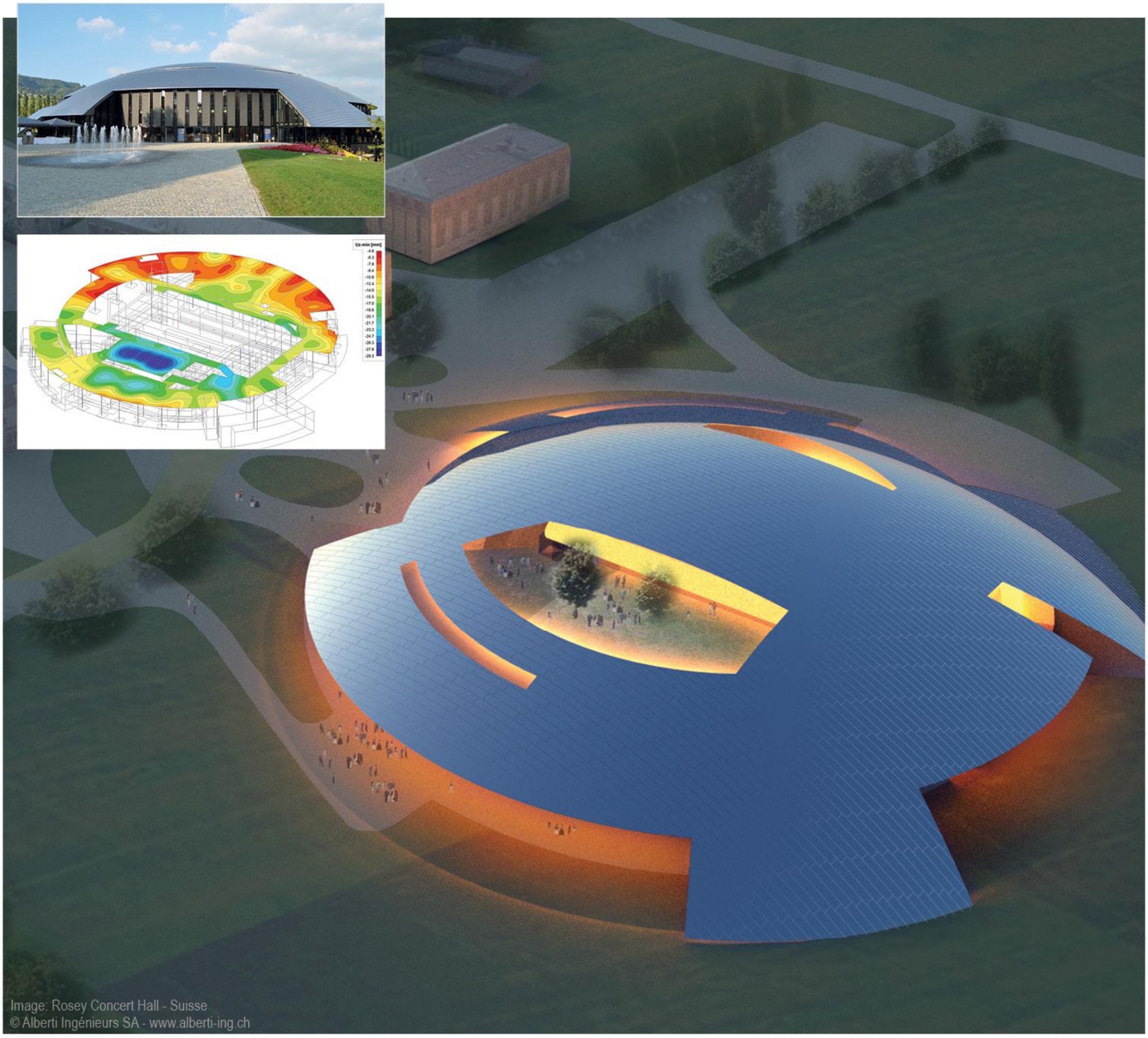
SCIA ENGINEER



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Advanced Concept Training

Steel Code Check

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Introduction

This course will explain the calculation of steel following the EN 1993-1-1: General rules and rules for buildings and EN 1993-1-2: Design of Steel Structures: Structural fire design.

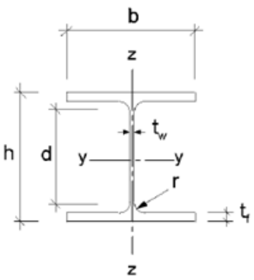
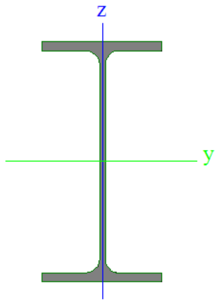
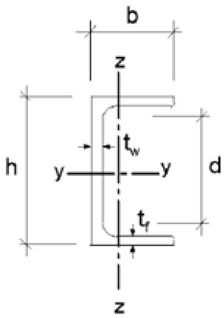
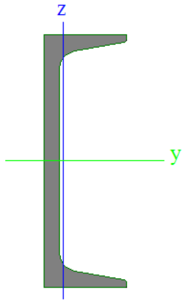
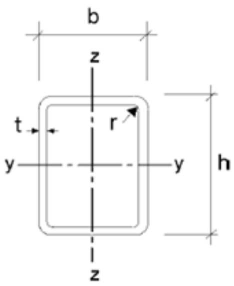
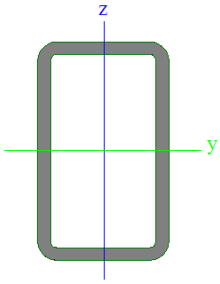
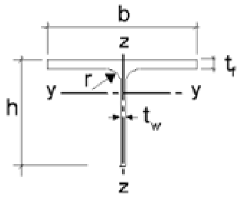
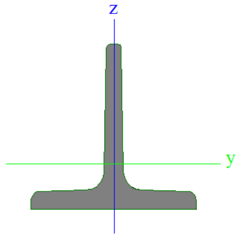
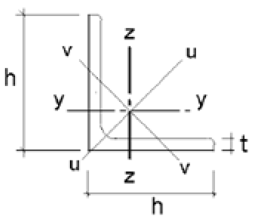
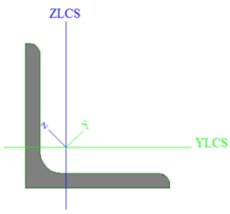
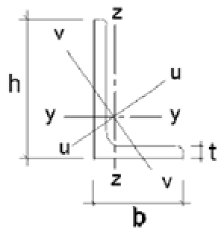
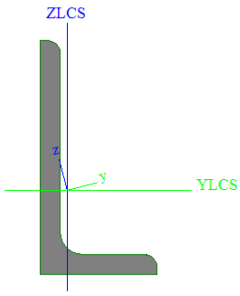
Most of the options in the course can be calculated/checked in SCIA Engineer with the **Concept edition**.

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

Cross-sections

SCIA Engineer will use the axes y-y and z-z respectively for the major and minor principal axes of the cross section.

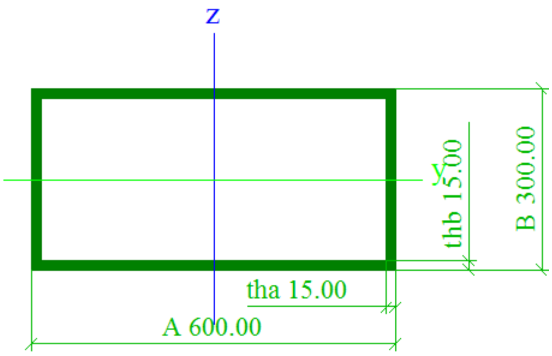
If the principal axes not coincide with the y-y and z-z axes following the EN 1993-1-1, also those axes are indicated:

EN 1993-1-1	SCIA Engineer	EN 1993-1-1	SCIA Engineer
			
			
			

In the steel code check, the strong axis will always be considered as the y-y axis. So when inputting a profile in which the local y-axis does not corresponds with the strong axis, the axes are switched in the steel code check and SCIA Engineer will give a message about the axes.

Example: Local Axes.esa

The strong axis of this cross section in the local z-axis:



This beam is only loaded by a line load in the z-direction, resulting in a moment My and a shear force Vz on the beam.

When looking at the steel code check, the axes are switched and the strong axis is taken as local y-axis:

The critical check is on position 0.000 m

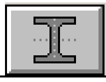
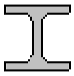
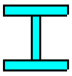
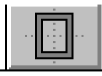
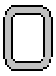
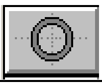
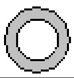
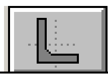
Axis definition :
- principal y- axis in this code check is referring to the principal z axis in Scia Engineer
- principal z- axis in this code check is referring to the principal y axis in Scia Engineer

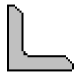
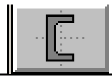
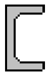
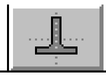
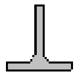
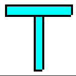
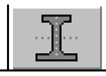
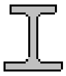
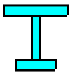
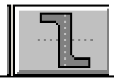
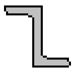
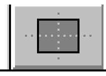
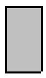
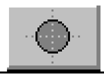
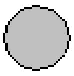
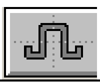
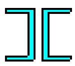
Internal forces	Calculated	Unit
N,Ed	0,00	kN
Vy,Ed	300,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	-300,00	kNm

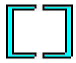
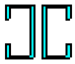
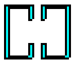
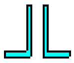
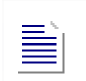
This is indicated in steel code check and also the internal forces are switched following the new local axis.

The following sections are classified during the steel code checks

So as I profile only the indicated profiles are recognized. The other I-forms are not recognized as an I-profile, and e.g. a classification cannot be executed for those forms.

I	Symmetric I shape	
	from library	
	thin walled geometric	
	sheet welded Iw	
RHS	Rectangular Hollow Section	
	from library	
	thin walled geometric	
CHS	Circular Hollow Section	
	from library	
	thin walled geometric	
L	Angle Section	
	from library	

	thin walled geometric	
U	Channel Section	
	from library	
	thin walled geometric	
T	T Section	
	from library	
	thin walled geometric	
	sheet welded Tw	
PPL	Asymmetric I shape	
	from library	
	thin walled geometric	
	sheet welded Iwn	
Z	Z Section	
	from library	
	thin walled geometric	
RS	Rectangular Sections	
	from library	
	thin walled geometric	
O	Solid Tube	
	from library	
	thin walled geometric	
Σ	Cold formed sections	
	from library	
	pairs 2CFUo	

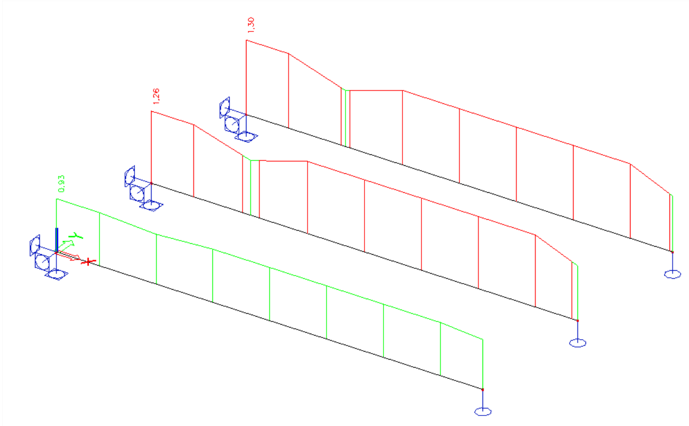
	pairs 2CFUc	
	pairs 2CFCo	
	pairs 2CFCc	
	pairs 2CFLT	
	General Cross Section: thin walled	
NUM	Numerical section	
	Numerical section	
COM	all others	

The standard profile sections (from library) have fixed sections properties and dimensions, which have to be present in the profile library. The section properties and the required dimension properties are described in the Theoretical Background (Ref.[32]).

Example: Cross Section.esa

- 3 cross sections:
  - HEA300 from library
  - HEA300 inputted as a general cross section (Imported from dwg)
  - HEA 300 inputted as a general shape
- Only the first profile is recognized as a symmetric I-shape
- The second and third profile are not recognized as a symmetric I-shape
- The first profile will be classified as an I-profile and a plastic check will be performed.

- The second and third profile cannot be classified, so an elastic check will be performed. This will results in different checks:



Remark:

The general cross section cannot be imported in SCIA Engineer with the concept edition. For this option the Professional or Expert edition is necessary or module esa.07.

Materials

For standard steel grades, the yield strength f_y and tensile strength f_u are defined according to the thickness of the element.

The standard steel grades as defined in Table 3.1 of the EN 1993-1-1 are:

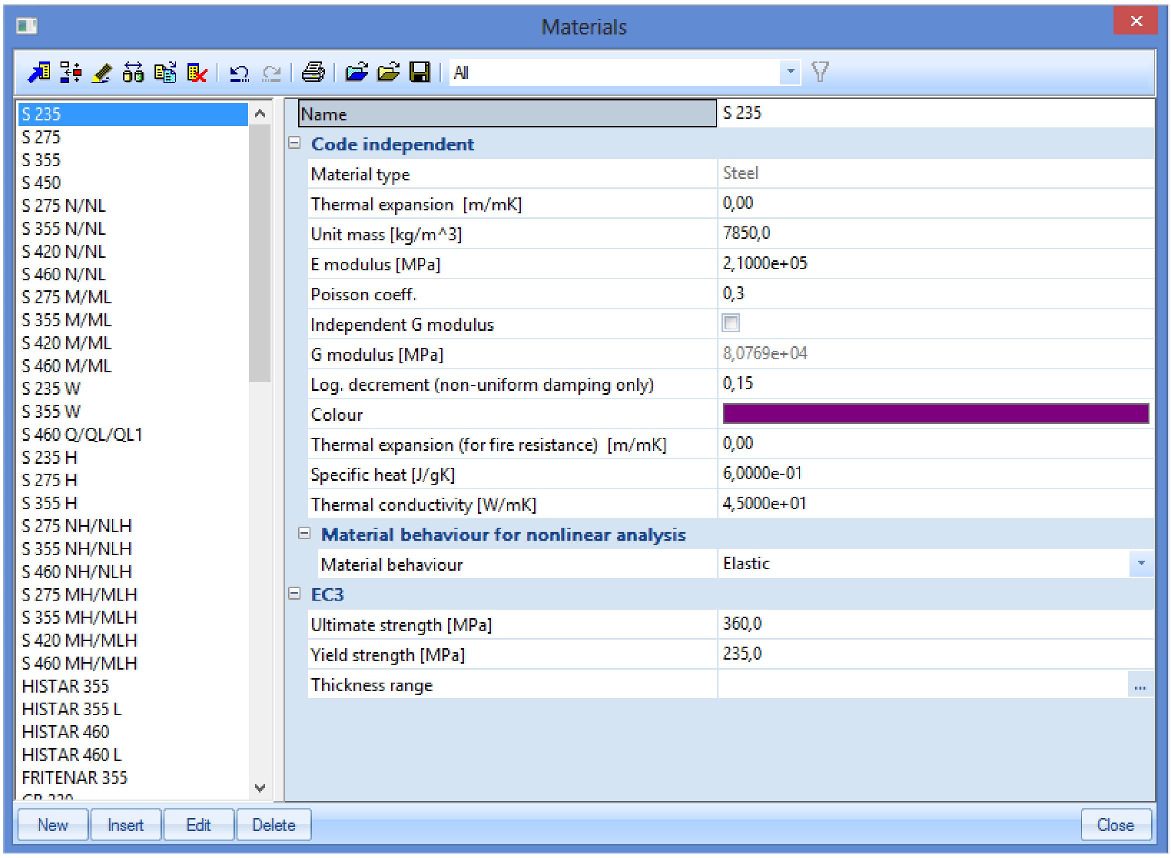
Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

Table 3.1 (continued): Nominal values of yield strength f_y and ultimate tensile strength f_u for structural hollow sections

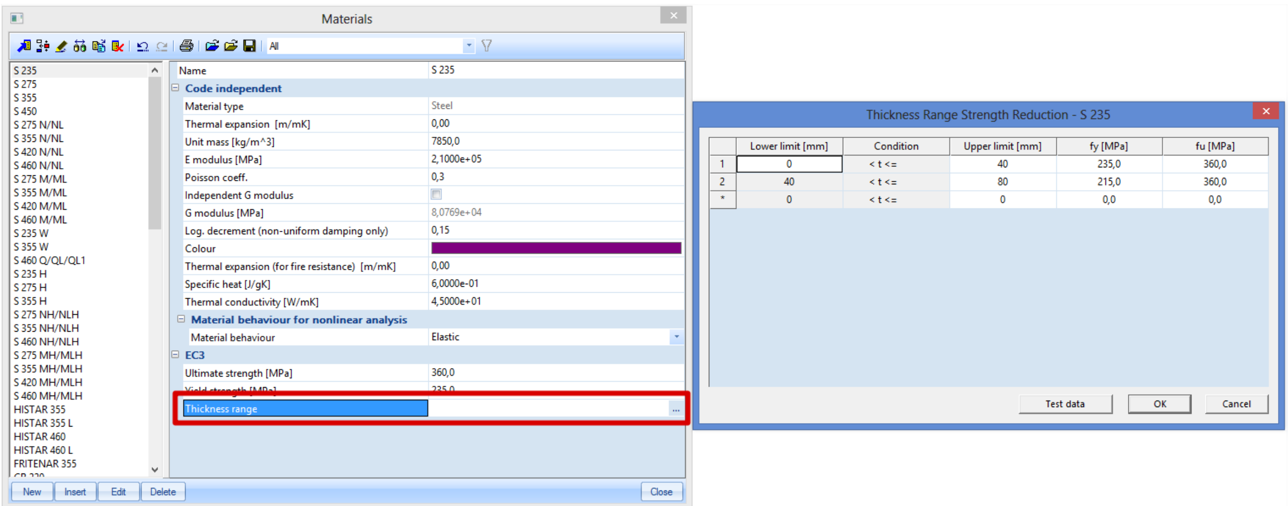
Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10210-1				
S 235 H	235	360	215	340
S 275 H	275	430	255	410
S 355 H	355	510	335	490
S 275 NH/NLH	275	390	255	370
S 355 NH/NLH	355	490	335	470
S 420 NH/NHL	420	540	390	520
S 460 NH/NLH	460	560	430	550
EN 10219-1				
S 235 H	235	360		
S 275 H	275	430		
S 355 H	355	510		
S 275 NH/NLH	275	370		
S 355 NH/NLH	355	470		
S 460 NH/NLH	460	550		
S 275 MH/MLH	275	360		
S 355 MH/MLH	355	470		
S 420 MH/MLH	420	500		
S 460 MH/MLH	460	530		

Those materials are included in SCIA Engineer:



The Histar and Fritenar steel grades have been implemented according to Arcelor.

With the option “Thickness range” the influence of the thickness on the yield strength f_y and tensile strength f_u are defined. When inputting a new user defined material in SCIA Engineer, also this thickness range for this material can be inputted:



Example: NA_Material_Strength_Application.esa

In this example a material of the National Annex of Belgium has been inputted manually: S 275 J2. This material has a lot of thickness range strength reductions:

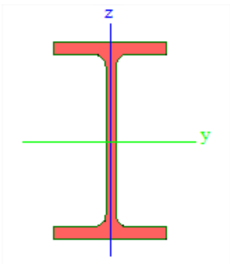
Thickness Range Strength Reduction - S 275 J2

	Lower limit [mm]	Condition	Upper limit [mm]	fy [MPa]	fu [MPa]
1	0,00	< t <=	16,00	275,0	430,0
2	16,00	< t <=	40,00	265,0	430,0
3	40,00	< t <=	63,00	255,0	430,0
4	63,00	< t <=	80,00	245,0	430,0
5	80,00	< t <=	100,00	235,0	430,0
6	100,00	< t <=	150,00	225,0	430,0
7	150,00	< t <=	200,00	215,0	430,0
8	200,00	< t <=	250,00	205,0	430,0
9	250,00	< t <=	400,00	195,0	430,0
*	0,00	< t <=	0,00	0,0	0,0

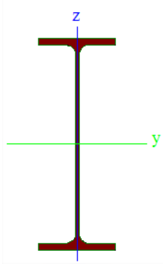
Test data | OK | Cancel

And for each beam the correct yield strength has been taken into account according to the inputted table above:

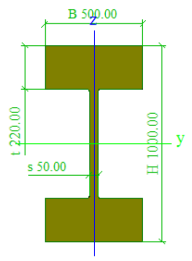
CS1



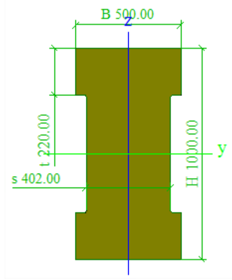
CS2



CS3



CS4



	CS1	CS2	CS3	CS4
Formcode	1 - I sections	1 - I sections	1 - I sections	1 - I sections
h [mm]	80,00	600,00	1000,00	1000,00
b [mm]	46,00	220,00	500,00	500,00
t [mm]	5,20	19,00	220,00	220,00
s [mm]	3,80	12,00	50,00	402,00
r [mm]	5,00	24,00	12,00	12,00
r1 [mm]	0,00	0,00	0,00	0,00
a [%]	0	0	0	0
W [mm]	0,00	116,00	0,00	0,00
wm [mm^2]	0,00	0,00	0,00	0,00

Material	CS1	CS2	CS3	CS4
Yield strength fy	275,0 MPa	265,0 MPa	205,0 MPa	195,0 MPa
Ultimate strength fu	430,0 MPa	430,0 MPa	430,0 MPa	430,0 MPa
Fabrication	Rolled	Rolled	Rolled	Rolled

Section classification

The classification is executed following EN 1993-1-1, art. 5.5.

