\frac{\sigma_1}{f_u / \gamma_{M_w}} = 0.17 \le 1$

In SCIA Engineer:

5. Weldsize calculation

5.1. Weldsize plate/beam

data		
Fu	360.00	MPa
beta	0.80	
a	4.00	mm
l1	139.00	mm
l2	130.00	mm
L	114.10	mm
g	104.10	mm
delta	0.30	
mu	0.16	
tau	0.40	
D	129.76	kN
N	-0.72	kN
M	14.81	kNm
for weld check 1 :		
sigma,1	127.57	MPa
tau,1	127.57	MPa
tau,2	70.89	MPa
for weld check 2 :		
sigma,1	61.61	MPa
tau,1	61.61	MPa
tau,2	147.84	MPa

Weld size Plate/Column

Weld size: a = 0.011 m

Normal force: N = -1440 N = -1.44 kN

Moment: $\mathrm{M = D \cdot L = 259.790 \cdot 0.082 = 21.3\ kNm}$

Stress calculation:

$$\sigma_1 = -\tau_1 = \frac{N}{2 \cdot \sqrt{2} \cdot a \cdot l} + \frac{M}{W} = \frac{N}{2 \cdot \sqrt{2} \cdot a \cdot l} + \frac{D \cdot L}{2 \cdot \sqrt{2} \cdot a \cdot \frac{h_{pl}^2}{6}} = 154{,}3\ N/mm^2$$

$$\tau_2 = \frac{D}{2 \cdot a \cdot l} = 76{,}68\ N/mm^2$$

Unity Check: $\dfrac{\sqrt{\sigma_1^2 + 3 \cdot \left(\tau_1^2 + \tau_2^2\right)}}{f_u / \beta_w \cdot \gamma_{M_w}} = 0.75 \le 1$ $\qquad$ $\dfrac{\sigma_1}{f_u / \gamma_{M_w}} = 0.42 \le 1$

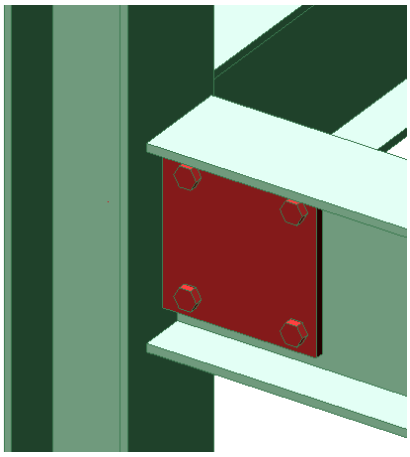
In SCIA Engineer:

5.2. Weldsize plate/column

data		
Fu	360.00	MPa
beta	0.80	
a	11.00	mm
L	82.00	mm
D	259.52	kN
N	-1.43	kN
M	21.28	kNm
sigma,1	155.21	MPa
tau,1	155.21	MPa
tau,2	72.37	MPa

10.2. Bolted fin plate connection

In this chapter we will show the calculation of a bolted fin plate connection using example **CON_010.esa, node N2** for combination C1.



10.2.1. Design shear resistance V_{Rd} for connection element

Transversal section of the plate: $A_v = 2 \cdot h \cdot t = 2 \cdot 188 \cdot 12 = 4512\ mm^2$ (2 plates)

$A_{v,net} = A - \#bolts \cdot (t \cdot d_0) = 4512 - 4 \cdot 12 \cdot 18 = 3648\ mm^2$

Normal stress: $\sigma_N = \frac{N}{A} = \frac{-1440}{4512} = -0.32\ N/mm^2$

Flexion module : $W = 2 \cdot \frac{t \cdot h^2}{6} = 141376\ mm^3$

Bolt centre : a = 34 mm (= X1 parameter of the bolt position)

The bolt holes are not taken into account when: $A_{v,net} \geq \frac{f_y}{f_u} \cdot A \rightarrow 3648 \geq 2945$ OK

When $A_{v,net}$ is less than this limit, an effective shear area of $A_v = \frac{f_u}{f_y} \cdot A_{v,net}$ may be assumed, else $A_v = A$

Design shear resistance:

$$V_{rd1} = \frac{A_v \cdot f_y}{\gamma_{M0} \cdot \sqrt{3}} = \frac{4512 \cdot 235}{1.00 \cdot \sqrt{3}} = 612176\ N = 612.18\ kN$$

$$V_{rd2} = \left[\frac{f_y}{f_u} \cdot \sigma_N\right] \cdot \frac{W}{a} = \left[\frac{235}{360} \cdot (-0.32)\right] \cdot \frac{141376}{34} = 978488\ N = 978.49\ kN$$

$$V_{rd} = \min(V_{rd1}; V_{rd2}) = (612.18; 978.49) = 612.18\ kN$$

In SCIA Engineer:

2. Design shear resistance

2.1. Design shear resistance VRd for connection element

data		
sigmaN	0.00	MPa
A	4512.00	mm^2
Av,net	3648.00	mm^2
W	141376.00	mm^3
a	34.00	mm
VRd - bending	977.16	kN
VRd - shear	612.18	kN

10.2.2. Design shear resistance V_{Rd} for beam

Shear area : $A_v = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f = 3912 - 2 \cdot 120 \cdot 9.8 + (6.2 + 2 \cdot 15) \cdot 9.8 = 1912.76\ mm^2$

Net area : $A_{v,net} = A_v - \#bolts\ per\ section \cdot t_w \cdot d_0 = 1912.76 - 2 \cdot 6.2 \cdot 18 = 1689.56\ mm^2$

The bolt holes are not taken into account when: $A_{v,net} \geq \frac{f_y}{f_u} \cdot A_v \rightarrow 1689.56\ mm^2 \geq 1249\ mm^2$ OK

When $A_{v,net}$ is less than this limit, an effective shear area of $A_{v1} = \frac{f_u}{f_y} \cdot A_{v,net}$ may be assumed, else $A_{v1} = A_v$

Design shear resistance: $V_{Rd} = \frac{A_{v1} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{1912.76 \cdot 235}{\sqrt{3} \cdot 1.00} = 259518\ N = 259.52\ kN$

In SCIA Engineer:

