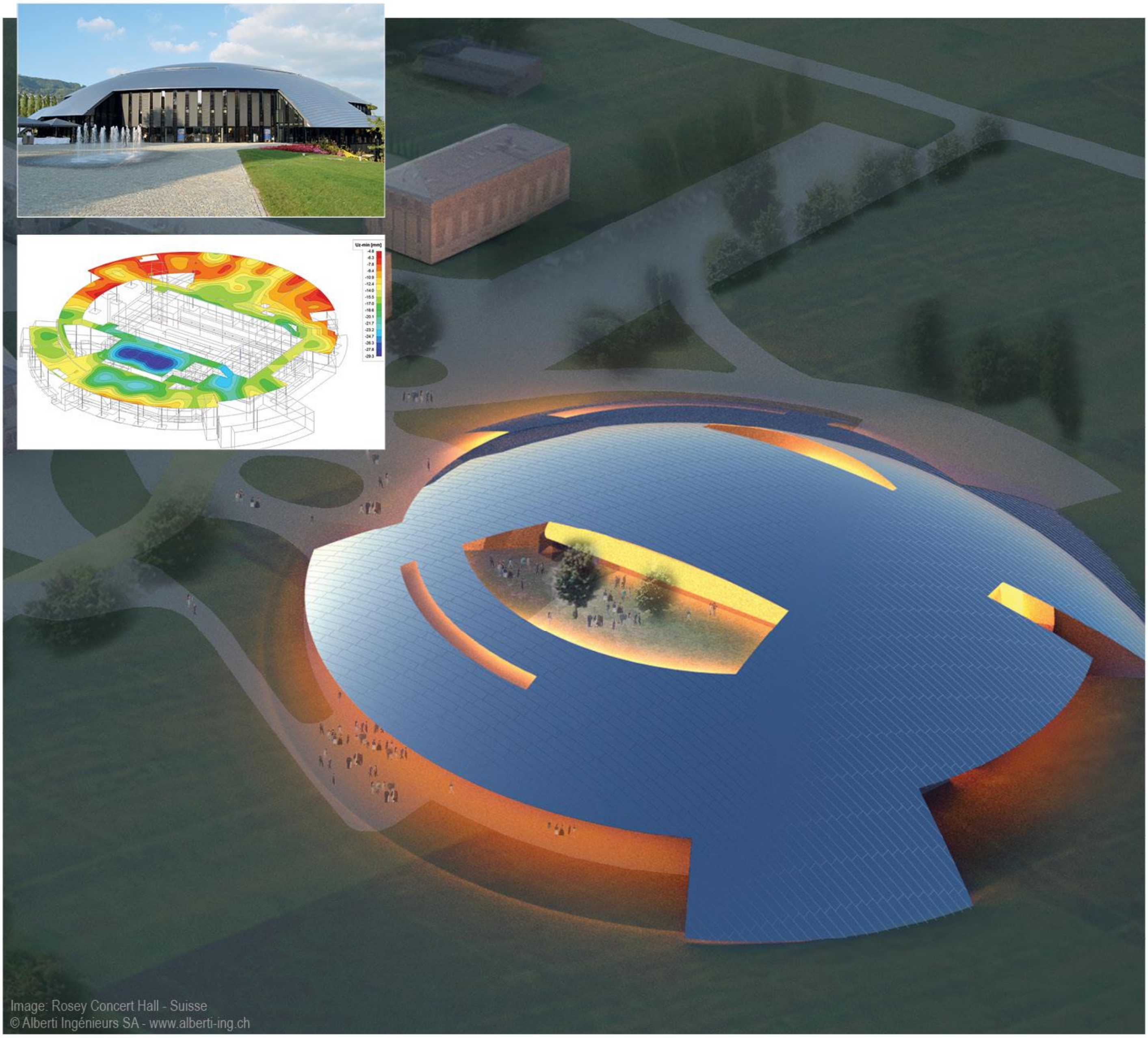
SCIA ENGINEER



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 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]).

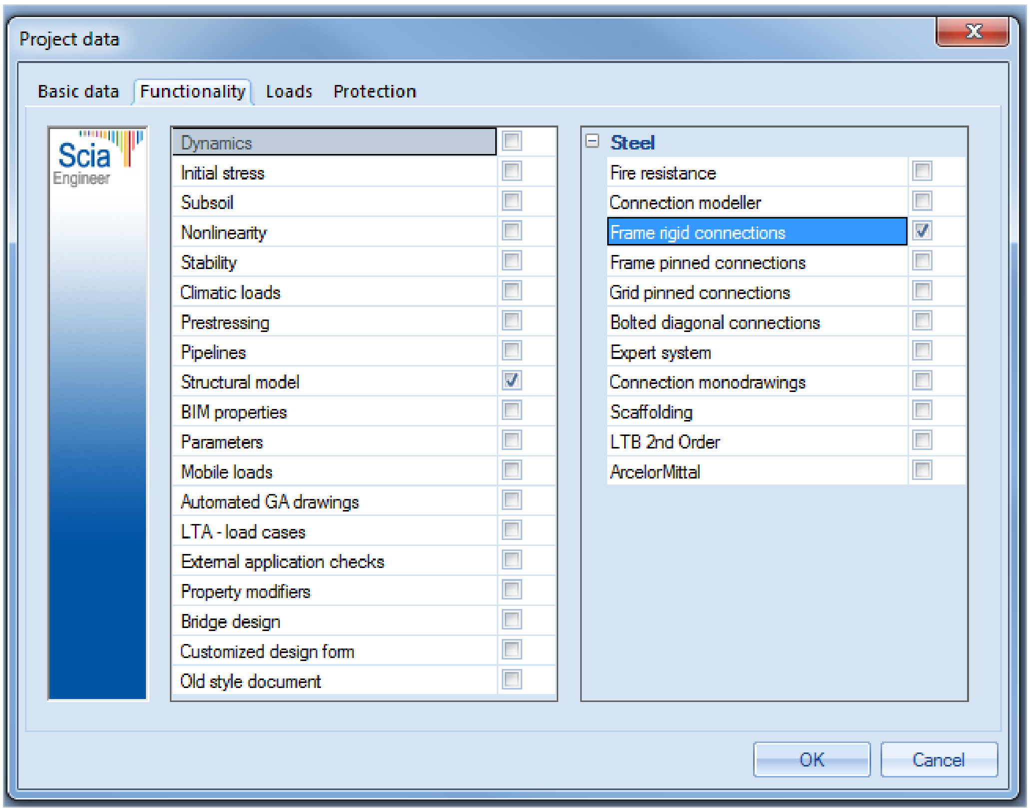
The following chapters are valid for the bolted and welded column-beam joints. The design methods for the beam-column joints are principally for moment-resisting joints between I or H sections in which the beams are connected to the flanges of the column. In this document we will describe the total procedure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

2. Creation of a small example in SCIA Engineer

2.1. Modeling 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.

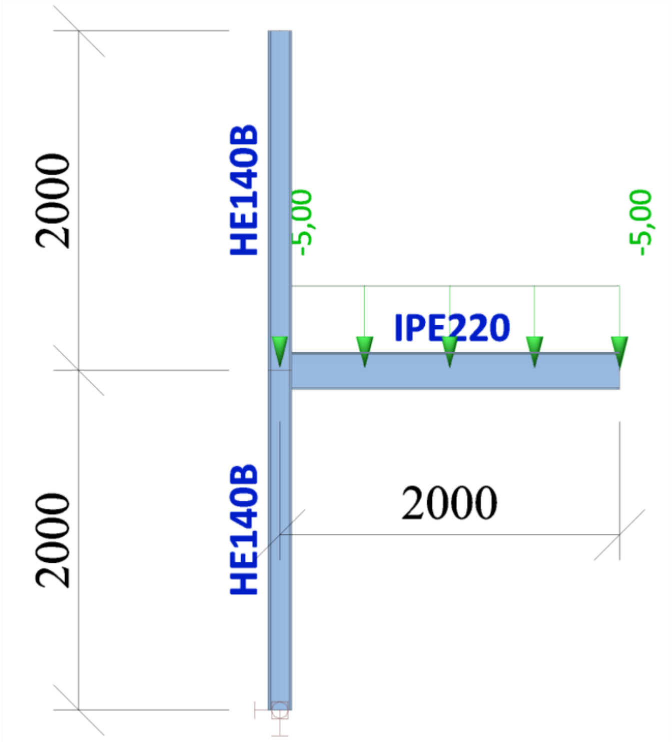
Create a new project in **Frame XYZ**, activate the material **Steel S235** and activate the **functionality Frame rigid connections**.



The following options are available for connections:

Connection Modeller:	With this option you can only model a connection and not run a calculation of a connection. If you want to calculate the connection, don't activate this functionality!
Frame rigid connections:	Calculation of bolted and welded (rigid and semi-rigid) connections.
Fame pinned connections:	Calculation of pinned connections
Grid pinned connections:	Calculation of pinned connection in the horizontal plane
Bolted diagonal connections:	Calculation of bolted diagonals
Expert system:	Use a library with default connections in SCIA Engineer or add your own connections to this library
Connection monodrawings:	Make some nice 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).

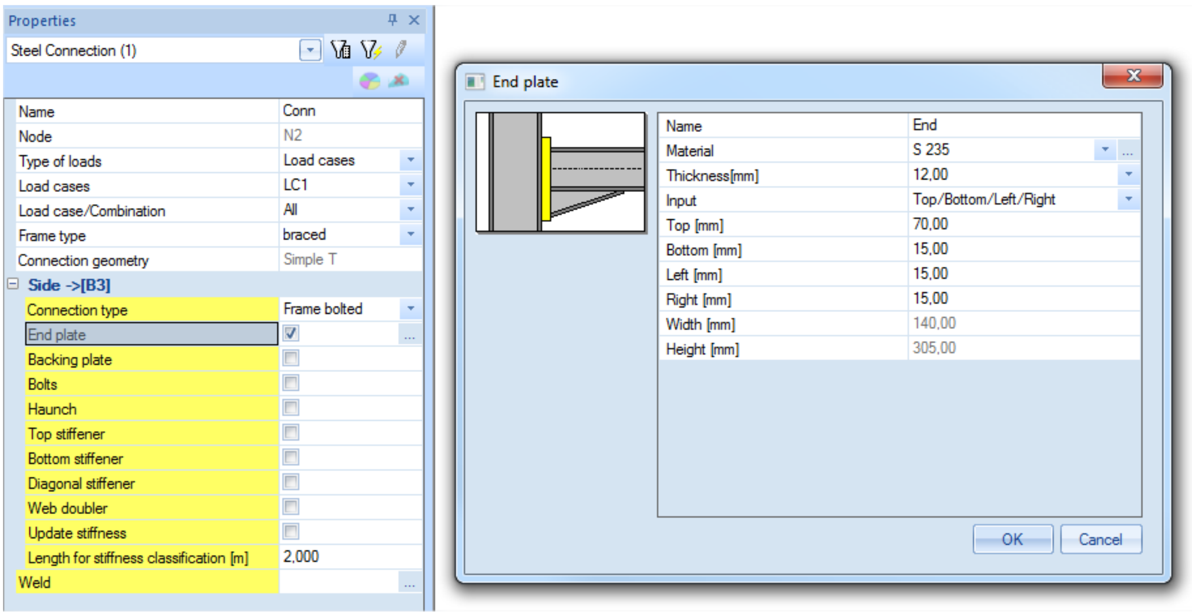


2.2. *Input of the connection*

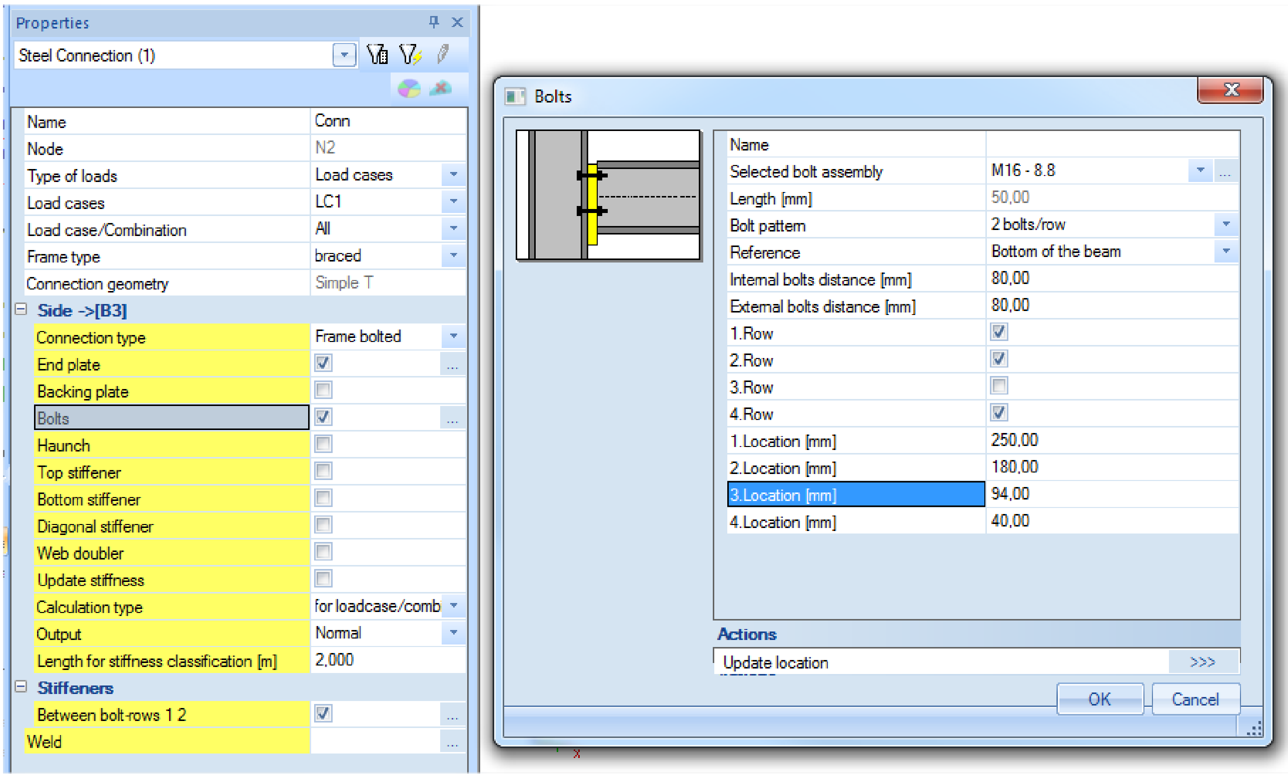
Calculate the model and go to the **Steel** menu.

The beam is connected with the strong axis of the column, so we choose in this menu for “Connections -> Frame bolted/welded-strong axis”. Double-click on this option and select the node between the column and the beam to input the connection.

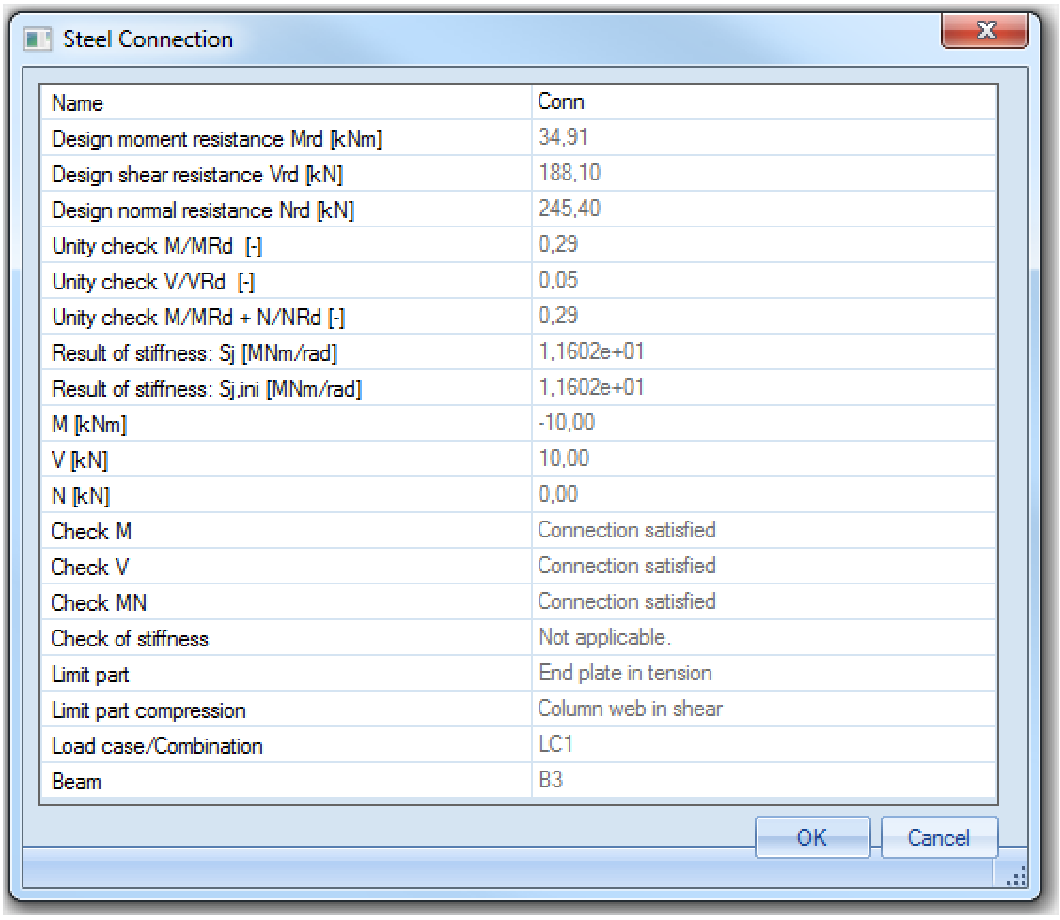
In the properties window of the connection, you can activate what you want to add at the connection. We choose for a **Frame bolted** connection and we add an end plate. By clicking on the three dots behind the endplate option, you can adapt the endplate and we change it into:



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



To check the connection, you have to click on refresh.
With the option “Results” you can have a short output of the connection:



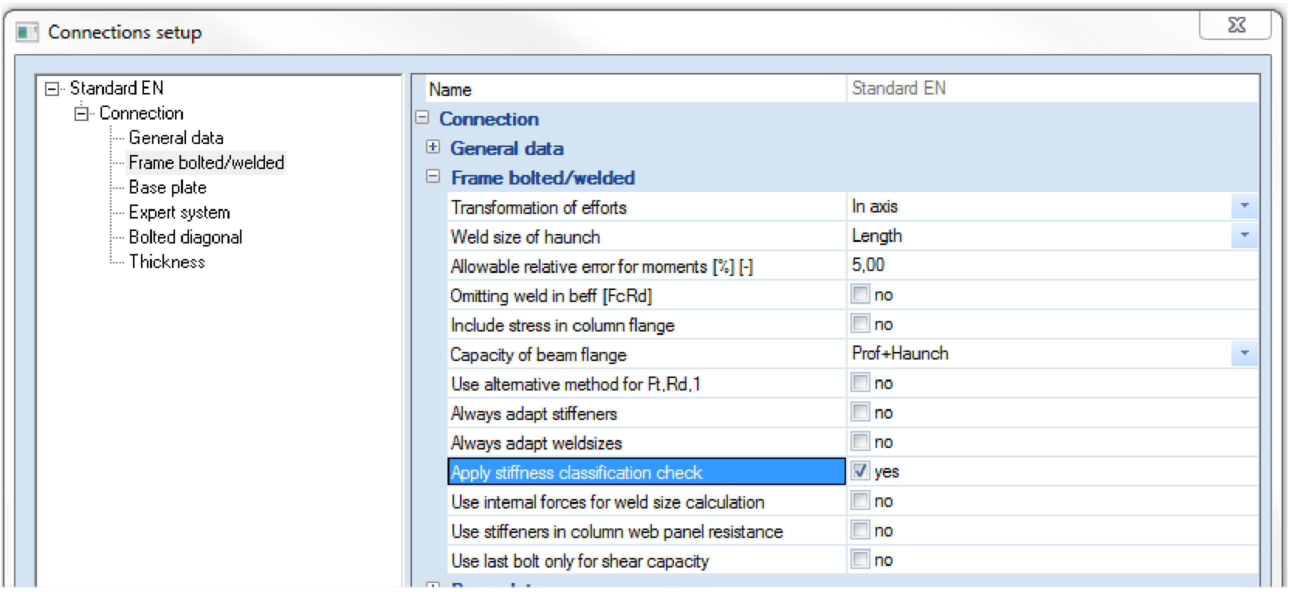
Name	Conn
Design moment resistance Mrd [kNm]	34,91
Design shear resistance Vrd [kN]	188,10
Design normal resistance Nrd [kN]	245,40
Unity check M/MRd [-]	0,29
Unity check V/VRd [-]	0,05
Unity check M/MRd + N/NRd [-]	0,29
Result of stiffness: Sj [MNm/rad]	1,1602e+01
Result of stiffness: Sj,ini [MNm/rad]	1,1602e+01
M [kNm]	-10,00
V [kN]	10,00
N [kN]	0,00
Check M	Connection satisfied
Check V	Connection satisfied
Check MN	Connection satisfied
Check of stiffness	Not applicable.
Limit part	End plate in tension
Limit part compression	Column web in shear
Load case/Combination	LC1
Beam	B3

Summary of the results shown in this window:

Name	Name of the connection in SCIA Engineer
Design moment resistance Mrd [kNm]	Moment resistance of the connection
Design moment resistance Vrd [kN]	Shear resistance of the connection
Design moment resistance Nrd [kN]	Normal force resistance of the connection
Unity check M/MRd	Unity check of the moment
Unity check V/VRd	Unity check of the shear force
Unity check M/MRd + N/NRd	Unity check of the moment and normal force
Result of stiffness: Sj [MNm/rad]	Stiffness of the connection at the moment Med
Result of stiffness: Sj,ini [MNm/rad]	Stiffness of the connection for small moments
M [kNm]	MEd of the connection
V [kN]	VEd of the connection
N [kN]	NEd of the connection
Check M	Shows if the unity check for the moment check will not exceed 1.00
Check V	Shows if the unity check for the shear force check will not exceed 1.00
Check MN	Shows if the unity check for the normal force check will not exceed 1.00
Check of stiffness	Checks if the stiffness taken into account in the calculation is between the boundaries of the real stiffness of the connection.
Limit part	Shows the limiting part for the tension component
Limit part compression	Shows the limiting part for the compression component
Beam	The check is done for the beam, shown here.