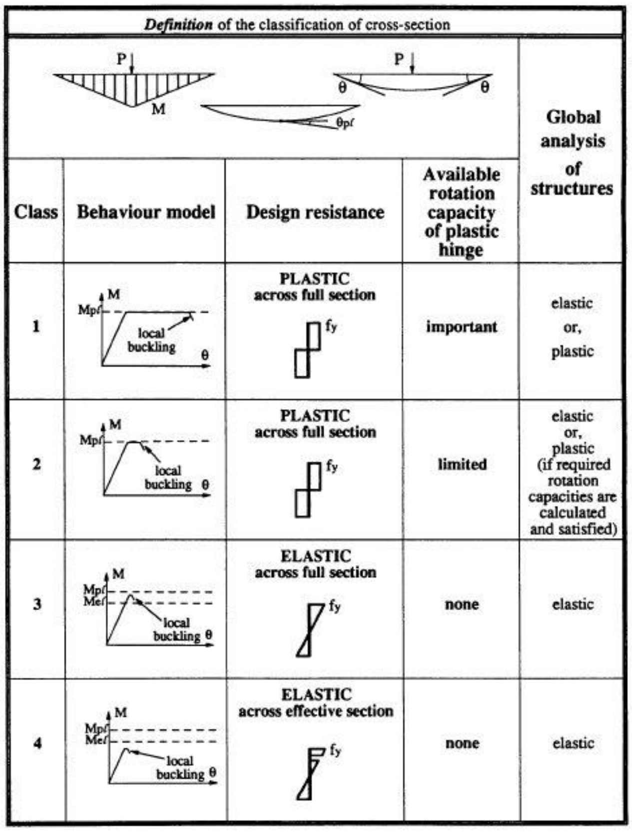
Four classes of cross sections are defined:

- **Class 1** (EC3, NEN) or PL-PL (DIN) section
  Cross sections which can form a plastic hinge with the rotation capacity required for plastic analysis

- **Class 2** (EC3, NEN) or EL-PL (DIN) section
  Cross sections which can develop their plastic moment resistance, but have limited rotation capacity.

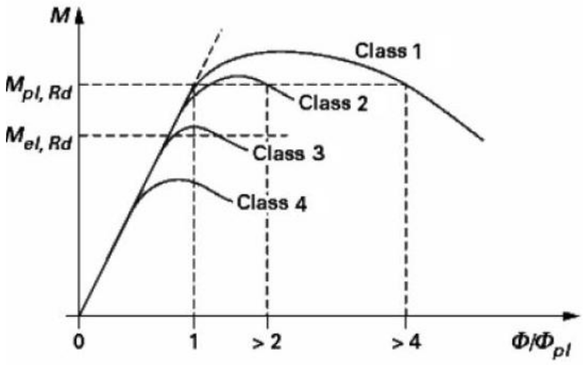
- **Class 3** (EC3, NEN) or EL-EL (DIN) section
  Cross sections in which the calculated stress in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.

- **Class 4** (EC3, NEN) or Slender section
  Cross sections in which it is necessary to make explicit allowance for the effects of local buckling when determining their moment resistance or compression resistance.



Definition of the classification of cross-section

Class	Behaviour model	Design resistance	Available rotation capacity of plastic hinge	Global analysis of structures
1	M, Mpl, local buckling, θ	PLASTIC across full section, fy	important	elastic or, plastic
2	M, Mpl, local buckling, θ	PLASTIC across full section, fy	limited	elastic or, plastic (if required rotation capacities are calculated and satisfied)
3	M, Mpl, Mel, local buckling, θ	ELASTIC across full section, fy	none	elastic
4	M, Mpl, Mel, local buckling, θ	ELASTIC across effective section, fy	none	elastic



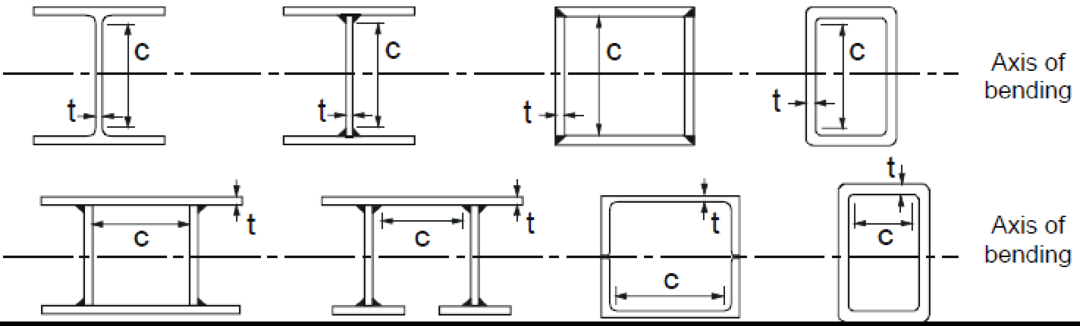
This classification depends on the proportions of each of its compression elements.
For each intermediary section, the classification is determined and the related section check is performed. The classification can change for each intermediary point.

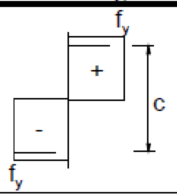
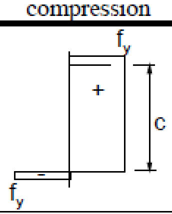
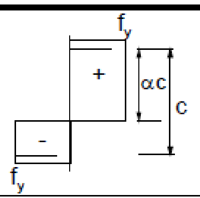
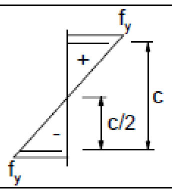
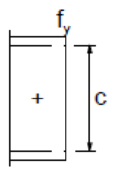
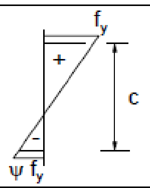
For each load case/combination, the critical section classification over the member is used to perform the stability check. So, the stability section classification can change for each load case/combination. However, for non-prismatic sections, the stability section classification is determined for each intermediary section.

The classification check in SCIA Engineer will be executed following tables 5.2 of the EN 1993-1-1:

Table 5.2 (sheet 1 of 3): Maximum width-to-thickness ratios for compression parts

Internal compression parts



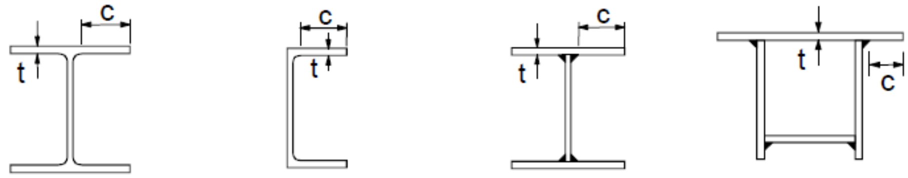
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1^{*)}$: $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

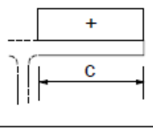
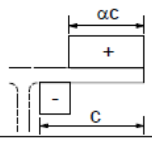
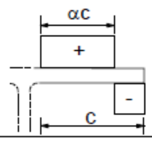
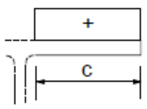
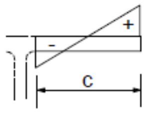
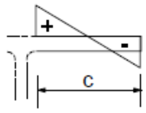
*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Table 5.2 (sheet 2 of 3): Maximum width-to-thickness ratios for compression parts

Outstand flanges



Rolled sections — Welded sections

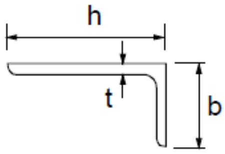
Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \frac{9\varepsilon}{\alpha}$	$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \frac{10\varepsilon}{\alpha}$	$c/t \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

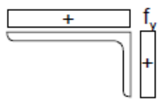
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

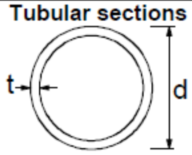
Refer also to "Outstand flanges" (see sheet 2 of 3)



Does not apply to angles in continuous contact with other components

Class	Section in compression
Stress distribution across section (compression positive)	
3	$h/t \le 15\varepsilon : \frac{b+h}{2t} \le 11{,}5\varepsilon$

Tubular sections



Class	Section in bending and/or compression
1	$d/t \le 50\varepsilon^2$
2	$d/t \le 70\varepsilon^2$
3	$d/t \le 90\varepsilon^2$ **NOTE** For $d/t > 90\varepsilon^2$ see EN 1993-1-6.

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51

If this classification results in a class 4 profile, the effective cross section will be calculated according to EN 1993-1-5.

For each load case and combination, the most critical effective area properties are saved:

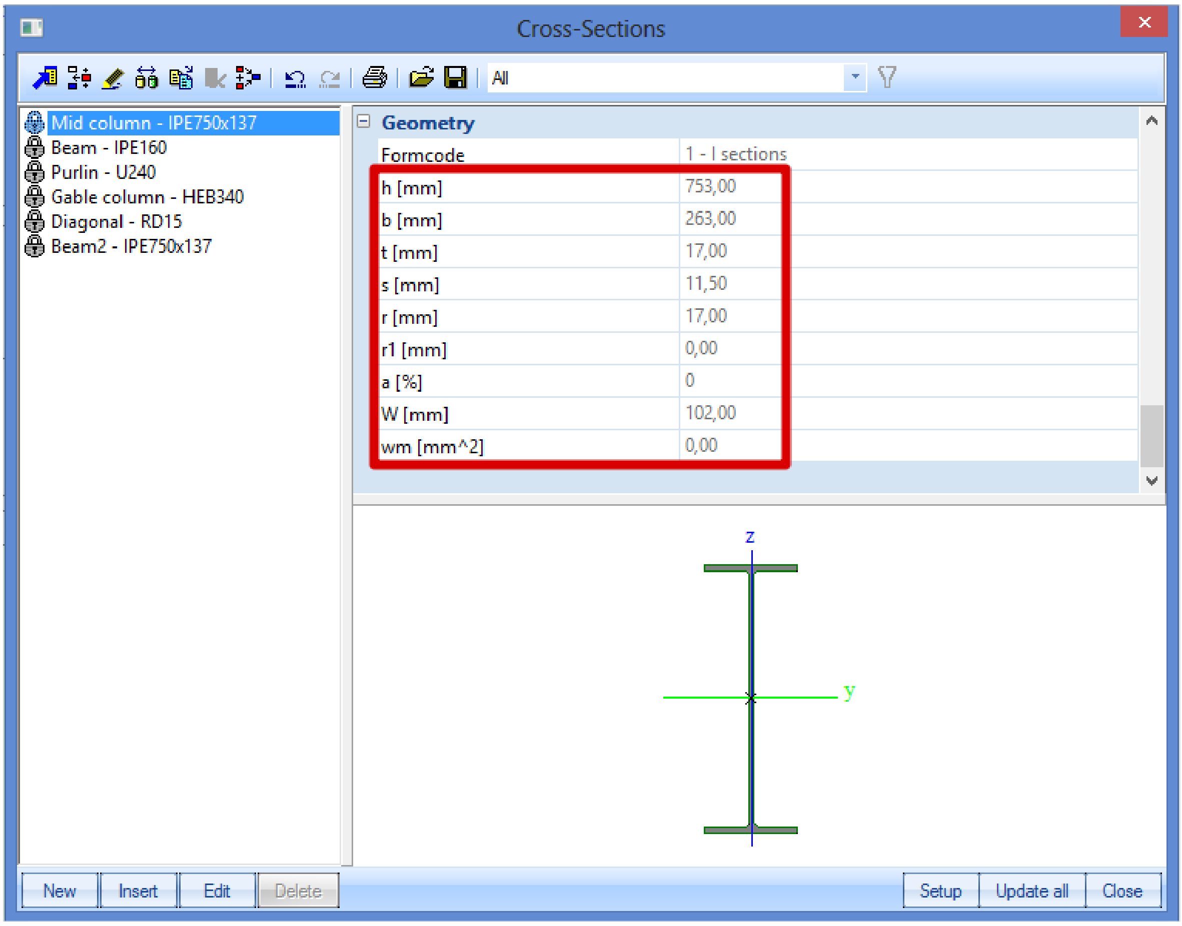
- A_{eff} is the effective area of the cross section when subject to uniform compression.
- W_{eff} is the effective section modulus of the cross-section when subject only to moment about the relevant axis.
- e_N is the shift of the relevant centroidal axis when the cross section is subject to uniform compression.

With these critical properties, the steel code check is performed.

Example: Industrial Hall.esa

In this example the classification will be done for an IPE750x137 profile, resulting in a class 4 cross section. And afterwards the calculation of the effective shape following EN 1993-1-5 will be given.

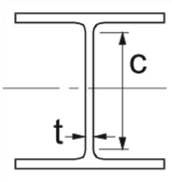
Consider column B28



The classification has been executed at the bottom of the column (position = 0.00m)

On this position a hinged support has been inputted, so on this position the column is not subjected to bending.

Table 5.2 (sheet 1):

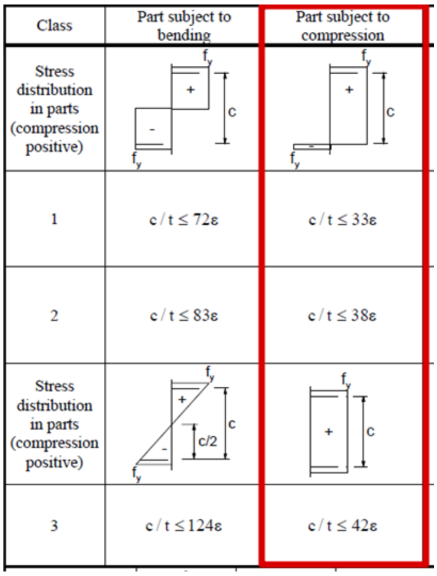


c = H – 2 * thickness flange – 2* radius

c = 753mm – 2x17mm – 2x17mm = 685mm

t = 11,5 mm

c/t = **59,57**



Class	Part subject to bending	Part subject to compression
Stress distribution in parts (compression positive)		
1	$c/t \leq 72\varepsilon$	$c/t \leq 33\varepsilon$
2	$c/t \leq 83\varepsilon$	$c/t \leq 38\varepsilon$
Stress distribution in parts (compression positive)		
3	$c/t \leq 124\varepsilon$	$c/t \leq 42\varepsilon$

With $\varepsilon = \sqrt{235/fy} = 1$

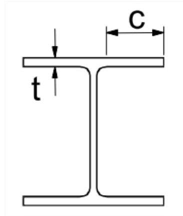
Maximum ratio class 1: 33

Maximum ratio class 2: 38

Maximum ratio class 3: 42

c/t = 59.57 > 42 => Class 4

Table 5.2 (sheet 2):

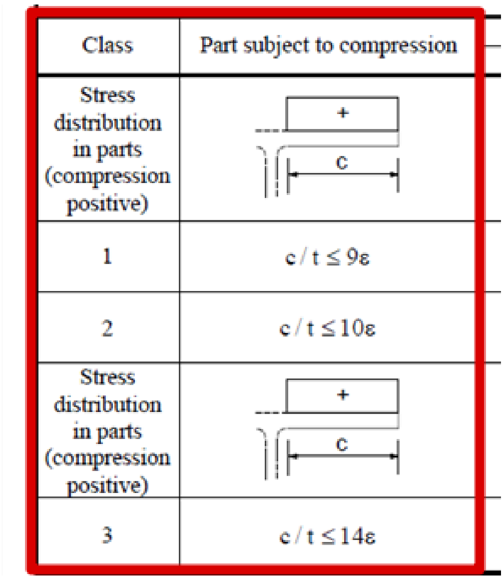


c = B/2 – thickness web/2 – radius

c = 263mm/2 – 11,5mm/2 – 17 = 109

t = 17 mm

c/t = **6,40**



Class	Part subject to compression
Stress distribution in parts (compression positive)	
1	$c/t \leq 9\varepsilon$
2	$c/t \leq 10\varepsilon$
Stress distribution in parts (compression positive)	
3	$c/t \leq 14\varepsilon$

With $\varepsilon = \sqrt{235/fy} = 1$

Maximum ratio class 1: 9

Maximum ratio class 2: 10

Maximum ratio class 3: 14

c/t = 6.40 < 9 => Class 1

In SCIA Engineer:

Classification for member buckling design

Decisive position for stability classification: 0,000 m

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	59,57
Class 1 Limit	33,00
Class 2 Limit	38,00
Class 3 Limit	42,00

=> Internal Compression parts Class 4

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6,40
Class 1 Limit	9,00
Class 2 Limit	10,00
Class 3 Limit	14,00

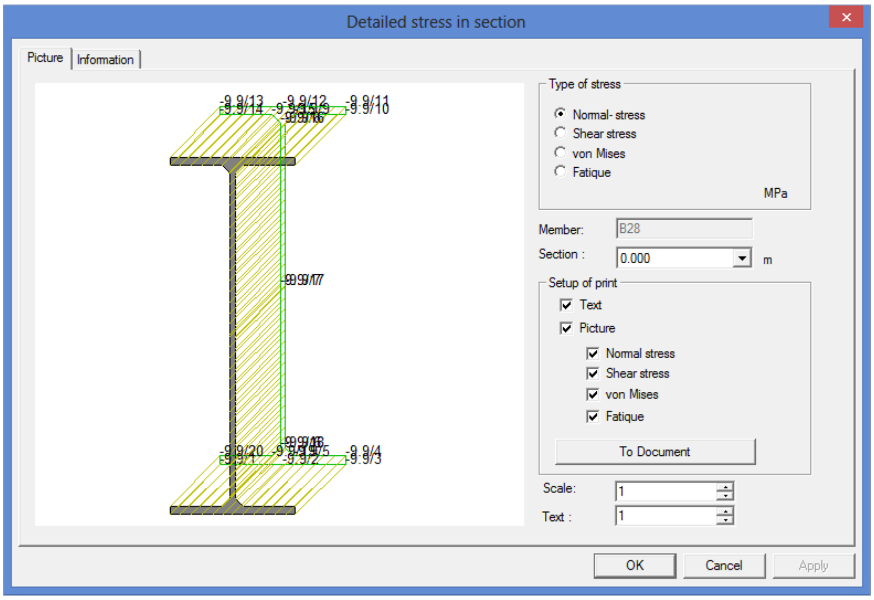
=> Outstand Flanges Class 1

=> Section classified as Class 4 for member buckling design

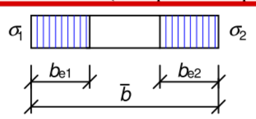
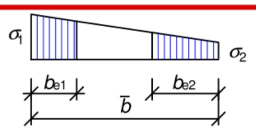
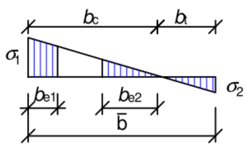
This cross section has a classification class 4 for the stability classification, so effective properties have to be calculated.
Those properties are also given in SCIA Engineer in the preview of the steel code check, just below the classification calculation.