2.2. Design shear resistance VRd for beam

data		
Av	1912.76	mm²
Av,net	1689.56	mm²
VRd	259.52	kN

10.2.3. Design shear resistance V_{Rd} for bolt in beam

The calculation of the shear resistance for bolt in beam is based on the following equation to be solve

$$V_{Rd}^2 \cdot \left(\frac{1}{n^2} + \frac{a^2 \cdot c^2}{I_p^2} + \frac{2 \cdot a \cdot c}{n \cdot I_p} + \frac{a^2 \cdot d^2}{I_p^2} \right) + V_{Rd} \cdot \left(\frac{2 \cdot a \cdot N \cdot d}{I_p \cdot n} \right) + \frac{N^2}{n^2} - Q^2 = 0$$

Where : $a = 0.0995m$ $b = 0.094m$ $c = 0.0655m$ $d = 0.07m$

$I_p = \sum r_i^2 = \sum_{i=1}^{4} 95.66^2 = 0.036761 m^2$

$Q = \min\left(2 \cdot F_{v,Rd}, \min\left(F_{b,Rd,plate}; F_{b,Rd,beam}\right)\right)$=31740.8256 N for two plates, where

- $F_{V_{Rd}} = \frac{0.6 \cdot f_{ub} \cdot A_s}{\gamma_{Mb}} = 30144N = 30.1kN$
- $F_{b,Rd,Beam} = \frac{2.5 \cdot \alpha_p \cdot f_u \cdot d \cdot t}{\gamma_{Mb}} = 31740.8256N = 31.7kN$

$$\text{with } \alpha_p = \min\left(\frac{e_1}{3d_0}; \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1\right) = 0.444$$

- $F_{b,Rd,plate} = \frac{2.5 \cdot \alpha_p \cdot f_u \cdot d \cdot t_{pl}}{\gamma_{Mb}} = 122867.712\,N$

$$\text{with } \alpha_p = \min\left(\frac{e_1}{3d_0}; \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1\right) = 0.444$$

By solving the second-degree equation, we find V_{Rd} = 124.28 kN

In SCIA Engineer:

2.3. Design shear resistance VRd for bolts in beam

Bolt resistance		
e1,el	24.00	mm
e1,bw	24.00	mm
p1	140.00	mm
p2	131.00	mm
k1 plate	2.03	
k1 beam	2.03	
Alfa_b plate	0.44	
Alfa_b beam	1.00	
Alfa_d plate	0.44	
Alfa_d beam	2.34	
Fb,el,Rd	99.94	kN
Fb,bw,Rd	58.09	kN
Fv,Rd	30.14	kN

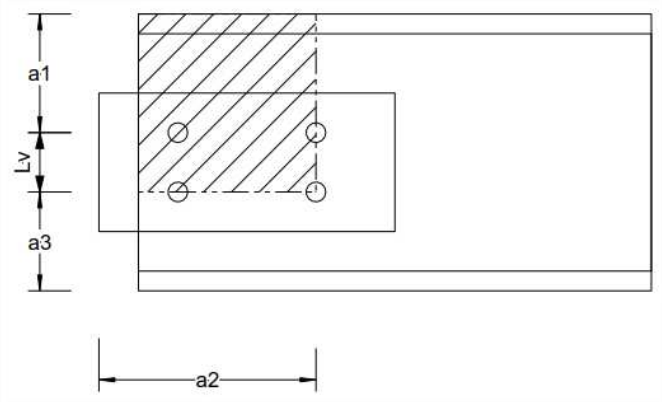
data		
a	99.50	mm
b	94.00	mm
d	70.00	mm
c	65.50	mm
Ip	36761.00	mm^2
VRd	124.28	kN

10.2.4. Design block shear resistance V_{Rd} beam

The design value of the effective resistance to block shear is determined by the following expression:

$$V_{eff,Rd} = \frac{A_{v,eff} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} \text{ with } A_{v,eff} = t \cdot L_{v,eff}$$

The values a1, a2, a3 and Lv are defined as follows:



We determined the effective shear area $A_{v,eff}$ as follows :

a_1 = 50 mm; a_2 = 165 mm; a_3 = 50 mm

$L_v = h - a_1 - a_3 = 240 - 50 - 50 = 140$ mm

$$L_3 = \min\left(L_v + a_1 + a_3 ; (L_v + a_1 + a_3 - n \cdot d_0) \cdot \frac{f_u}{f_y} \right)$$
$$= \min(240;257,36) = 240mm$$
$$L_1 = \min(a_1 ; 5 \cdot d_0) = \min(50;90) = 50mm$$
$$L_2 = (a_2 - k \cdot d_0) \cdot \frac{f_u}{f_y} = (165 - 2,5 \cdot 18) \cdot \frac{360}{235} = 183,83mm$$
$$\text{with k} = 2.5 \text{ for 2 bolt - rows}$$
$$L_{v,eff} = \min(L_v + L_1 + L_2 ; L_3) = \min(373,83;240) = 240mm$$

$A_{v,eff} = t \cdot L_{v,eff} = 6 \cdot 240 = 1440 \text{ mm}^2$

$V_{eff,Rd} = \frac{A_{v,eff} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = 195375 \text{ N} = 195.38 \text{ kN}$

In SCIA Engineer:

2.4. Design block shear resistance VRd - Beam

data		
Ant	793.60	mm²
Anv	1016.80	mm²
VRd	252.24	kN

10.2.5. Design block shear resistance V_{Rd} connection element (beam side)

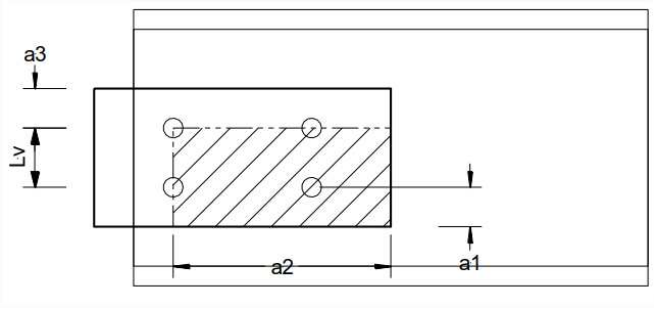
The design value of the effective resistance to block shear is determined by the following expression:

$$V_{eff,Rd} = \frac{A_{v,eff} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} \text{ with } A_{v,eff} = n \cdot t \cdot L_{v,eff}$$