2.3. Check and update of stiffness

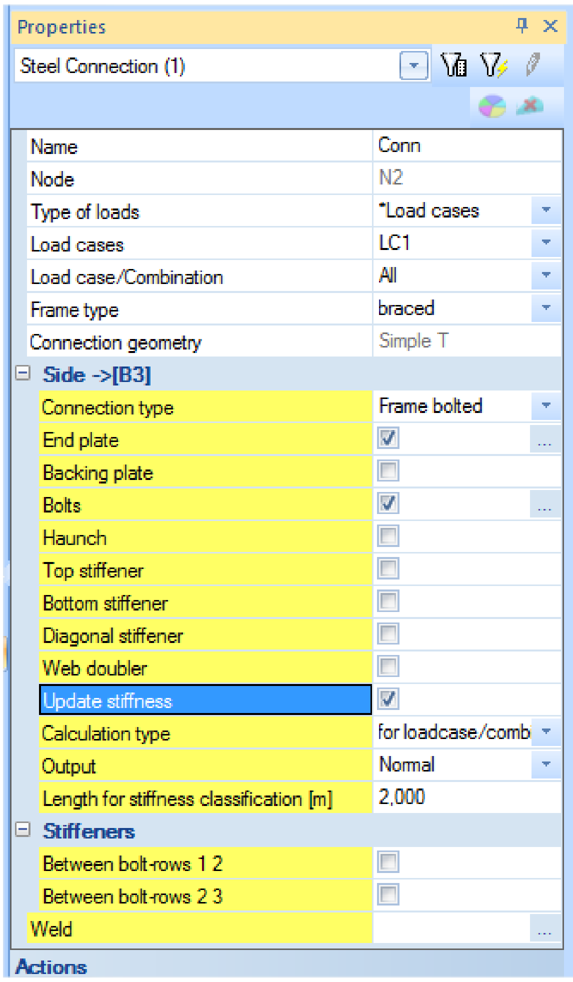
The check of stiffness is shown here as “Not applicable” because this check is not activated. You can activate the stiffness check under the chapter “Connections Setup -> Frame bolted/welded -> Apply stiffness classification check”



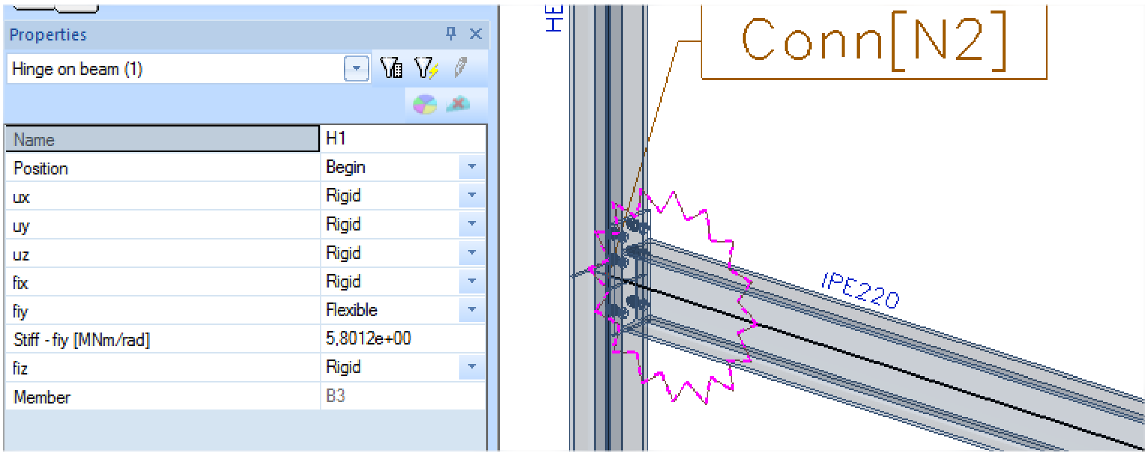
With this check you can check if the stiffness taken into account in the calculation is between the boundaries of the real stiffness of the connection.

More information about the check of the stiffness will be given further in this document.

To take into account the real stiffness of the connection in the calculation, you can activate the option “Update stiffness” in the properties window of the connection.

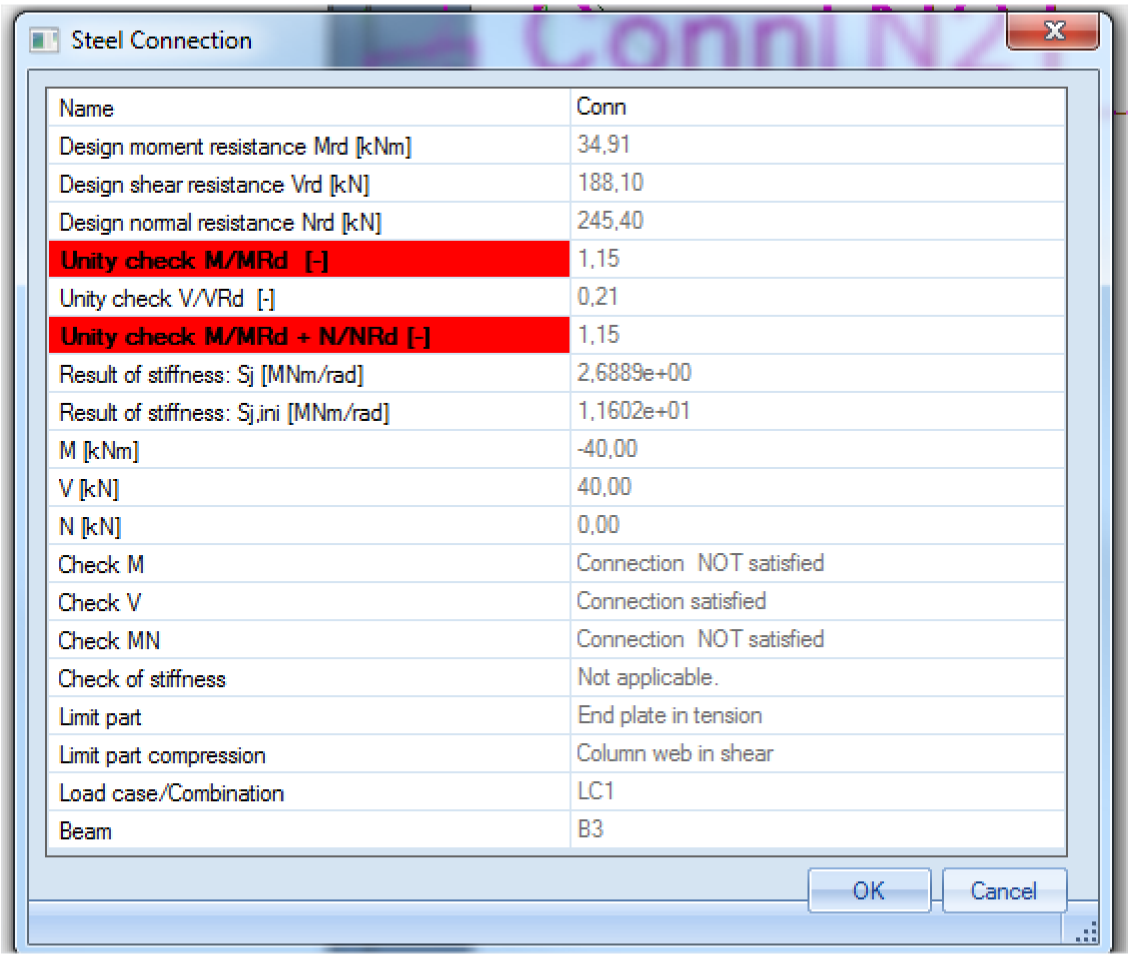


If this option is activated, a hinge will be input on the connection with the stiffness of the connection after reculating the project.



2.4. Connection does not satisfy the checks

If the check is not okay, this will be indicated by a red color as. When changing the load from 5 kN/m to 20 kN/m, the following result will be shown for this same connection:



Name	Conn
Design moment resistance Mrd [kNm]	34,91
Design shear resistance Vrd [kN]	188,10
Design normal resistance Nrd [kN]	245,40
Unity check M/MRd [-]	1,15
Unity check V/VRd [-]	0,21
Unity check M/MRd + N/NRd [-]	1,15
Result of stiffness: Sj [MNm/rad]	2,6889e+00
Result of stiffness: Sj,ini [MNm/rad]	1,1602e+01
M [kNm]	-40,00
V [kN]	40,00
N [kN]	0,00
Check M	Connection NOT satisfied
Check V	Connection satisfied
Check MN	Connection NOT satisfied
Check of stiffness	Not applicable.
Limit part	End plate in tension
Limit part compression	Column web in shear
Load case/Combination	LC1
Beam	B3

In this case the Unity check for the moment and for the moment + normal force is not okay.

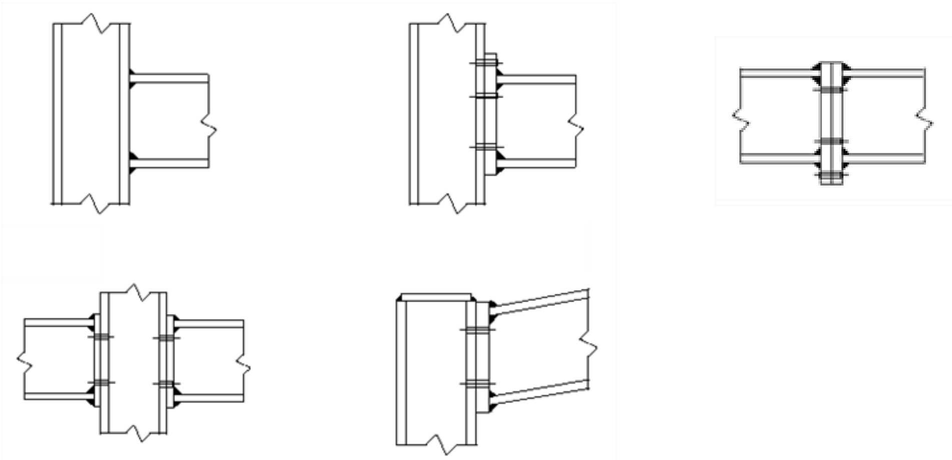
If the connection does not satisfy the check, you can see the limiting parts. So for this connection we have to adapt or the endplate (for the tension side) or we have to stiffen the column web (for the compression checks).

I will change the load again to 5 kN/m afterwards, because all checks are explained in the next chapter using this example.

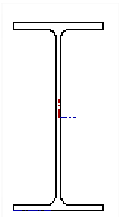
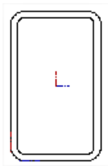
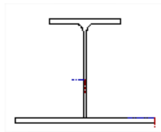
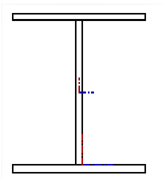
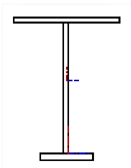
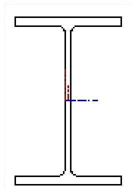
3. Possible connections in SCIA Engineer

The design methods for the column-beam joints are taken from EN 1933-1-8. More detailed information about the applied rules and specific implementations are found in Ref.[1].

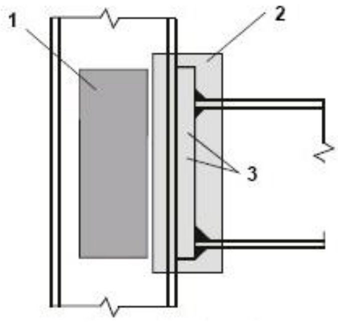
The following column-beam and beam-beam connections are possible in SCIA Engineer:



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

Rolled I beam 	RHS – Rolled hollow section 	Built up I setion (made of a flat and T section) 
Symmetrical welded I section (made of three flats) 	Asymmetrical welded I section (made of three flats) 	I section with a haunch (elements with variable height) 

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

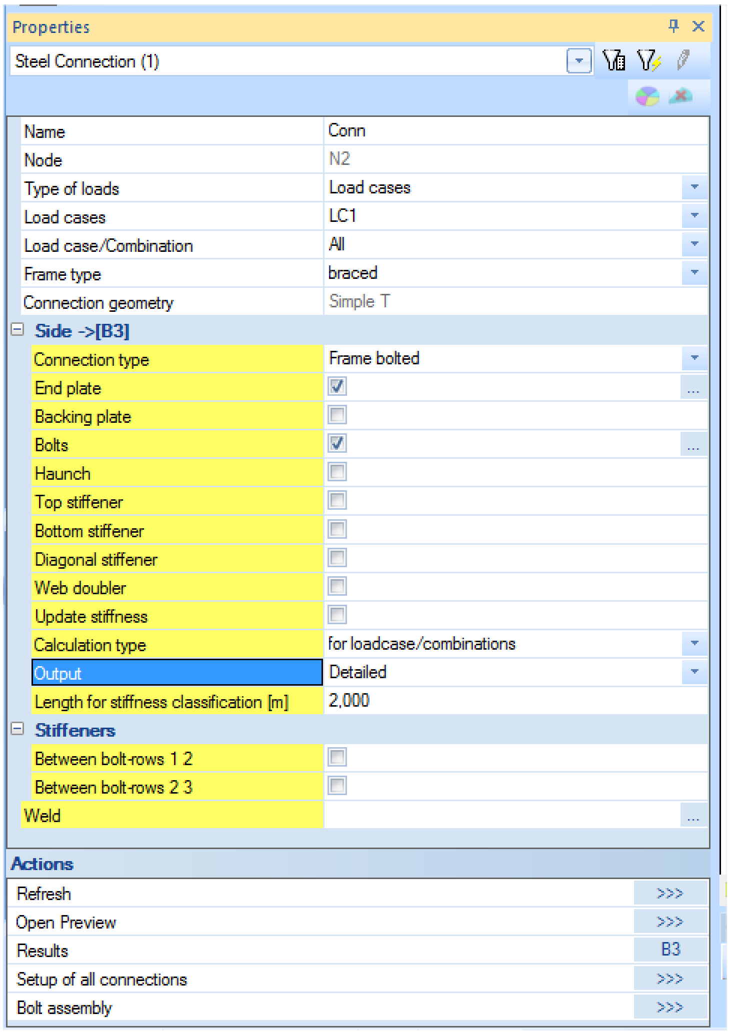
2 - connection

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

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].

4.1. General data

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

Name	Conn								
Node	N2								
Connection type	Frame bolted								
Connection geometry	Simple T								
Connected beams	Name	CrossSection	Length [m]	Shape	Beg. node	End node	Type	FEM type	Layer
	B1	CS1 - HE140B	2,000	Line	N1	N2	column (100)	standard	Layer1
	B2	CS1 - HE140B	2,000	Line	N2	N3	column (100)	standard	Layer1
	B3	CS2 - IPE220	2,000	Line	N2	N4	beam (80)	standard	Layer1

Type Name		Bolts
Internal bolts distance [mm]		80,00
External bolts distance [mm]		80,00
Selected bolt assembly		M16 - 8.8
Length [mm]		50,00
1.Location [mm]		250,00
2.Location [mm]		180,00
4.Location [mm]		40,00
Bolt pattern		2 bolts/row
Bolt assembly	Type Name	Bolt assembly
	Name	M16 - 8.8
	Selected bolt	M16 - ISO 4017
	Selected washer	M16 - ISO 7089
	Selected nut	M16 - ISO 4034
	Washer at the nut	✓
	Washer at the head	×
	Nut grade	8
	Bolt grade	8.8
	Ultimate tensile strength [N/mm^2]	800,000
	Type	Normal
	Delta [mm]	7,00
Bolts	Type Name	Bolts
	Name	M16 - ISO 4017
	Bore hole [mm]	18,00
	Diameter of wrench required for bolt [mm]	80,00
	Head diameter [mm]	24,00
	Diagonal head diameter [mm]	26,75
	Head height [mm]	10,00
	Gross cross-section area [mm^2]	201,00
	Tensile stress area [mm^2]	157,00
	Diameter [mm]	16,00
Nut	Type Name	Nut
	Name	M16 - ISO 4034
	Diameter [mm]	24,00
	Diagonal diameter [mm]	26,20
	Height [mm]	13,00
Washer	Type Name	Washer
	Name	M16 - ISO 7089
	Internal diameter [mm]	17,00
	External diameter [mm]	30,00
	Thickness [mm]	3,30

Name	End
Material	S 235
Thickness[mm]	12,00
Top [mm]	70,00
Bottom [mm]	15,00
Left [mm]	15,00
Right [mm]	15,00
Width [mm]	140,00
Height [mm]	305,00

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

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		
N	0.00	kN
Vz	10.00	kN
My	-10.00	kNm

Tension top

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, 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.

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:

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

Shear area of the column:

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

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

In SCIA Engineer:

2.1. Design resistance of basic components

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

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

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

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

Above the bottom flange, there is sufficient room to allow 45° dispersion
Below the bottom flange, there is NOT sufficient room. Thus the dispersion is limited.

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

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

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

$k_{wc} = 1$

$$\boldsymbol{F_{c,wc,Rd}} = \frac{\omega \cdot k_{wc} \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M0}} = \frac{0{,}71 \cdot 1 \cdot 162{,}27 \cdot 7 \cdot 235 \cdot 10^{-3}}{1} = \mathbf{190{,}7\ \boldsymbol{kN}}$$

In SCIA Engineer:

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

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

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): $\boldsymbol{F_{c,fb,Rd}} = \frac{M_{c,Rd}}{(h-t_{fb})} = \frac{W_{pl} \cdot f_{yb}}{\gamma_{M0} \cdot (h-t_{fb})} = \frac{285 \cdot 10^3 \cdot 235 \cdot 10^{-3}}{1 \cdot (220-9{,}2)} = \boldsymbol{317{,}7\ kN}$

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

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

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

In SCIA Engineer:

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

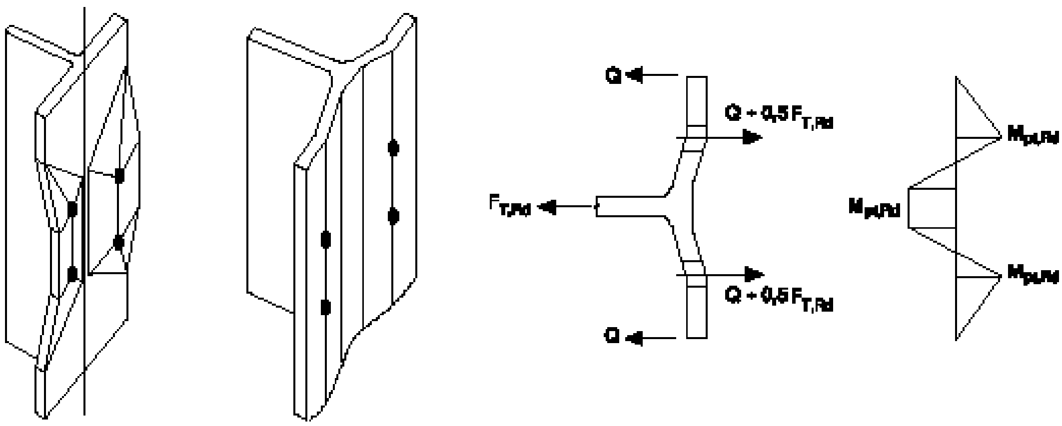
Beam flange in compression (Fc,fb,Rd) data		
Beam flange in compression (Fc,fb,Rd)	317.72	kN
section class	1	
Mc,Rd	66.98	kNm
hb-tfb	210.80	mm

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:

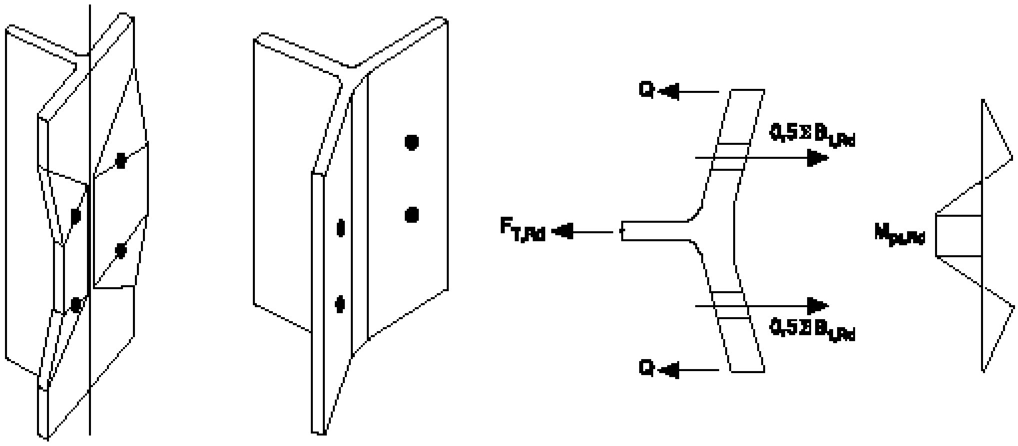
1. **complete flange yielding**



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

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

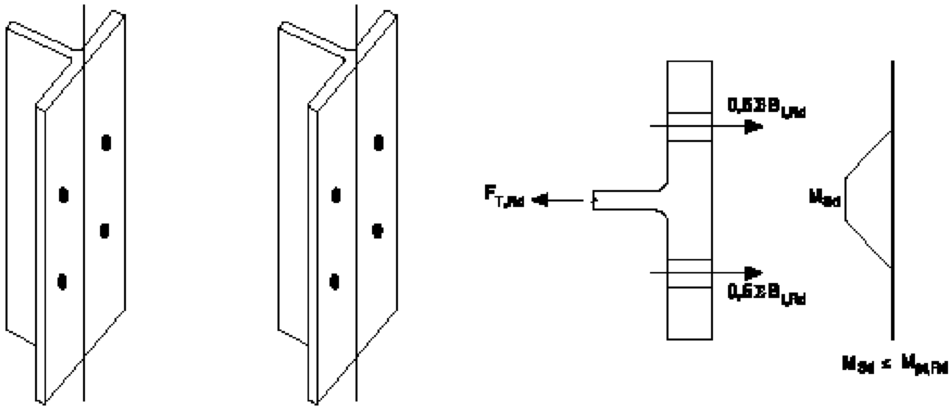
2. bolt failure with flange yielding



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

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

3. bolt failure



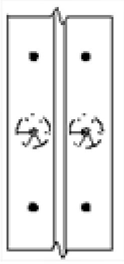
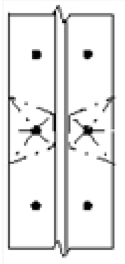
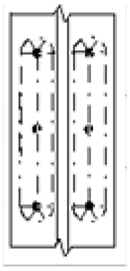
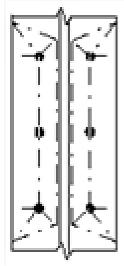
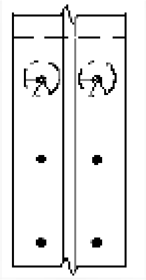
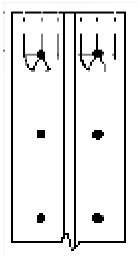
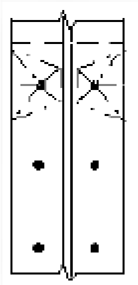
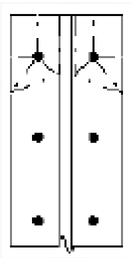
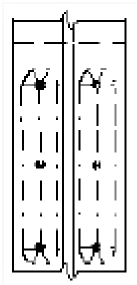
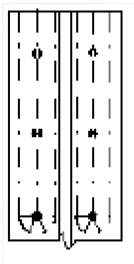
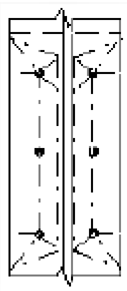
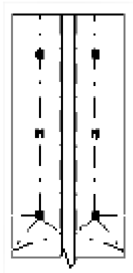
The bolts brake. But there is no influence on the column flange (or end plate).