Properties					
sectional area A eff	1.5269e+04	mm^2			
Shear area Vy eff	8.9420e+03	mm^2	Vz eff	6.3269e+03	mm^2
radius of gyration iy eff	320.39	mm	iz eff	58.15	mm
moment of inertia Iy eff	1.5674e+09	mm^4	Iz eff	5.1634e+07	mm^4
elastic section modulus Wy eff	4.1630e+06	mm^3	Wz eff	3.9265e+05	mm^3
Eccentricity eny	0.00	mm	enz	0.00	mm

The calculation of the sectional area A_{eff} is given in below.
In this section there is a uniform compression force over the web:



The calculation of the effective section will be performed following EN 1993-1-5:2006, Tabel 4.1 and EN 1993-1-5:2006/AC:2009 article 9). In Table 4.1 the uniform compression situation will be used in this example:

Stress distribution (compression positive)	Effective^p width b_{eff}
	$\underline{\psi = 1}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = 0,5\ b_{eff}$ $\quad b_{e2} = 0,5\ b_{eff}$
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = \dfrac{2}{5-\psi} b_{eff}$ $\quad b_{e2} = b_{eff} - b_{e1}$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$ $b_{e1} = 0,4\ b_{eff}$ $\quad b_{e2} = 0,6\ b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi > -3$
Buckling factor k_σ	4,0	$8,2 / (1,05 + \psi)$	7,81	$7,81 - 6,29\psi + 9,78\psi^2$	23,9	$5,98\ (1 - \psi)^2$

$$\overline{\lambda_p} \leq 0{,}5 + \sqrt{0.085 - 0.055\psi} \quad = 0{,}673 \qquad \rho = 1$$
$$\overline{\lambda_p} > 0{,}5 + \sqrt{0.085 - 0.055\psi} \quad = 0{,}673 \qquad \rho = (\,\overline{\lambda_p} - 0.22)/\,\overline{\lambda_p}^{\,2}$$

$$\overline{\lambda_p} = [f_y/\sigma_{cr}]^{0,5} = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}}$$

ε = 1,00 (S235)

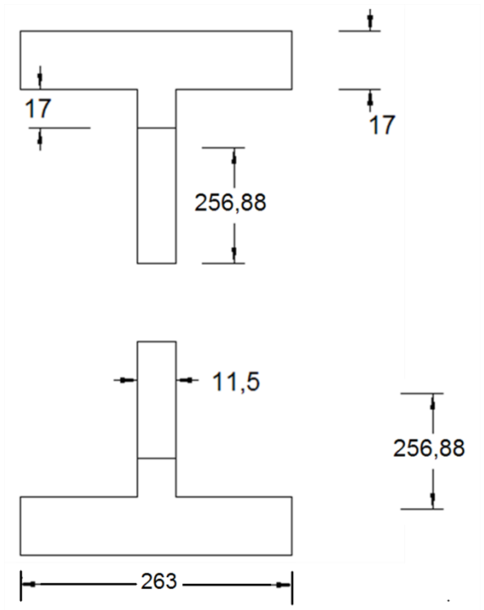
k_σ = 4,0 (Tabel 4.1 of EN 1993-1-5:2006)

$$\overline{\lambda_p} = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{59.57}{28.4 \cdot 1.00 \cdot \sqrt{4.0}} = 1.05$$

$$\rho = \frac{1.05 - 0.22}{(1.05)^2} = \ 0{,}75$$

$$\mathrm{b_{e1}} = \mathrm{b_{e2}} = \ 0.5 \cdot \mathrm{b_{eff}} = 0{,}5 \cdot \rho \cdot \bar{\mathrm{b}} = 0{,}5 \cdot 0{,}75 \cdot 685\ \mathrm{mm} \ = \ 256{,}88\ \mathrm{mm}$$

$$A_{eff} = [263 \cdot 17 + 256{,}88 \cdot 11{,}5 + 17 \cdot 11{,}5] \cdot 2 \ = \ 15152\ \mathrm{mm}^2$$

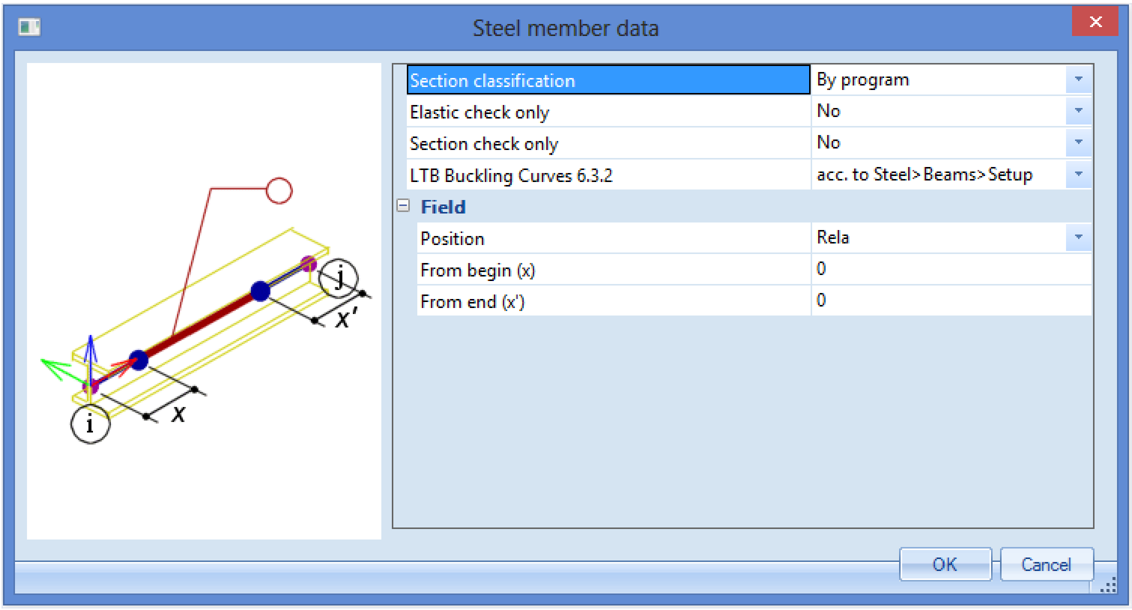


In this calculation the roundings at the corners between the flanges and the web are not taken into account. Therefore the result in SCIA Engineer will be a bit higher:

Properties					
sectional area A eff	1.5269e+04	mm²			
Shear area Vy eff	8.9420e+03	mm²	Vz eff	6.3269e+03	mm²
radius of gyration iy eff	320.39	mm	iz eff	58.15	mm
moment of inertia Iy eff	1.5674e+09	mm⁴	Iz eff	5.1634e+07	mm⁴
elastic section modulus Wy eff	4.1630e+06	mm³	Wz eff	3.9265e+05	mm³
Eccentricity eny	0.00	mm	enz	0.00	mm

Also the other properties of this effective cross section can be calculated.

The calculated classification in SCIA Engineer can be overruled by 2 settings in the **Steel member data**:



Section classification

The user can choose between a classification calculation **By program**, or can overwrite this and choose for class 1, 2 or 3. Since classification 4 is not described for all cross section in the Eurocode, this option can't be chosen.

Elastic Check Only

The user can choose to perform an elastic check. This corresponds with a class 3 check.

Classification in SCIA Engineer

Following classes are possible per profile type in SCIA Engineer:

Section	Classification	Class 1	Class2	Class3	Class 4	Warping check Class 1 & 2	Warping check Class 3
I	x	x	x	x	x	x	x
RHS	x	x	x	x	x		
CHS	x	x	x	x			
L	x			x	x		
U	x			x	x		x
T	x			x			
PPL	x			x	x		x
RS	x			x			
Z	=3			x			
O	=3			x			
Σ	x			x	x		x
NUM	=3			x			
COM	=3			x			

And in the chapter "Cross sections" an explanation is given which sections are recognized under which form type (I, RHS, L, CHS, ...).

ULS Section Check

In this chapter, first the Partial safety factors are explained and afterwards a short explanation of all the section checks is given.

The section check can be found in SCIA Engineer under “Steel -> Beams -> ULS check -> Check”. In this menu the user can choose to look at the “Brief” output and or the “Detailed output”.

With the brief output the results are shown in one line.
With the detailed output, the results of all the unity checks are shown including a reference to the used formula in the EN 1993-1-1 for each check.
In below the Section Check will be explained and the detailed results are shown.

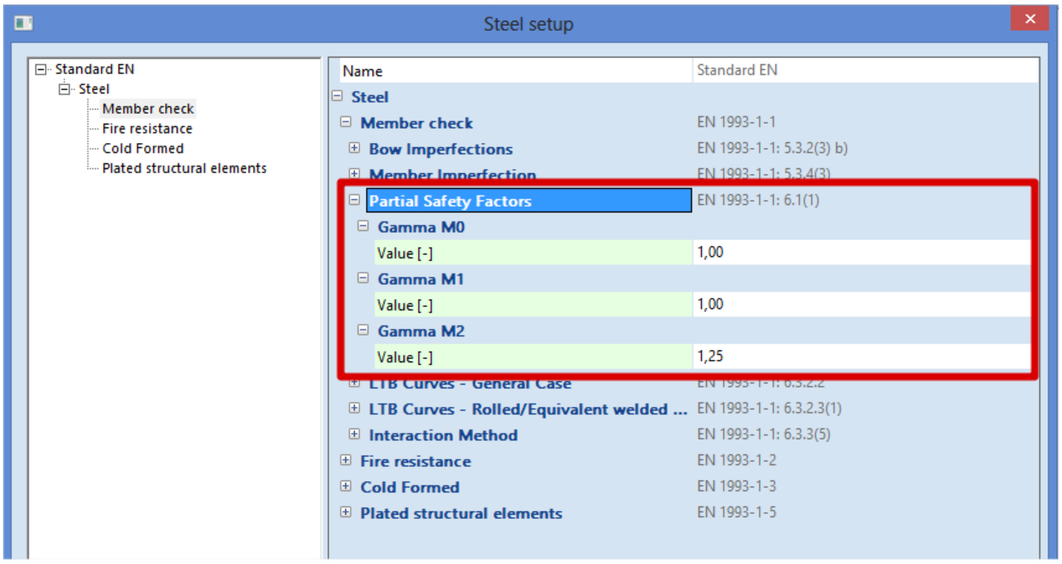
Partial Safety factors

The partial safety factors are taken from EN 1993-1-1 art. 6.1.

And the following safety factors are taken into account:

γ_{M0} = 1,00	Resistance of cross-section
γ_{M1} = 1,00	Resistance of members to instability accessed by member checks
γ_{M2} = 1,25	Resistance of cross-section in tension to fracture

Those factors can also be found in the **National Annex** of EN 1993-1-1 in SCIA Engineer:



Tension

The tension check will be executed following EN 1993-1-1 **art. 6.2.3**.

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1$$

$N_{t,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$: the design plastic resistance of the cross-section

---

Example: Industrial Hall.esa

Consider bracing B234 (for load case 3DWind1). In this load case the bracing in under tension:

$$N_{t,Rd} = \frac{1,7663 \cdot 10^2 mm^2 \cdot 235MPa}{1.0} = 41,51kN$$

$$Unity\ Check = \frac{29,77kN}{41,51kN} = 0,72$$

Internal forces	Calculated	Unit
N,Ed	29,77	kN
Vy,Ed	0,00	kN
Vz,Ed	0,00	kN
T,Ed	0,00	kNm
My,Ed	0,00	kNm
Mz,Ed	0,00	kNm

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

A	1,7663e+02	mm^2
Npl,Rd	41,51	kN
Nu,Rd	45,78	kN
Nt,Rd	41,51	kN
Unity check	0,72	-

Compression

The compression check will be executed following EN 1993-1-1 **art. 6.2.4**.

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1$$

Where

- $N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ For class 1, 2 or 3 cross-sections
- $N_{c,Rd} = \frac{A_{eff} \cdot f_y}{\gamma_{M0}}$ For class 4 cross-sections

Example: Industrial Hall.esa

Consider column B28 (for combination CO1-ULS).

The critical check is on position 1.150 m

Internal forces	Calculated	Unit
N,Ed	-159,86	kN
Vy,Ed	-0,04	kN
Vz,Ed	-101,11	kN
T,Ed	0,00	kNm
My,Ed	-116,27	kNm
Mz,Ed	-0,04	kNm

The classification on this position is Class 1 (column under compression and bending).

$$N_{c,Rd} = \frac{1,7500 \cdot 10^4 mm^2 \cdot 235MPa}{1.0} = 4112,5\ kN$$

$$Unity\ Check = \frac{159.86kN}{4112.5kN} = 0,04$$

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	1,7500e+04	mm^2
Nc,Rd	4112,50	kN
Unity check	0,04	-

Bending moment

The bending moment check for My and Mz will be executed following EN 1993-1-1 **art. 6.2.5**.

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1$$

Where

- $M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}}$ For class 1 or 2 cross-sections
- $M_{c,Rd} = M_{el,Rd} = \frac{W_{el,min} \cdot f_y}{\gamma_{M0}}$ For class 3 cross-sections
- $M_{c,Rd} = \frac{W_{eff} \cdot f_y}{\gamma_{M0}}$ For class 4 cross-sections

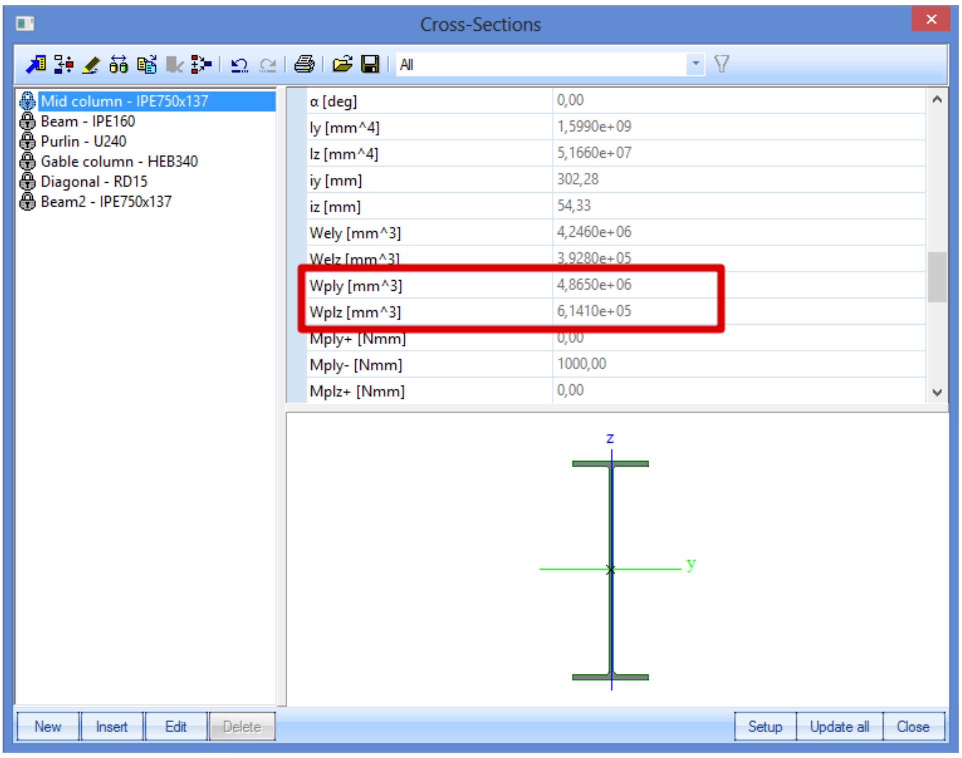
Example: Industrial Hall.esa

Consider column B28 (for combination CO1-ULS).

The critical check is on position 1.150 m

Internal forces	Calculated	Unit
N,Ed	-159,86	kN
Vy,Ed	-0,04	kN
Vz,Ed	-101,11	kN
T,Ed	0,00	kNm
My,Ed	-116,27	kNm
Mz,Ed	-0,04	kNm

The classification on this position is Class 1 (column under compression and bending).



$$M_{y,c,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = \frac{4.865 \cdot 10^6 mm^3 \cdot 235MPa}{1.0} = 1{,}14328 \cdot 10^9\ Nmm = 1143{,}28\ kNm$$

$$M_{z,c,Rd} = \frac{W_{pl,z} \cdot f_y}{\gamma_{M0}} = \frac{6.141 \cdot 10^5 mm^3 \cdot 235MPa}{1.0} = 1{,}4431\ \cdot 10^8\ Nmm = 144{,}31\ kNm$$

$$Unity\ Check\ My = \frac{116.27\ kNm}{1143{,}28\ kNm} = 0{,}10$$

$$Unity\ Check\ Mz = \frac{0.04\ kNm}{144{,}31\ kNm} = 0{,}00$$

Bending moment check for My

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	4,8650e+06	mm^3
Mpl,y,Rd	1143,28	kNm
Unity check	0,10	-

Bending moment check for Mz

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,z	6,1410e+05	mm^3
Mpl,z,Rd	144,31	kNm
Unity check	0,00	-

Shear

The shear check for Vy and Vz will be executed following EN 1993-1-1 **art. 6.2.6**.

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1$$

For **plastic design** $V_{c,Rd}$ the absence of torsion, is the design plastic shear resistance $V_{pl,Rd}$:

$$V_{pl,Rd} = \frac{A_v \cdot (f_y/\sqrt{3})}{\gamma_{M0}}$$

With:
A_v: the shear area.
The formula for A_v depends on the cross-section (see EN 1993-1-1 article 6.2.6(3)).

For **elastic design** $V_{c,Rd}$ is the design elastic shear resistance. The following criterion for a critical point of the cross-section may be used unless the buckling verification in section 5 of EN 1993-1-5 applies:

$$\frac{\tau_{Ed}}{f_y \cdot \left(\sqrt{3} \cdot \gamma_{M0}\right)} \leq 1$$

Where $\tau_{Ed} = \frac{V_{Ed} \cdot S}{I\, t}$

Where the shear force is combined with a torsional moment, the plastic resistance $V_{pl,Rd}$ should be reduced as specified in the next paragraph.

Example: Industrial Hall.esa

Consider column B28 (for combination CO1-ULS).

The critical check is on position 1.150 m

Internal forces	Calculated	Unit
N,Ed	-159,86	kN
Vy,Ed	-0,04	kN
Vz,Ed	-101,11	kN
T,Ed	0,00	kNm
My,Ed	-116,27	kNm
Mz,Ed	-0,04	kNm

The classification on this position is Class 1 (column under compression and bending).

Shear check for Vy
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	9,2698e+03	mm^2
Vpl,y,Rd	1257,69	kN
Unity check	0,00	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	9,9222e+03	mm^2
Vpl,z,Rd	1346,22	kN
Unity check	0,08	-

Torsion

EN 1993-1-1 **article 6.2.7**.

$$\frac{T_{Ed}}{T_{Rd}} \leq 1$$

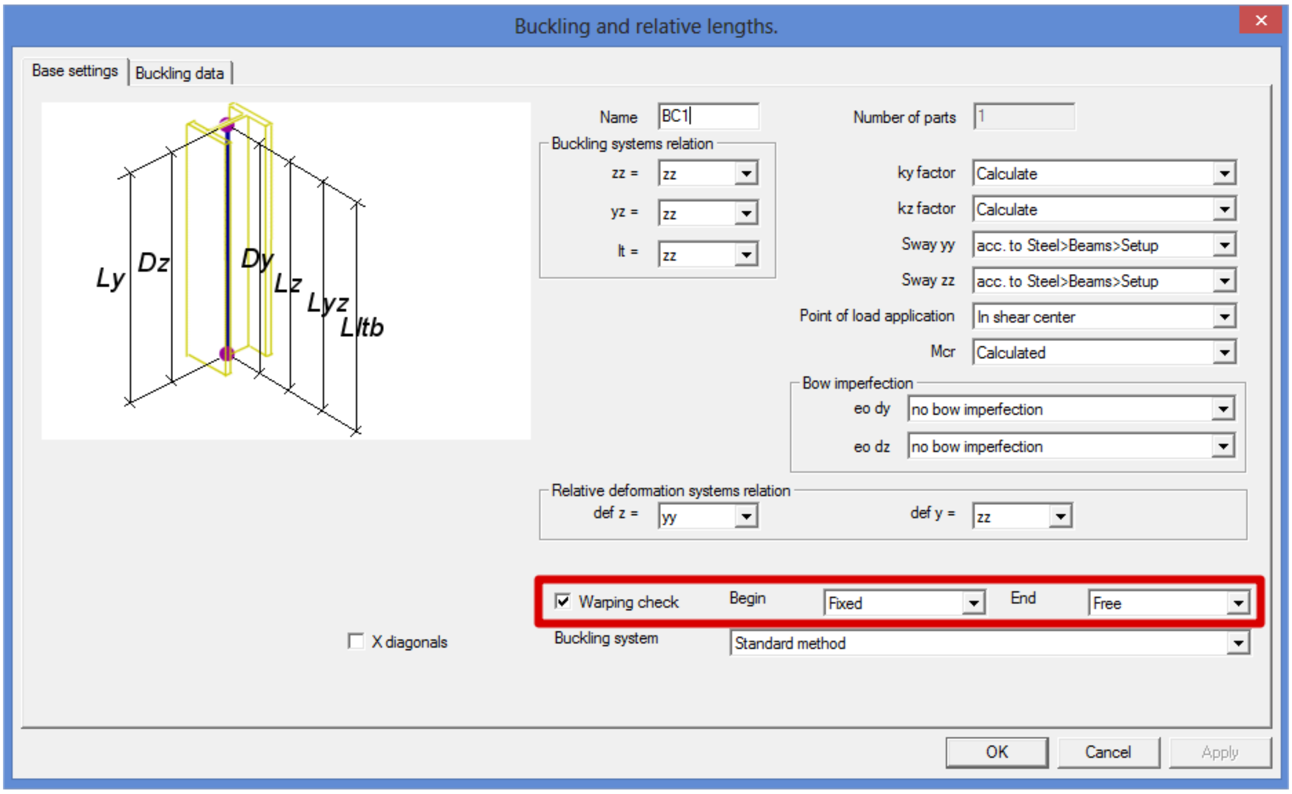
Where T_{Rd} is the design torsional resistance of the cross-section.