With:

- n number of plate, cleat
- t thickness of plate, cleat

The values a_1, a_2, a_3 and L_v are defined as follows:



We determined the effective shear area $A_{v,eff}$ as follows :

a_1 = 24 mm; a_2 = 155 mm; a_3 = 24 mm

$L_v = h_{plate} - a_1 - a_3 = 188 - 24 - 24 = 150$ mm

$$L_3 = \min\left(L_v + a_1 + a_3 ; (L_v + a_1 + a_3 - n \cdot d_0) \cdot \frac{f_u}{f_y} \right)$$
$$= \min(188;177,7) = 177,7mm$$
$$L_1 = \min(a_1 ; 5 \cdot d_0) = \min(24;90) = 24mm$$

$$L_2 = (a_2 - k \cdot d_0) \cdot \frac{f_u}{f_y} = (155 - 2{,}5 \cdot 18) \cdot \frac{360}{235} = 168{,}51 mm$$

with k = 2.5 for 2 bolt - rows

$$L_{v,eff} = \min(L_v + L_1 + L_2; L_3) = \min(342{,}51; 177{,}7) = 177{,}7 mm$$

$A_{v,eff} = n \cdot t \cdot L_{v,eff} = 2 \cdot 12 \cdot 177.7 = 4264.8\ mm^2$

$V_{eff,Rd} = \frac{A_{v,eff} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = 578637\ N = 578.64\ kN$

In SCIA Engineer:

2.5. Design block shear resistance VRd - Connection element on beam side

data		
Ant	3312.00	mm²
Anv	3288.00	mm²
VRd	923.04	kN

10.2.6. Design compression/tension resistance N_{Rd} for connection element

Area : $A = 2 \cdot h \cdot t = 2 \cdot 188 \cdot 12 = 4512\ mm^2$

Net area : $A_{net} = A - 2 \cdot t \cdot 2 \cdot d_0 = 4512 - 2 \cdot 12 \cdot 2 \cdot 18 = 3648\ mm^2$

Compression resistance: $N_{Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{4512 \cdot 235}{1.00} = 1060320\ N = 1060.32\ kN$

In SCIA Engineer:

3.1. Design compression/tension resistance NRd for connection element

data		
A	4512.00	mm²
A,net	3648.00	mm²
NRd	945.56	kN

10.2.7. Design compression/tension resistance N_{Rd} for beam

Area: A = 3910 mm²

Net area: $A_{net} = A - 2 \cdot t \cdot d_0 = 3910 - 2 \cdot 6.2 \cdot 18 = 3686.8\ mm^2$

Tension resistance : $N_{Rd} = \min\left(\frac{A \cdot f_y}{\gamma_{M0}}, \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M1}}\right) = \min\left(\frac{3910 \cdot 235}{1.00}, \frac{0.9 \cdot 3686.8 \cdot 360}{1.25}\right) = 918.85\ kN$

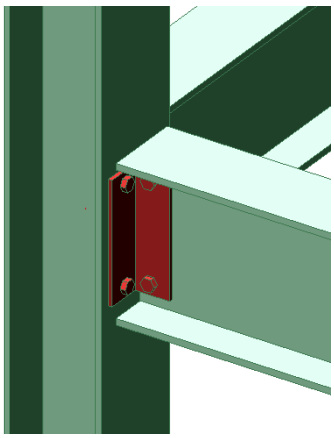
In SCIA Engineer:

3.2. Design compression/tension resistance NRd for beam

data		
A	3910.00	mm²
A,net	3686.80	mm²
NRd	918.85	kN

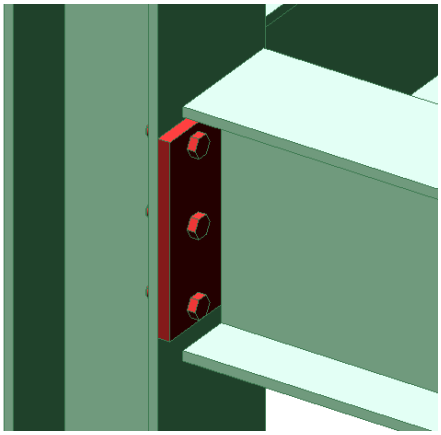
10.3. Bolted cleat connection

An example of this connection can be found in example **CON_011.esa, node N2**.



10.4. Flexible end plate connection

An example of this connection can be found in example **CON_012.esa, node N2**.



11.Hollow section joints

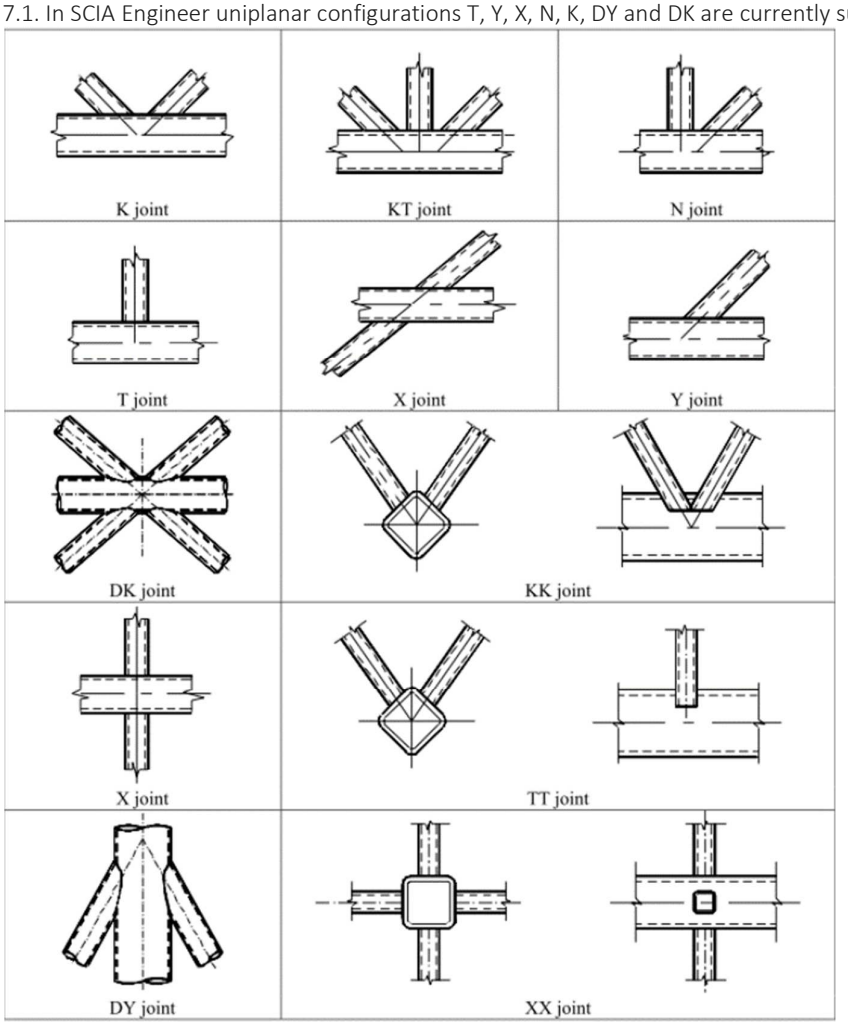
The design of Hollow Section Joints within SCIA Engineer is implemented according to EN 1993-1-8:2005-05, including Corrigendum EN 1993-1-8:2005/AC:2009-07, however for certain parts also different references are taken into account. Such references will be mentioned where relevant.