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

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. $L_b \leq L_b^*$		No prying forces
Mode 1	Method 1	Method 2 (alternative method)	$F_{T,1\text{-}2,Rd} = \frac{2M_{pl,1,Rd}}{m}$
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:

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

2.1.4. Design tension resistance of bolt-row

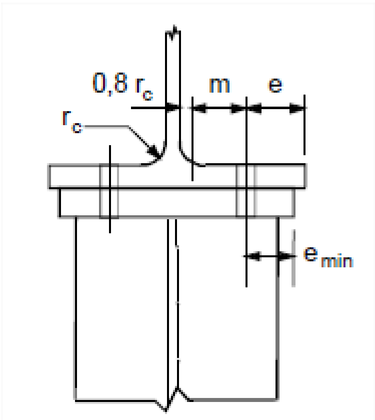
Ft,Rd data		
fub	800.00	MPa
As	157.00	mm^2
k2	0.90	-
Ft,Rd	90.43	kN
Lb	38.80	mm

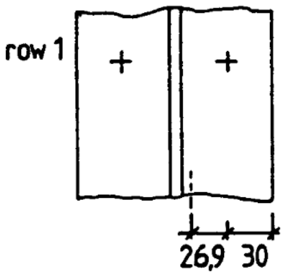
Note: The bolt-rows are numbered starting from the bolt-row farthest from the centre of compression as specified in EN 1993-1-8 Article 6.2.7.2 (1).

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:





$\boldsymbol{e = 30\ mm}$

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

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

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

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

$e_{min} = 30\ mm$

$n = e_{min}$

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

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

$\boldsymbol{n = 30mm}$

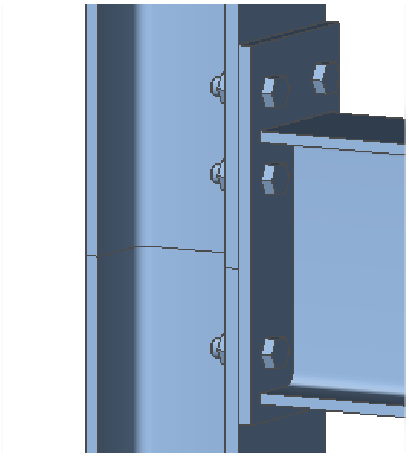
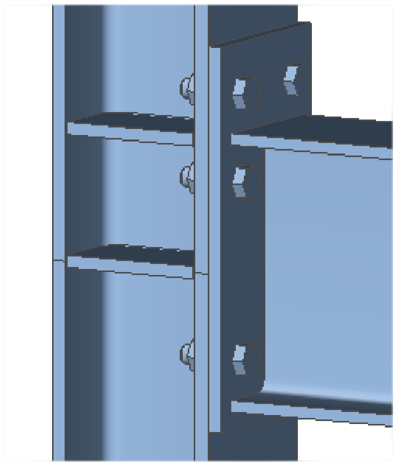
Row	p (p_1 + p_2)
1	0.0 + 35.0
2	35.0 + 70.0
3	70.0 + 0.0

And this is also shown in SCIA Engineer:

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

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

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

Row 1 and Row 3: End bolt-row
Row 2: 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	Bolt-row location
1	Other end bolt-row
2	Other inner bolt-row
3	Other end bolt-row

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

The calculation of leff 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])

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

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 $L_b \leq L_b^*$
A is the tensile stress area of the bolt A_s

$L_b^* = \frac{8{,}8\, m^3 A_s}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8{,}8\,(26{,}9)^3 \cdot 157}{145{,}10 \cdot (12)^3} \cdot 1 = 107\ mm$ (see formula in Table 6.2 of EN 1993-1-8 (Ref.[1]))

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

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

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

⇨ $\boldsymbol{F_{T,fc,Rd} = 138{,}5\ kN}$(smalles of the three modes)

All those results are shown in SCIA Engineer:

For individual bolt-row :

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

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

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

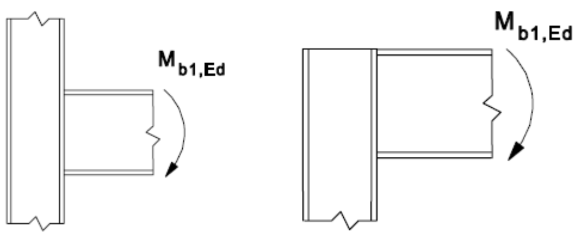
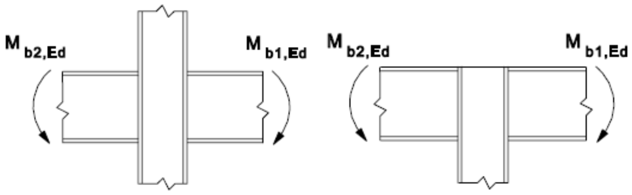
And ω, to allow for the possible effects of shear in the column web panel, should be determined from Table 6.3 (EN 1993-1-8):

Table 6.3: Reduction factor ω for interaction with shear

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

And:

Table 5.4: Approximate values for the transformation parameter β

<table>
<tr><th>Type of joint configuration</th><th>Action</th><th>Value of β</th></tr>
<tr><td></td><td>$M_{b1,Ed}$</td><td>$\beta \approx 1$</td></tr>
<tr><td rowspan="4"></td><td>$M_{b1,Ed} = M_{b2,Ed}$</td><td>$\beta = 0$ *)</td></tr>
<tr><td>$M_{b1,Ed} / M_{b2,Ed} > 0$</td><td>$\beta \approx 1$</td></tr>
<tr><td>$M_{b1,Ed} / M_{b2,Ed} < 0$</td><td>$\beta \approx 2$</td></tr>
<tr><td>$M_{b1,Ed} + M_{b2,Ed} = 0$</td><td>$\beta \approx 2$</td></tr>
<tr><td colspan="3">*) In this case the value of β is the exact value rather than an approximation.</td></tr>
</table>

In this example:
β = 1
ω = ω1

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

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

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

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

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

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

In SCIA Engineer:

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

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	-	-
3	-	-	224.51	142.55	-	-

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

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

Row 1-2:
$\sum l_{eff,cp} = 154.10 + 154.50 = 309.02$
$\sum l_{eff,nc} = 107.55 + 107.55 = 215.10$
Mode 1 = Mode 2 : l_{eff} = 215.10

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

Row 1-3:
$\sum l_{eff,cp} = 154.51 + 210.00 + 224.51 = 589.02$
$\sum l_{eff,nc} = 107.55 + 105.00 + 142.55 = 355.10$
Mode 1 = Mode 2 : l_{eff} = 355.10

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

Prying forces may develop if $L_b \leq L_b^*$
L_b = 38,8mm

Row 1-2:
$$L_b^* = \frac{8{,}8\ m^3 A_S}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8{,}8\ (26{,}9)^3 \cdot 157}{215{,}10 \cdot (12)^3} \cdot 2 = 145\ mm$$
(with n_b = number of bolt rows)

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

Row 1-3:
$$L_b^* = \frac{8{,}8\ m^3 A_S}{\sum l_{eff} t_f^3} \cdot n_b = \frac{8{,}8\ (26{,}9)^3 \cdot 157}{355{,}10 \cdot (12)^3} \cdot 3 = 131\ mm$$
(with n_b = number of bolt rows)

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

Row 1-2:

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

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

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

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

Row 1-3:

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

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

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

⇨ $\boldsymbol{F_{T,Rd} = 391,7\ kN}$

In SCIA Engineer:

For bolt group :

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

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

Row 1-2:

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

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

•

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

Row 1-3:

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

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

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

In SCIA Engineer:

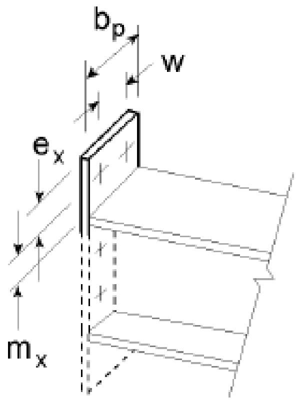
group	beff,t,wc	omega 1	omega 2	omega	Ft,wc,Rd,g
1- 1	145.10	0.75	0.49	0.75	178.95
1- 2	215.10	0.61	0.36	0.61	214.86
1- 3	355.10	0.42	0.23	0.42	245.40

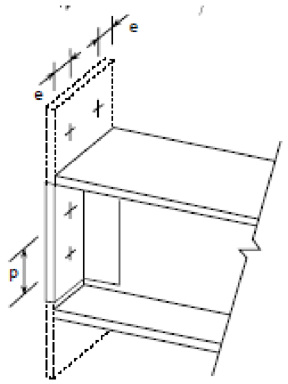
4.5.4. End plate

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.

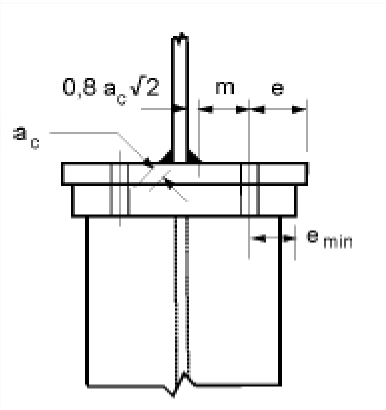
For the end-plate extension, use e_x and m_x in place of e and m when determining the design resistance of the equivalent T-stub flange.

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

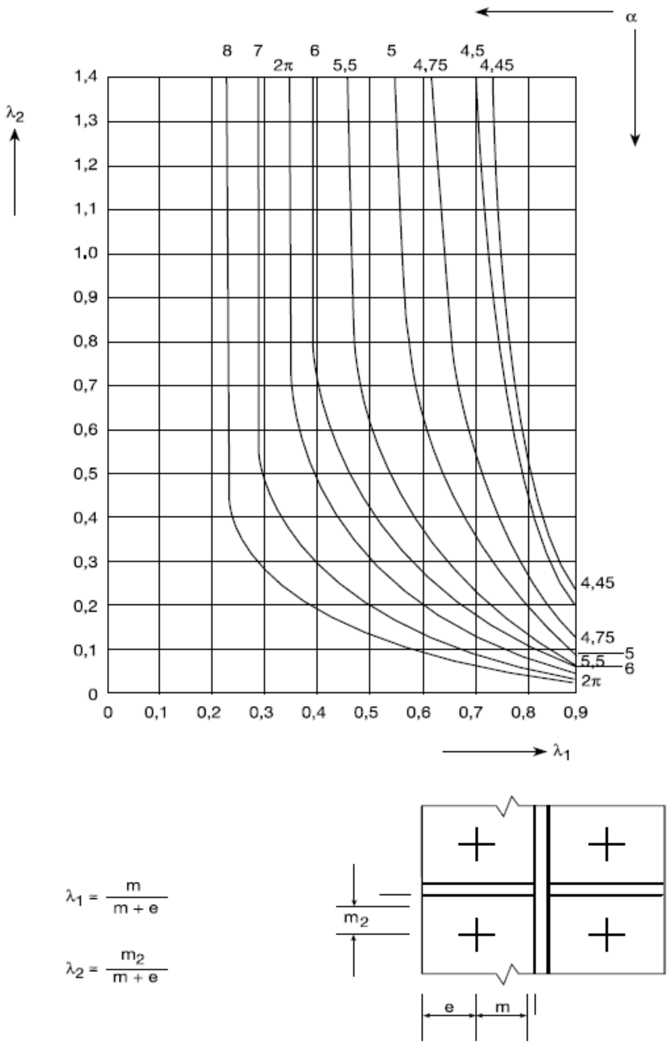
$a_f\ =\ 0{,}5 \cdot t_{fb}\ =\ 0{,}5 \cdot 9{,}2 = 4{,}6$ => $a_f\ =\ 5\ mm$
$m_x\ =\ Top - e_x - 0{,}8 \cdot a \cdot \sqrt{2}$ (see also EN1993-1-8 (Figure 6.10))
$m_x\ = (305 - 220 - 15) - 40 - 0{,}8 \cdot 5 \cdot \sqrt{2} = 24{,}34$



$n = e_{min}$ = 40mm
$\leq 1{,}25 \cdot m = 1{,}25 \cdot 24{,}34 = 30{,}42mm$
$\boldsymbol{n = 30{,}42mm}$
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	$a_f \geq 0.7\ t_{fb}$ $a_w \geq 0.7\ t_{wb}$

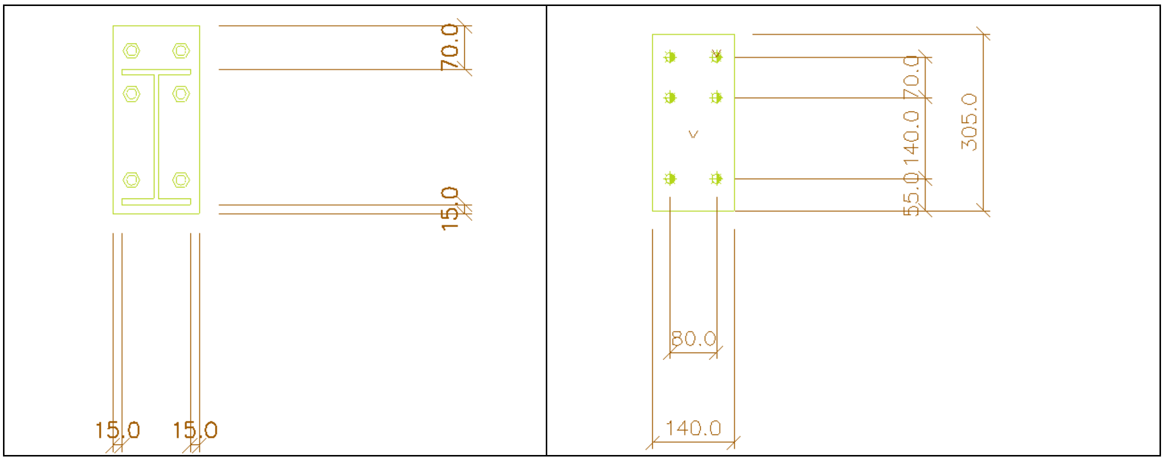
$a_w\ =\ 0{,}5 \cdot t_{wb}\ =\ 0{,}5 \cdot 5{,}9 = 3{,}0$

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

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

$n = e_{min}$ = 30mm
$\leq 1{,}25 \cdot m = 1{,}25 \cdot 33{,}66 = 42{,}01 mm$

$\boldsymbol{n = 30\ mm}$



$$m_{2,row2} = e_x - t_f - 0{,}8 \cdot a_f \cdot \sqrt{2}$$
$$m_{2,row2} = (35 + \tfrac{9{,}2}{2}) - 9{,}2 - 0{,}8 \cdot 5 \cdot \sqrt{2} = 24{,}74 \text{ mm}$$
$$m_{2,row3} = h_{row3} - t_f - 0{,}8 \cdot a_f \cdot \sqrt{2}$$
$$m_{2,row3} = 35 + \tfrac{9{,}2}{2}) - 9{,}2 - 0{,}8 \cdot 5 \cdot \sqrt{2} = 24{,}74\text{mm}$$
$$\lambda_1 = \frac{m}{m+e} = \frac{33{,}66}{33{,}66+30} = 0{,}53$$
$$\lambda_{2,row2} = \lambda_{2,row3} = \frac{m_{2,row2}}{m+e} = \frac{24{,}74}{33{,}66+30} = 0{,}39$$

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

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

In SCIA Engineer:

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

To calculate the end plate we will use Table 6.6 of the EN 1993-1-8 - Ref.[1].

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	πm + 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 $\ell_{eff,1} \leq \ell_{eff,cp}$		$\sum\ell_{eff,1} = \sum\ell_{eff,nc}$ but $\sum\ell_{eff,1} \leq \sum\ell_{eff,cp}$	
Mode 2:	$\ell_{eff,2} = \ell_{eff,nc}$		$\sum\ell_{eff,2} = \sum\ell_{eff,nc}$	
α should be obtained from Figure 6.11.				

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

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

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

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

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

4.5.4.2. Bolt rows considered individually

<u>**Row 1:**</u>

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:

l_{eff} circular patterns = $2\pi m$ = 2*3.14*33,66 = 211,49
l_{eff} non circular patterns: αm = 5,99 * 33,66 = 201,62

Row 3:

l_{eff} circular patterns = $2\pi m$ = 2*3.14*33,66 = 211,49
l_{eff} non circular patterns: 4m + 1,25e = 4 * 33,66 + 1.25 * 30 = 172,14mm

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

In SCIA Engineer:

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

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:

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 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:

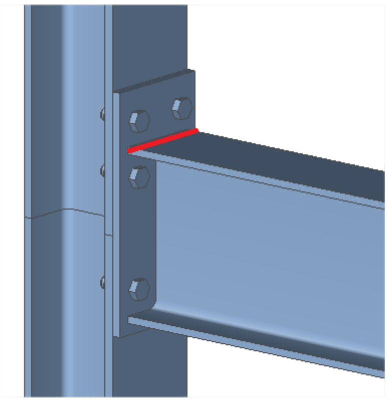
For individual bolt-row :

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

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

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 leff 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 + 5,99*33,66 – (2*33,66 + 0,625*30) = 185,55

Row 3:

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

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

Summary of values:

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

In SCIA Engineer:

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

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

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

Row 2-3:

$$\sum l_{eff,cp} = 245{,}73 + 245{,}73 = 491{,}46$$
$$\sum l_{eff,nc} = 185{,}51 + 185{,}51 = 341{,}57$$

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

In SCIA Engineer:

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

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

Prying forces may develop if $L_b \leq L_b^*$
L_b = 38,8mm

Row 2-3:

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

(with n_b = number of bolt rows)

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

Row 2-3:

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

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

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

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

In SCIA Engineer:

For bolt group :

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

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

Row 2-3:

⇨ $F_{T,wb,Rd} = \frac{b_{eff,t,wb} t_{wb} f_{y,wb}}{\gamma_{M0}} = 341{,}57 \cdot 5{,}9 \cdot 235 \cdot 10^{-3}/1$
⇨ $\boldsymbol{F_{T,wc,Rd} = 473{,}6\ kN}$

In SCIA Engineer:

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

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	138.51	178.95	178.95	97.31	97.31	-	-	97.31
2	138.51	157.37	178.95	117.55	138.82	138.82	279.47	279.47	117.55
3	138.51	176.82	178.95	30.54	130.99	143.72	238.65	356.04	30.54

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

Row 1: 97,31 kN (End plate failure)
Row 2: 117,55 kN (Column flange failure)
Row 3: 30,54 kN (Column flange failure)

This will be used in the calculation of MRd in the next chapter.