T_{Ed} should be considered as the sum of two internal effects:

$$T_{Ed} = T_{t,Ed} + T_{w,Ed}$$

Where $T_{t,Ed}$ is the internal St. Venant torsion
$T_{w,Ed}$ is the internal warping torsion

SCIA Engineer will take into account the St. Venant torsion automatically. If the user would like to calculate also with **warping torsion**, this option should be activated in the “Buckling and relative lengths” of this beam:



The check box **Warping check** should be activated and for the beginning and end of the beam, the user should indicate if this end is Fixed or Free for warping.

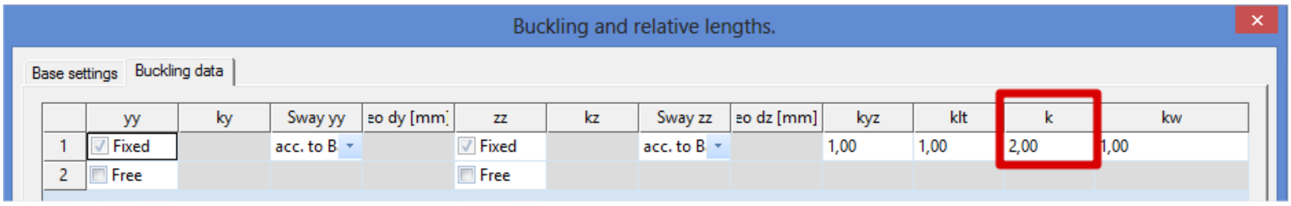
Example: Warping.esa

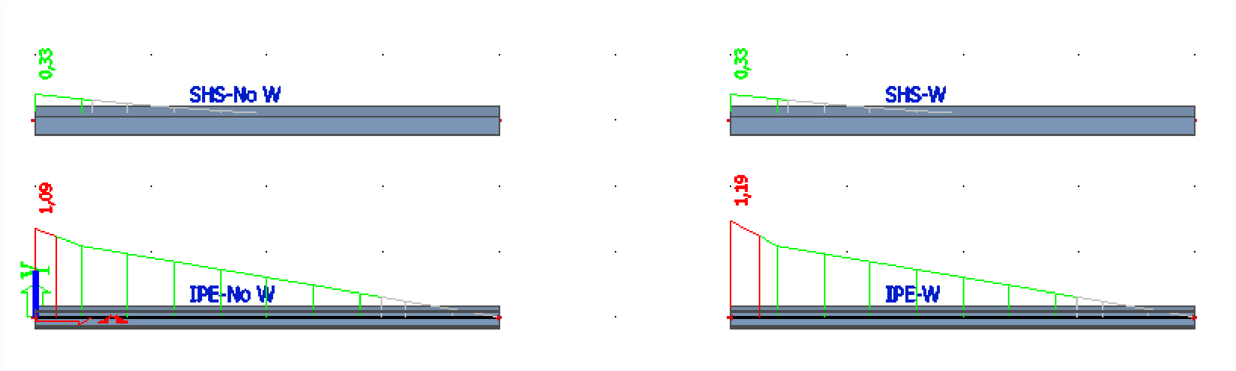
As quoted in the EN 1993-1-1, art. 6.2.7(7), for closed hollow sections the effects of torsional warping can be neglected and in the case of a member with an open cross section, such as I of H, it may be assumed that the effects of St.Venant torsion can be neglected.

In this example 4 beams are inputted:

- IPE 180 – No Warping has been activated
- SHS 180/180/10.0 – No Warping has been activated
- IPE 180 – Warping has been activated in the buckling data
- SHS 180/180/10.0 –Warping has been activated in the buckling data

On all beams, a line force of -4kN is inputted with an eccentricity ey of 0,050 m. The k factor has been changed into 2 in the buckling data.





For the **SHS profiles**, there is no difference between the option “Warping” activated or not. This was expected as warping can be neglected for closed hollow section.

For **IPE profiles**, the warping torsion cannot be neglected. So when this is activated, it has a big influence on the resistance value for torsion:

Torsion check IPE 180 without warping activated:

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Tau,t,Ed	133,6	MPa
Tau,Rd	135,7	MPa
Unity check	0,98	-

Combined Shear and Torsion check for Vz and Tau,t,Rd

According to EN 1993-1-1 article 6.2.6 & 6.2.7 and formula (6.25),(6.26)

Vpl,T,z,Rd	70,02	kN
Unity check	0,23	-

Torsion check IPE 180 with warping activated:

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Tau,t,Ed	133,6	MPa
Tau,Rd	135,7	MPa
Unity check	0,98	-

Combined Shear and Torsion check for Vz and Tau,t,Rd

According to EN 1993-1-1 article 6.2.6 & 6.2.7 and formula (6.25),(6.26)

Vpl,T,z,Rd	70,02	kN
Unity check	0,23	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.7
According to I. Vayas, Stahlbau 69 (2000)
"Interaktion der plastischen Grenzschnittgroessen doppelsymmetrischer I-Querschnitte", Table 1.

Warping conditions at the extremities of the buckling system

Extremity	Condition
Begin	Fixed
End	Free

Decomposition of torsional moment

x [m]	Mxp,Ed [kNm]	Mxs,Ed [kNm]	Mw,Ed [kNm²]
0,000	0,00	0,80	-0,43
0,400	0,29	0,43	-0,19
0,800	0,41	0,23	-0,06
1,200	0,44	0,12	0,00
1,600	0,42	0,06	0,04
2,000	0,37	0,03	0,06
2,000	0,37	0,03	0,06
2,400	0,31	0,01	0,06
2,800	0,25	-0,01	0,06
3,200	0,19	-0,03	0,05
3,600	0,14	-0,06	0,04
4,000	0,12	-0,12	0,00

Internal forces			
St. Venant torsion	Mxp,Ed	0,00	kNm
Warping torsion	Mxs,Ed	0,80	kNm
Bimoment	Mw,Ed	-0,43	kNm ²

Force ratios		
Cross-section flange quotient	alpha,f	0,61
Cross-section web quotient	alpha,w	0,39
Normal force ratio	n	0,00
Moment ratio for My	my	-0,82
Moment ratio for Mz	mz	0,00
Shear ratio for Vy	vy	0,00
Shear ratio for Vz	vz	0,12
Moment ratio for Mw	mw	0,61
Moment ratio for Mxp	mxp	0,00
Moment ratio for Mxs	mxs	0,05

Influence factors		
Interaction ratio for shear stresses in flanges	beta,y	0,05
Reduction factor for shear stresses in flanges	pf	0,00
Yield strength reduction for shear stresses in flanges	sf	1,00
Interaction ratio for shear stresses in web	beta,z	0,12
Reduction factor for shear stresses in web	pw	0,00
Yield strength reduction for shear stresses in web	sw	1,00
Additional moment coefficient	lambda,s	0,00
Additional moment coefficient	delta	0,00

Unity checks		
Unity check (42)	**1,19**	-

In this case the difference between the option “warping” activated or not, give a difference in the unity check.

The **torsional properties of the cross sections** are not calculated correctly in a lot of cases. The next example give an explanation about this calculation.

Example: Torsion.esa

In this example two beams are inputted, with the same properties, only the torsional properties of the cross sections are different:

CS1

The option “2D FEM analysis” is not activated:

Fibres and Parts	
Fibre text zoom	1.0
Edit named items	
2D FEM analysis	
Use 2D FEM analysis	☐

So the value for It will be calculated with a general formula, which does only give an estimation of the It, but not a correct value. The value for Iw will not be calculated and Iw will taken as 0.00 mm^6.

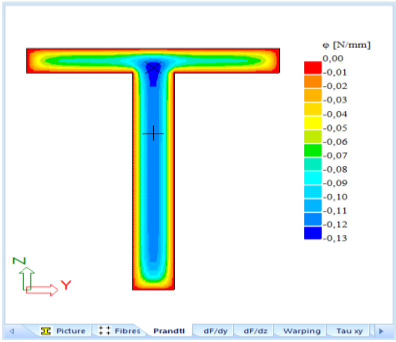
Property	
A [mm^2]	7,5000e+03
Ay [mm^2]	7,5000e+03
Az [mm^2]	7,5000e+03
AL [m^2/m]	7,0000e-01
AD [m^2/m]	7,0000e-01
cYUCS [mm]	13
cZUCS [mm]	130
α [deg]	0,00
Iy [mm^4]	3,0250e+07
Iz [mm^4]	5,8594e+06
iy [mm]	64
iz [mm]	28
Wely [mm^3]	2,3269e+05
Welz [mm^3]	7,8125e+04
Wply [mm^3]	4,1250e+05
Wplz [mm^3]	1,4063e+05
Mply+ [Nmm]	0,00
Mply- [Nmm]	0,00
Mplz+ [Nmm]	0,00
Mplz- [Nmm]	0,00
dy [mm]	0
dz [mm]	0
It [mm^4]	2,5672e+06
Iw [mm^6]	0,0000e+00
β y [mm]	0
β z [mm]	0

CS2

The option “2D FEM analysis” is activated:

Fibres and Parts	
Fibre text zoom	1.0
Edit named items	
2D FEM analysis	
Use 2D FEM analysis	☑
Mesh size [mm]	0
Min. point distance [mm]	0
Shear area Ay	With τ xz
Shear area Az	With τ xy

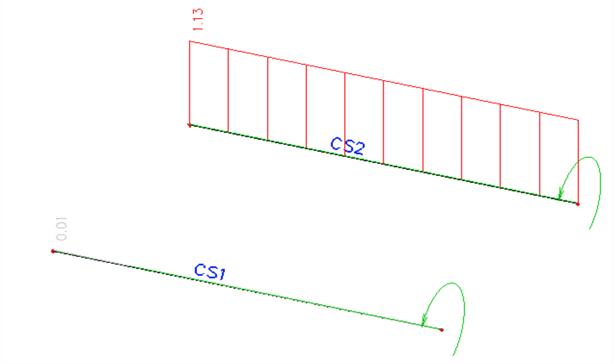
So the value for It and Iw will be calculated with a FEM analysis:



Property	
A [mm^2]	7,5000e+03
Ay [mm^2]	3,7071e+03
Az [mm^2]	4,4171e+03
AL [m^2/m]	7,0000e-01
AD [m^2/m]	7,0000e-01
cYUCS [mm]	75
cZUCS [mm]	130
α [deg]	0,00
Iy [mm^4]	3,0250e+07
Iz [mm^4]	5,8594e+06
iy [mm]	64
iz [mm]	28
Wely [mm^3]	2,3269e+05
Welz [mm^3]	7,8125e+04
Wply [mm^3]	4,1250e+05
Wplz [mm^3]	1,4063e+05
Mply+ [Nmm]	96937500,00
Mply- [Nmm]	96937500,00
Mplz+ [Nmm]	33046875,00
Mplz- [Nmm]	33046875,00
dy [mm]	0
dz [mm]	54
It [mm^4]	1,3076e+06
Iw [mm^6]	2,8054e+09
β y [mm]	-134
β z [mm]	0

On the two beams a torsional moment (8 kNm) has been inputted and a small line load (-0.1 kN/m).

This result in the following Section Check:



The difference in the results is in the torsion check. For the first beam no torsion check can be executed, since no results for torsional stiffnesses can be found.

For **beam B2** (CS2) the torsion check can be executed:

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	0,00	kN
Vy,Ed	0,00	kN
Vz,Ed	0,40	kN
T,Ed	-8,00	kNm
My,Ed	-0,80	kNm
Mz,Ed	0,00	kNm

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Tau,t,Ed	153,0	MPa
Tau,Rd	135,7	MPa
Unity check	**1,13**	-

Combined check: Bending, shear and axial force

This check will be executed following EN 1993-1-1 **articles 6.2.8 – 6.2.9 -6.2.10**.

For bi-axial bending the following criterion may be used:

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1$$

in which α and β are defined as follows:

- I and H sections:

  $\alpha = 2$; $\beta = 5n$ but $\beta \geq 1$

- Circular hollow sections:

  $\alpha = 2$; $\beta = 2$

- Rectangular hollow sections:

  $\alpha = \beta = \frac{1{,}66}{1-1{,}13n^2}$ but $\alpha = \beta \leq 6$

  Where $n = \frac{N_{Ed}}{N_{pl,Rd}}$

The values $M_{N,y,Rd}$ and $M_{N,z,Rd}$ depends on the moment resistance, reduced with a factor depending on "*n*", the check of the normal force:

$$M_{N,y,Rd} = M_{pl,y,Rd}(1-n)/(1-0{,}5a_w)$$

$$M_{N,z,Rd} = M_{pl,z,Rd}(1-n)/\left(1-0{,}5a_f\right)$$

And $M_{pl,Rd}$ depends on the yield strength f_y.

If $V_{Ed} \geq 0{,}5\, V_{pl,T,Rd}$

The yield strength will be reduced with a factor ρ.

$$(1-\rho)f_y$$

Where

$$\rho = \left(\frac{2\,V_{Ed}}{V_{pl,Rd}} - 1\right)^2$$

And

$$V_{pl,Rd} = \frac{A_v \cdot \left(f_y/\sqrt{3}\right)}{\gamma_{M0}}$$

When torsion is present ρ should be obtained from

$$\rho = \left(\frac{2\,V_{Ed}}{V_{pl,T,Rd}} - 1\right)^2$$

Class 3 cross-sections

EN 1993-1-1 **article 6.2.9.2**.

In absence of shear force, for Class 3 cross-sections the maximum longitudinal stress should satisfy the criterion:

$$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}}$$

Class 4 cross-sections

EN 1993-1-1 **article 6.2.9.3**.

In absence of shear force, for Class 4 cross-sections the maximum longitudinal stress should satisfy the criterion:

$$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}}$$

The following criterion should be met:

$$\frac{N_{Ed}}{A_{eff} \cdot \dfrac{f_y}{\gamma_{M0}}} + \frac{M_{y,Ed} + N_{Ed} \cdot e_{Ny}}{W_{eff,y,min} \cdot \dfrac{f_y}{\gamma_{M0}}} + \frac{M_{z,Ed} + N_{Ed} \cdot e_{Nz}}{W_{eff,z,min} \cdot \dfrac{f_y}{\gamma_{M0}}} \leq 1$$

Where A_{eff} is the effective area of the cross-section when subjected to uniform compression

$W_{eff, min}$ is the effective section modulus of the cross-section when subjected only to moment about the relevant axis

e_N is the shift of the relevant centroidal axis when the cross-section is subjected to compression only.

Example: Industrial hall.esa

Consider column B28 (for combination CO1-ULS).

Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	1,7500e+04	mm^2
Nc,Rd	4112,50	kN
Unity check	0,04	-

⇨ n = 0,04

Shear check for Vy

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	9,2698e+03	mm^2
Vpl,y,Rd	1257,69	kN
Unity check	0,00	-

Shear check for Vz

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1,20	
Av	9,9222e+03	mm^2
Vpl,z,Rd	1346,22	kN
Unity check	0,08	-

⇨ Unity check for shear force is smaller than 0,5, thus no reduction of the yield strength for the combined check.

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Mpl,y,Rd	1143,28	kNm
Alpha	2,00	
Mpl,z,Rd	144,31	kNm
Beta	1,00	

Unity check (6.41) = 0,01 + 0,00 = 0,01 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.
Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

ULS Stability Check

The stability check can be found in SCIA Engineer under “Steel -> Beams -> ULS check -> Check”. In this menu the user can choose to look at the “Brief” output and or the “Detailed output”.

With the brief output the results are shown in one line.
With the detailed output, the results of all the unity checks are shown including a reference to the used formula in the EN 1993-1-1 for each check.
In below the Stability Check will be explained and the detailed results are shown.

Not all stability checks are inputted in SCIA Engineer for all cross sections. In the table below an overview of the possible checks for each section is given:

Section	Buckling	LTB	Shear buckling
I	x	x	x
RHS	x	x	
CHS	x	x	
L	x	(*2)	
U	x	(*2)	x
T	x	(*2)	
PPL	x	x	x
RS	x	(*2)	
Z	x	(*2)	
O	x	(*2)	
Σ	(*1)	(*2)	x
NUM	(*1)	(*2)	
COM	(*1)	(*2)	

(*1) Buckling curve are introduced manually by user

(*2) General formula for M_{cr} – see chapter ‘Lateral Torsional Buckling’

And in the chapter “Cross sections” an explanation is given which sections are recognized under which form type (I, RHS, L, CHS, ...).

Classification

There is a difference on the section of classification in the Section check and the Stability check:

Classification in the section check:
Here the classification is done for each section on the member and afterwards the Section check will be executed with the classification and the internal forces on this section.

Classification in the Stability Check
In the stability check the highest classification along the member is used for the stability check on all the sections.

So when having a beam with the highest check on 2m from the beginning of the beam:

- The section check will take into account the internal forces on 2m from the beginning of the beam and perform a classification with those internal forces.

- The stability check will perform a stability check with the internal forces on 2m of the beginning of the beam, but not with the classification on this place. Perhaps this section has the highest moment in comparison with a compression force. The classification can result here in Class one.

  But in the beginning of the beam, the bending moment is equal to zero, so perhaps the classification here will be Class 4. The stability check will thus take the internal forces of section 2m on the member, but take into account the classification Class 4 (from 0m on the member).

Flexural buckling check

The flexural buckling check will be executed following EN 1993-1-1 **art. 6.3.1**.

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1$$

Where

- $N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$ For class 1, 2 or 3 cross sections
- $N_{b,Rd} = \frac{\chi \cdot A_{eff} \cdot f_y}{\gamma_{M1}}$ For class 4 cross sections

The reduction factor χ will be calculated as follows:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}}$$ but $\chi \leq 1{,}0$

with

Φ $0{,}5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right]$

$\bar{\lambda}$ $\sqrt{\frac{A \cdot f_y}{N_{cr}}}$ For class 1, 2 or 3 cross-sections

$\sqrt{\frac{A_{eff} \cdot f_y}{N_{cr}}}$ For class 4 cross-sections

N_{cr} Critical normal force (Euler force)

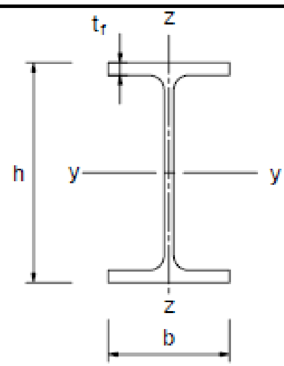
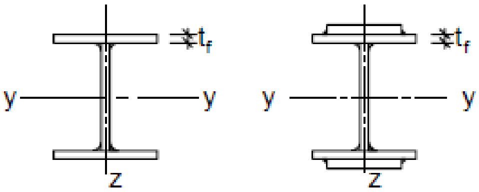
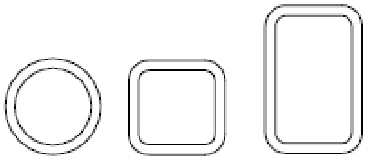
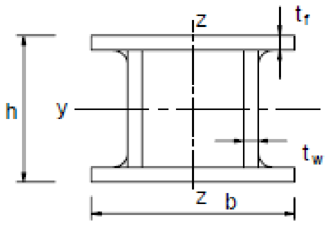
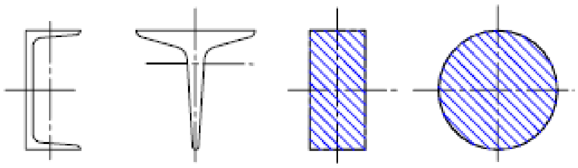
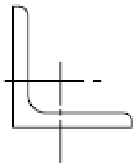
$$N_{cr} = \frac{\pi^2 EI}{k^2 L^2}$$

α Imperfection depending on the buckling curve:

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

The selection of the buckling curve for a cross section is done with EN 1993-1-1, Table 6.2

Table 6.2: Selection of buckling curve for a cross-section

Cross section		Limits		Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S 460
Rolled sections		$h/b > 1,2$	$t_f \le 40$ mm	y – y z – z	a b	a_0 a_0
			40 mm < $t_f \le 100$	y – y z – z	b c	a a
		$h/b \le 1,2$	$t_f \le 100$ mm	y – y z – z	b c	a a
			$t_f > 100$ mm	y – y z – z	d d	c c
Welded I-sections		$t_f \le 40$ mm		y – y z – z	b c	b c
		$t_f > 40$ mm		y – y z – z	c d	c d
Hollow sections		hot finished		any	a	a_0
		cold formed		any	c	c
Welded box sections		generally (except as below)		any	b	b
		thick welds: $a > 0,5t_f$ $b/t_f < 30$ $h/t_w < 30$		any	c	c
U-, T- and solid sections				any	c	c
L-sections				any	b	b

In SCIA Engineer the calculation of the buckling coefficient k_y or k_z can be done automatically or can be inputted by the user.