Used terms and symbols are in line with the main code.

Open example **CON_13.esa.**

11.1. Joint configuration

The design procedure defined by the EN 1993-1-8 Article 7, may be applied to joint types specified in EN 1993-1-8 Article 7.1.2 Table 7.1. In SCIA Engineer uniplanar configurations T, Y, X, N, K, DY and DK are currently supported.

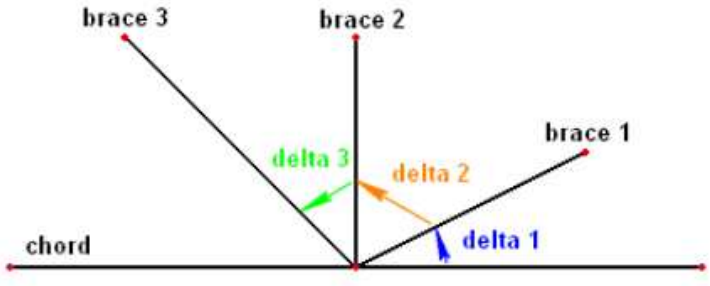


11.1.1. Automatic recognition

The **automatic recognition of joint configuration is based on the Structural model**, taking into account all members selected during input of the joint object. The recognition is processed in several steps:

- List of all selected members is created.
- Members with the highest value of Structure type are marked as chord members. User may use for example type **truss chord** with value of 95.
- Remaining members are marked as brace members. User may use for example type **truss diagonal** with value of 90.
- For each brace member respective delta angle between the brace member and the chord member is measured .
- The brace member with the smallest delta angle is marked as brace 1.

- Depending on the number of brace members, an additional delta angles between brace members are measured. The angles are measured in a continuous direction from angle delta 1. See example picture below.



- The algorithm indicated on the picture below is used for the final determination of the joint configuration.

An additional per-plane validation tests need to be fulfilled in the structural model:

- Axes of all members has to be in one plane.
- Chord members has to be in line, which in terms means use the same eccentricity
- For T, Y, X, DY and DK joint configurations, all the members need to have one intersection point.

An additional cross-section validation tests need to be fulfilled:

- For Circular Hollow Section joints, the cross-sections marked by SCIA Engineer as formcode 3 cross-section, are supported .
- The cross-section id of the chord members have to be identical.

An additional geometry validation tests need to be fulfilled in the structural model:

- Continuous chord members are supported.
- Continuous brace members are not supported.
- The curved members are not supported.

If the configuration is not recognized, an error massage is displayed during the input of the joint object. User may also modify the geometry or member attributes when a joint already exists and in such case recognized joint configuration is changed to "Not Recognized".

In SCIA Engineer:

1. Joint members info

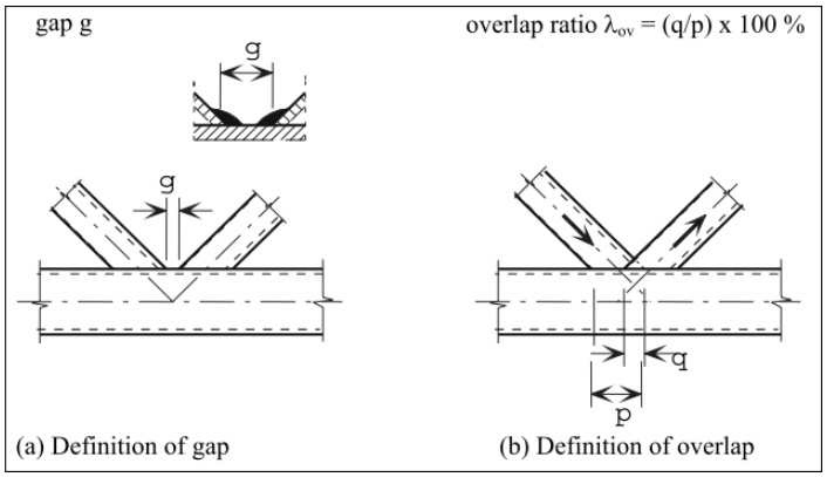
Chord		
Members	B115, B116	
Cross-section	CHS219.1/12.5	
d_0	219,10	mm
t_0	12,50	mm
Material	S 355	
$f_{y,0}$	355,0	MPa
A_0	8110	mm^2
$W_{el,y}$	397000	mm^3
$W_{el,z}$	397000	mm^3

Brace 1		
Member	B117	
Cross-section	CHS139.7/5.0	
d_1	139,70	mm
t_1	5,00	mm
θ_1	45,00	deg
Material	S 355	
$f_{y,1}$	355,0	MPa
A	2120	mm^2
$W_{el,y}$	68800	mm^3
$W_{el,z}$	68800	mm^3

Brace 2		
Member	B125	
Cross-section	CHS114.3/3.6	
d_2	114,30	mm
t_2	3,60	mm
θ_2	45,00	deg
Material	S 355	
$f_{y,2}$	355,0	MPa
A	1250	mm^2
$W_{el,y}$	33600	mm^3
$W_{el,z}$	33600	mm^3

11.1.2. Gap / Overlap

Geometric definition of gap and overlap is given by EN 1993-1-8 Article 1.5 Figure 1.3. Within SCIA Engineer the Gap / Overlap definition parameters are only valid and displayed for N, K and DK joint configurations.