4.6. Calculation of MRd

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

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

Ft,min for each boltrow:

Row 1: 97,31 kN (End plate failure)
Row 2: 117,55 kN (Column flange failure)
Row 3: 30,54 kN (Column flange failure)

Following §6.2.7.2 (6) and (8)

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

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

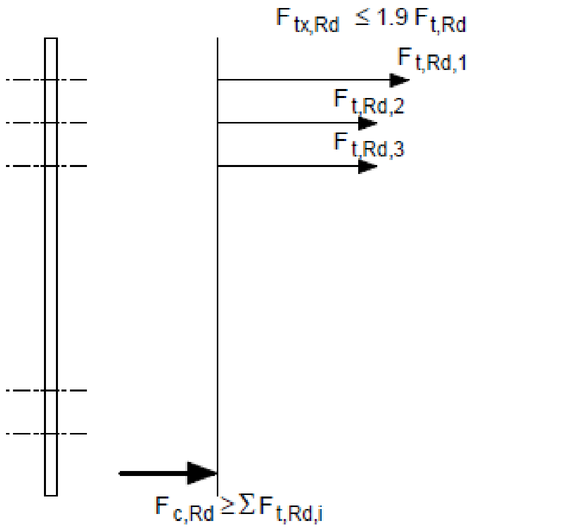
This is also shown in SCIA Engineer:

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

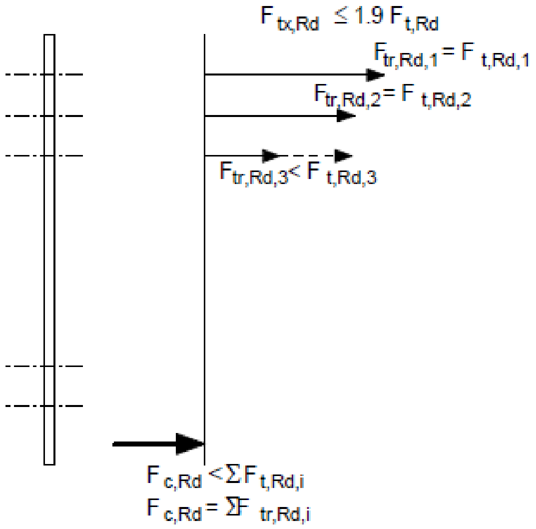
Column web in shear (Vwp,Rd/Beta)	160.21	kN
Column web in compression (Fc,wc,Rd)	190.56	kN
Beam flange and web in compression (Fc,fb,Rd)	317.72	kN

Limiting resistance = 160.21 kN

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

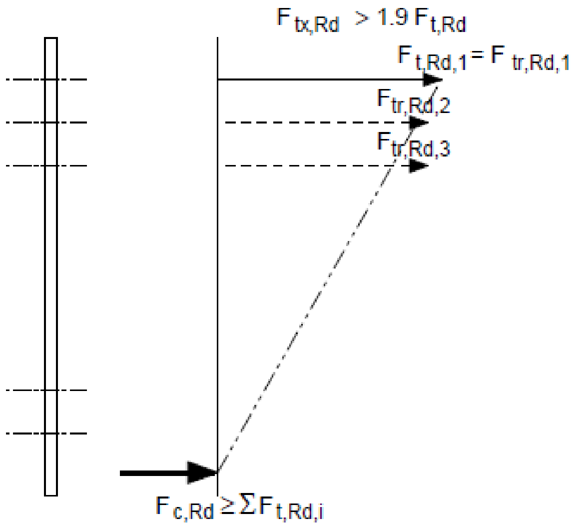


(a) Plastic distribution

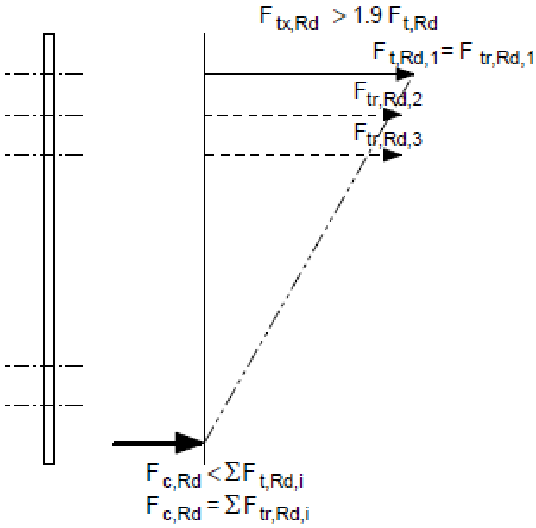


(b) Modified plastic distribution

- 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 ($F_{t,Rd,i}$)
- 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



(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 $F_{tx,Rd} > 1{,}9\ F_{t,Rd}$ 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 boltrow $F_{t,Rd,1}$ = 97,31kN.

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

And row 3 will not take any resistance.

This principle is shown on the next page.

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

This is also shown in SCIA Engineer:

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

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

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

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

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

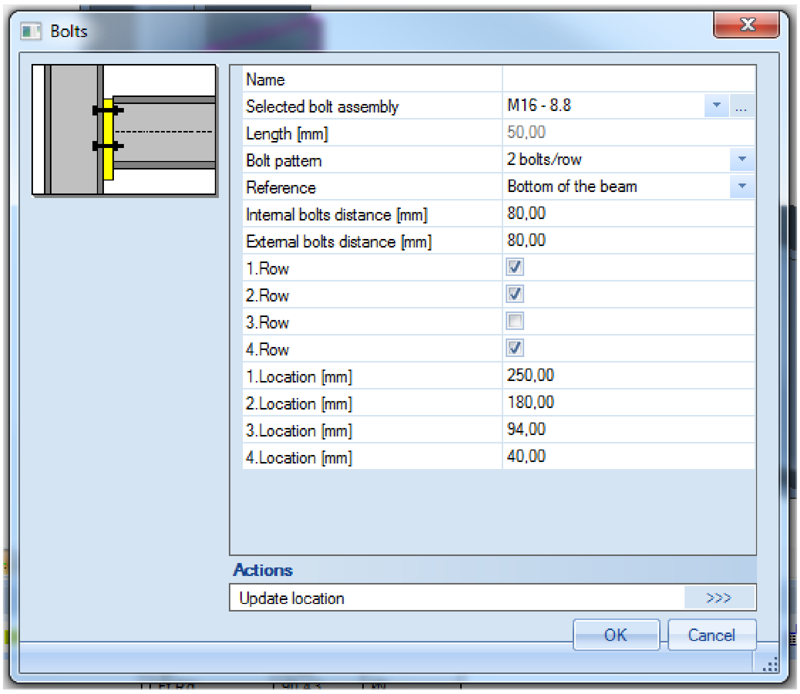
So $M_{j,Rd}$ can be calculated with the following values:

$h_{row\ 1}$ = 250 – 9,2/2 = 245.4 mm

$h_{row\ 2}$ = 180 – 9,2/2 = 175.4 mm

$h_{row\ 3}$ = 40 – 9,2/2 = 35,4 mm

Those values are calculated as the distance from the bolt to the middle of the bottom flange. 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 (=9,2mm/2) of this distance.



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

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

In SCIA Engineer:

2.3. Determination of Mj,Rd

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

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

Mj,Rd = 34.91 kNm

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, $N_{j,Rd} \leq N_{pl,Rd}$.

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

⇨ **245, 40 kN**

Beam Web in tension:

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

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

⇨ **473,58 kN**

Endplate in bending:

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 bolt group :

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

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

Column Flange in tension:

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

For bolt group :

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

⇨ **391,68 kN**

Bolts in Tension:

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

⇨ 6 x 90,43 kN = **542,58 kN**

$N_{j,Rd}$

⇨ **Minimum of all previous values**

⇨ **245,40 kN**

In SCIA Engineer:

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

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

Nj,Rd = 245.40 kN

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} = 800MPa

A is the tensile stress area of the bolt A_s

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

⇨ **$F_{v,Rd}$ = 60,29 kN**

Following the NOTE of §6.2.2 (2) (EN 1993-1-8):
As a simplification, bolts required to resist in tension may be assumed to provide their full design resistance in tension when it can be shown that the design shear force does not exceed the sum of

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

4 bolts (row 1 and 2) are required to resist tension, 2 bolts (of row 3) are not required to resist tension. The value 0,4/1,4 will be simplified in SCIA Engineer by the value 0,28:

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

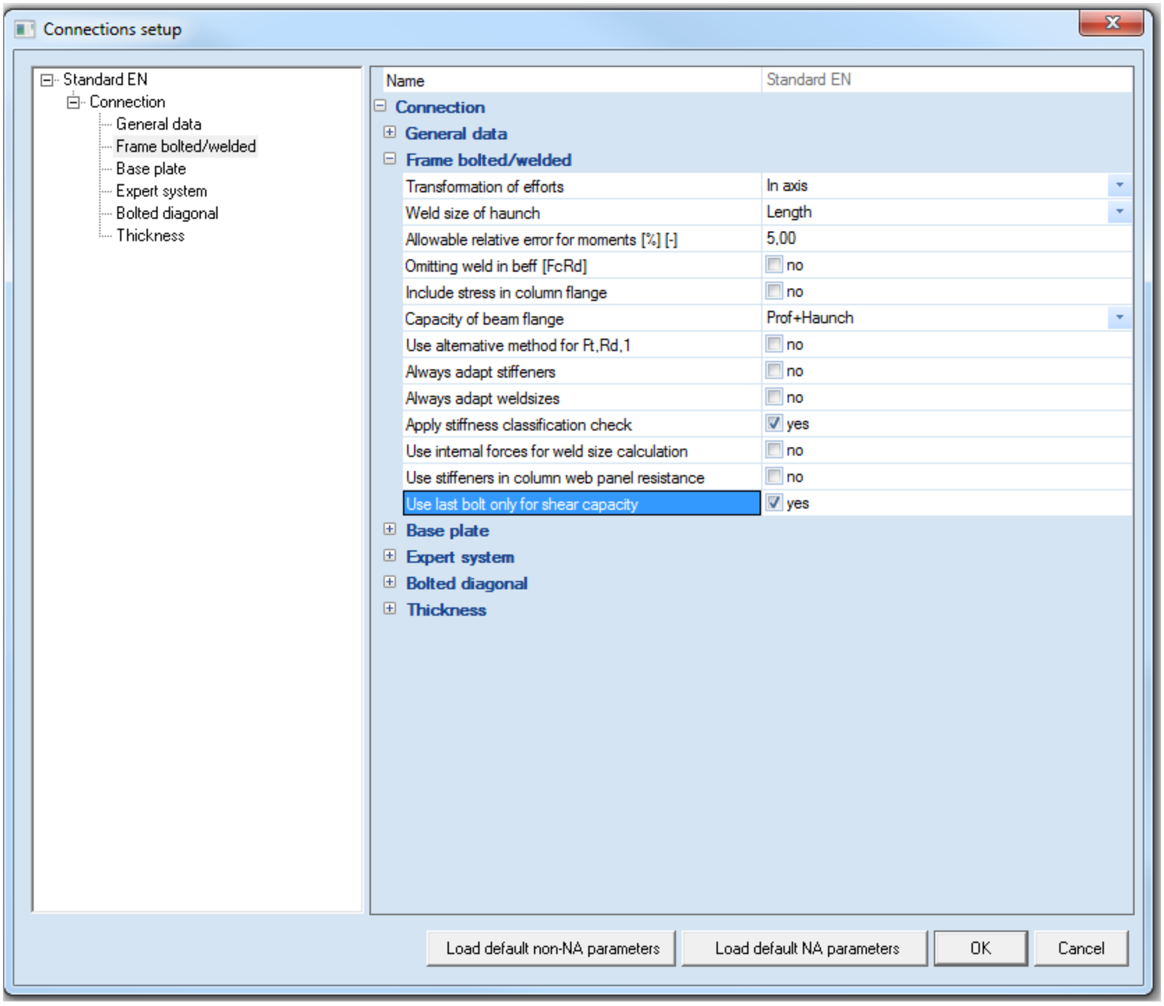
In SCIA Engineer:

3. Design shear resistance VRd

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

Remark

If the unity check for the Moment and Moment + Normal force is okay, but the unity check for the shear force is bigger than one, it could be interesting to use the last bolt row only for the shear force resistance.
You can activate the option “Use last bolt only for shear capacity”.



This last bolt-row has a small lever arm for the moment resistance, so the influence on the moment check will be small.

4.9. Unity checks

4.9.1. 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 :

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

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

4.9.2. General unity checks

Assume following internal forces in this connection:

N_{Sd} = 0 kN
V_{Sd} = 10 kN
$M_{y,Sd}$ = 10 kNm

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

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

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

In SCIA Engineer:

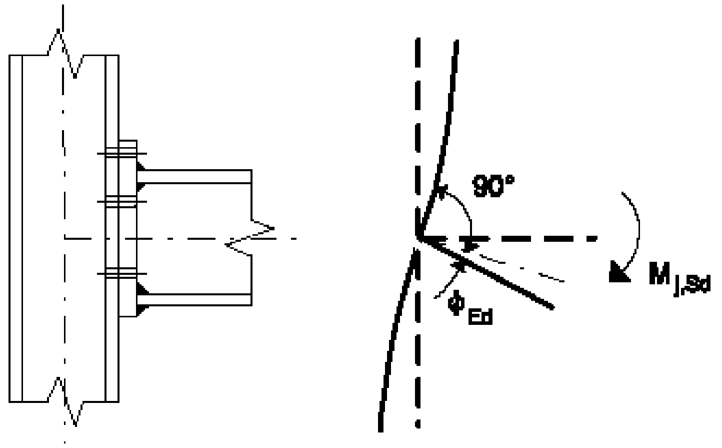
5. Unity checks

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

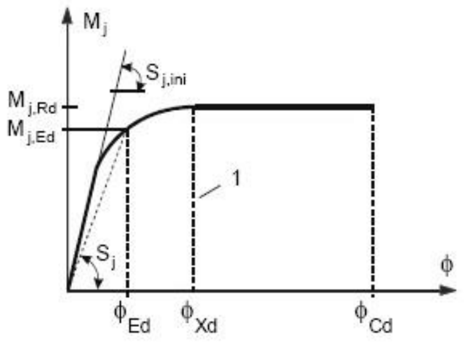
5. Stiffness of the connection

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.

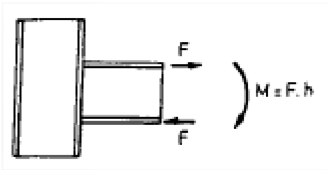


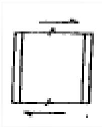
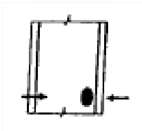
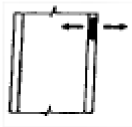
This moment-rotation characteristic defines three main properties:
- the moment resistance $M_{j,Rd}$
- the rotational stiffness S_j
- the rotation capacity ϕ_{Cd}

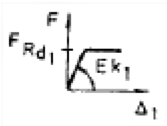
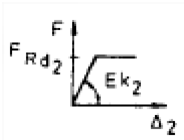
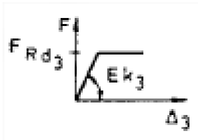


The general analytical procedure which is used for determining the resistance and stiffness properties of a joint, is the so-called component method. The component method considers any joint as a set of individual basic components. Each of these basic components possesses its own strength and stiffness.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

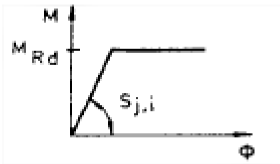
1. identification of the active components in the joint being considered
2. evaluation of the stiffness and/or resistance characteristics for each individual basic component
3. assembly of all the constituent components and evaluation of the stiffness and/or resistance characteristics of the whole joint

<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:

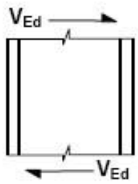
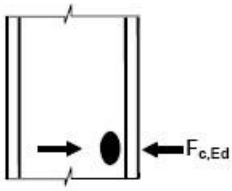
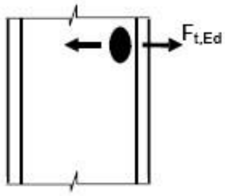
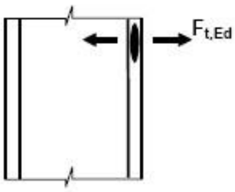
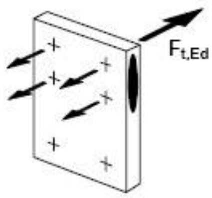
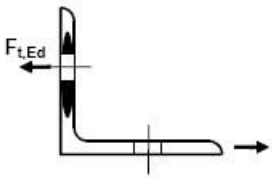
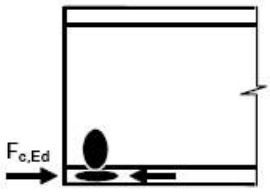
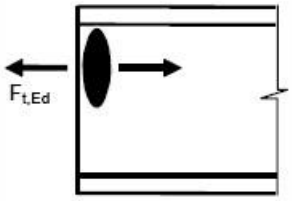
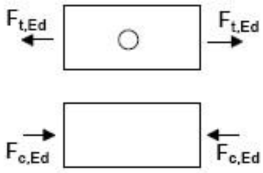
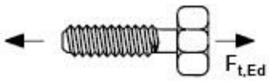
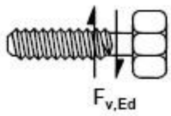
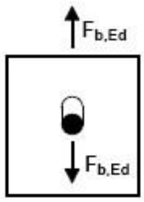
Response of the components</td><td>
Stiffness coefficient k_i of each component

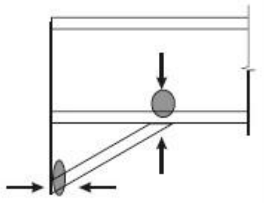
Resistance F_{Rd} of Each component</td></tr>
<tr><td>Third Step:

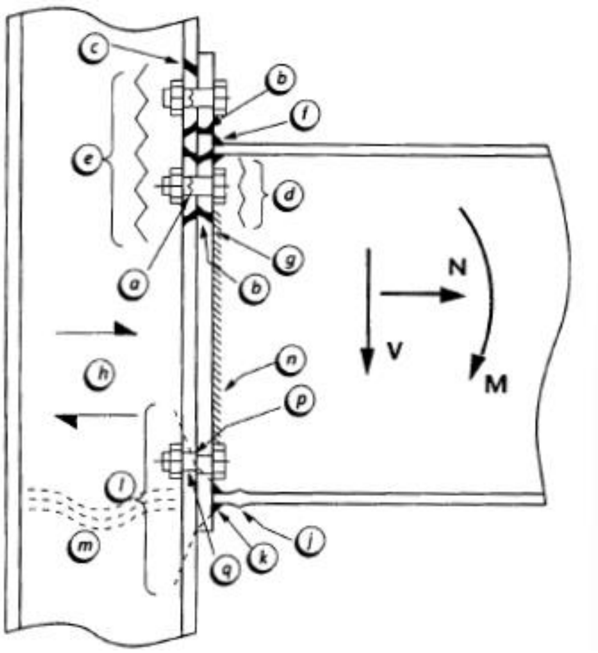
Assembling of the components</td><td>
Stiffness of the joint $S_{j,i} = E\ h^2 \ / \ \sum l/k_i$

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

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: $k_1 = \dfrac{0{,}38 \cdot A_{VC}}{\beta z}$	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_6	Flange cleat in bending	$k_6 = \frac{0{,}9 \cdot l_{eff} t_a^3}{m^3}$	
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)	$k_{10} = 1{,}6\, A_s/L_b$	
k_{11} (or k_{17})	Bolts in shear	$k_{11}(or\ k_{17}) = \frac{16 \cdot n_b \cdot d^2 \cdot f_{ub}}{E \cdot d_{M16}}$	
k_{12} (or k_{18})	Bolts in bearing (for each component *j* on which the bolts bear)	$k_{12}(or\ k_{18}) = \frac{24 \cdot n_b \cdot k_b \cdot k_t \cdot d \cdot f_{ub}}{E}$	
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

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

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

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

⇨ $b_{eff,t,wc,row1} = 107{,}55$

⇨ $b_{eff,t,wc,row2} = 105$

$$\boldsymbol{k_{3,row1}} = \frac{0{,}7 \cdot 107{,}55 \cdot 7}{92} = \mathbf{5{,}73\ mm}$$
$$\boldsymbol{k_{3,row2}} = \frac{0{,}7 \cdot 105 \cdot 7}{92} = \mathbf{5{,}59\ mm}$$

In SCIA Engineer:

4.1. Design rotational stiffness

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

5.2.2.2. Column flange in bending: k_4

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

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

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

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

In SCIA Engineer:

4.1. Design rotational stiffness

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

5.2.2.3. End-plate in bending: k_5

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

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

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

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

In SCIA Engineer:

4.1. Design rotational stiffness

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

5.2.2.4. Bolts in tension: k_{10}

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

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

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

In SCIA Engineer:

4.1. Design rotational stiffness

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

5.2.2.5. Equivalent stiffness

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

$k_{eff,r} = 1/\sum_i(\frac{1}{k_{i,r}})$ (see also formula (6.30) of EN 1993-1-8 – 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 fork_3, k_4, k_5 and k_{10}.