In the next paragraph the calculation of the Buckling factors is explained.

Example: Buckling Curves.esa

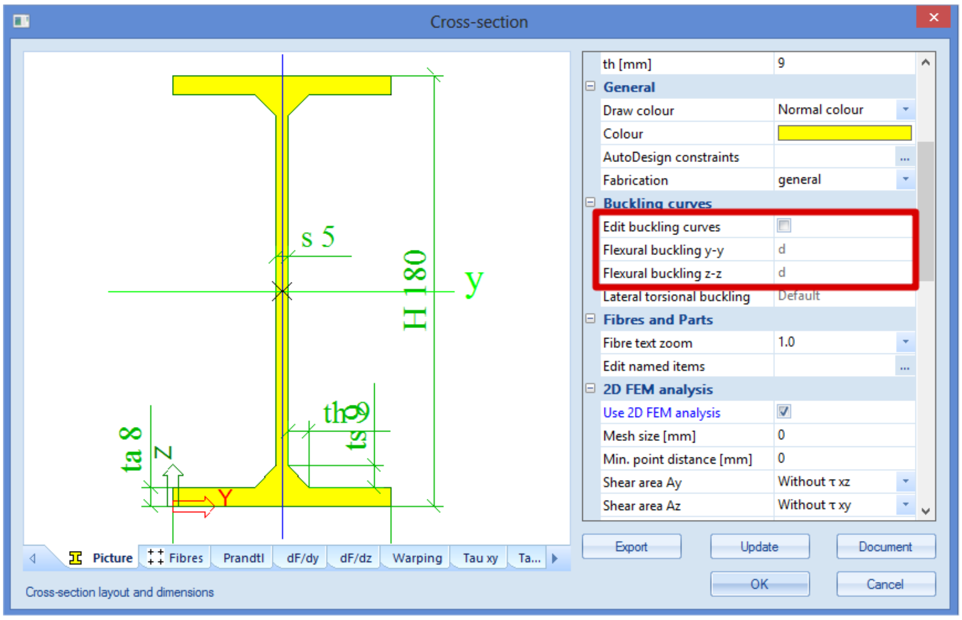
In Table 6.2 general buckling curves are given for the most common profiles. For some cross-section types, SCIA Engineer will automatically use these curves. The cross-section types that are supported are:

- Profile library
- Haunch
- Sheet welded
- Build-in beams
- Thin-walled geometric
- Fabricated

For the other cross-section types, the buckling curves for both directions are by default set to d. This can be changed manually by changing the properties of the cross-section.

In this example 2 beams are inputted with 2 different cross-sections:

- B1: CS1 – IPE 180:
  - buckling curve a for y-y according to code
  - Stability check uses curve a, according to code
- B2: CS2 – I form
  - Non standard section: no buckling curve according to code. Buckling curve d is used.
  - Stability check uses curve d



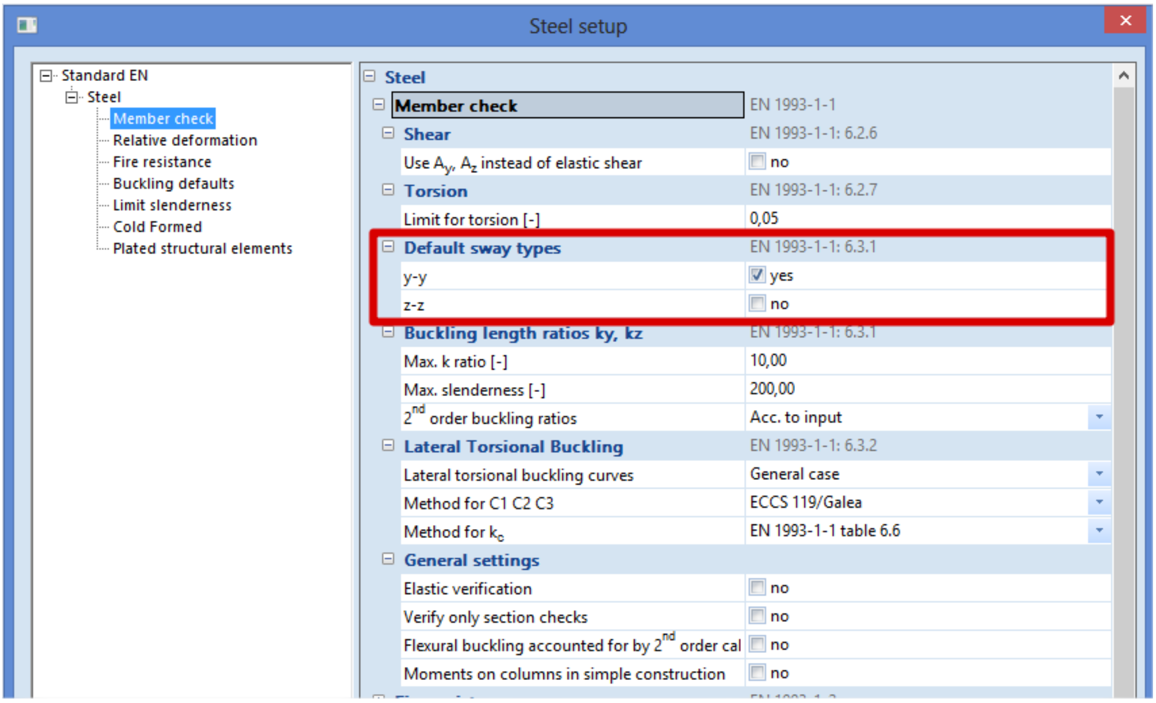
Buckling factors

General method

By default, for the calculation of the buckling ratios, two approximate formulas are used: one formula for a non sway structure (resulting in a buckling factor smaller (or equal) than 1) and one formula for a sway structure (resulting in a buckling factor higher (of equal) than 1.

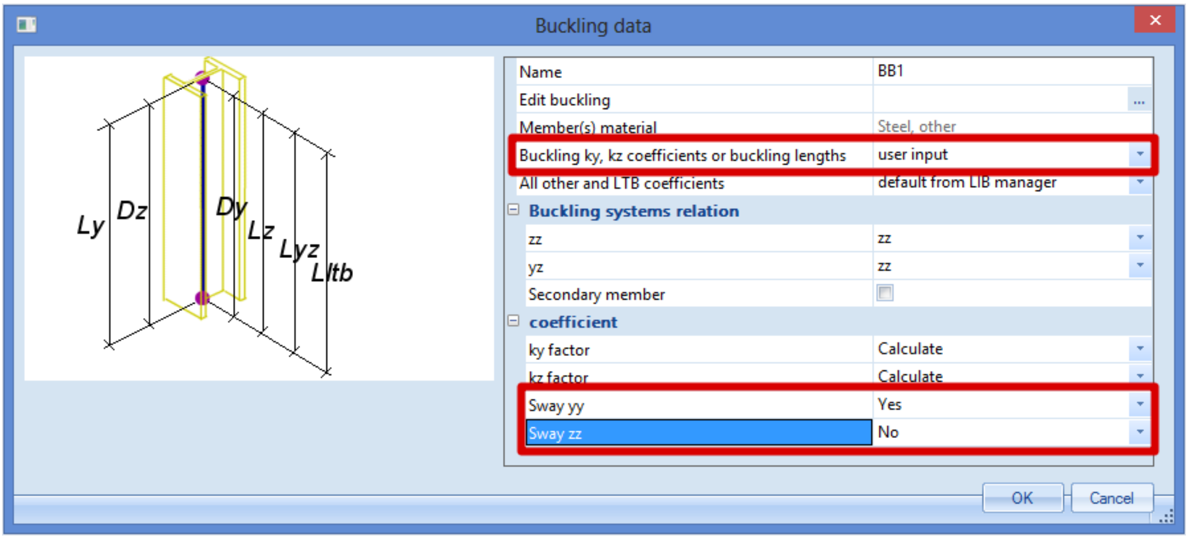
So it is important for this method that the user chooses the correct option sway or non sway for the two local directions:
yy: buckling around the local y-axis (so deformation in the direction of the local z-axis)
zz: buckling around the local z-axis (so deformation in the direction of the local y-axis)

The option “sway” or “non sway” can be chosen in the menu “Steel -> Beams -> Steel Setup” for the whole structure:



It the option is checked (as here for buckling around the y-axis), it is indicated that the construction is not braced enough and can be considered as “sway”.

This can also be changed for each beam separately, using the buckling data. This can be found under “Steel -> Beams -> Member Check data -> Member buckling data”:



For Sway yy and Sway zz there are 3 options:

- According to Steel>Beams>Setup: The same option will be taken as in the steel setup as shown above.
- Yes: this beam will be considered as “sway” in this direction.
- No: this beam will be considered as “non sway” in this direction.

The following formulas are used for the buckling ratios:

- for a non sway structure :

$$k = \frac{(\rho_1\rho_2+5\rho_1+5\rho_2+24)(\rho_1\rho_2+4\rho_1+4\rho_2+12)2}{(2\rho_1\rho_2+11\rho_1+5\rho_2+24)(2\rho_1\rho_2+5\rho_1+11\rho_2+24)}$$

- for a sway structure :

$$k = x\sqrt{\frac{\pi^2}{\rho_1 x}+4}$$

with	k	the buckling factor
	L	the system length
	E	the modulus of Young
	I	the moment of inertia
	C_i	the stiffness in node i
	M_i	the moment in node i
	ϕ_i	the rotation in node i

$$x = \frac{4\rho_1\rho_2 + \pi^2\rho_1}{\pi^2(\rho_1+\rho_2)+8\rho_1\rho_2}$$

$$\rho_i = \frac{C_i L}{EI}$$

$$C_i = \frac{M_i}{\phi_i}$$

The values for M_i and ϕ_i are approximately determined by the internal forces and the deformations, calculated by load cases which generate deformation forms, having an affinity with the buckling form. So when performing a linear calculation, in the background 2 additional load cases are calculated, just to calculated the buckling factors for the elements.

This calculation is automatically done when calculating the construction linearly. **So when calculating non linear, the user should also perform a linear calculation otherwise no buckling factors are calculated and no steel code check can be performed.**

The following load cases are considered in the linear calculation for the calculation of the buckling factors:

- load case 1:
  - on the beams, the local distributed loads qy=1 N/m and qz=-100 N/m are used
  - on the columns the global distributed loads Qx =10000 N/m and Qy =10000 N/m are used.

- load case 2:
  - on the beams, the local distributed loads qy=-1 N/m and qz=-100 N/m are used
  - on the columns the global distributed loads Qx =-10000 N/m and Qy=-10000 N/m are used.

The used approach gives good results for frame structures with **perpendicular rigid or semi-rigid beam** connections. **For other cases, the user has to evaluate the presented bucking ratios**.

Example: Buckling Factor.esa

Consider Column B1:

- L = 4000mm
- Set as sway
- In node N1 : My = 0 kNm => $C_2 = \rho_2 = 0.0$
- This node N1 defines ρ_2 because ρ_2 is always the smallest of the two.
- In node N2 for Loadcase LC1:
  - M_{y1} = 79833 kNm
  - ϕ_1 = fiy = 1530.6 mrad
  - $C_1 = M_{y1}/\phi_1$ = 79833 kNm / 1530.6 mrad = 52.158 kNm/mrad
    = 5,2158 x 10^{10} Nmm/rad
  - E = 210 000 N/mm²
  - Iy = 162700000 mm^4
  - $\rho_1 = \frac{C_i L}{EI} = \frac{5.2158 \cdot \frac{10^{10} Nmm}{rad} \cdot 4000mm}{210000 \frac{N}{mm^2} \cdot 162700000\ mm^4} = 6.106$
  - $x = \frac{4\rho_1\rho_2 + \pi^2\rho_1}{\pi^2(\rho_1+\rho_2)+8\rho_1\rho_2} = \frac{4 \cdot 6.106 \cdot 0.0 + \pi^2 \cdot 6.106}{\pi^2(6.106+0.0)+8 \cdot 6.106 \cdot 0.00} = 1.0$
  - $k = x\sqrt{\frac{\pi^2}{\rho_1 x}+4} = 1.0\sqrt{\frac{\pi^2}{6.11 \cdot 1.00}+4} = 2.37$
  - $N_{cr} = \frac{\pi^2 EI}{k^2 L^2} = \frac{\pi^2 \cdot 210000 N/mm^2 \cdot 162700000\ mm^4}{(2.37)^2(4000)^2} = 3752200N = 3752.2\ kN$

Those values can also be found in SCIA Engineer:

Under “Steel -> Beams -> Steel slenderness” the buckling length k_y will be found:

Steel slenderness

Linear calculation

Member	CS Name	Part	Sway y Sway z	Ly [m] Lz [m]	ky [-] kz [-]	ly [m] lz [m]	Lam y [-] Lam z [-]	lyz [m]	l LTB [m]
B1	CS1	1	Yes	4,000	2,37	9,466	63,28	4,000	4,000
			Yes	4,000	1,00	4,000	105,61		

This value can also be found in the stability check through “Steel -> Beams -> ULS Check -> Check” under the buckling parameters. And here also the critical normal force Ncr can be found:

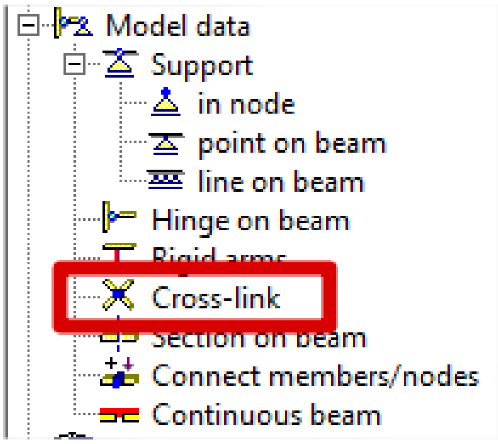
Buckling parameters	yy
Sway type	sway
System length L	4,000
Buckling factor k	2,37
Buckling length Lcr	9,466
Critical Euler load Ncr	3763,44
Slenderness Lambda	63,28
Relative slenderness Lambda,rel	0,67
Limit slenderness Lambda,rel,0	0,20

Calculation of the buckling factors for crossing diagonals

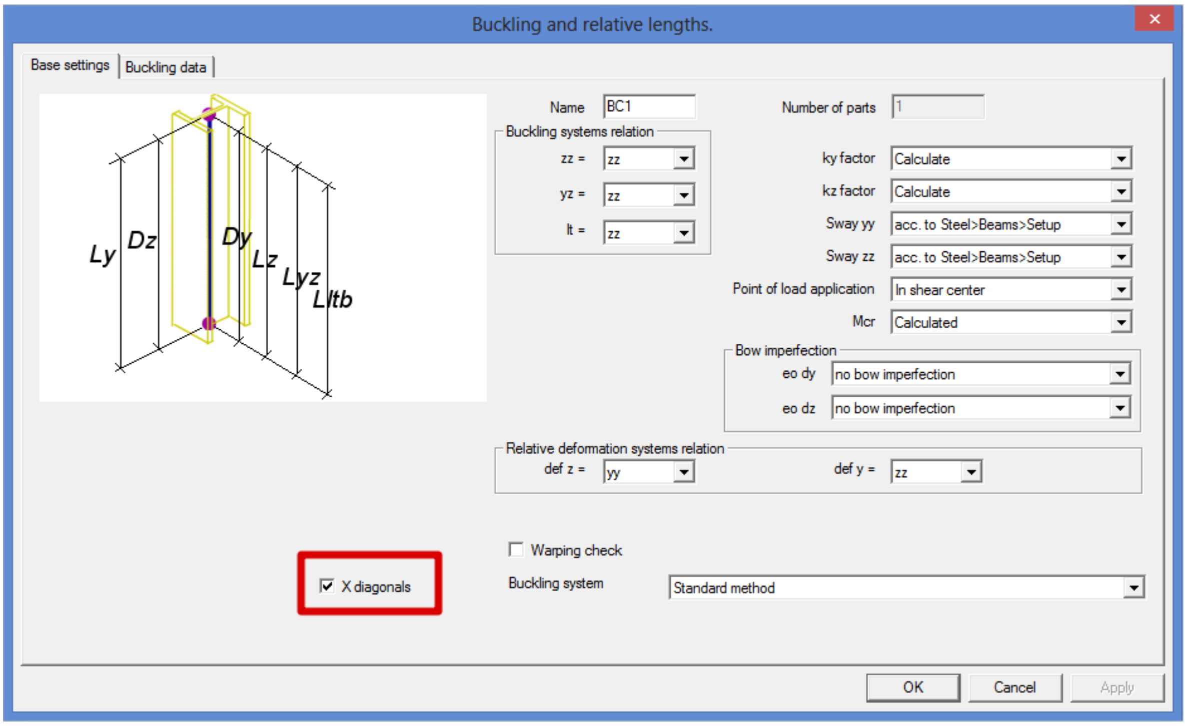
As the previous default method is only valid for perpendicular connection, this can't be used for the calculation of the buckling factors for diagonals.

In the DIN 1880 Teil 2, Table 15 a method is given for the calculation of the buckling factor for crossing diagonals. In SCIA Engineer this option is also implemented. In this method the buckling length s_K is calculated in function of the load distribution in the element and s_K is not a purely geometrical data.

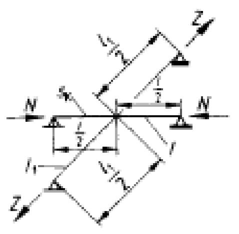
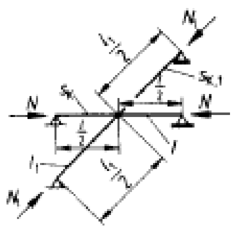
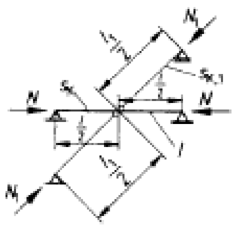
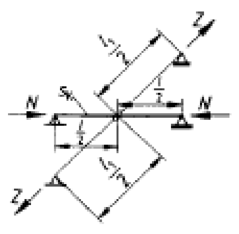
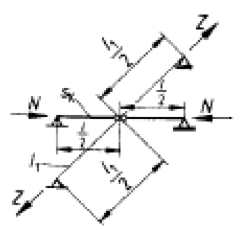
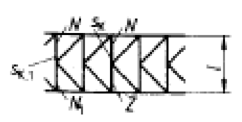
This method is only applicable for 2 diagonals with a hinged or rigid connection in the middle. To use this functionality in SCIA Engineer, the user has to connect the two diagonals with the option "Cross-link", which can be found in the menu Structure:



When connecting two beams with this option, in the buckling and relative lengths properties of the two beams the option **X-diagonals** can be checked:



If this option is used, SCIA Engineer will use the method from the DIN to calculate the buckling factors for the diagonals.

	1	2	3
1		$s_K = l\sqrt{\dfrac{1-\frac{3}{4}\,\frac{Z\cdot l}{N\cdot l_1}}{1+\frac{I_1\cdot l^3}{I\cdot l_1^3}}}$ jedoch $s_K \ge 0{,}5\,l$	
2		$s_K = l\sqrt{\dfrac{1+\frac{N_1\cdot l}{N\cdot l_1}}{1+\frac{I_1\cdot l^3}{I\cdot l_1^3}}}$ jedoch $s_K \ge 0{,}5\,l$	$s_{K,1} = l_1\sqrt{\dfrac{1+\frac{N\cdot l_1}{N_1\cdot l}}{1+\frac{I\cdot l_1^3}{I_1\cdot l^3}}}$ jedoch $s_{K,1} \ge 0{,}5\,l_1$
3		durchlaufender Druckstab $s_K = l\sqrt{1+\frac{\pi^2}{12}\cdot\frac{N_1\cdot l}{N\cdot l_1}}$	gelenkig angeschlossener Druckstab $s_{K,1} = 0{,}5\,l_1$ wenn $(E\cdot I)_d \ge \frac{N_1\cdot l^3}{\pi^2\cdot l_1}\left(\frac{\pi^2}{12}+\frac{N\cdot l_1}{N_1\cdot l}\right)$
4		$s_K = l\sqrt{1-0{,}75\,\frac{Z\cdot l}{N\cdot l_1}}$ jedoch $s_K \ge 0{,}5\,l$	
5		$s_K = 0{,}5\,l$ wenn $\frac{N\cdot l_1}{Z\cdot l} \le 1$ oder wenn gilt $(E\cdot I_1)_d \ge \frac{3\,Z\cdot l_1^2}{4\,\pi^2}\left(\frac{N\cdot l_1}{Z\cdot l}-1\right)$	
6		$s_K = l\left(0{,}75-0{,}25\left\lvert\frac{Z}{N}\right\rvert\right)$ jedoch $s_K \ge 0{,}5\,l$	$s_{K,1} = l\left(0{,}75+0{,}25\,\frac{N_1}{N}\right)$ $N_1 < N$

With:

s_K	buckling length
L	member length
l_1	length of supporting diagonal
I	moment of inertia (in the buckling plane) of the member
I_1	moment of inertia (in the buckling plane) of the supporting diagonal
N	compression force in member
N_1	compression force in supporting diagonal

Z	tension force in supporting diagonal
E	elastic modulus

Calculation of the buckling length for a VARH-element

For a VARH element, SCIA Engineer will use another calculation for the buckling length.