Either gap or overlap is calculated automatically by default using the structural model geometry and the value is shown in the properties of the joint object with reference to identified brace members. The formulas for calculation of gap, overlap and also eccentricity are based on a CIDECT publication "Design guide for circular hollow section (CHS) joints under predominantly static loading", 2008:

$$g = (e + \frac{d_0}{2}) * \frac{\sin(\theta_1 + \theta_2)}{\sin\theta_1 * \sin\theta_2} - \frac{d_1}{2 * \sin\theta_1} - \frac{d_2}{2 * \sin\theta_2}$$

$$e = (\frac{d_1}{2 * \sin\theta_1} + \frac{d_2}{2 * \sin\theta_2} + g) * \frac{\sin\theta_1 * \sin\theta_2}{\sin(\theta_1 + \theta_2)} - \frac{d_0}{2}$$

Eccentricity of the joint is defined in EN1993-1-8 Article 5.1.5 Figure 5.3.

The user is also allowed to input direct values of gap or overlap or define it through manual input of eccentricity. This may be done through "Definition" combo-box item in the properties where the user may select from several options:

- From structural model - Default option. Both gap and overlap is calculated automatically using the structural model geometry. Eccentricity, gap or overlap items are disabled for editing.
- Eccentricity - Direct input of eccentricity. Gap or overlap is calculated based on the defined eccentricity value and is disabled for editing. Valid input range is <-1;1>[m].
- Gap - Direct input of gap. Eccentricity is calculated based on the defined gap value and is disabled for editing. Valid input range is <0;1>[m].
- Overlap - Direct input of overlap. Eccentricity is calculated based on the defined overlap value and is disabled for editing. Valid input range is <0;110>[%].

In SCIA Engineer:

2. Joint configuration

Gap / Overlap [Brace 1, Brace 2]		
Eccentricity	0,00	mm
Gap	39,49	mm

Joint configuration	K
Design tables	7.2, 7.5

11.2. Validation tests

11.2.1. Redistribution of bending moment caused by eccentricity of brace members

As specified in EN 1993-1-8 Article 5.1.5 (6), when brace **eccentricities are within the limits given in 5.1.5(5)**, the moments resulting from the eccentricities should be taken into account in the design of **compression chord members**.

In SCIA Engineer the redistribution for such case is done in several steps:

- The moment caused by eccentricity of braces is calculated as:
  $M_{y,0,ecc} = \sum N_i * \cos\theta_i * e$
  With
  N_i design normal force in the brace
  θ_i angle between the brace and chord members
  e brace eccentricity
- The chord sides in compression are identified. The side is recognized to be in compression if the formula below produces a positive result. The internal forces from the respective section closest to the the joint node are used in the formula.
  $$\sigma = \frac{N_{Ed}}{A_0} + \frac{|M_{y,0,Ed}|}{W_{el,y,0}} + \frac{|M_{z,0,Ed}|}{W_{el,z,0}}$$
  With
  N_{Ed} design normal force in the chord at a joint
  A_0 chord cross-section area
  $M_{y,0,Ed}$ design bending moment around y-axis in the chord at a joint
  $W_{el,y,0}$ elastic section modulus of chord cross-section around y-axis
  $M_{z,0,Ed}$ design bending moment around z-axis in the chord at a joint
  $W_{el,z,0}$ elastic section modulus of chord cross-section around z-axis

  If no chord side in compression is found, no redistribution is done and a warning is displayed in the detailed the output.
- Ratio for each chord side in compression is calculated based on the number of chord sides in compression. If only one side of chord is in compression the ratio for that side is set to 1 however if two sides of chord is in compression are found, the moment caused by eccentricity of braces is equally redistributed to both sides using coefficient 0,5.

  $I_{y,I}$ moment of inertia for 1st chord side in compression
  L_i system length of the member on 1st chord side in compression
  $I_{y,(i+1)}$ moment of inertia for 2nd chord side in compression
  $L_{(i+1)}$ system length of the member on 2nd chord side in compression
- Additional moments for each of the compression sides of a chord are calculated and added to initial internal forces of the respective side. As specified by EN 1993-1-8 Article 5.1.5 (8) these additional moments, resulting in additional stresses, are taken into account in the determination of the factors k_m , k_n and k_p used in the design of the joint.

As specified in EN 1993-1-8 Article 5.1.5 (7), when the **eccentricities are outside the limits given in 5.1.5(5)**, the moments resulting from the eccentricities should be taken into account in the design of **the joints and the members**. In this case the moments produced by the eccentricity should be distributed between all the members meeting at the joint, on the basis of their relative stiffness coefficients I/L .

In SCIA Engineer for such case no redistribution is done and warning is displayed in the detailed output. The reason for this is that the redistribution of additional moment should be done in more elaborated way (FEM analysis).

11.2.2. General scope and field of application

As specified in EN 1993-1-8 Article 7.1.1 the method application is valid both for hot hollow sections to EN 10210 and for cold formed hollow sections to EN 10219. This is not checked by SCIA Engineer and is responsibility of the user to ensure that this is fulfilled.

Following items are verified by SCIA Engineer.

General criteria for the joint members given by EN 1993-1-8 Article 7.1.1:

- The nominal yield strength should not exceed 460 MPa. If yield strength higher than 355 MPa , the static design resistances should be reduced by factor 0,9.
- The nominal wall thickness of hollow sections should not be less than 2.5 mm.
- The nominal wall thickness of hollow section chord should not be greater than 25 mm unless special measures have been taken.