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

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

In SCIA Engineer:

4.1. Design rotational stiffness

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

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

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

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

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

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

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

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

And those values are also given in SCIA Engineer:

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

5.2.2.6. Column web panel in shear: k_1

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

z is the lever arm from Figure 6.15

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

⇨ $z = z_{eq} = 216{,}8 = 216{,}8$mm

β is the transformation parameter from 5.3 (7)

⇨ β = 1

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

In SCIA Engineer:

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

5.2.2.7. Column web in compression: k_2

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

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

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

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

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

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

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

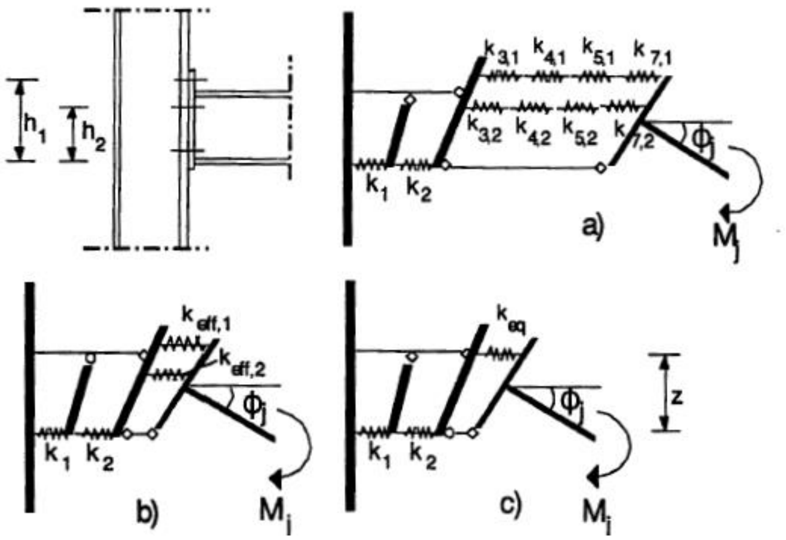
$$\boldsymbol{k_2} = \frac{\mathbf{0{,}7 \cdot 163{,}3 \cdot 7}}{\mathbf{92}} = \mathbf{8{,}70\ mm}$$

In SCIA Engineer:

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

5.2.2.8. Design rotational stiffness

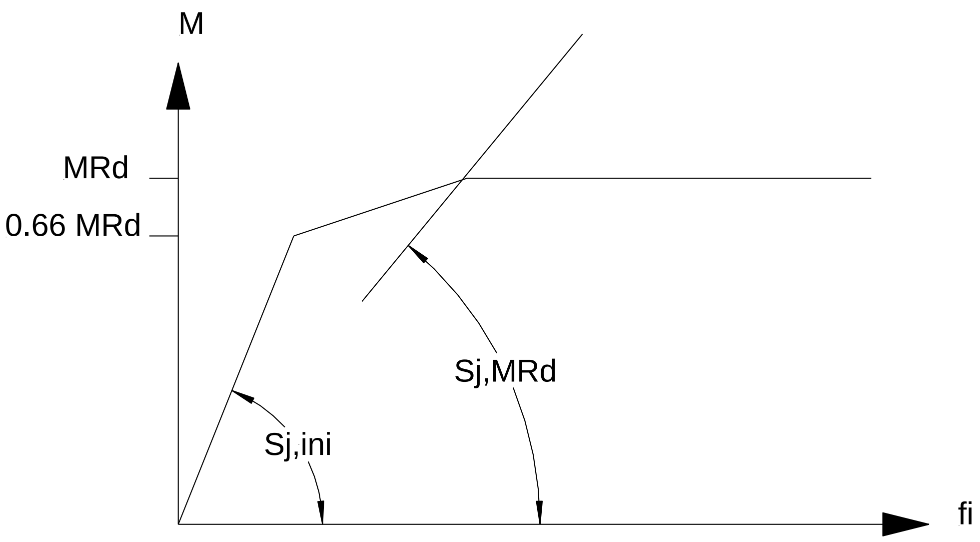
When all different stiffness of all components are known, we can assembly this to one stiffness for the joint.



The program will calculate 3 stiffnesses :

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

The moment-rotation diagram is based on the values of Sj,ini and Sj,MRd.



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

⇨ z = 216,4 mm

⇨ μ is the stiffness ration $S_{j,\,ini}$ / S_j
- 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

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

⇨ $\mu = 1$

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

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

In SCIA Engineer:

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

5.3. The classification on stiffness

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

If Sj,ini >= Sj,rigid, the joint is rigid.

If Sj,ini <= Sj,pinned, the joint is classified as pinned.

If Sj,ini<Sj,rigid and Sj,ini>Sj,pinned, the joint is classified as semi-rigid.

For braced frames:

$$Sj,rigid = 8\frac{EI_b}{L_b}$$

$$Sj,pinned = 0.5\frac{EI_b}{L_b}$$

For unbraced frames:

$$Sj,rigid = 25\frac{EI_b}{L_b}$$

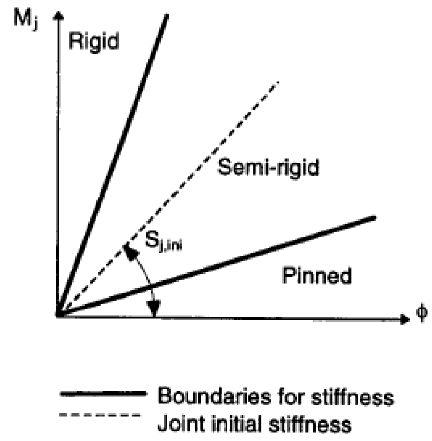
$$Sj,pinned = 0.5\frac{EI_b}{L_b}$$

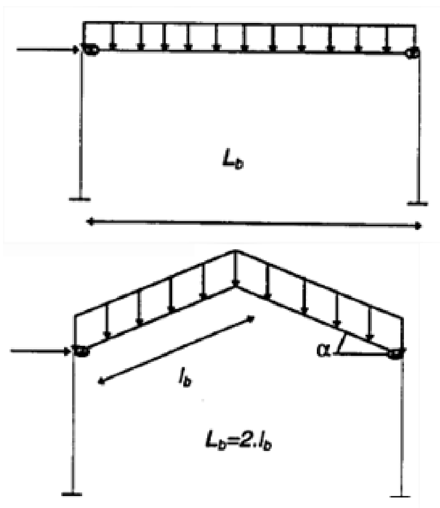
For column base joints:

$$Sj,rigid = 15\frac{EI_c}{L_c}$$

$$Sj,pinned = 0.5\frac{EI_c}{L_c}$$

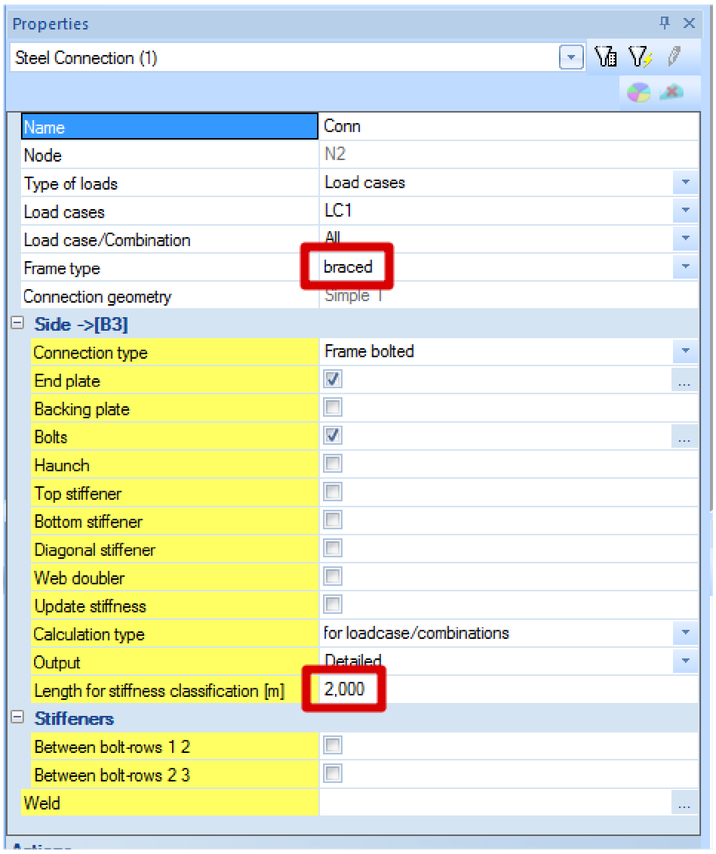
with	I_b	the second moment of area of the beam
	L_b	the span of the beam
	I_c	the second moment of area of the column
	L_c	the storey height of the column
	E	the Young modulus





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:



$$S_{j,rigid} = 8\frac{E \cdot I_b}{L_b} = 8\,\frac{(210000\frac{N}{mm^2})\cdot(2{,}772\cdot10^7\ mm^4)}{2000\ mm} = 23{,}28\ \text{MNm/rad}$$

$$S_{j,pinned} = 0{,}5\frac{E \cdot I_b}{L_b} = 0{,}5\,\frac{(210000\frac{N}{mm^2})\cdot(2{,}772\cdot10^7\ mm^4)}{2000\ mm} = 1{,}46\ \text{MNm/rad}$$

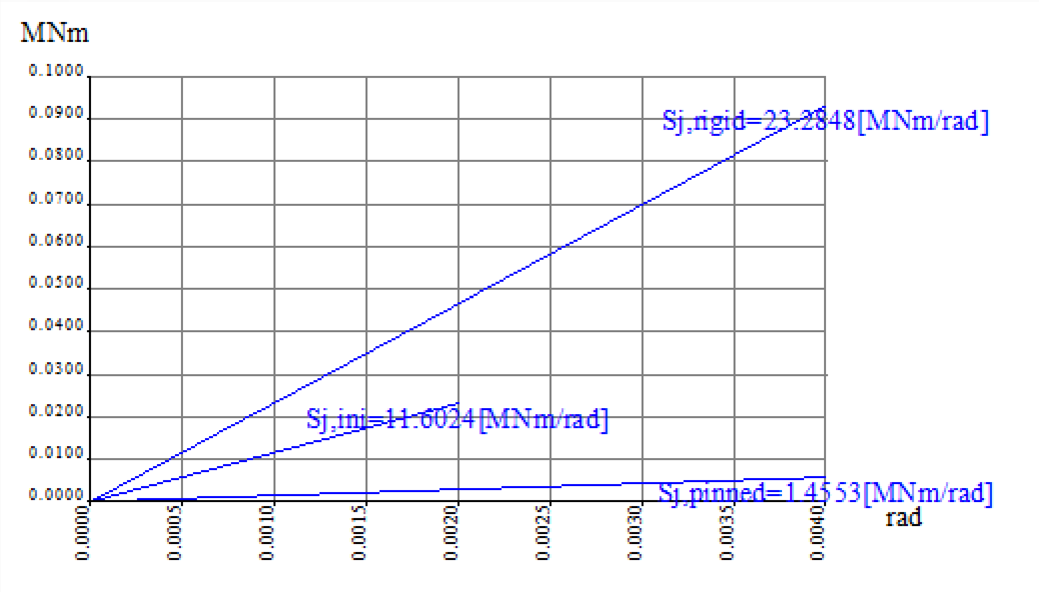
In SCIA Engineer:

4.2. Stiffness classification

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

System SEMI RIGID

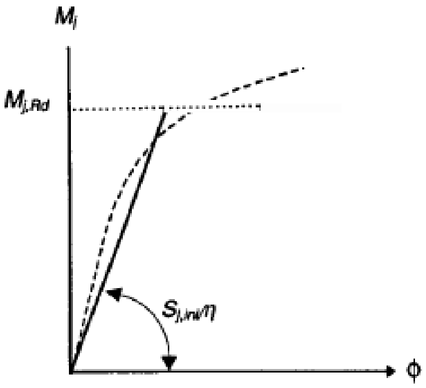
And this is also given in SCIA Engineer in a picture:



5.4. Transferring the joint stiffness to the analysis model

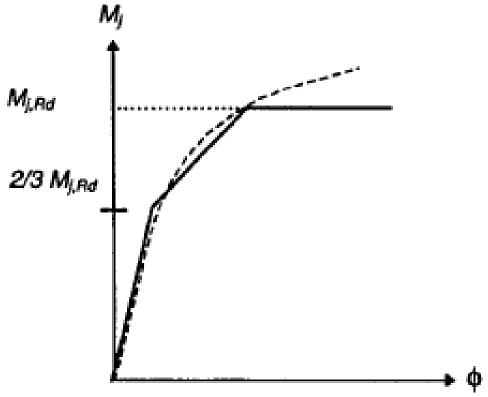
When requested, the actual stiffness of the joint can be transferred to the analysis model. The linear spring value for <**fi y**> (in the hinge dialog) is taken as Sj,ini divided by the stiffness modification coefficient η.

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 Sj,ini (from both directions) divided by the stiffness modification coefficient η:

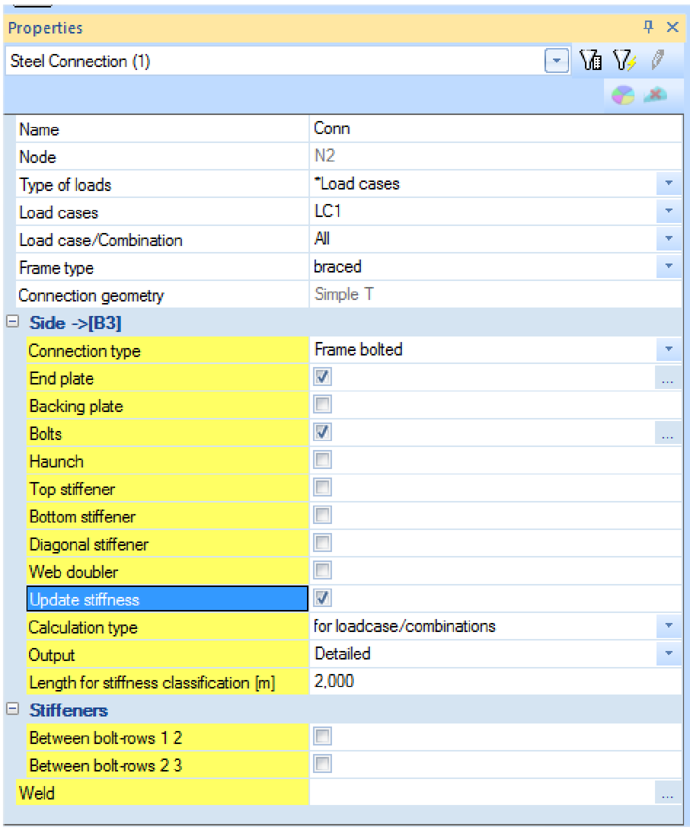


bolted beam-to-column	2
welded beam-to-column	2
welded plate-to-plate	3
column base	3

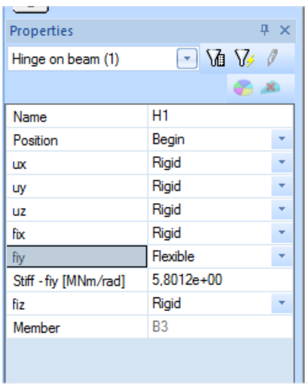
At the same time, a non-linear function is generated, representing the moment-rotation diagram.

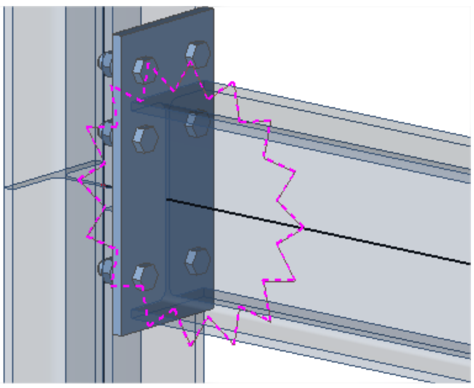


You can add this stiffness to the connection by activating the option “Update Stiffness”:

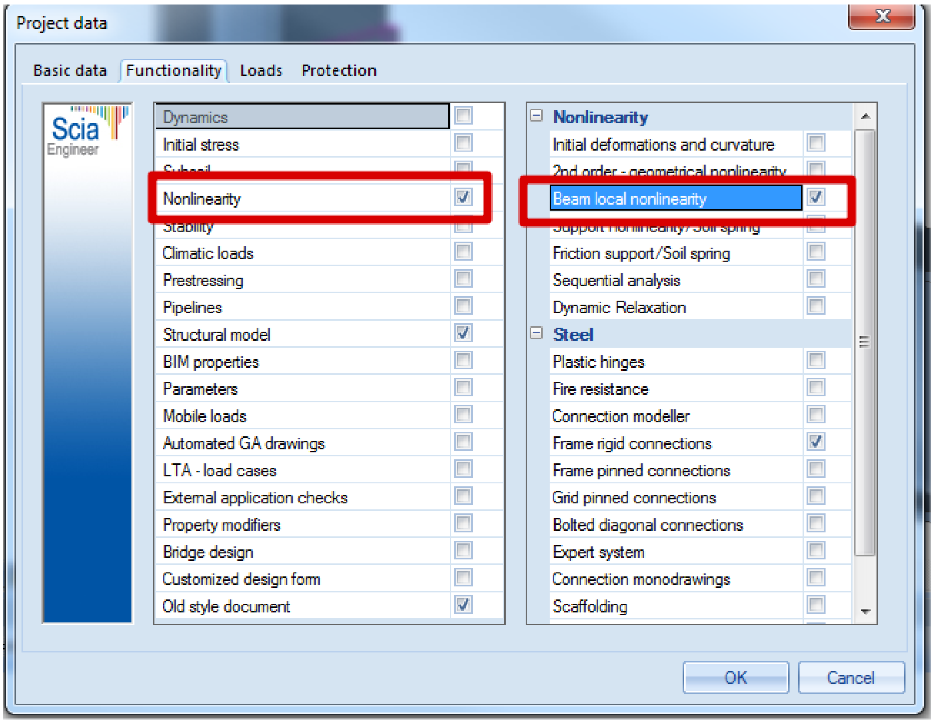


After recalculating the project, a hinge will be added to this connection with this stiffness $S_{j,\,ini}/\eta$.



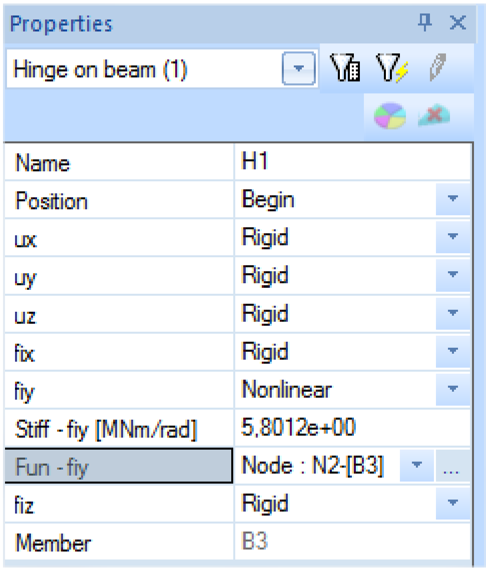


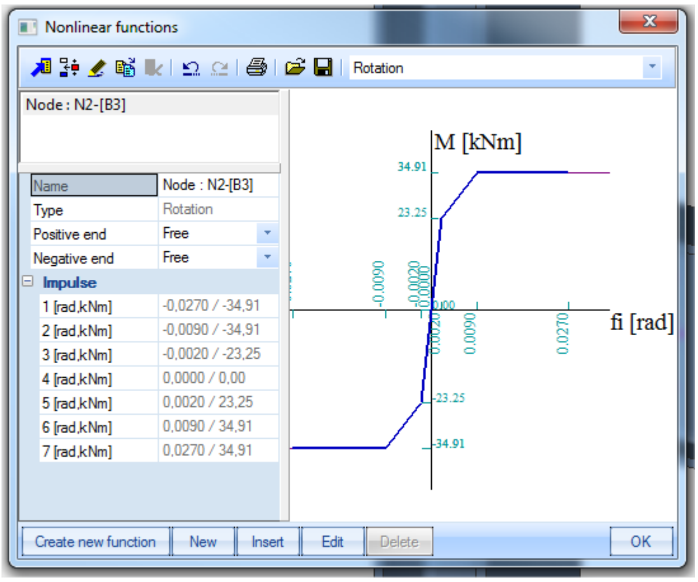
If you want to take into account the non linear stiffness of the connection, you have to activate the following functionality in the Project Data menu:



And after the calculation, you can select the input hinge and choose here for a non linear stiffness.

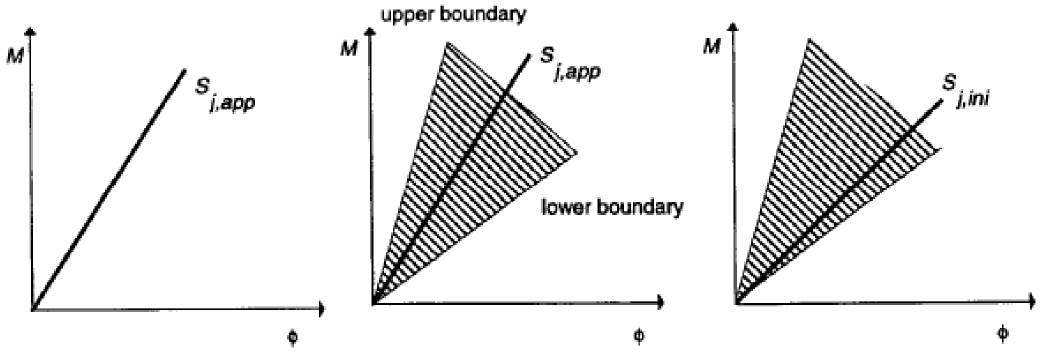
The stiffness function will be registered automatically for each node in SCIA Engineer. So you only have to choose the corresponding node for each connection.





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 Sj,ini >= Sj,low and Sj,ini<=Sj,upper, the actual joint stiffness is conform with the applied Sj,app in the analysis model.

When a non-linear function is used during the analysis model, we check the following :

When Sj >= Sj,low and Sj<=Sj,upper, the actual joint stiffness is conform with the applied Sj,app in the analysis model.

The boundaries are calculated with the following formulas:

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

And for a column base connection:

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

with	I_b	the second moment of area of the beam
	L_b	the span of the beam
	I_c	the second moment of area of the column
	L_c	the storey height of the column
	E	the Young modulus
	Sj,app	the approximate joint stiffness
	Sj,ini	the actual initial joint stiffness
	Sj,low	the lower boundary stiffness
	Sj,upper	the upper boundary stiffness
	Sj	the actual joint stiffness

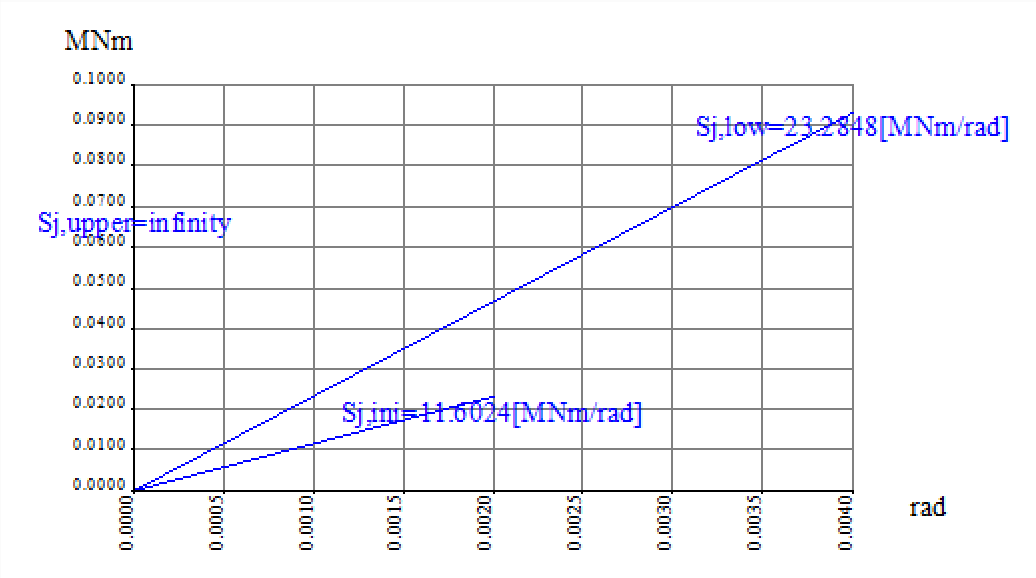
In this case we have a braced system and we did not take into account any stiffness, so the upper boundary equals infinity and the lower boundary is the boundary for a rigid connection:

4.3 Check of stiffness requirement

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

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

And also in the graph you can see that Sj,ini is not between the boundaries:



When activating the option “Update stiffness” and recalculating the project, the value for Sj,app equals Sj,ini. The stiffness taken into account in the calculation equals Sj,ini/2 because η = 2 for a beam-column connection.