A VARH element is defined as follows:
The member has the properties of a symmetric I section where only the height is linear variable along the member. The system length for buckling around the local yy axis (strong axis), is equal to the member length.
For a VARH element (form node i to node j), we can define

L	beam length
I_i, I_j	moment of inertia at end i and j
A_i, A_j	sectional area at end i and j
E	modulus of Young
N_{cr}	critical Euler force
R_i, R_j	beam stiffness at end i and j

The stiffness R and R' is given by:

$$R = \frac{M}{\Phi}$$

$$R_{i'} = R_i \frac{L}{EI_i}$$

$$R_{j'} = R_j \frac{L}{EI_i}$$

$$\xi = \sqrt{\frac{I_j}{I_i}}$$

The critical Euler force is given by:

$$N_{cr} = \frac{\alpha^2 E I_i}{k^2 L^2}$$

To calculate α, the next steps are followed:

1. Calculate L, I_i, I_j, R_i, R_j, R'_i, R'_j, ξ

2. We suppose that

$$\frac{\overline{\alpha}}{\xi - 1} > \frac{1}{2}$$

3. Calculate a, b, c and d as follows

$$\beta = \sqrt{\frac{\alpha^2}{(\xi - 1)^2} - \frac{1}{4}}$$

$$a = \frac{1}{\alpha^2}[1 + (\xi - 1)(\frac{1}{2} - \beta \mathrm{cotg}(\beta \ln \xi)]$$

$$b = c = \frac{1}{\alpha^2}[1 - \frac{\beta}{\sin(\beta \ln \xi)} \frac{\xi - 1}{\sqrt{\xi}}]$$

$$d = \frac{1}{\alpha^2}[1 + \frac{(\xi - 1)}{\xi}(\frac{1}{2} + \beta \mathrm{cotg}(\beta \ln \xi)]$$

4. For a beam in non-sway system, we solve

$$1 + a\,R_{i'} + d\,R_{j'} + (ad - bc)\,R_{i'}\,R_{j'} = 0$$

For a beam in sway system, we solve

$$R_{i'}(1 - a\,\alpha^2) + R_{j'}(1 - d\,\alpha^2) - \alpha^2 + R_{i'}\,R_{j'}(a - b - c + d - \alpha^2(ad - bc)) = 0$$

5. When a solution is found, we check if

$$\frac{\overline{\alpha}}{\xi - 1} > \frac{1}{2}$$

6. If not, then recalculate a,b,c en d as follows :

$$a = \frac{1}{\alpha^2}[1 + \frac{(\xi - 1)((\frac{1}{2} - \beta)\,\xi^{\beta} - (\frac{1}{2} + \beta)\,\xi^{-\beta})}{\xi^{\beta} - \xi^{\beta}}]$$

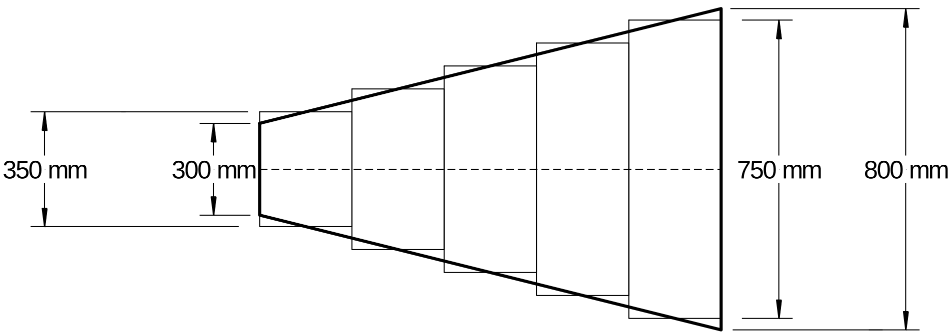
$$b = c = \frac{1}{\alpha^2}[1 - \frac{2\beta(\xi - 1)}{\sqrt{\xi}(\xi^{\beta} - \xi^{-\beta})}]$$

$$d = \frac{1}{\alpha^2}[1 + \frac{(\xi - 1)((\frac{1}{2} - \beta)\,\xi^{-\beta} - (\frac{1}{2} + \beta)\,\xi^{\beta})}{\xi(\xi^{\beta} - \xi^{-\beta})}]$$

and resolve the proper equation of 4.

Example:

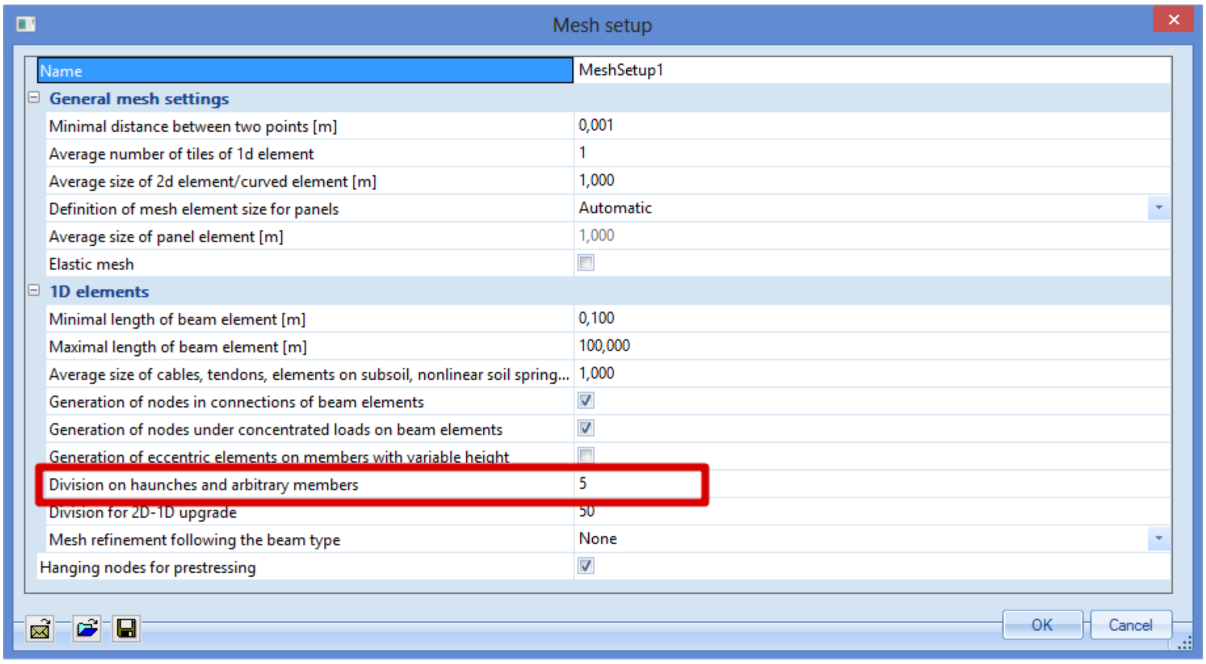
VARH.esa



Consider column B1

The VARH element going from 800mm to 300mm

The VARH is internally divided into a number of prismatic members. In this case, in 5 parts, but this can be changed in the Mesh setup:



- in node N1:
  - hi = 350 mm
  - Ii = 2.403 10^{-4} m^4
  - M=0 → R_i, R_i' = 0.0
- in node N1:
  - fiy = 1512 mrad
  - My = 79818 kNm
  - E = 210000 N/mm²
  - hj = 750 mm
  - Ii = 1.363 x 10^9 mm^4
  - Rj = 5.28 x 10^{10}
  - R'j = 4.2
  - ξ = 2.38
  - α = 1.75

⇨ $N_{cr} = 9626.16\ kN$

Buckling parameters	yy	zz	
Sway type	sway	sway	
System length L	4,000	4,000	m
Buckling factor k		1,00	
Buckling length Lcr	7,190	4,000	m
Critical Euler load Ncr	9785,96	7001,21	kN
Slenderness Lambda	48,58	57,43	
Relative slenderness Lambda,rel	0,52	0,61	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	b	c	
Imperfection Alpha	0,34	0,49	
Reduction factor Chi	0,88	0,78	
Buckling resistance Nb,Rd	2294,75	2038,29	kN

Buckling length

In the previous paragraph the general calculation of the buckling factors has been explained for all type of elements. With this buckling factor the buckling length of the beam will be calculated as follows:

$$l = k \cdot L$$

With:

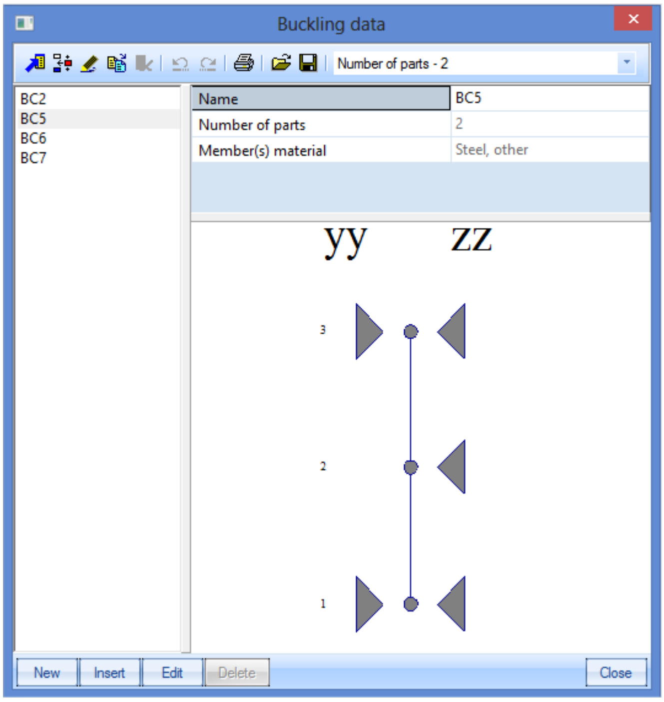
l	the buckling length
k	the buckling factor
L	the system length

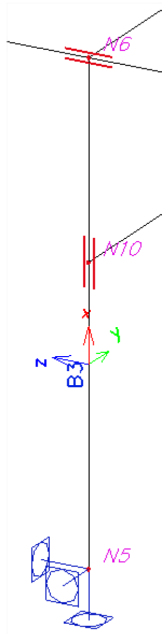
The system length of a beam is defined by the property “Buckling and relative lengths” of the beam.

Example: Buckling Length.esa

Consider column B3. This column has 3 nodes: N5, N10 and N6. The local X direction goes from the bottom to the top of the column, so internal for SCIA Engineer N5 is the first node of this column and N6 the last one.

In the properties window of this column the user can choose for “Buckling and relative lengths”. With this option, the system length of the beam can be inputted. Default the following option will appear:





Explanation of the System Lengths:

- The first node (according to the local x-axis) is node N5, the last one is node N6.
- yy direction:
  - This means **around** the local y-axis. So the column will deform in the z-direction.
  - Around the y-axis, node N5 is supported. In node N10 no beam can be found in the local z-direction in this point, thus column B3 is not supported around the y-axis in node N10. In node N6 a horizontal beam in the local z-direction can be found and the column will be supported around the local y-axis (yy) in node N6. This is indicated with the triangles in this window:
    - Supported in node N5
    - Not supported in node N10
    - Supported in node N6

- zz directions:
    - This means **around** the local z-axis. So the column will deform in the y-direction.
    - Around the z-axis, node N5 is supported. In node N10 a horizontal beam in the local y-direction can be found and the column will be supported around the local z-axis (zz) in node N10. Also in node N6 a horizontal beam in the local y-direction can be found and the column will also be supported around the local z-axis (zz) in node N6. This is indicated with the triangles in this window:
        - Supported in node N5
        - Supported in node N10
        - Supported in node N6

- The system length will be taken as follows:
    - Around the y-axis: the length between node N5 and N6: so 3m
    - Around the z-axis: the length between node N5 and N10 for the first part of the beam (1,8m) and the length between N10 and N6 for the second part of the beam: so 1.2m.
    - This can also be found in the menu “Steel -> Beams -> Steel slenderness”:

Steel slenderness

Linear calculation

Member	CS Name	Part	Sway y Sway z	Ly [m] Lz [m]	ky [-] kz [-]	ly [m] lz [m]	Lam y [-] Lam z [-]	lyz [m]	l LTB [m]
B3	CS3	1	Yes	3,000	1,09	3,277	18,56	1,800	1,800
			No	1,800	0,51	0,919	26,79		
B3	CS3	2	Yes	3,000	1,09	3,277	18,56	1,200	1,200
			No	1,200	0,57	0,681	19,85		

    - In this window the user can easily check the system length (Ly and Lz), the buckling factors (ky and kz) and the buckling length (ly = ky x Ly and lz = kz x Lz).

- When clicking on **Edit** in the buckling data window, the triangles indicating the system lengths of each part of a beam can be changed in the second tab “Buckling data”:

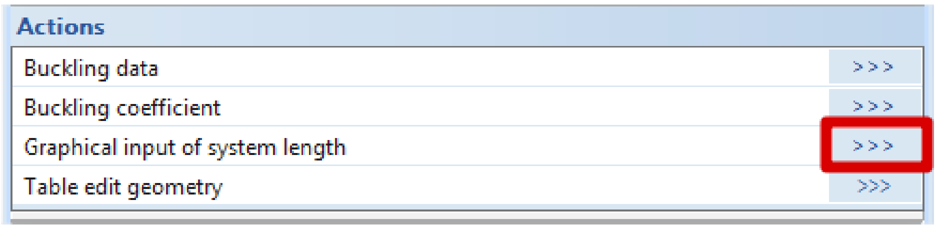
Buckling and relative lengths.

Base settings | Buckling data

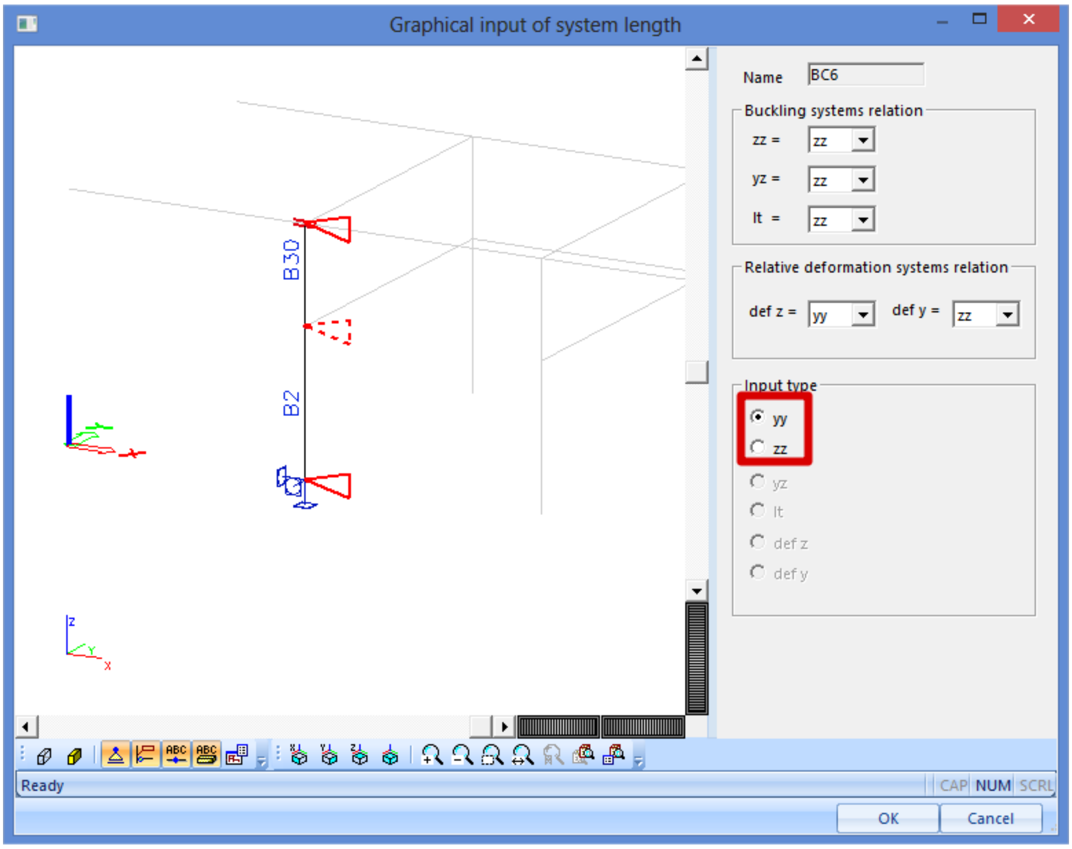
	yy	ky	Sway yy	ɛo dy [mm]	zz	kz	Sway zz	ɛo dz [mm]	kyz	klt	k	kw
1	Fixed		acc. to B		Fixed		acc. to B		1,00	1,00	1,00	1,00
2	Free				Fixed		acc. to B		1,00	1,00	1,00	1,00
3	Fixed				Fixed							

- Compare beam B3 and beams (B2+B30) with each other: they should have exactly the same system lengths. The only difference between those columns is that beam B3 was inputted as a beam of 3m and beams (B2+B30) are divided in two parts. SCIA Engineer will consider those two beams also as one buckling system:
    - When the local axes are exactly in the same direction (so in this case the local x-axis is in the same direction and the angle between the beams is exactly 180°).
    - If no hinge has been inputted between the two beams.

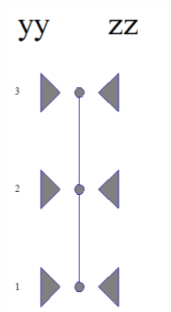
- There is also a graphical representation of the system lengths:
    - Select beam B2 (or B30)
    - Go to the actions menu and click on **Graphical input of system length**:



    - And now the user can choose between the yy and the zz axis:

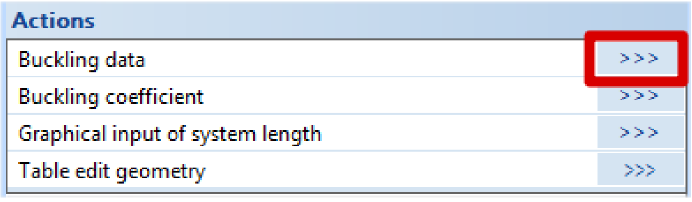


- Now take a look at beam B13. The system lengths are the following (as expected, because there are horizontal beams in the two directions on each node):

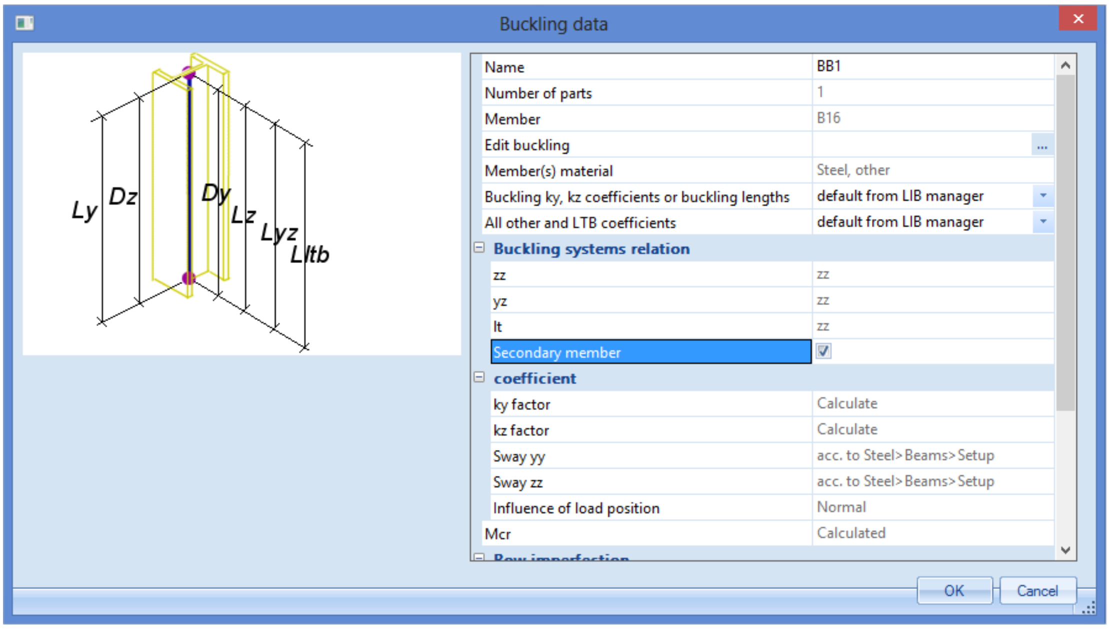


When looking at the rendered view, it will be clear that beam B16 is too weak to have an influence on the system length of beam B13. In SCIA Engineer there is possibility to exclude a beam from a buckling system.