General field of application for joint members are given in EN 1993-1-8 Article 7.1.2:

- The compression elements of the members should satisfy the requirements for Class 1 or Class 2 given in EN 1993-1-1 for the condition of axial compression.
- The angle θ_i between the chords and the brace members, and between adjacent brace members, should be equal or bigger than 30 degrees.
- In gap joints, the gap between the brace members should not be less than (t_1+t_2). Based on TATA Steel publications also maximum limit of 12*t_0 is checked.
- In overlap type joints, the overlap should be at least 25% in order to be bale place welds.
- If the overlap is larger than 60% in case hidden seam of the overlapped brace is not welded or larger than 80% in case hidden seam of the overlapped brace is welded, a warning requesting additional shear check is displayed.
- Where overlapping brace members have different thickness and/or different strength grades, the member with the lower (t_i*$f_{y,i}$) value should overlap the other member.
- Where overlapping brace members are of different widths, the narrower member should overlap the wider one.

Not fulfilling any of the tests above will result in a displayed error after a chapter with geometry tests and termination of the check part! The exception are the tests for the yield strength of 355 MPa, eccentricity and maximum overlap limit. These exceptions will result in warning messages in the respective validation parts.

11.2.3. Range of validity for CHS chord and CHS brace members

The EN 1993-1-8 Article 7.4.1 Table 7.1 specifies an additional range of validity tests for the CHS chord joints.

Diameter ratio		$0.2 \le d_i/d_0 \le 1{,}0$
Chords	tension	$10 \le d_0/t_0 \le 50$ (generally), but:
	compression	Class 1 or 2 and $10 \le d_0/t_0 \le 50$ (generally), but:
Braces	tension	$d_i/t_i \le 50$
	compression	Class 1 or 2
Overlap		$25\% \le \lambda_{ov} \le \lambda_{ov,lim.}$, see 7.1.2 (6)
Gap		$g \ge t_1 + t_2$

All the items above are verified by SCIA Engineer.

Not fulfilling any of the tests above will result in a displayed error after a chapter with geometry tests and termination of the check part! The exception is the test for maximum overlap limit. This exceptions will result in warning message in the validation part.

In SCIA Engineer:

5. Validity checks

EN 1993-1-8 Article 5.1.5 (5)

$-0,55 * d_0 <= e <= 0,25 * d_0$

Location	Min limit[mm]	Max limit[mm]	Eccentricity[mm]	Result
Brace 1, Brace 2	-120,50	54,77	0,00	OK

$M_{y,0,ecc}$	0,00	kNm

Chord	Position[m]	Ratio[-]	$M_{y,0,ecc}$[kNm]
B115	3,000	0,50	0,00
B116	0,000	0,50	0,00

Note: Eccentricity is within the limits, therefore additional moments are taken into account for compression chord members.

EN 1993-1-8 Article 7.1.1 (4)

$f_{y,i} <= 355MPa$

$f_{y,i} <= 460MPa$

Member	Property	Value[MPa]	Warning limit Result	Max limit Result
Chord	$f_{y,0}$	355,0	-	OK
Brace 1	$f_{y,1}$	355,0	OK	OK
Brace 2	$f_{y,2}$	355,0	OK	OK

EN 1993-1-8 Article 7.1.1 (5)

$t_i >= 2.50$ mm

Member	Property	Value[mm]	Result
Chord	t_0	12,50	OK
Brace 1	t_1	5,00	OK
Brace 2	t_2	3,60	OK

EN 1993-1-8 Article 7.1.1 (5)

$t_0 <= 25.00$ mm

Member	Property	Value[mm]	Result
Chord	t_0	12,50	OK

EN 1993-1-8 Article 7.1.2 (3)

$30 \text{ deg} <= \theta_i =< 90 \text{ deg}$

Member	Property	Value[deg]	Result
Brace 1	θ_1	45,00	OK
Brace 2	θ_2	45,00	OK

EN 1993-1-8 Article 7.1.2 (5)

$t_1 + t_2 <= g <= 12 * t_0$

Location	Min limit[mm]	Max limit[mm]	Gap[mm]	Result
Brace 1, Brace 2	8,60	150,00	39,49	OK

EN 1993-1-8 Article 7.4.1 (3) Table 7.1

$0.20 <= d_i/d_0 <= 1.00$

Member	Property	Value[-]	Result
Brace 1	d_1/d_0	0,64	OK
Brace 2	d_2/d_0	0,52	OK

EN 1993-1-8 Article 7.4.1 (3) Table 7.1

$10.00 <= d_0/t_0 <= 50.00$

Class 1 or 2

Chord	Loading effect	Property	Value	Cross-section	Result
B115	Compression	d_0/t_0	17,53	OK	OK
B116	Compression	d_0/t_0	17,53	OK	OK

EN 1993-1-8 Article 7.4.1 (3) Table 7.1

Class 1 or 2

Member	Loading effect	Cross-section
Brace 1	Compression	OK

EN 1993-1-8 Article 7.4.1 (3) Table 7.1

$d_i/t_i <= 50.00$

Member	Loading effect	Property	Value[-]	Result
Brace 2	Tension	d_2/t_2	31,75	OK

11.3. Design resistance

The general criteria for steel member present in the hollow section joint are specified in EN 1993-1-8 Article 7.2.1 (1-2).

- The design values of the internal axial forces both in the brace members and in the chords at the ultimate limit state shall not exceed the design resistances of the members determined from EN 1993-1-1. In order to check this the user should run steel check.
- The design values of the internal axial forces in the brace members at the ultimate limit state shall also not exceed the design resistances of the joints given in design tables as appropriate. This is calculated and verified by the hollow section joint.