In SCIA Engineer we have:

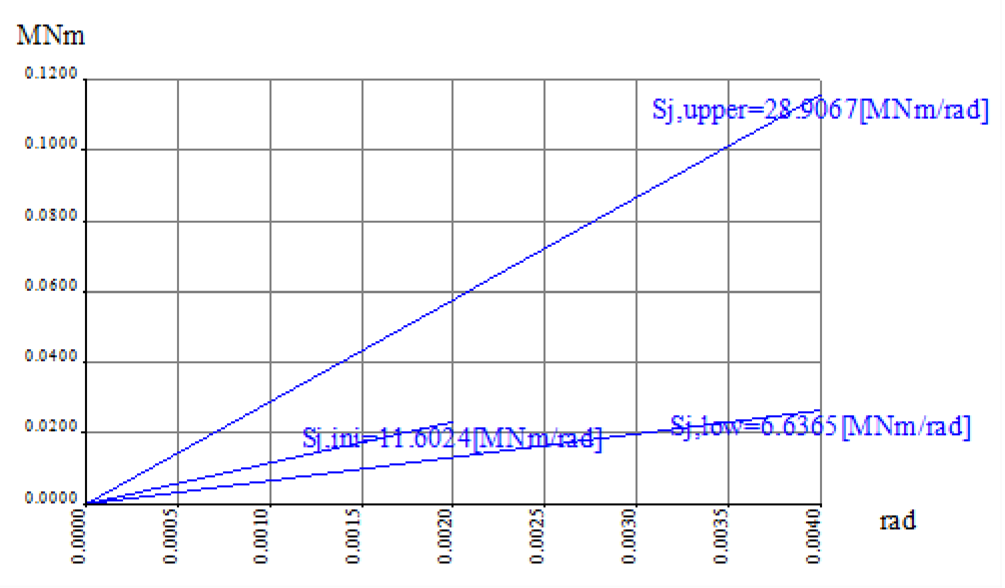
Fi y	The stiffness taken into account in the calculation, thus: Sj,ini/η = Sj,ini/2 = 11,60 MNm/rad /2 = 5,80 MNm/rad
Stiffness modification coef.	Factor η and η = 2 for a beam-column connection
Sj,app	In this case Sj,ini = 11,60 MNm
Sj,lower boundary	For a braced system: $= \frac{8 \cdot S_{j,app} \cdot E \cdot I_b}{10 \cdot E \cdot I_b + S_{j,app} \cdot L_b}$ $= \frac{8 \cdot \frac{11{,}60 MNm}{rad} \cdot 210000 MPa \cdot 2{,}77E-05\ m^4}{10 \cdot 210000 MPa \cdot 2{,}77E-05\ m^4 + 11{,}60 MNm/rad \cdot 2m}$ = 6,64 MNm/rad
Sj,upper boundary	For a braced system: First we have to check if Sj,app is bigger or smaller than $\frac{8 \cdot E \cdot I_b}{L_b} = \frac{8 \cdot 210000\ MPa \cdot 2{,}77E-05\ m^4}{2m} = 23{,}3\ MPa$ Thus $S_{j,app} = 11{,}60 \leq \frac{8 \cdot E \cdot I_b}{L_b}$ And now the upper boundary can be calculated with the following formula: $= \frac{10 \cdot S_{j,app} \cdot E \cdot I_b}{8 \cdot E \cdot I_b - S_{j,app} \cdot L_b}$ $= \frac{10 \cdot \frac{11{,}60 MNm}{rad} \cdot 210000 MPa \cdot 2{,}77E-05\ m^4}{(8 \cdot 210000 MPa \cdot 2{,}77E-05) - (11{,}60 MNm/rad \cdot 2m)}$ = 28,90 MNm/rad

4.3 Check of stiffness requirement

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

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

And now Sj,ini will be in between the two boundaries on the graph also:



6. Calculation of welds

6.1. Default method

The default values for the double fillet welds to the beam flange a_f and for the double fillet welds to the beam web a_w, are as follows:

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

With:
- a_f The throat thickness of weld at beam flange (fillet weld)
- a_w The throat thickness of weld at beam web (fillet weld)
- t_{fb} The thickness of the beam flange
- t_{wb} The thickness of the beam web

In the example **CON_004.esa**:

t_{fb} = 9,2 mm
t_{wb} = 5,9 mm

And the material S235 has been used. So:

$a_f \geq 0{,}5\ t_{fb} = 4{,}6$ mm

⇨ a_f will be taken as 5 mm

In SCIA Engineer:

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

data		
MRd	34.91	kNm
Gamma	1.40	
h	210.80	mm
FRd	231.86	kN
NT,Rd	237.82	kN
N	231.86	kN
Fu	360.00	N/mm^2
BetaW	0.80	
minimum af	4.14	mm
af	5.00	mm
Minimum th	8.97	mm

$a_w \geq 0{,}5\ t_{wb} = 2{,}95$ mm

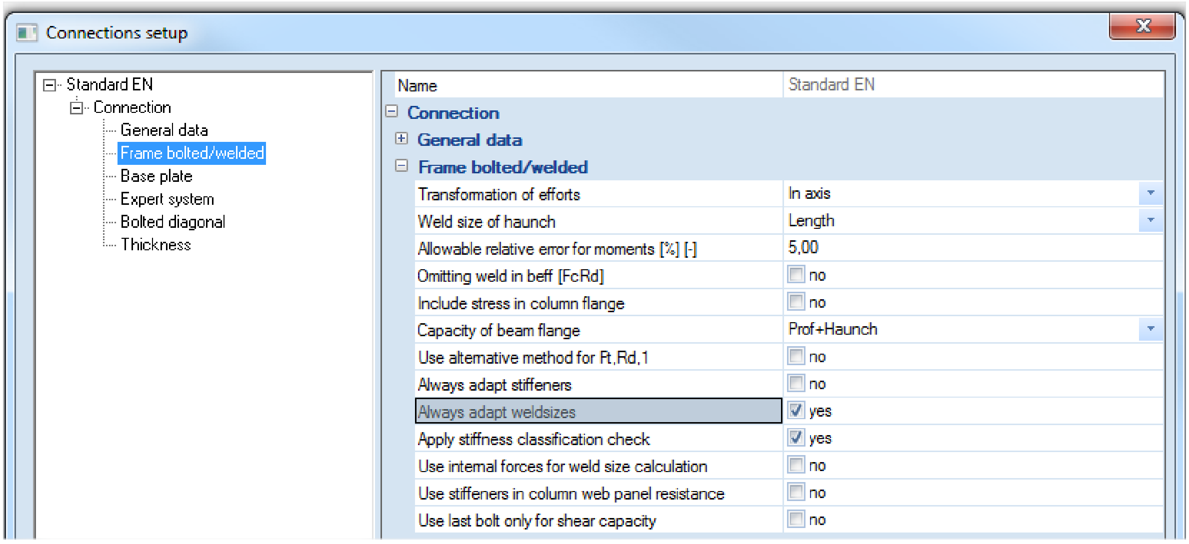
⇨ a_w will be taken as 3 mm

In SCIA Engineer:

6.2. Calculation aw

data		
Ft	62.90	kN
Fv	7.23	kN
lw	201.57	mm
Fu	360.00	N/mm^2
BetaW	0.80	
minimum aw (a2)	1.00	mm
aw	3.00	mm

Default this method is used.
It is also possible to calculate the welds with the formulas given in the next chapters, but then the option “Always adapt weldsizes” has to be activation in the menu “Steel -> Connections -> Connections Setup”.



6.2. Calculation of a_f

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:

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

With: F_{Rd} The design force in the beam flange
M_{Rd} The design moment resistance of the connection
H The lever arm of the connection

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:

γ = 1.7 for unbraced frames
γ = 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 $N_{t.Rd}$:

$$N_{t,Rd} = \frac{b_f \cdot t_{fb} \cdot f_{yb}}{\gamma_{M0}}$$

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

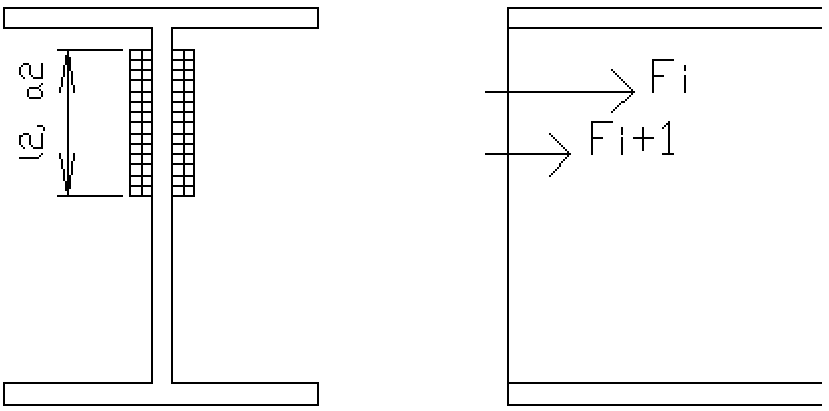
So, we have
$F_w = \min(N_{t.Rd}, \gamma F_{Rd})$

The weld size design for a_f:

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

With F_w The design resistance of the weld
b_w The beam flange width
f_u The ultimate tensile strength of the weaker part
β_W The correlation factor
γ_{Mw} The partial safety factor for welds

6.3. Calculation of a_w



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

On the weld 2 x l2 x a2, 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:

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

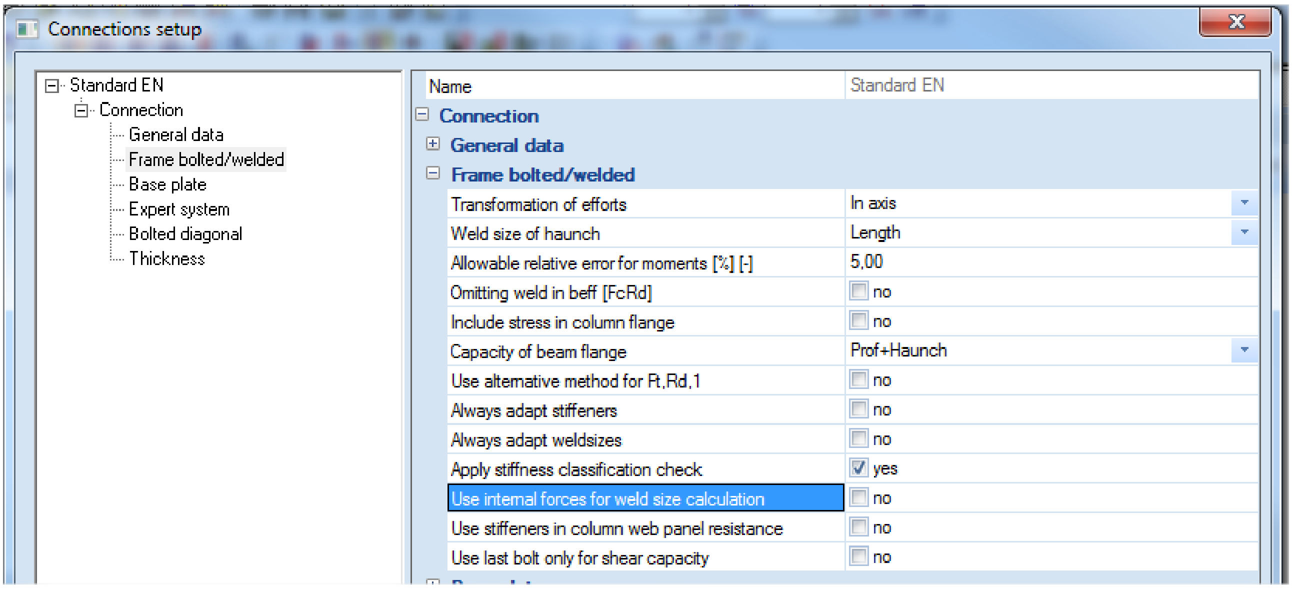
$$\sigma_1 = \tau_2 = \left(\frac{N}{A}\right)\frac{1}{\sqrt{2}}$$

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

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

6.4. 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 “Steel -> Connections -> Connections setup -> Frame bolted/welded” and here with the option “Use internal forces for weld size calculation”.



7. Ductility class

7.1. Ductility classes

The following classification is valid for joints:

Class 1 joint: Mj,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: Mj,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.

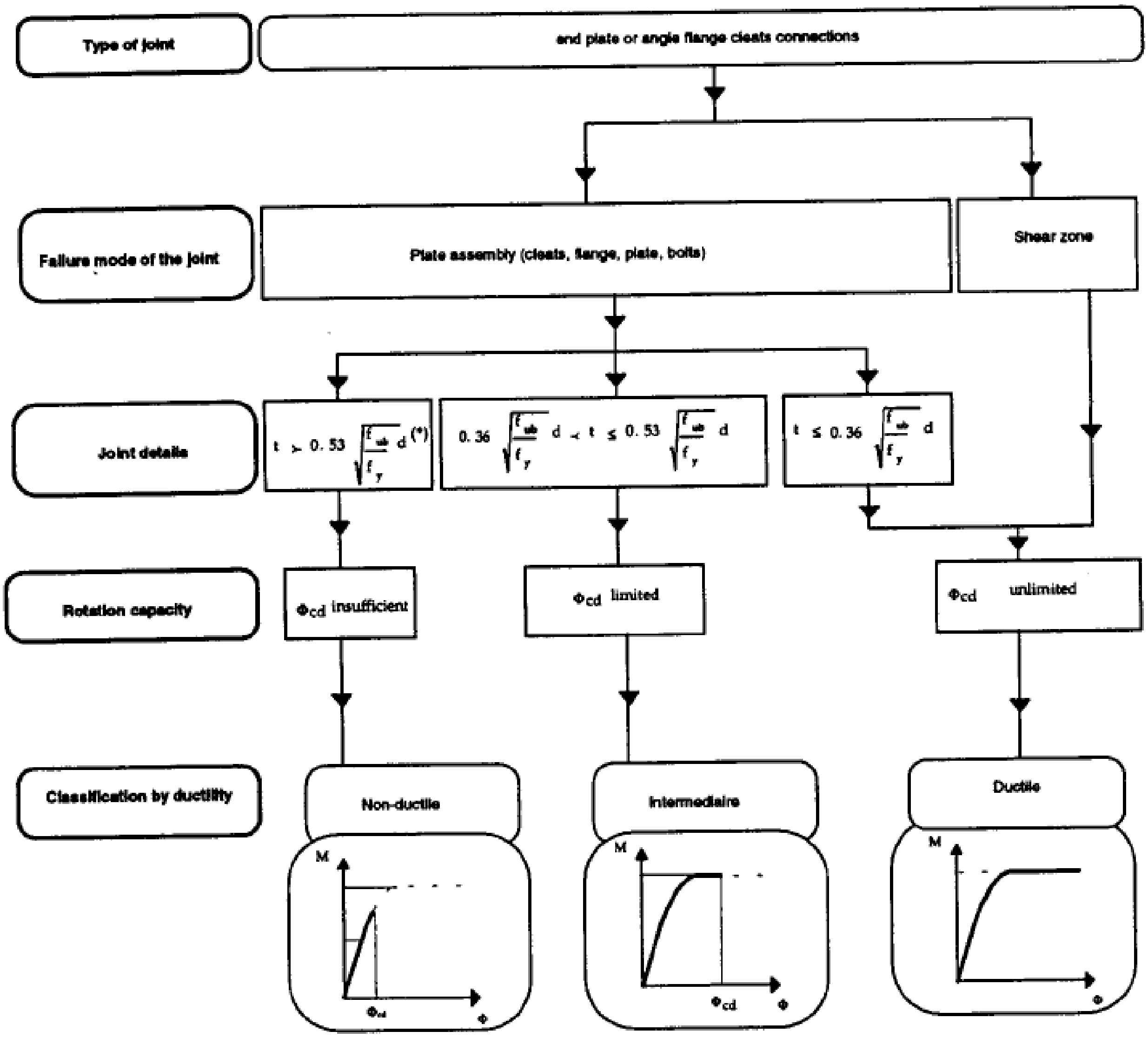
7.2. Ductility classification for bolted joints

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
$t \leq 0.36\sqrt{\frac{f_{ub}}{f_y}}d$	Ductile	1
$0.36\sqrt{\frac{f_{ub}}{f_y}}d < t \leq 0.53\sqrt{\frac{f_{ub}}{f_y}}d$	Intermediary	2
$t > 0.53\sqrt{\frac{f_{ub}}{f_y}}d$	Non-ductile	3

with	t	the thickness of either the column flange or the endplate
	d	the nominal diameter of the bolts
	f_{ub}	the ultimate tensile strength of the bolt
	f_y	the 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 MRd). 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

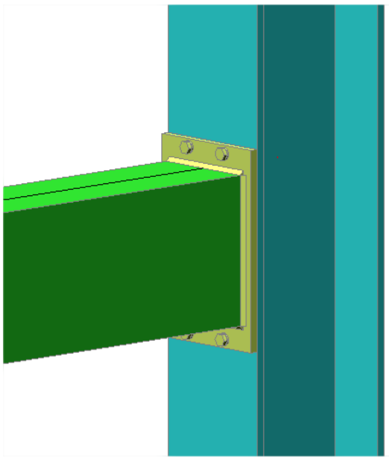
If the failure mode of the joint is the situated in the shear zone of the column web, the joint is classified as a ductile, i.e. a class 1 joint. If the failure mode is not in the shear zone, the joint is classified as intermediary for ductility, i.e. a class 2 joint.

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**.

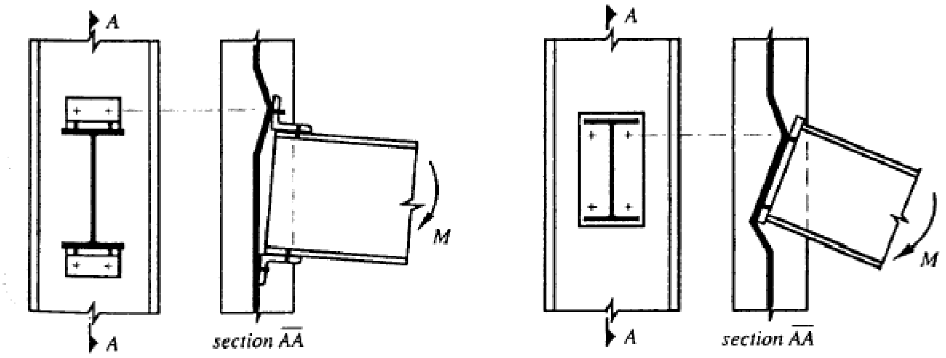


8.2. Column in minor axis configuration

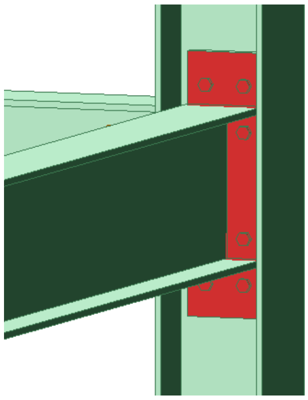
In beam-to-column minor-axis joints, the beam is directly connected to the web of an I-section column, causing bending about the minor-axis of the column section. In order to determine the strength of a column web in bending and punching, the following failure mechanisms are considered:

1. Local mechanism : the yield pattern is localised in the compression zone or in the tension zone
2. 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**.

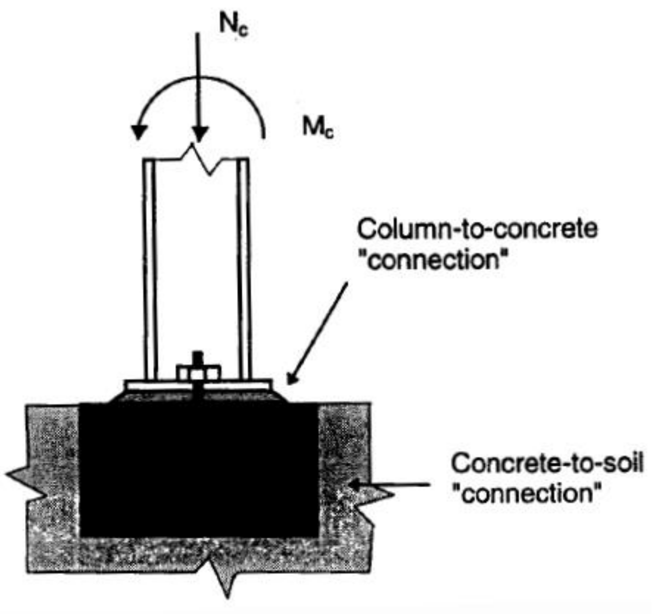


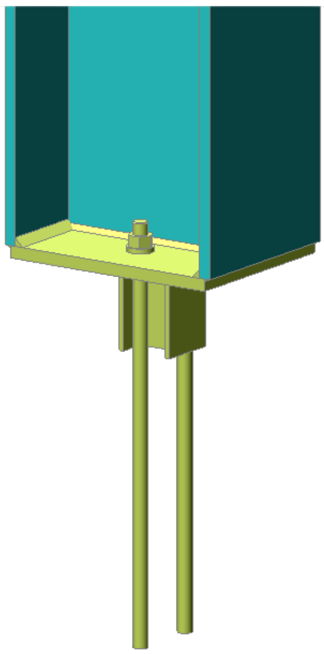
8.3. Base plate connections: shear iron, flange wideners

In a column base, 2 connection deformabilities 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.