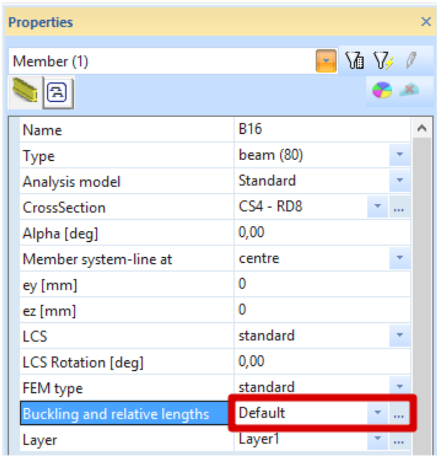
So select beam B16 and go in the Actions menu to **Buckling data**:

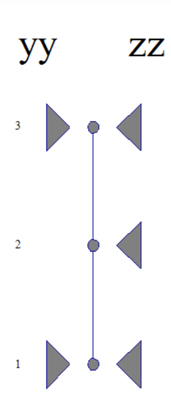


In this menu it is possible to indicate that beam B16 is a secondary beam and should not be taken into account in the system lengths:



When looking now at member B13 and changing the Buckling and relative lengths back to “defaults” the member B16 will not be included in the system lengths:



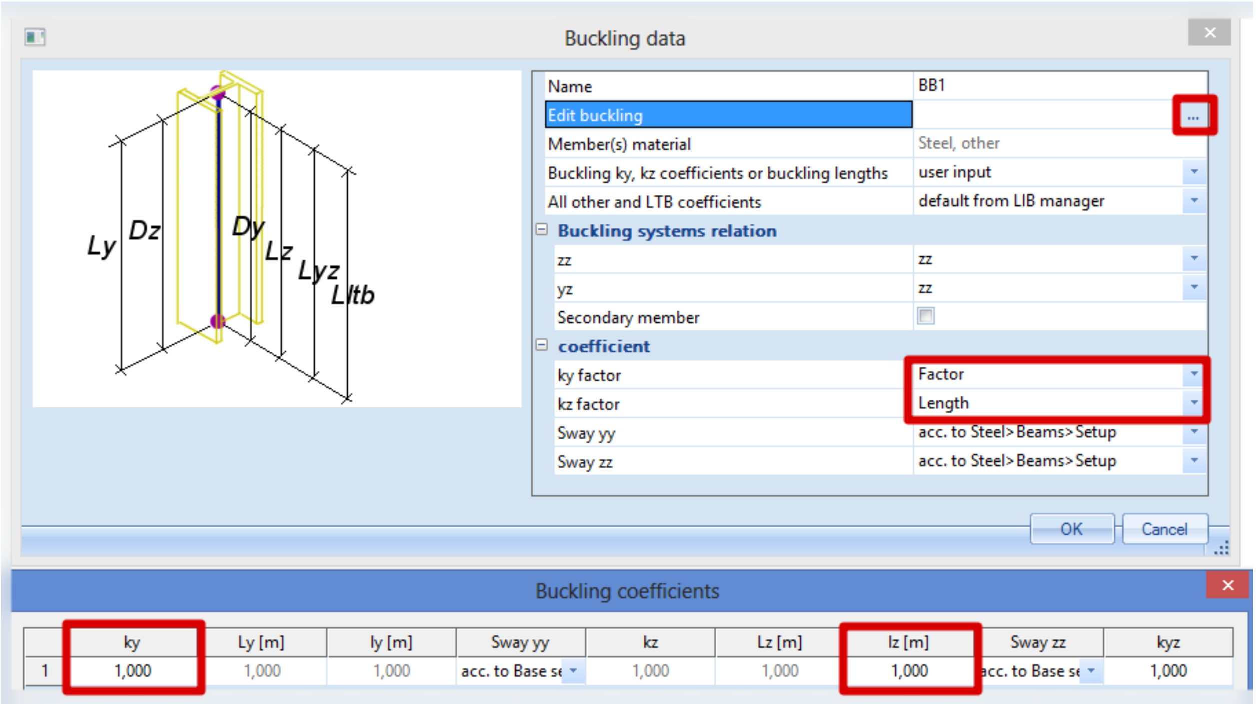


Buckling factors/lengths: manual input

The principles of the buckling factor and buckling length are explained in the previous paragraphs. Those default settings can be changed in the menu “Buckling and relative lengths” or in the steel menu with the “member buckling data”. This last one has a higher priority, so SCIA Engineer will first look if there are “member buckling data” on a beam. If yes, the program will take into account those properties, if no, the program will look at the properties in “buckling and relative lengths” of the properties window of the member.

In the menu “Steel -> Beams -> Member Check data -> Member buckling data” the user can change the “Buckling ky, kz coefficients or buckling lengths” on “user input”.

And now the user can choose for ky and kz if those coefficients are calculated (default in SCIA Engineer) or input the buckling factor or even directly the buckling length:



And those properties can be placed on one or more members.

Flexural buckling check in SCIA Engineer

Once all the buckling factors and system lengths have been inputted correctly, the Flexural buckling check can be executed in SCIA Engineer.

Example: Industrial hall.esa

Consider column B28:

The classification of beam B28 is class 4, so an effective cross section has been calculated:

Properties					
sectional area A eff	1.5269e+04	mm^2			
Shear area Vy eff	8.9420e+03	mm^2	Vz eff	6.3269e+03	mm^2
radius of gyration iy eff	320.39	mm	iz eff	58.15	mm
moment of inertia Iy eff	1.5674e+09	mm^4	Iz eff	5.1634e+07	mm^4
elastic section modulus Wy eff	4.1630e+06	mm^3	Wz eff	3.9265e+05	mm^3
Eccentricity eny	0.00	mm	enz	0.00	mm

SCIA Engineer will first show the Buckling parameters of this beam:

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	6,900	6,900	m
Buckling factor k	3,25	0,99	
Buckling length Lcr	22,415	6,832	m
Critical Euler load Ncr	6596,08	2293,84	kN
Slenderness Lambda	74,15	125,75	
Relative slenderness Lambda,rel	0,74	1,25	
Limit slenderness Lambda,rel,0	0,20	0,20	
Buckling curve	a	b	
Imperfection Alpha	0,21	0,34	
Reduction factor Chi	0,83	0,45	
Buckling resistance Nb,Rd	2976,05	1619,63	kN

In below the results of the yy direction are explained.

This direction has been set on "**sway**" in the steel Setup and with this option the **Buckling factor k** is calculated.
The length of the column is 6.900m. So the buckling length = 3.35 x 6.900 m = 22.4 m

With this buckling length the Critical Euler load Ncr can be calculated. Afterwards the slenderness and the Relative slenderness Lambda can be calculated with the critical Euler load.

An IPE750 profile has a buckling curve a, resulting in an Imperfection factor alpha = 0.21.

With those properties the reduction factor χ will be calculated, which will filled in the following formula:

$$N_{b,Rd} = \frac{\chi \cdot A_{eff} \cdot f_y}{\gamma_{M1}}$$

This result in a buckling resistance Nb,Rd = 2976.05 kN for flexural buckling around the local y-axis.

Exactly the same principle can be repeated for flexural buckling around the local z-axis.
This will result in a lower buckling resistance: Nb,Rd = 1619.63 kN.

The lowest buckling resistance will be used in the flexural buckling check:

Flexural Buckling verification		
Cross-section effective area Aeff	1,5269e+04	mm^2
Buckling resistance Nb,Rd	1619,63	kN
Unity check	0,10	-

Torsional Buckling

The check on torsional buckling can be important for profiles in which the position of the shear centre is not the same as the centre of gravity of this section.

The design buckling resistance $N_{b,Rd}$ for torsional or torsional-flexural buckling (according to EN 1993-1-1, art 6.3.1.4(3)) shall be obtained using the buckling curve of the z-axis, and with relative slenderness given by:

$$\bar{\lambda} = \sqrt{\frac{f_{yb}}{\sigma_{cr}} \beta_A}$$

$$\sigma_{cr} = \min\left(\sigma_{cr,T}, \sigma_{cr,TF}\right)$$

$$\sigma_{cr,T} = \frac{1}{A_g i_0^2}\left(GI_t + \frac{\pi^2 E C_m}{l_T^2}\right)$$

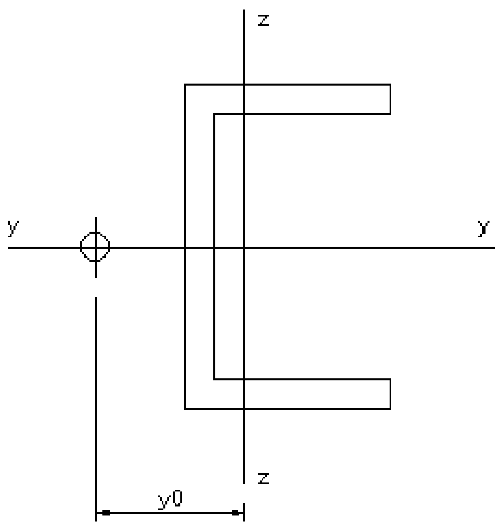
$$i_0^2 = i_y^2 + i_z^2 + y_0^2$$

$$\sigma_{cr,TF} = \frac{1}{2\beta}\left[\left(\sigma_{cr,y} + \sigma_{cr,T}\right) - \sqrt{\left(\sigma_{cr,y} + \sigma_{cr,T}\right)^2 - 4\beta\sigma_{cr,y}\sigma_{cr,T}}\right]$$

$$\sigma_{cr,y} = \frac{\pi^2 E}{\left(\frac{l_y}{i_y}\right)^2}$$

$$\beta = 1 - \left(\frac{y_0}{i_0}\right)^2$$

with	β_A	the ratio A_{eff}/A
	f_{yb}	the basic yield strength
	σ_{cr}	the critical stress
	$\sigma_{cr,T}$	the elastic critical stress for torsional buckling
	$\sigma_{cr,TF}$	the elastic critical stress for torsional-flexural buckling
	G	the shear modulus
	E	the modulus of elasticity
	I_T	the torsion constant of the gross section
	C_M	the warping constant
	i_y	the radius of gyration about yy-axis
	i_z	the radius of gyration about zz-axis
	l_T	the buckling length of the member for torsional buckling (= Lyz)
	y_0	the position of the shear centre
	l_y	the buckling length for flexural buckling about the yy-axis



Example: Flexural Torsional buckling.esa

Look at the steel code check for member B1:

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Torsional buckling length Lcr	3,600	m
Elastic critical load Ncr,T	2084,37	kN
Elastic critical load Ncr,TF	1302,24	kN
Relative slenderness Lambda,rel,T	0,98	
Limit slenderness Lambda,rel,0	0,20	
Buckling curve	c	
Imperfection Alpha	0,49	
Reduction factor Chi	0,55	
Cross-section area A	5,3300e+03	mm^2
Buckling resistance Nb,Rd	690,40	kN
Unity check	0,14	-

Lateral Torsional Buckling

General

General case

The flexural buckling check will be executed following EN 1993-1-1 **art. 6.3.2**.

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1$$

Where

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}}$$

$W_y = W_{pl,y}$ for class 1 or 2 cross-sections
$W_y = W_{el,y}$ for class 3 cross-sections
$W_y = W_{effy}$ for class 4 cross-sections

The reduction factor χ_{LT} will be calculated as follows:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}}$$ but $\chi \leq 1{,}0$

with

$$\Phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0{,}2\right) + \bar{\lambda}_{LT}^2\right]$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

M_{cr} Critical bending moment (see next paragraph for this calculation)

α Imperfection depending on the buckling curves:

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0,21	0,34	0,49	0,76

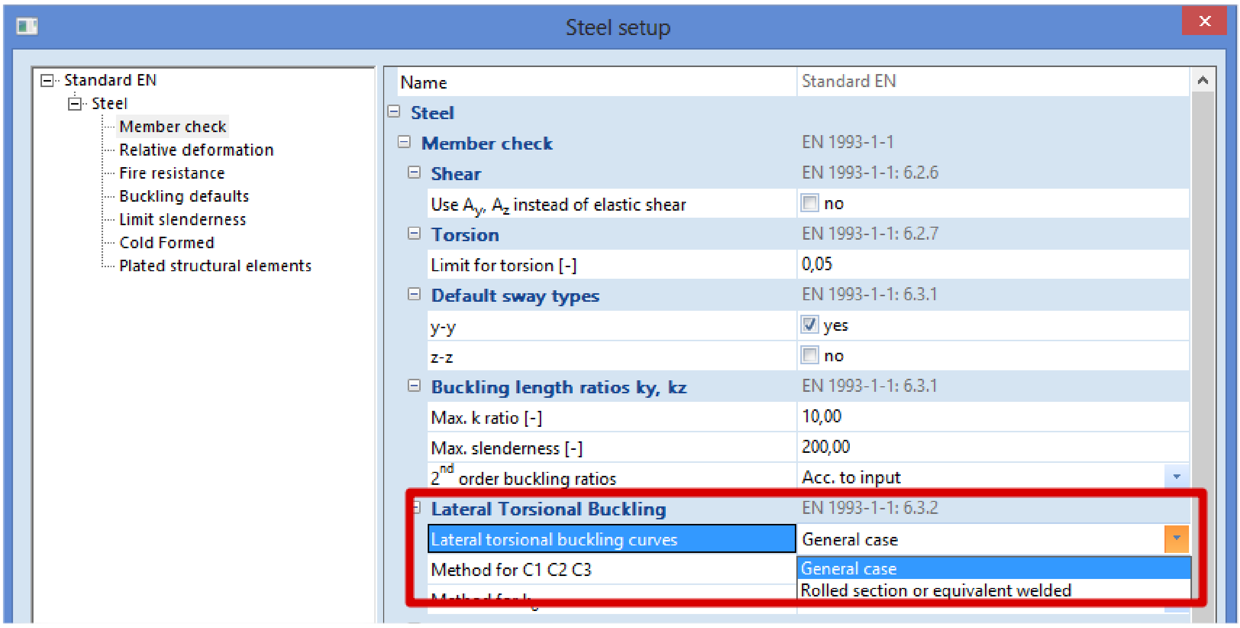
With the following recommended buckling curves for lateral torsional buckling:

Cross-section	Limits	Buckling curve
Rolled I-sections	$h/b \leq 2$ $h/b > 2$	a b
Welded I-sections	$h/b \leq 2$ $h/b > 2$	c d
Other cross-sections	-	d

For slendernesses $\bar{\lambda}_{LT} \leq 0{,}2$ of for $\frac{M_{Ed}}{M_{cr}} \leq 0{,}2^2$ lateral torsional buckling effects may be ignored.

Lateral torsional buckling for rolled sections or equivalent welded sections

In the EN 1993-1-1 a distinction is made between lateral torsional buckling for general cases and for rolled sections or equivalent welded sections.
This distinction can also be chosen in SCIA Engineer through “Steel -> Beams -> Steel setup” this can be chosen for all the beams:



With this option the reduction factor for Lateral Torsional buckling is calculated a bit differently:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}}$$

But: $\chi_{LT} \leq 1{,}0$

$\chi_{LT} \leq \frac{1}{\bar{\lambda}_{LT}^2}$

$$\Phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}\right) + \beta\bar{\lambda}_{LT}^2\right]$$

And

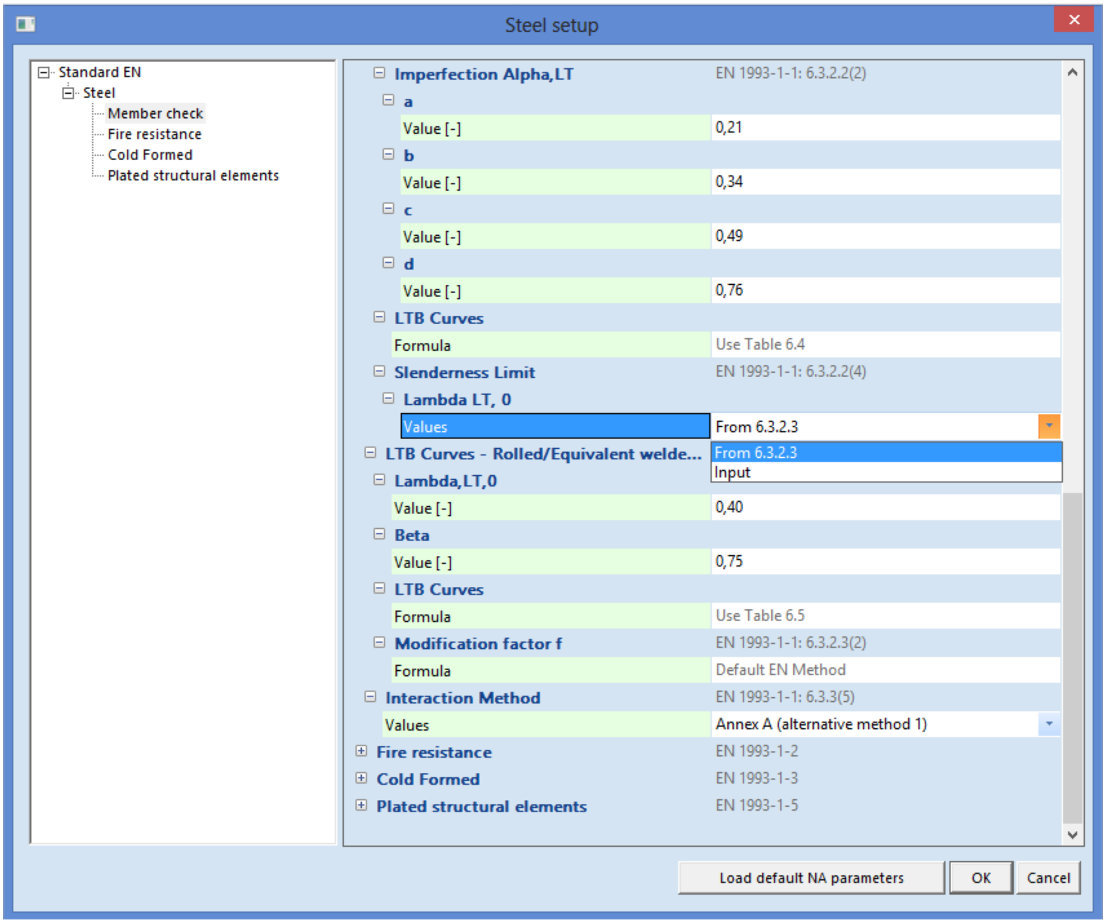
$\bar{\lambda}_{LT,0}$= 0,4 (maximum value)

β= 0,75 (minimum value)

Cross-section	Limits	Buckling curve
Rolled I-sections	$h/b \leq 2$ $h/b > 2$	b c
Welded I-sections	$h/b \leq 2$ $h/b > 2$	c d

For slendernesses $\bar{\lambda}_{LT} \leq \bar{\lambda}_{LT,0}$ of for $\frac{M_{Ed}}{M_{cr}} \leq \bar{\lambda}_{LT,0}{}^2$ lateral torsional buckling effects may be ignored.

Following EN 1993-1-1 article 6.3.2.3: $\bar{\lambda}_{LT,0}$= 0.4 but this can be adapted in the national annex of a country and also in SCIA Engineer:



The reduction factor χ_{LT} may be modified as follows:

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f}$$

f may be defined in the National Annex.