In SCIA Engineer:

6. Design resistances

γ	8,76	-
$\sigma_{p,Ed}$	123,3	MPa
n_p	0,35	-
k_p	0,86	-
$f_{y,0}$	355,0	MPa
t_0	12,50	mm
d_0	219,10	mm
k_g	1,76	-

6.1. Brace 1

Brace 1		
θ_1	45,00	deg
d_1	139,70	mm
β	0,58	-

$d_1 <= d_0 - 2.00*t_0$

Max limit[mm]	d_1[mm]	Result
194,10	139,70	OK

6.1.1. Design axial resistance

According to EN 1993-1-8 Article 7.4 Table 7.2

Chord face failure		
$N_{1,Rd}$	987,19	kN

Punching shear failure		
$N_{1,Rd}$	1919,48	kN

$N_{1,Rd}$ = 987,19 kN

6.1.2. Bending moment resistance

According to EN 1993-1-8 Article 7.4 Table 7.5

Chord face failure		
$M_{y,1,Rd}$	-	kNm
$M_{z,1,Rd}$	47,95	kNm

Punching shear failure		
$M_{y,1,Rd}$	78,03	kNm
$M_{z,1,Rd}$	92,68	kNm

$M_{y,1,Rd}$ = 78,03 kNm
$M_{z,1,Rd}$ = 47,95 kNm

6.1.3. Utilization

According to EN 1993-1-8 Article 7.4 Table 7.5

$N_{1,Ed} / N_{1,Rd}$	0,51
$M_{y,1,Ed} / M_{y,1,Rd}$	0,00
$M_{z,1,Ed} / M_{z,1,Rd}$	0,00
$N_{1,Ed} / N_{1,Rd} + [M_{y,1,Ed} / M_{y,1,Rd}]^2 + M_{z,1,Ed} / M_{z,1,Rd}$	0,51

6.2. Brace 2

Brace 2		
θ_2	45,00	deg
d_2	114,30	mm
β	0,58	-

$d_2 <= d_0 - 2.00*t_0$

Max limit[mm]	d_2[mm]	Result
194,10	114,30	OK

6.2.1. Design axial resistance

According to EN 1993-1-8 Article 7.4 Table 7.2

Chord face failure		
$N_{2,Rd}$	846,61	kN

Punching shear failure		
$N_{2,Rd}$	1570,49	kN

$N_{2,Rd}$ = 846,61 kN

6.2.2. Bending moment resistance

According to EN 1993-1-8 Article 7.4 Table 7.5

Chord face failure		
$M_{y,2,Rd}$	-	kNm
$M_{z,2,Rd}$	39,23	kNm

Punching shear failure		
$M_{y,2,Rd}$	52,24	kNm
$M_{z,2,Rd}$	62,04	kNm

$M_{y,2,Rd}$ = 52,24 kNm
$M_{z,2,Rd}$ = 39,23 kNm

6.2.3. Utilization

According to EN 1993-1-8 Article 7.4 Table 7.5

$N_{2,Ed} / N_{2,Rd}$	0,47
$M_{y,2,Ed} / M_{y,2,Rd}$	0,00
$M_{z,2,Ed} / M_{z,2,Rd}$	0,00
$N_{2,Ed} / N_{2,Rd} + [M_{y,2,Ed} / M_{y,2,Rd}]^2 + M_{z,2,Ed} / M_{z,2,Rd}$	0,47

7. Results

Member	Unity check
Brace 1	**0,51**
Brace 2	0,47

11.3.1. Uniplanar joints

EN 1993-1-8 Article 7.4.2 (1) specifies that in brace member connections subject only to axial forces, the design internal axial force $N_{i,Ed}$ shall not exceed the design axial resistance of the welded joint $N_{i,Rd}$ obtained from appropriate design table.

EN 1993-1-8 Article 7.4.2 (2) specifies that brace member connections subject to combined bending and axial force should satisfy:

$$\frac{N_{i,Ed}}{N_{i,Rd}} + \left[\frac{M_{y,i,Ed}}{M_{y,i,Rd}}\right]^2 + \frac{|M_{z,i,Ed}|}{M_{z,i,Rd}} \leq 1$$

With:

- $N_{i,Ed}$ design axial force
- $N_{i,Rd}$ design axial force resistance
- $M_{y,i,Ed}$ design in-plane bending moment
- $M_{y,i,Rd}$ design in-plane bending moment resistance
- $M_{z,i,Ed}$ design out-of-plane bending moment
- $M_{z,i,Rd}$ design out-of-plane bending moment resistance

11.3.2. Chord stress

For the calculation of the stresses in chord member, used by the joint design routine, the extended formulas to those given in EN 1993-1-8 Article 7.2.1 (3) are used in SCIA Engineer:

$$\sigma_{p,Ed} = \frac{N_{p,Ed}}{A_0} + \frac{|M_{y,0,Ed}|}{W_{el,y,0}} + \frac{|M_{z,0,Ed}|}{W_{el,z,0}}$$

$$\sigma_{0,Ed} = \frac{N_{0,Ed}}{A_0} + \frac{|M_{y,0,Ed}|}{W_{el,y,0}} + \frac{|M_{z,0,Ed}|}{W_{el,z,0}}$$

With:

- $N_{p,Ed}$ maximum compressive normal force in the chord at a joint excluding the stress due to the components parallel to the chord axis of the axial forces in the braces at that joint
- $N_{0,Ed}$ maximum compressive normal force in the chord at a joint
- A_0 chord cross-section area
- $M_{y,0,Ed}$ design bending moment around y-axis in the chord at a joint
- $W_{el,y,0}$ elastic section modulus of chord cross-section around y-axis
- $M_{z,0,Ed}$ design bending moment around z-axis in the chord at a joint
- $W_{el,z,0}$ elastic section modulus of chord cross-section around z-axis

For the CHS-CHS hollow section joints the set of internal forces used in calculation of $\sigma_{p,Ed}$ is presented in the chapter of internal forces in the detailed output.

11.3.3. Identification of brace members

Integer subscript i for joint members are described by EN 1993-1-8 Article 1.5 (4), where subscript i = 0 denotes a chord, subscripts i = 1, 2, 3 and 4 denote brace members.

In joints with two brace members, i = 1 normally denotes the compression brace and i = 2 the tension brace. For a single brace i = 1 whether it is subject to compression or tension.