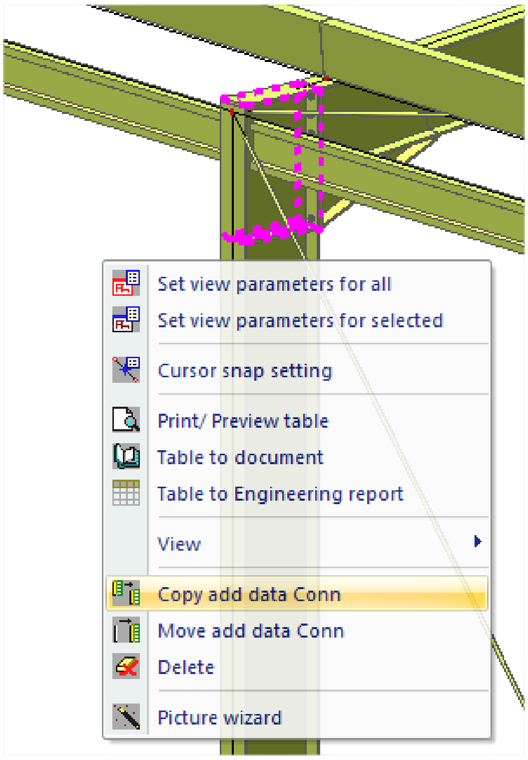
For more info about base plate design (shear irons, etc…), we refer to Ref.[2]. An example of a base plate connection in SCIA Engineer is given in **Example CON_005.esa, Node N9**.

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 “Check” in the menu “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.

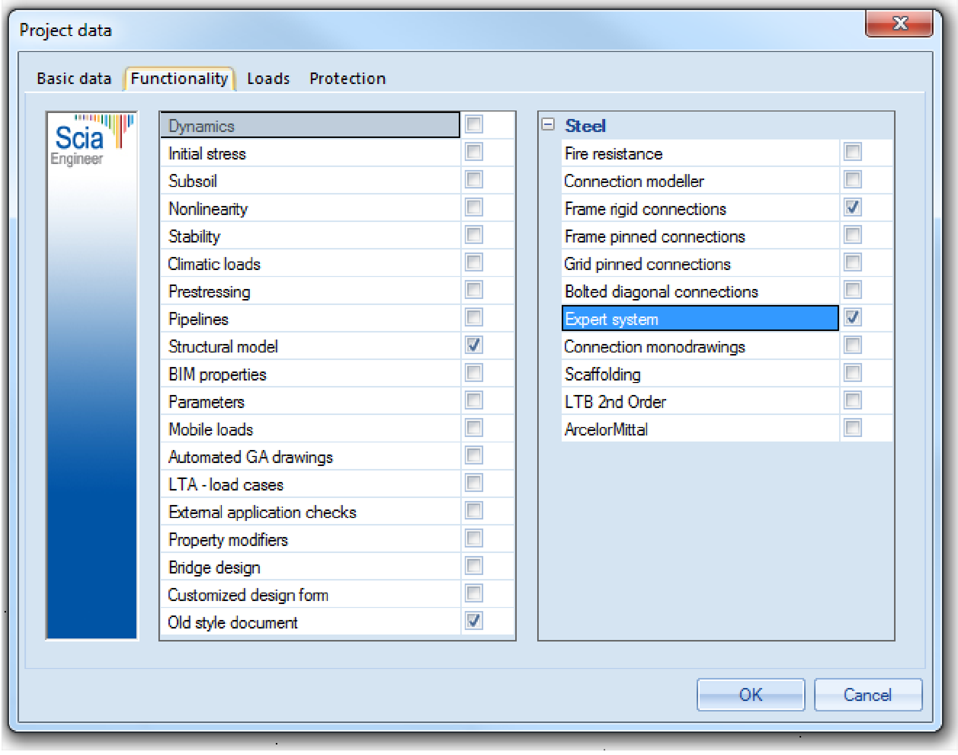
Check of connection

Steel	
Node	N2
Lc/Combi	NC2
Beam	B3
Unity check M/MRd [-]	0,74
Unity check V/VRd [-]	0,52
Unity check M/MRd + N/NRd [-]	0,81
Stiffness	Not applicable.
Node	N42
Lc/Combi	NC9
Beam	B35
Unity check M/MRd [-]	0,26
Unity check V/VRd [-]	0,05
Unity check M/MRd + N/NRd [-]	0,27
Stiffness	Not applicable.
Node	N44
Lc/Combi	NC2
Beam	B36
Unity check V/VRd [-]	0,06
Unity check M/MRd [-]	0,19
Unity check M/MRd + N/NRd [-]	0,19
Stiffness	Not applicable.
Node	N55
Lc/Combi	NC9
Beam	B43
Unity check M/MRd [-]	0,27
Unity check V/VRd [-]	0,05
Unity check M/MRd + N/NRd [-]	0,28
Stiffness	Not applicable.

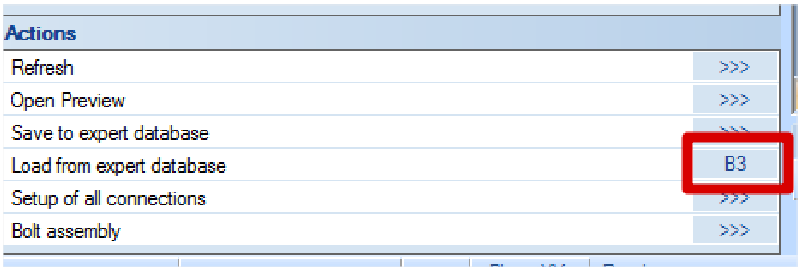
8.4.3. Expert system

For this option, an extra module is needed, more specific module esasd.07.

Open example **CON_004.esa** and go to the functionalities and activate the option “Expert system”:

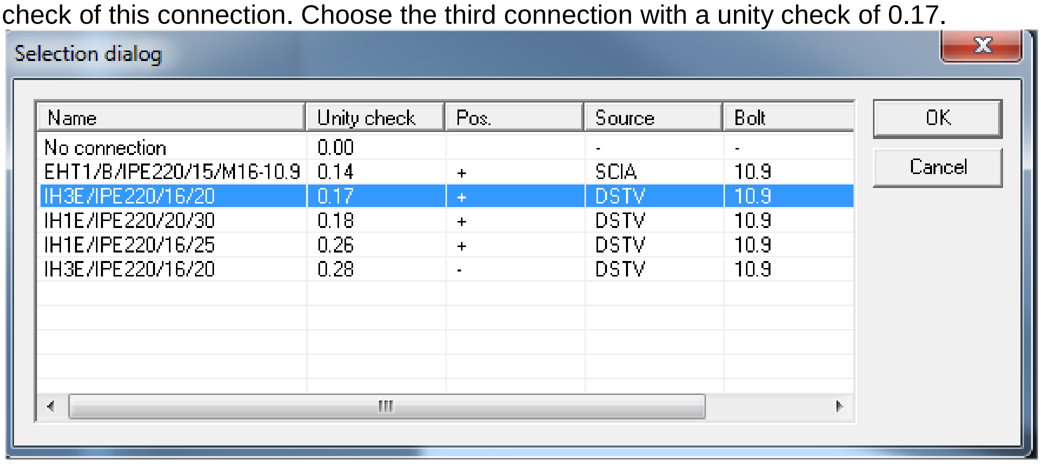


Now delete the connection “Conn” in this example and add a new connection to this node. In the Actions menu, you will the option “Load from expert database” and choose for this option:

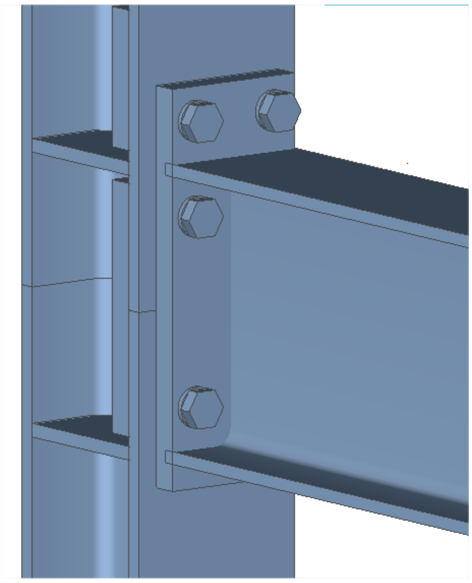


This option is only available if you did not add anything yet to the connection.

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.

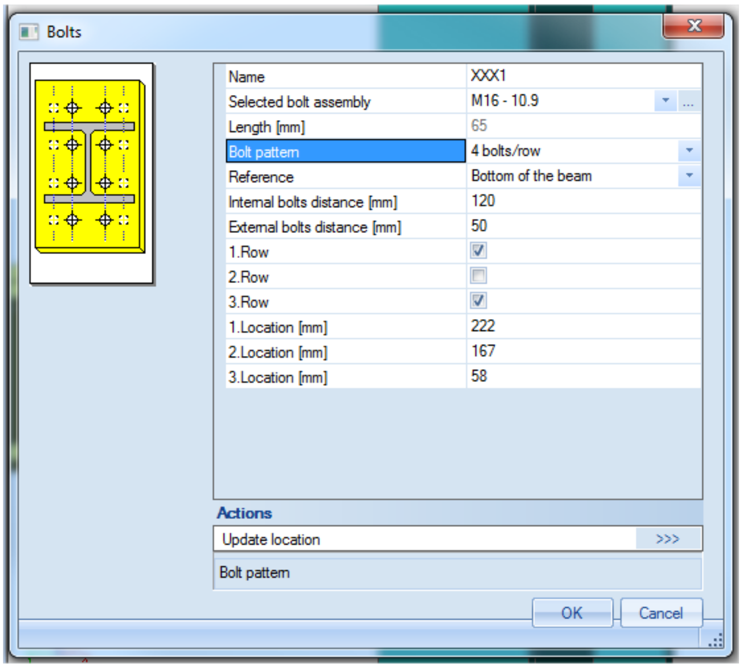
8.4.4. The use of 4 bolts / row

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:

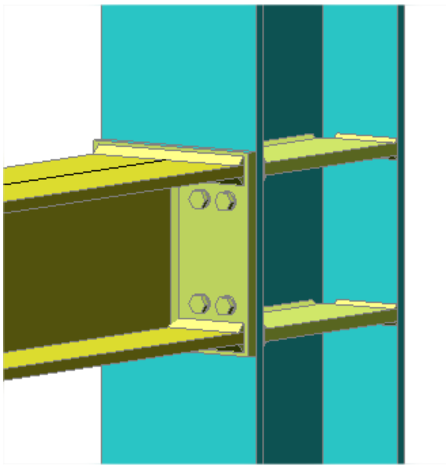
- the capacity of the inner two bolts is equal to the bolt tension resistance (failure mode 3) or is defined by a circular pattern
- the bolt row / group is stiffened
- the bolt group contains only 1 bolt row

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 three dots behind the bolts.



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

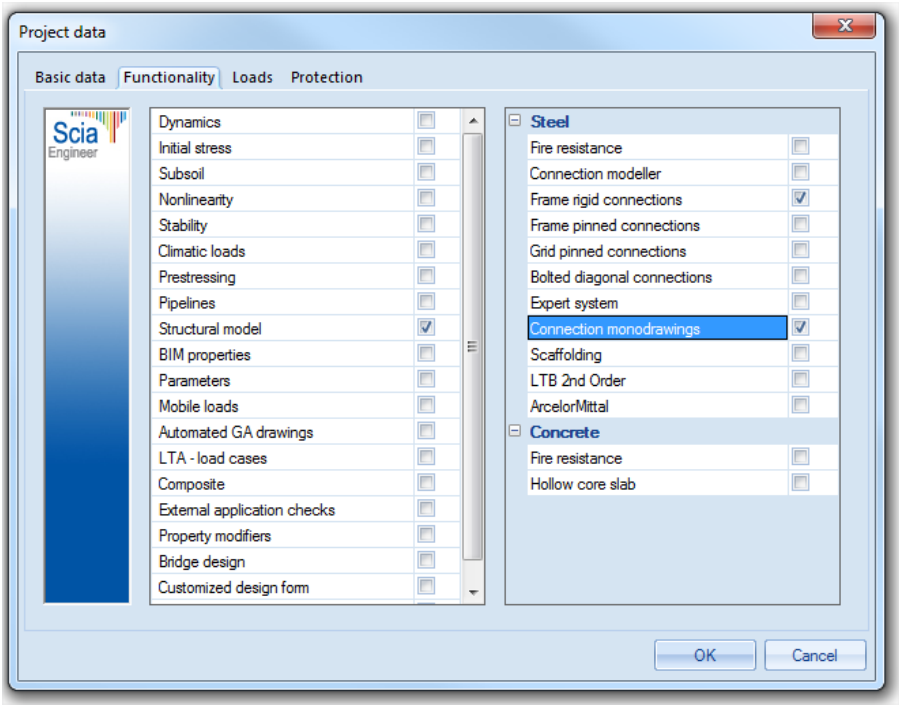


Additional capacity for 4 bolts/row

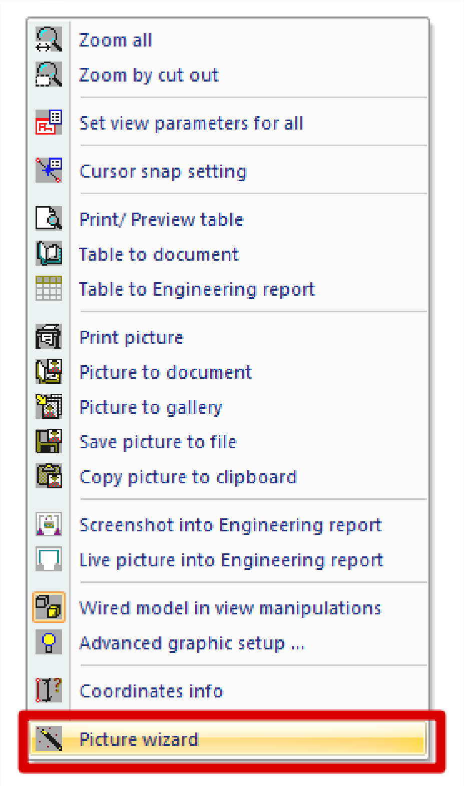
row	(Ft,fc,Rd)	group	(Ft,fc,Rd)
1	168.03	1- 1	168.03
2	0.00	1- 2	0.00

8.4.5. Monodrawings

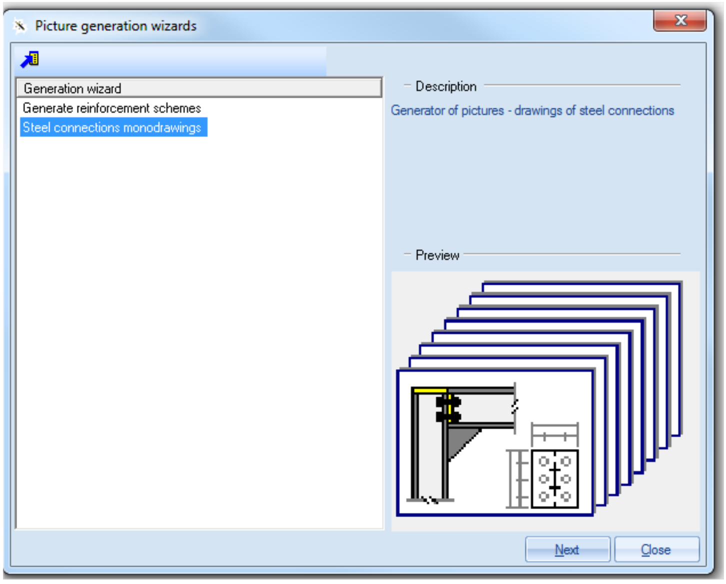
It is possible to make automatic connection drawings in SCIA Engineer. To use this option, it is necessary to select the functionality “Connection monodrawings”:



When this option is activated and you have one or more connections in the project, you can right click on the screen and choose for the “Picture wizard”:

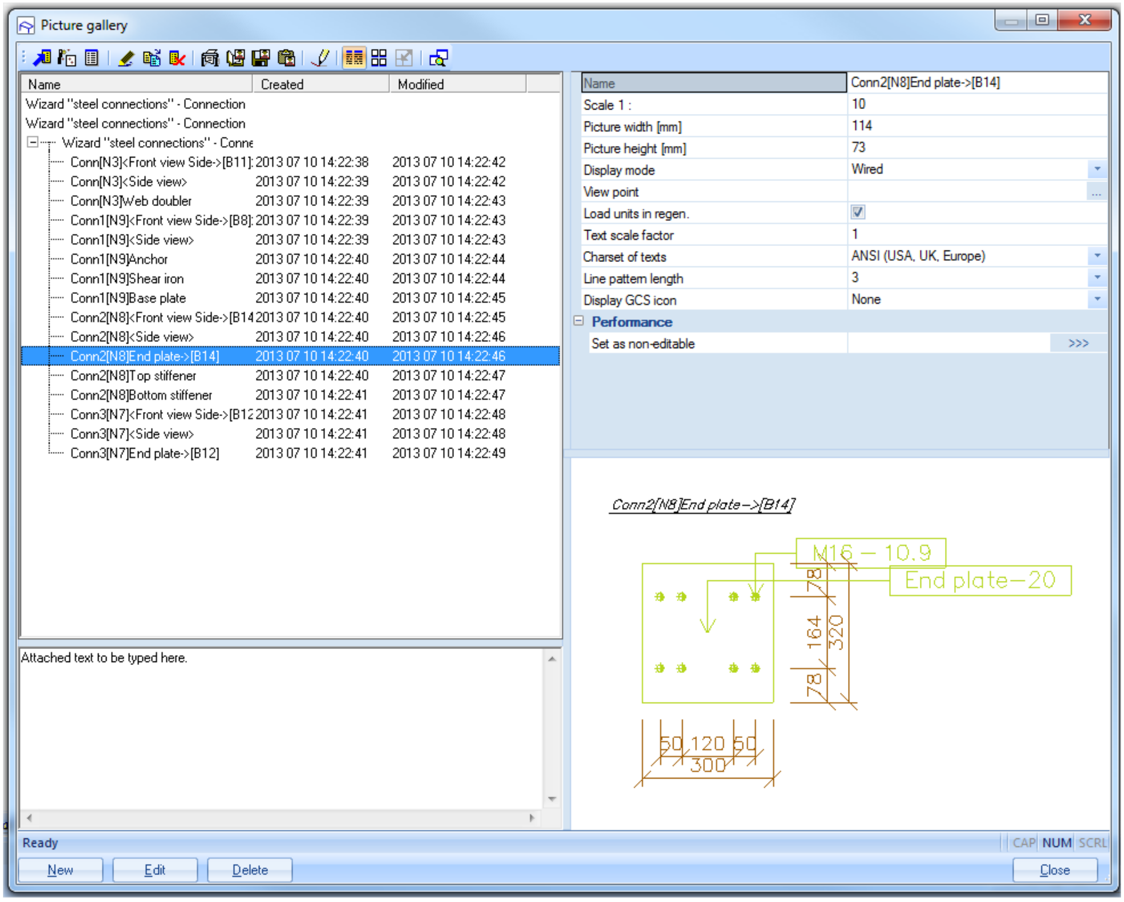


And choose to generate "Steel connections monodrawings":



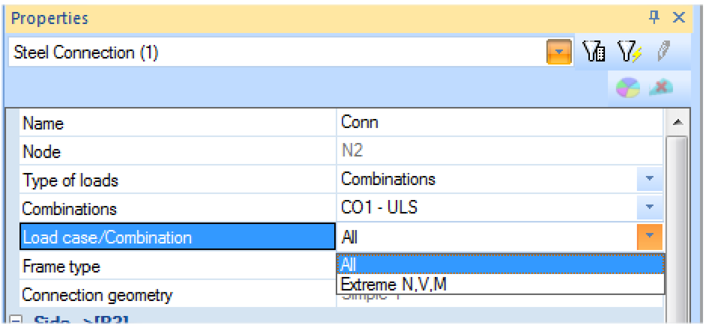
When the drawings are finished, they can be found in the Picture gallery:

And here all generated pictures can be found:



8.4.6. Options in the properties window

Load case/Combination



All

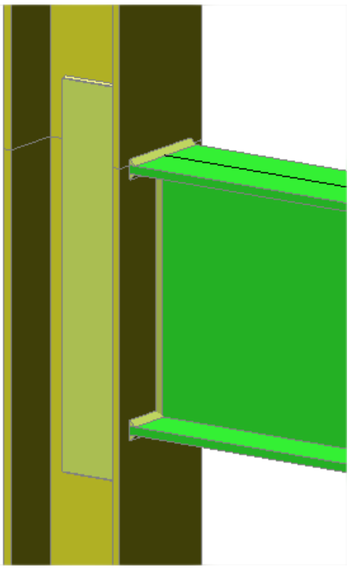
All combinations are checked.

Extreme N,V,M

In this case only 6 combinations are checked, more specific the combinations results in the biggest positive and biggest negative value for N, V and M.
Those biggest values are not combined together, but always with the real internal forces. This check will go a bit faster than when choosing for “All”.

With the option All, it is possible that not the biggest moment or biggest normal check will cause the biggest check, but a combination of the two smaller values in another combination. This check will not be shown when choosing for “Extreme N,V,M”.