The above also applies to SCIA Engineer with several extensions for:

- X and DY joints, where the first brace in the list uses subscript i = 1, the second brace in the list uses subscript i = 2
- DK joint with all members in either tension or compression, where the first brace in the list uses subscript i = 1, brace on the same side of chord as brace 1 uses i = 2, brace opposing to brace 1 uses i = 3 and brace opposing to brace 2 uses i = 4
- DK joint with tension or compression in opposing braces, where the first compression brace in the list uses subscript i = 1, brace on the same side of chord as brace 1 uses i = 2, brace opposing to brace 1 uses i = 3 and brace opposing to brace 2 uses i = 4

As noted above, in some joint configurations the identification of braces depends on the actual internal forces present in the members. Following the specifications of EN 1993-1-8 Article 7.4.2 Table 7.6 certain combinations of internal forces may not be designable according to EN 1993-1-8 and will result in unity check of 999. This also applies to K and N joint configurations when loading effect on both braces is both tensile or compressive.

If zero internal forces are found on both braces in a K or N joint configurations for a certain combination, such combination will be evaluated with zero unity check and a note will be displayed in the detailed output.

11.3.4. Welded joints between CHS members

Provided that the geometry of the joints is within the range of validity given in EN 1993-1-8 Article 7.4.1 Table 7.1, the design resistances of uniplanar welded joints between circular hollow section members may be determined using EN 1993-1-8 Article 7.4.2. For joints within the range of validity given in Table 7.1, only chord face failure and punching shear need be considered. The design resistance of a connection should be taken as the minimum value for these two failure modes.

11.3.4.1. Axial force resistance

Design axial resistances of welded joints between CHS brace members and CHS chords is calculated according to EN 1993-1-8 Article 7.4.2 Table 7.2 and Table 7.6.

11.3.4.2. Bending moment resistance

Design resistance moments of welded joints between CHS brace members and CHS chords is calculated according to EN 1993-1-8 Article 7.4.2 Table 7.5.

11.3.5. Special types of welded joints

As specified by EN 1993-1-8 Article 7.4.2 (5), the special types of welded joints indicated in Table 7.6 should satisfy the appropriate design criteria specified for each type in that table.

Two combinations of internal forces are supported for DK joint configurations:

- All bracing members should always be in either compression or tension. Condition given by the table was modified and an additional joint unity check is calculated as.

$$\frac{|N_{1-3,Ed}| * \sin\theta_{1-3}}{N_{x,Rd} * \sin\theta_x} + \frac{|N_{2-4,Ed}| * \sin\theta_{2-4}}{N_{x,Rd} * \sin\theta_x} \leq 1$$

  With:

  - $N_{1\text{-}3,Ed}$ average value of design normal force in brace 1 and brace 3
  - $\theta_{1\text{-}3}$ angle between the brace and chord member for brace 1 and brace 3
  - $N_{2\text{-}4,Ed}$ average value of design normal force in brace 2 and brace 4
  - $\theta_{2\text{-}4}$ angle between the brace and chord member for brace 2 and brace 4
  - $N_{x,Rd}*\sin\theta_x$ maximum vertical component of brace resistance determined from all braces

- Opposing braces either in tension or compression. An additional joint unity check is calculated for cases when there are gaps at both sides of the chord as.

$$\left(\frac{N_{0,Ed}}{N_{pl,0,Rd}}\right)^2 + \left(\frac{V_{0,Ed}}{V_{pl,0,Rd}}\right)^2 \leq 1$$

$$N_{0,Ed} = \frac{N_{left,Ed} + N_{right,Ed}}{2}$$

$$N_{pl,0,Rd} = \frac{f_{y,0} * A_0}{\gamma_{M5}}$$

$$V_{0,Ed} = Max(|N_{1-3,Ed}| * \sin\theta_{1-3}; |N_{2-4,Ed}| * \sin\theta_{2-4})$$

$$V_{pl,0,Rd} = \frac{f_{y,0} * A_{v,0}}{\sqrt{3} * \gamma_{M5}}$$

$$A_{v,0} = \frac{2 * A_0}{\pi}$$

  With:

  - $N_{0,Ed}$ design normal force in the section
  - $N_{left,Ed}$ design normal force in the chord from the left section at the joint
  - $N_{right,Ed}$ design normal force in the chord from the right section at the joint
  - $N_{pl,0,Rd}$ plastic normal force resistance of the chord
  - $f_{y,0}$ yield strength of the chord material
  - A_0 chord cross-section area
  - γ_{M5} partial safety factor for the resistance of joints
  - $V_{0,Ed}$ design shear force in the section
  - $N_{1\text{-}3,Ed}$ average value of design normal force in brace 1 and brace 3
  - $\theta_{1\text{-}3}$ angle between the brace and chord member for brace 1 and brace 3

- $N_{2\text{-}4,Ed}$ average value of design normal force in brace 2 and brace 4
- $\theta_{2\text{-}4}$ angle between the brace and chord member for brace 2 and brace 4
- $V_{pl,0,Rd}$ plastic shear force resistance of the chord
- $A_{v,0}$ chord cross-section shear area

11.3.6. Design of welds

Calculation of weld size is based on the Ref. [35] - ECCS N° 126, where the formula is derived for double fillet weld. Since only one weld will be present on the brace member, the result value has to be multiplied by 2. The final formula for weld size used for each brace is derived as:

$$a \geq 2 \cdot \frac{f_y \cdot \beta_w \cdot \gamma_{M2}}{\sqrt{2} \cdot f_u \cdot \gamma_{M0}} \cdot t$$

With:

- f_y yield strength of the brace material
- f_u ultimate tensile strength of the brace material
- β_w correlation factor given by EN 1993-1-8 Article 4.5.3.2 Table 4.1
- γ_{M0} partial safety factor for material
- γ_{M2} partial safety factor for welds
- t thickness of the brace

The weld size is calculated separately for all braces, is rounded up and compared with the minimum value from the setup. The final weld size is maximum of these two values.

In SCIA Engineer:

8. Welds

Member	t[mm]	f_y[MPa]	f_u[MPa]	β_w	a_{calc}[mm]	a_{min}[mm]	a[mm]
Brace 1	5,00	355,0	490,0	0,90	2,88	3,00	3,00
Brace 2	3,60	355,0	490,0	0,90	2,07	3,00	3,00

12.References and literature

[1] Eurocode 3
Design of steel structures
Part 1 - 8 : Design of joints
EN 1993-1-8: 2005

[2] SCIA Engineer Steel_Connections_Theory_enu.pdf
Theoretical Background of SCIA Engineer
Release : 13.0