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		
N	147.75	kN
Vz	85.00	kN
My	-123.03	kNm

Tension top

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

Calculation $V_{wp,Rd}$: Column web panel in shear

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

When a web doubler is used:

$$A_{vc}' = A_{vc} + b_s t_s$$
$$A_{vc} = A - 2bt_f + (t_w + 2r)t_f$$
$$A_{vc} = 13140 - 2 \cdot 280 \cdot 18 + (10.5 + 2 \cdot 24)18$$
$$A_{vc} = 4113 mm^2$$
$$A_{vc}' = 4113 + 172 \cdot 10.5 = 5919 mm^2$$
$$\text{Vwp,Rd} = \frac{0.9 \cdot 235 \cdot 5919}{\sqrt{3} \cdot 1.0} = 727{,}77 kN$$

And the same is shown in SCIA Engineer:

2.1. Design resistance of basic components
2.1.1. Column web panel in shear (EN 1993-1-8 art. 6.2.6.1)

Column web in shear (Vwp,Rd) data		
Column web in shear (Vwp,Rd)	722.77	kN
Beta	1.00	
Avc	5919.00	mm^2

Calculation $F_{c,wc,Rd}$: Column web in compression

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

$$t_{wc} = 1.5 t_w = 1.5 \cdot 10.5 = 15.75 mm$$

$$b_{eff} = t_{fb} + 2\sqrt{2}a + 5(t_{fc} + r)$$

$$b_{eff} = 17.2 + 2\sqrt{2} \cdot 9 + 5(18 + 24) = 252{,}66 mm$$

$$\rho = \rho_1$$

$$\rho_1 = \frac{1}{\sqrt{1 + 1.3\left(\frac{b_{eff} t_{wc}}{A_{vc}'}\right)^2}} = 0.79$$

$$F_{c,wc,Rd} = \frac{0{,}79 \cdot 252{,}66 \cdot 15{,}75 \cdot 235}{1{,}0} = 674 kN = 738{,}77 kN$$

And in SCIA Engineer:

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

Column web in compression (Fc,wc,Rd) data		
Column web in compression (Fc,wc,Rd)	742.18	kN
beff,c,wc	252.66	mm
twc	15.75	mm
omega 1	0.79	
omega 2	0.55	
omega	0.79	
kwc	1.00	
lambda rel	0.44	
reduction factor for plate buckling	1.00	
dwc	196.00	mm

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

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

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

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

In SCIA Engineer:

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

Beam flange in compression (Fc,fb,Rd) data		
Beam flange in compression (Fc,fb,Rd)	1226.16	kN
section class	1	
Mc,Rd	653.30	kNm
hb-tfb	532.80	mm

Calculation $F_{t,fc,Rd}$: Column flange in bending

$$F_{c,fc,Rd} = (t_{wc} + 2r_c + 7kt_{fc})\frac{t_{wc} f_y}{\gamma_{M0}}$$

$$F_{c,fc,Rd} = (10.5 + 2 \cdot 24 + 7 \cdot 1 \cdot 18)\frac{17.20 \cdot 235}{1.0} = 745kN$$

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

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

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

$$b_{eff} = t_{fb} + 2\sqrt{2}a + 5(t_{fc} + r)$$

$$b_{eff} = 17.2 + 2\sqrt{2} \cdot 9 + 5(18 + 24) = 252mm$$

$$\rho = \rho_1$$

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

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

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)	709.84	kN
beff	252.66	mm
twc	14.70	mm
omega 1	0.81	
omega 2	0.57	
omega	0.81	

Calculation MRd : Design moment resistance

705 kN x 0.532 m = 375 kNm

In SCIA Engineer:

2.3. Determination of Mj,Rd
According to EN 1993-1-8 Article 6.2.7.1 (4)

Mj,Rd data		
F	709.84	kN
h	532.80	mm
Mj,Rd	378.20	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:

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

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

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:

γ = 1.7 for sway frames
γ = 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 $N_{t.Rd}$:

$$N_{t,Rd} = \frac{b_f \cdot t_{fb} \cdot f_{yb}}{\gamma_{M_0}}$$

$$N_{t,Rd} = \frac{210 \cdot 17.2 \cdot 235}{1.0} = 849kN$$

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

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

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

$$a_f \geq \frac{849000 \cdot 1.25 \cdot 0.8}{360 \cdot 210 \cdot \sqrt{2}} = 7.94mm$$

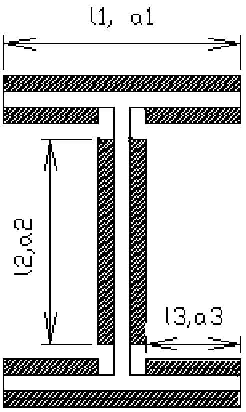
We take a_f=9 mm.

In SCIA Engineer

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

data		
MRd	378.20	kNm
Gamma	1.40	
h	532.80	mm
FRd	993.78	kN
NT,Rd	848.82	kN
N	848.82	kN
Fu	360.00	MPa
BetaW	0.80	
minimum af	7.94	mm
af	9.00	mm
Minimum th	17.20	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 MRd 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	the ultimate tensile strength of the weaker part
	β_W	the correlation factor
	γ_{Mw}	the partial safety factor for welds

In SCIA Engineer:

6.2. Calculation aw

data		
M	378.20	kNm
N	147.75	kN
V	85.00	kN
Fu	360.00	MPa
BetaW	0.80	
a1	9.00	mm
a3	9.00	mm
l1	210.00	mm
l2	450.40	mm
l3	75.45	mm
A	7397.00	mm^2
I	481611467.86	mm^4
minimum aw (a2)	1.00	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

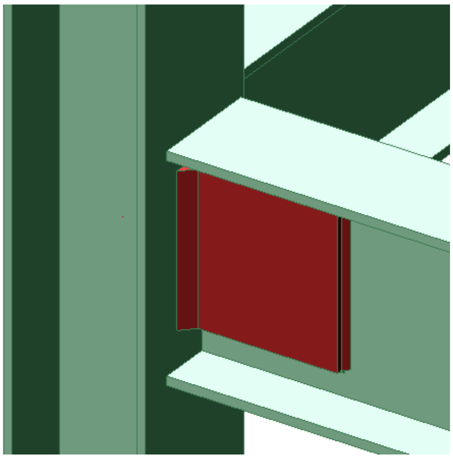
The design compression/tension resistance is calculated for the following failure modes:

- 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	-1.44	kN
Vz	7.97	kN
My	0.00	kNm

Calculation Design Shear Resistance VRd for Connection Element

Transversal section of the plate: $A_{pl} = 2 \cdot h_{pl} \cdot t_{pl} = 0.003912 m^2$ (2 plates)

Normal stress: $\sigma_N = \dfrac{N}{A_{pl}} = \dfrac{262.4939}{0.003912} = 67099.667 \, {}^{N}\!/_{m^2}$

Flexion module: $W_{pl} = 2 \cdot \dfrac{t_{pl} \cdot h_{pl}^2}{6} = 0.000106276 m^3$

Design Shear Resistance: *a= 0.082 m* is the centre

$$V_{Rd} = \frac{-\dfrac{2\cdot\sigma_N\cdot a}{W_{pl}} + \sqrt{\left(\dfrac{2\cdot\sigma_N\cdot a}{W_{pl}}\right)^2 - 4\cdot\left(\dfrac{a^2}{N^2}\right)\cdot\left(\sigma_N^2 - \dfrac{f_y^2}{\gamma_{M_0}^2}\right)}}{2\cdot\left(\dfrac{a^2}{W_{pl}^2} + \dfrac{3}{A_{pl}^{\ 2}}\right)} = 240087.66N = 240kN$$

Calculation Design Shear Resistance VRd for Beam

Shear Area : $A_v = A - 2\cdot b\cdot t_f + (t_w + 2\cdot r)\cdot t_f = 0.00191276 m^2$

Shear Resistance : $VRd = \dfrac{A_v \cdot f_y}{\sqrt{3}\cdot\gamma_{M_0}} = 235925.57N = 236kN$

Calculation Design Tension Resistance NRd for Connection Element

Area of the element : $A_{pl} = 2\cdot h_{pl}\cdot t_{pl} = 0.003912 m^2$

Tension Resistance : $N_{Rd} = \dfrac{A_{pl}\cdot f_y}{\gamma_{M_0}} = 835745.45N = 835kN$

Calculation Design Tension Resistance NRd for Beam

Area of the Beam : $A = 0.003910 m^2$

Tension Resistance : $N_{Rd} = \dfrac{A\cdot f_y}{\gamma_{M_0}} = 835318.18N = 835kN$

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=4mm* and for weld between plate and column, the weld size is *a=10mm*.

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

Weld Length: $l_1 = h_{pl} - 2\cdot t_{pl} = 0.163 - 2\cdot 0.12 = 0.139m$

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

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

By EC3: f_{uw}=360000000N/m² and β_w=0.8. The parameters are:

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

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

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

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

$$L = 10 + g$$

Shear force on one plate: $D = \frac{V_{Rd}}{2} = 117962.789\text{N}$ (for one plate)

Normal force on one plate: $N = \frac{N}{2} = 131.24\text{N}$

Moment on the plate: $M = D \cdot L = 13459.781\text{Nm}$

Weld Check 1:

$$\tau_1 = \sigma_1 = \frac{6 \cdot \mu \cdot M}{\sqrt{2} \cdot a \cdot l_1^2} + \frac{\Gamma \cdot N}{\sqrt{2} \cdot a \cdot l_1} == 115363040.74 \, {}^{N}\!/\!_{m^2}$$

$$\tau_2 = \frac{\delta \cdot D}{a \cdot l_1} = 64449431.72 \, {}^{N}\!/\!_{m^2}$$

Unity Check: $\frac{\sqrt{\sigma_1^2 + 3 \cdot (\tau_1^2 + \tau_2^2)}}{f_u / \beta_w \cdot \gamma_{M_w}} = 0.316 \leq 1$ and $\frac{\sigma_1}{f_u / \gamma_{M_w}} = 0.4 \leq 1$

Weld Check 2:

$$\sigma_1 = \tau_1 = \frac{(1-\delta) \cdot D}{2 \cdot \sqrt{2} \cdot a \cdot l_2} = 55840293.8731 \, {}^{N}\!/\!_{m^2}$$

$$\tau_2 = \left(\frac{(1-\mu) \cdot M}{h \cdot a \cdot l_2} + \frac{(1-\Gamma) \cdot N}{2 \cdot a \cdot l_2} \right) = 134095932.161 \, {}^{N}\!/\!_{m^2}$$

Unity Check: $\frac{\sqrt{\sigma_1^2 + 3 \cdot (\tau_1^2 + \tau_2^2)}}{f_u / \beta_w \cdot \gamma_{M_w}} = 0.715 \leq 1$ and $\frac{\sigma_1}{f_u / \gamma_{M_w}} = 0.193 \leq 1$

Weld size Plate/Column

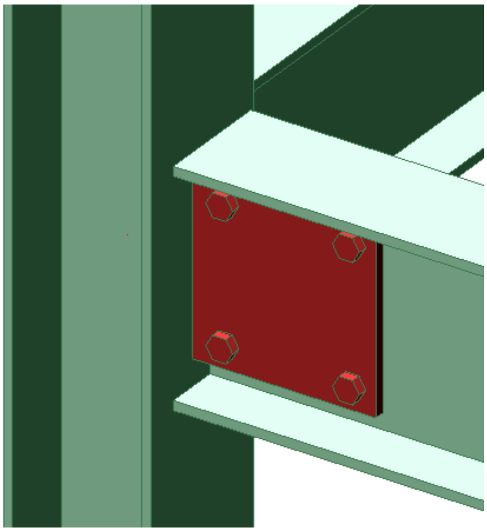
Weld size: a=0.01 m
Normal Force: N=262.4939N
Moment: $M = D \cdot L = 235925.57 \cdot 0.082 = 19345.89674\text{Nm}$
Stress Calculation:

$$\sigma_1 = -\tau_1 = \frac{N}{2 \cdot \sqrt{2} \cdot a \cdot l} + \frac{M}{W} = \frac{N}{2 \cdot \sqrt{2} \cdot a \cdot l} + \frac{D \cdot L}{2 \cdot \sqrt{2} \cdot a \cdot \frac{h_{pl}^2}{6}} = 154518316.96 \, {}^{N}\!/\!_{m^2}$$

$$\tau_2 = \frac{D}{2 \cdot a \cdot l} = 72369806.7485 \, {}^{N}\!/\!_{m^2}$$

Unity Check: $\frac{\sqrt{\sigma_1^2 + 3 \cdot (\tau_1^2 + \tau_2^2)}}{f_u / \beta_w \cdot \gamma_{M_w}} = 0.92 \leq 1$ $\qquad \frac{\sigma_1}{f_u / \gamma_{M_w}} = 0.53 \leq 1$

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 N7**.

Calculation Design Shear Resistance VRd for Connection Element

Transversal section of the plate:

$A = 2 \cdot h \cdot t = 2 \cdot 0.188 \cdot 0.012 = 0.004512 m^2$ (2 plates)

Normal Stress: $\sigma_N = \frac{N}{A} = \frac{262.4939}{0.004512} = 58176.8395 \, N/m^2$

Flexion Module : $W = 2 \cdot \frac{t \cdot h^2}{6} = 0.000141376 m^3$

Bolt Centre : a=0.0995m

Design Shear Resistance:

$$V_{Rd} = \frac{-\frac{2 \cdot \sigma_N \cdot a}{W} + \sqrt{\left(\frac{2 \cdot \sigma_N \cdot a}{W}\right)^2 - 4 \cdot \left(\frac{a^2}{N^2}\right) \cdot \left(\sigma_N^2 - \frac{f_y^2}{\gamma_{M_0}^2}\right)}}{2 \cdot \left(\frac{a^2}{W^2} + \frac{3}{A^2}\right)} = 266422.015N = 266kN$$

Calculation Design Shear Resistance VRd for Beam

Shear Area : $A_v = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f = 0.00191276 m^2$

Net Area : $A_{net} = A_v - 2 \cdot t_w \cdot d_0 = 0.001689.56m$

For the calculation of VRd in the beam, we use A_v because $A_{net} \geq \frac{f_y}{f_u} \cdot A_v = 0.00124860 m^2$

Shear Resistance: $VRd = \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{M_0}} = 235925.57N = 236kN$

Calculation Design Tension Resistance NRd for Connection Element

Area : $A = 2 \cdot t \cdot h = 0.004512 m^2$

Net Area : $A_{net} = A - 2 \cdot t \cdot 2 \cdot d_0 = 0.003648 m^2$

Tension Resistance :

$$N_{Rd} = \min\left(\frac{A \cdot f_y}{\gamma_{M_0}}, \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M_1}}\right) = \min(963927.27, 1074501.81) = 963927.27N = 963kN$$

Calculation Design Tension Resistance NRd for Beam

Area : $A = 0.003910 m^2$

Net Area : $A_{net} = A - 2 \cdot t \cdot d_0 = 0.0036868 m^2$

Tension Resistance : $N_{Rd} = \min\left(\frac{A \cdot f_y}{\gamma_{M_0}}, \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M_1}}\right)$

$= \min(835318.18, 1085930.18) = 835318.18N = 836kN$

Calculation Design Shear Resistance VRd for Bolt in Beam

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

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

Where : $a = 0.0995\text{m}$ $b = 0.094\text{m}$ $c = 0.0655\text{m}$ $d = 0.07\text{m}$

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

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

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

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

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

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

By solving the second-degree equation, we find $VRd = 67907.89\text{N} = 67.9\text{kN}$

Calculation Design Block Shear Resistance

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

$$V_{eff,Rd} = \frac{f_y \cdot A_{v,eff}}{\sqrt{3} \cdot \gamma_{M_0}} \quad \text{with } A_{v,eff} = t \cdot L_{v,eff}$$

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

$a_1 = 0.049\text{m}$ $a_2 = 0.155\text{m}$ $a_3 = 0.051\text{m}$

$L_v = h - a_1 - a_2 = 0.14\text{m}$

$$L_3 = \min\left(L_v + a_1 + a_3; \left(L_v + a_1 + a_3 - n \cdot d_0\right) \cdot \frac{f_u}{f_y} \right)$$

$$= \min\left(0.24; 0.2849\right) = 0.24\text{m}$$

$L_1 = \min\left(a_1; 5 \cdot d_0\right) = 0.049\text{m}$

$$L_2 = \left(a_2 - k \cdot d_0\right) \cdot \frac{f_u}{f_y} = 0.1685\text{m}$$

$$\text{with } k = 2.5 \text{ for 2 rows bolt}$$

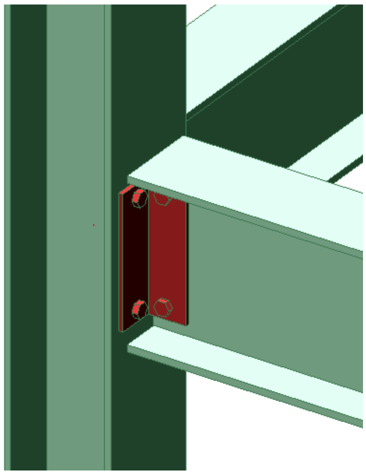
$L_{v,eff} = \min\left(L_v + L_1 + L_2; L_3\right) = 0.24\text{m}$

$A_{v,eff} = 0.001488\text{m}^2$

$$V_{eff,Rd} = \frac{f_y \cdot A_{v,eff}}{\sqrt{3} \cdot \gamma_{M0}} = 183534.40\text{N} = 185\text{kN}$$

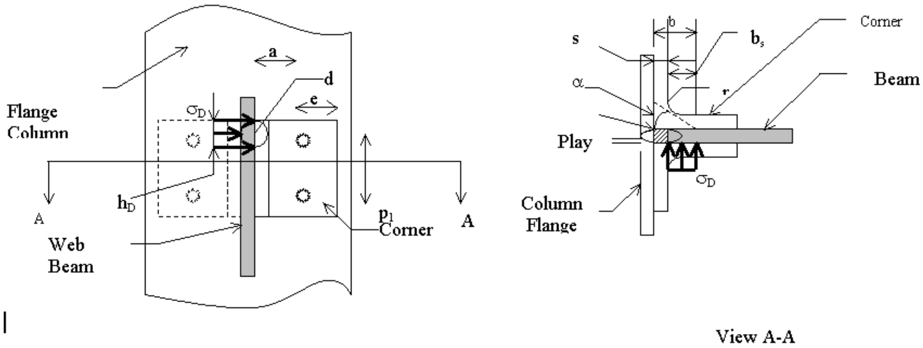
10.3. Bolted cleat connection

In this chapter we will show the calculation of a bolted cleat connection using example **CON_011.esa, node N2**.



To determine the design shear resistance for bolts in the flange of the column, we use a iterative process with h_D as parameter until we reach a equilibrium :

$$\sigma_D = \min\left(\frac{f_{y,beam}}{\gamma_{M_0}}, \frac{f_{y,cor}}{\gamma_{M_0}}\right)$$



We'll only consider the check for the final value of h_D. We have the following data:

$r = 0.008\text{m}$ $\quad a = 0.03\text{m}$ $\quad s = 0.006\text{m}$

$b = 0.4227 \cdot r + 1.577 \cdot s = 0.0128436\text{m}$ $\quad b_s = b - \text{Play} = 0.0028436\text{m}$

$h_D = 0.011\text{m}$ $\quad I_{pD} = \sum r_{i_D}^2 = 0.029194\text{m}^2$

We compute: $K = \dfrac{a}{I_{p_D} - n \cdot a^2} = 1.0951$

We define x_j=0.03m and z_j=0.165m respectively as the maximum horizontal distance between bolts and d and the maximum vertical distance between the bolts and d. It corresponds to the further bolts how is submitted to the higher force.

$$A = \frac{1}{n} + K \cdot (a - x_j) = 0.5 \qquad B = K \cdot z_j = 0.18069$$

$$Q = \min\left(F_{b,Rd}, F_{v,Rd} \cdot \left(1 - \frac{N}{1.4 \cdot n \cdot F_{t,Rd}}\right)\right) = 16224\text{N} \quad \text{where:}$$

$$F_{V_{Rd},cor} = \frac{0.6 \cdot f_{ub} \cdot A_s}{\gamma_{Mb}} = 16224\text{N} \quad \text{and} \quad F_{t,Rd} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{Mb}} = 24336\text{N}$$

$$F_{b,Rd,cor} = \frac{2.5 \cdot \alpha_p \cdot f_u \cdot d \cdot t_{cor}}{\gamma_{Mb}} = 22187.2N$$

$$\text{with } \alpha_p = \min\left(\frac{e_1}{3d_0}; \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1\right) = 0.428$$

$$F_{b,Rd,flange} = \frac{2.5 \cdot \alpha_p \cdot f_u \cdot d \cdot t_{flange}}{\gamma_{Mb}} = 77.8kN$$

$$\text{with } \alpha_p = \min\left(\frac{e_1}{3d_0}; \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1\right) = 1.0$$

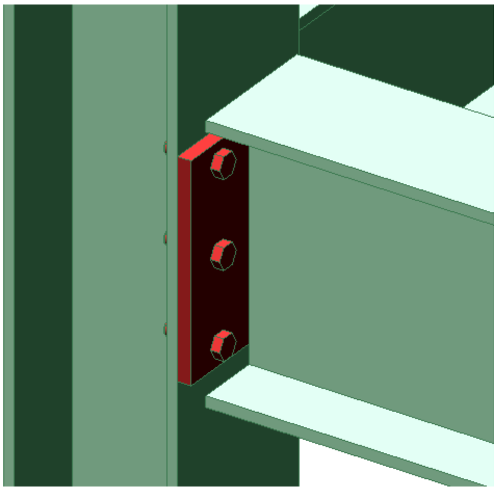
With those values, we have:

$$V_{Rd,ColFlange} = \frac{2 \cdot Q}{\sqrt{A^2 + B^2}} = 61032.94N \qquad \sum Q_h = V_{Rd} \cdot K \cdot \sum z_j = 5948.64N$$

$$\sigma_D = \frac{\sum Q_h}{h_D \cdot b_D} = 209194259.89 \, {}^{N}\!/\!{}_{m^2}$$

10.4. Flexible end plate connection

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



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