The following minimum values are recommended:

$$f = 1 - 0{,}5(1 - k_c)[1 - 2{,}0(\bar{\lambda}_{LT} - 0{,}8)^2]$$

But f $\leq$ 1,0

With k_c by default taken from the next table:

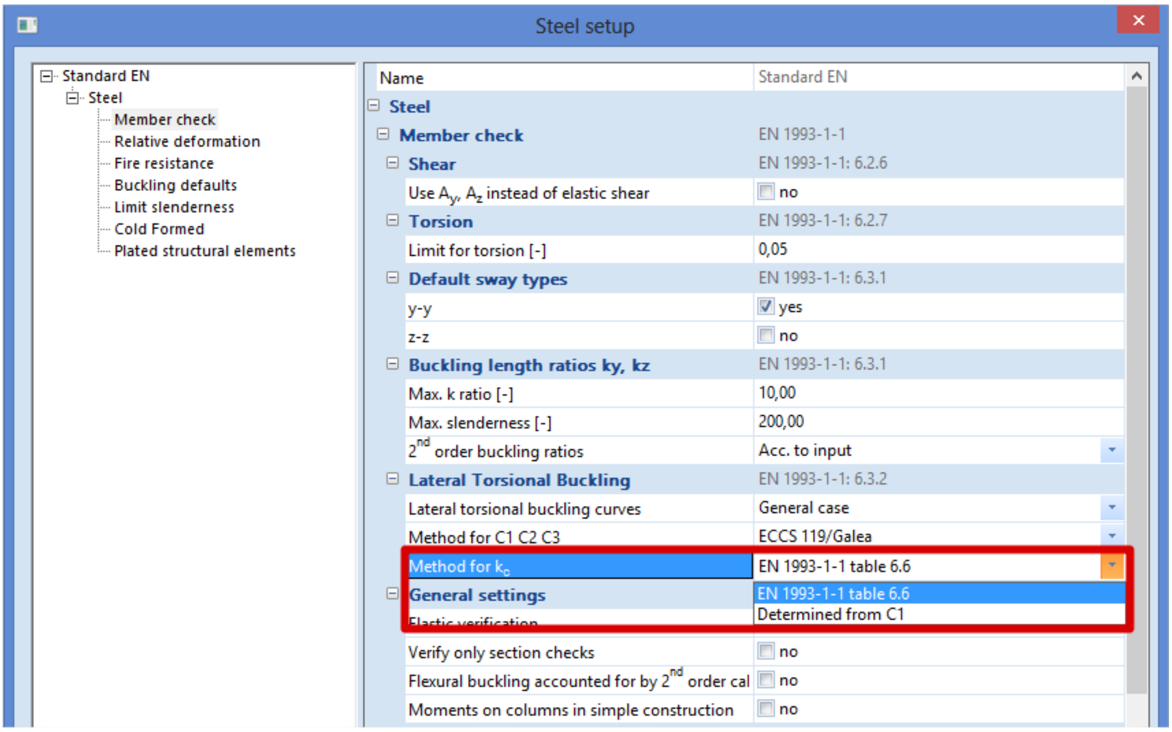
Table 6.6: Correction factors k_c

Moment distribution	k_c
$\psi = 1$	1,0
$-1 \leq \psi \leq 1$	$\frac{1}{1{,}33 - 0{,}33\psi}$
	0,94
	0,90
	0,91
	0,86
	0,77
	0,82

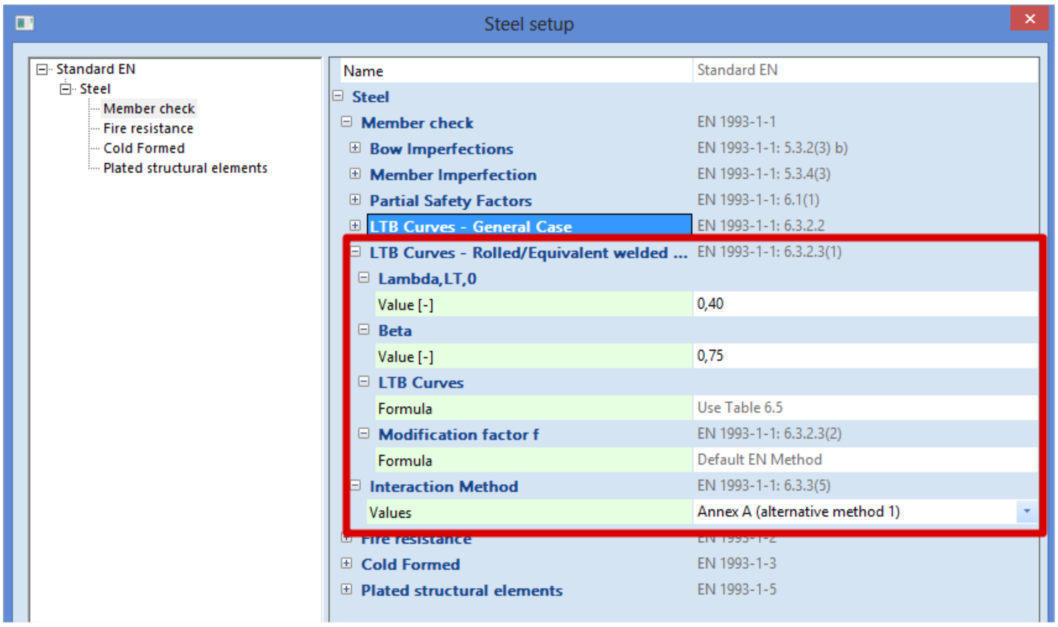
But alternatively k_c can also be calculated from the factor C_1:

$$k_c = 1/\sqrt{C_1}$$

In SCIA Engineer the user can choose between the standard method or the calculation of k_c in function of C_1 (by default k_c will be taken from Table 6.6) in “Steel -> Beams -> Steel setup”:



And also the values for beta and Lambda$_{LT,0}$ can be adapted in SCIA Engineer:



Lateral Torsional Buckling Check in SCIA Engineer

This LTB check will also be executed in SCIA Engineer.

Example: Industrial hall.esa

Consider beam B114:

The default method for the calculation of the C-factors has been used. This is also indicated in the preview of the check results:

First the Lateral Torsional Buckling Check will be shown:

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters		
Method for LTB curve	General case	
Cross-section plastic modulus Wpl,y	3,5800e+05	mm^3
Elastic critical moment Mcr	55,82	kNm
Relative slenderness Lambda,rel,LT	1,23	
Relative slenderness Lambda,rel,T	0,07	
Relative slenderness Lambda,rel,EXTRA	1,30	
Limit slenderness Lambda,rel,LT,0	0,20	
LTB curve	a	
Imperfection Alpha,LT	0,21	
Reduction factor Chi,LT	0,47	
Design buckling resistance Mb,Rd	39,49	kNm
Unity check	0,46	-

And afterwards the parameters for the calculation of M$_{cr}$ will be shown.

Mcr parameters		
LTB length L	6,000	m
Influence of load position	no influence	
Correction factor k	1,00	
Correction factor kw	1,00	
LTB moment factor C1	1,13	
LTB moment factor C2	0,45	
LTB moment factor C3	0,53	
Shear center distance d,z	0,00	mm
Distance of load application z,g	0,00	mm
Mono-symmetry constant beta,y	0,00	mm
Mono-symmetry constant z,j	0,00	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Below this "Mcr Parameters"-window the calculation method for the C parameters will be indicated.

Calculation of M_{cr}

General formula for I sections

For **I sections (symmetric and asymmetric), and Rectangular Hollow Sections (RHS)**, the elastic critical moment for Lateral Torsional Buckling M_{cr} is calculated by the following formula:

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(k_{LT} \cdot L)^2} \left\{ \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(k_{LT} \cdot L)^2 G I_t}{\pi^2 E I_z} + \left(C_2 z_g - C_3 z_j\right)^2} - \left[C_2 z_g - C_3 z_j\right] \right\}$$

Where

E	the Youn modulus of elasticity (E = 210000 N/mm² for steel)
G	the shear modulus (G = 80770 N/mm² for steel)
$k_{LT} \cdot L$	the lateral torsional buckling length of the beam between points which have lateral restraint (= l_{LTB}).
Iw	the warping constant
It	the torsional constant
Iz	the moment of inertia about the minor axis
z_g	the distance between the point of load application and the shear center
k_w	A factor which refers to end warping. Unless special provision for warming fixity is made, k_w should be taken as 1,0.
z_j	$z_j = z_s - 0{,}5 \int_A (y^2 + z^2) \frac{z}{I_y} dA$
	For doubly symmetric cross-sections: $z_j = 0$
z_s	the coordinate of the shear center

C_1, C_2 and **C_3** are factors depending on the loading and end restraint conditions.

In SCIA Engineer are different methods implemented for the calculation of those C_1 and C_2 factors. Those methods are explained further in this chapter.

Haunched sections (I+Ivar, Iw+Plvar, Iw+Iwvar, Iw+Ivar, I+Iwvar) and composed rail sections (Iw+rail, Iwn+rail, I+rail, I+2PL+rail, I+PL+rail, I+2L+rail, I+Ud+rail) are considered as equivalent asymmetric I sections.

The formula for Mcr uses the following parameters:

- **C1, C2, C3: calculated according to ENV, ECCS or Lopez**
- **LTB length: klt*L**
- **k and kw: factors related to the end fixity**
- **zg: load position**
- **zj: asymmetry of the section**

More details about each parameter are given in separate chapters

LTBII
It is also possible to calculate M_{cr} with a more precise calculation, a second order Lateral Torsional Buckling calculation. This will be explained further in this chapter.

General – calculation of C_1, C_2 and C_3 factors

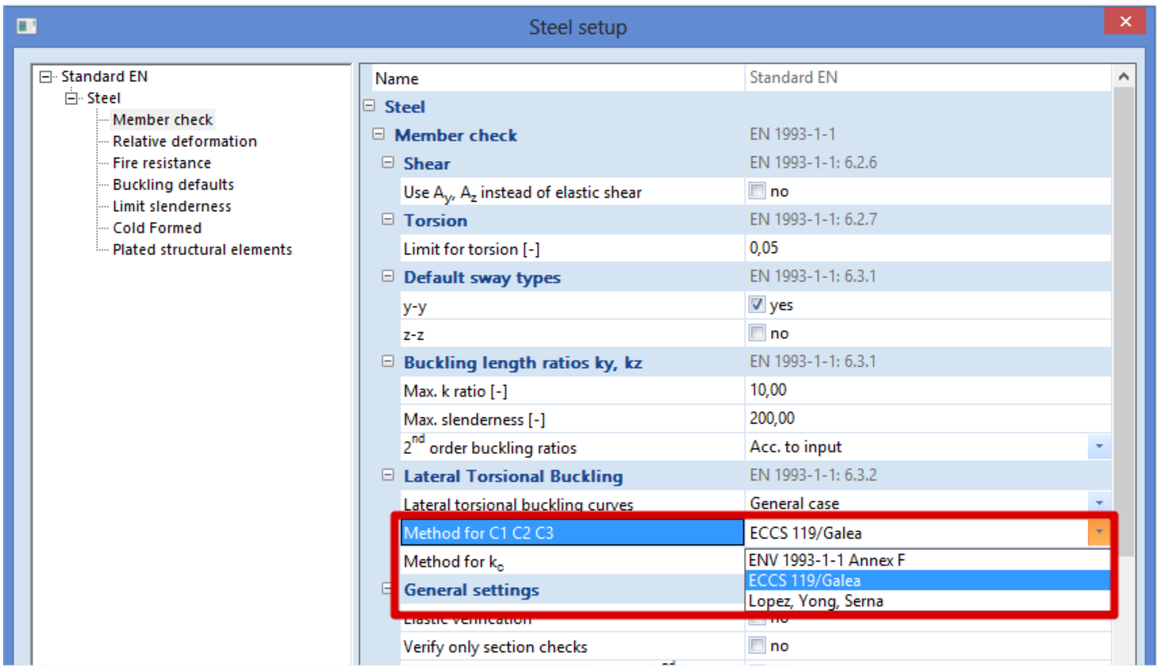
C1 : takes into account the shape of the moment diagram
C2 : takes into account the position of the loading
C3 : takes into account the asymmetry of the cross section

The coefficients **C1**, **C2** and **C3** can be calculated in SCIA Engineer according to three different methods:

- ENV 1993-1-1 Annex F
- ECCS 119/Galea
- Lopez, Young, Serna

By default the method according to ECCS 119/Galea is applied. The following paragraphs give more information on these methods.

The user can choose between those 3 methods in “Steel -> Beams -> Steel setup”:



ENV 1993-1-1 Annex F

When this setting is chosen, the moment factors are determined according to ENV 1993-1-1 Annex F Ref.[5].

For determining the moment factors (EN 1993-1-1: C1, C2 and C3) for lateral torsional buckling (LTB), we use the standard tables.
The current moment distribution is compared with some standard moment distributions. This standard moment distributions are moment lines generated by a distributed q load, a nodal F load, or where the moment line reach a maximum at the start or at the end of the beam.

The standard moment distributions which is closest to the current moment distribution, is taken for the calculation of the factors C1, C2 and C3.

ECCS 119/Galea

When this setting is chosen, the moment factors are determined according to ECCS 119 Annex B Ref.[34].
The figures given in this reference for C1 and C2 in case of combined loading originate from Ref.[28] which in fact also gives the tabulated values of those figures as well as an extended range.

The actual moment distribution is compared with several standard moment distributions. These standard moment distributions are moment lines generated by a distributed q load, a nodal F load, or where the moment line is maximum at the start or at the end of the beam.

The standard moment distribution which is closest to the actual moment distribution, is taken for the calculation of the factors C_1 and C_2.

Galea gives results only for C_1 and C_2 factors.
C_3 is taken from ECCS 119 Annex B tables 63 and 64. The C_3 is determined based on the case of which the C_1 value most closely matches the table value.

Lopez, Yong, Serna

When this method is chosen, the moment factors are determined according to Lopez, Yong, Serna Ref[35].

When using this method the coefficient **C_2** and **C_3** are set to **zero**.

The coefficient **C_1** is calculated as follows:

$$C_1 = \frac{\sqrt{\sqrt{k}A_1 + \left[\frac{(1-\sqrt{k})}{2}A_2\right]^2} + \frac{(1-\sqrt{k})}{2}A_2}{A_1}$$

Where:

$$k = \sqrt{k_1 k_2}$$

$$A_1 = \frac{M_{max}^2 + \alpha_1 M_1^2 + \alpha_2 M_2^2 + \alpha_3 M_3^2 + \alpha_4 M_4^2 + \alpha_5 M_5^2}{(1 + \alpha_1 + \alpha_2 + \alpha_3 + \alpha_4 + \alpha_5)M_{max}^2}$$

$$A_2 = \left|\frac{M_1 + 2M_2 + 3M_3 + 4M_4 + 5M_5}{9M_{max}}\right|$$

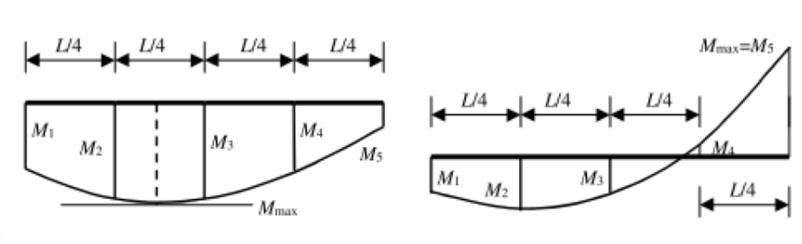
$$\alpha_1 = 1 - k_2$$

$$\alpha_2 = 5\frac{k_1^3}{k_2^2}$$

$$\alpha_3 = 5\left(\frac{1}{k_1} + \frac{1}{k_2}\right)$$

$$\alpha_4 = 5\frac{k_2^3}{k_1^2}$$

$$\alpha_1 = 1 - k_1$$



With:

k_1	Taken equal to k_w
k_2	Taken equal to k_w
M_1, M_2, M_3, M_4, M_5	The moments M_y determined on the buckling system in the given sections as shown on the above figure. These moments are determined by dividing the beam into 10 parts (11 sections) and interpolating between these sections.
M_{max}	The maximal moment My along the LTB system.

This method is only supported in case both k and k_w equal 0.50 or 1.00.

Comparison of the 3 calculation methods

In below an example in SCIA Engineer in which the three methods are calculated:

Example: Cfactors.esa

In the steel setup the chosen calculation method for the C factors has been changed. In below an overview of the results for those factors for the three methods:

ENV 1993-1-1 Annex F

Mcr parameters	
LTB length L	12,000
Influence of load position	stabilising
Correction factor k	1,00
Correction factor kw	1,00
LTB moment factor C1	1,89
LTB moment factor C2	0,33
LTB moment factor C3	2,64
Shear center distance d,z	0,00
Distance of load application z,g	-245,00
Mono-symmetry constant beta,y	0,00
Mono-symmetry constant z,j	0,00

Mcr = 1576.03 kNm

ECCS 119/Galea

Mcr parameters	
LTB length L	12,000
Influence of load position	stabilising
Correction factor k	1,00
Correction factor kw	1,00
LTB moment factor C1	1,26
LTB moment factor C2	0,45
LTB moment factor C3	0,41
Shear center distance d,z	0,00
Distance of load application z,g	-245,00
Mono-symmetry constant beta,y	0,00
Mono-symmetry constant z,j	0,00

Mcr = 1118.50 kNm

Lopez, Yong, Serna

Mcr parameters	
LTB length L	12,000
Influence of load position	stabilising
Correction factor k	1,00
Correction factor kw	1,00
LTB moment factor C1	1,20
LTB moment factor C2	0,00
LTB moment factor C3	0,00
Shear center distance d,z	0,00
Distance of load application z,g	-245,00
Mono-symmetry constant beta,y	0,00
Mono-symmetry constant z,j	0,00

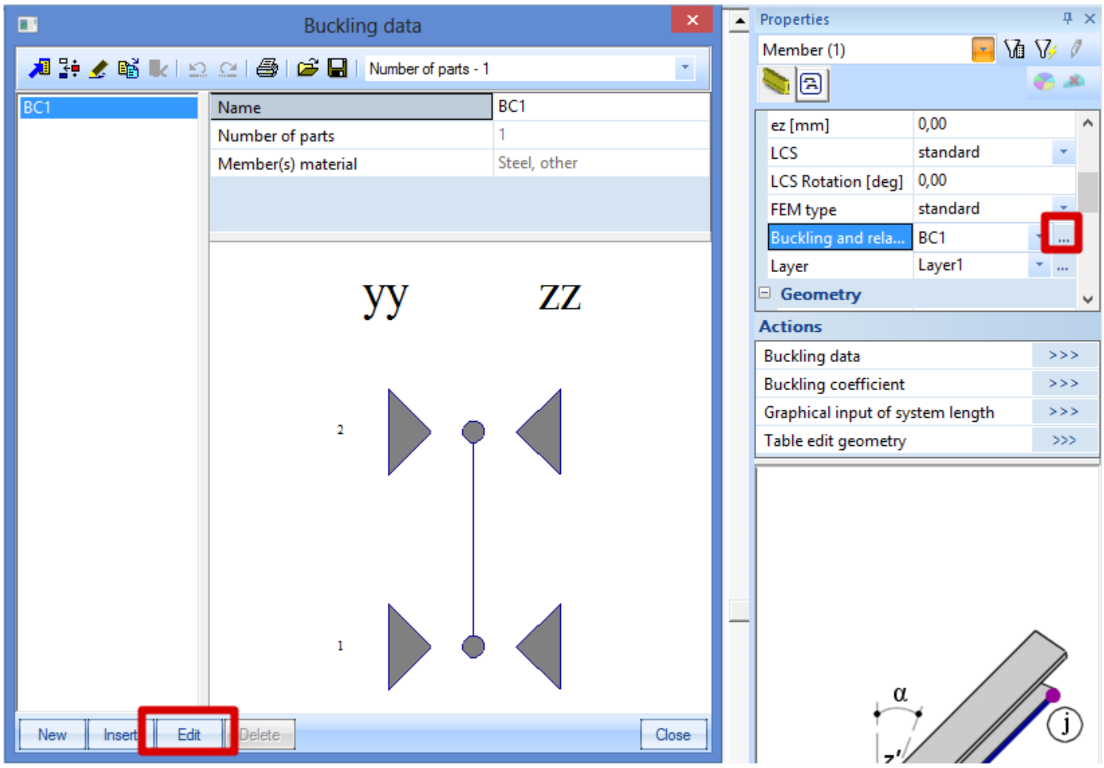
Mcr = 842.64 kNm

There can be a big difference between the three calculation models.
In the method following "Lopez, Yong, Serna" the values for C2 and C3 are always taken equal to zero.
When comparing the C1 factors, the method following "ECCS 119/Galea" and "Lopez, Yong, Serna" are approximately the same (1.26 and 1.20 respectively), but the C1 factor following the "ENV 1993-1-1 Annex F" results in total different value: 1.89.

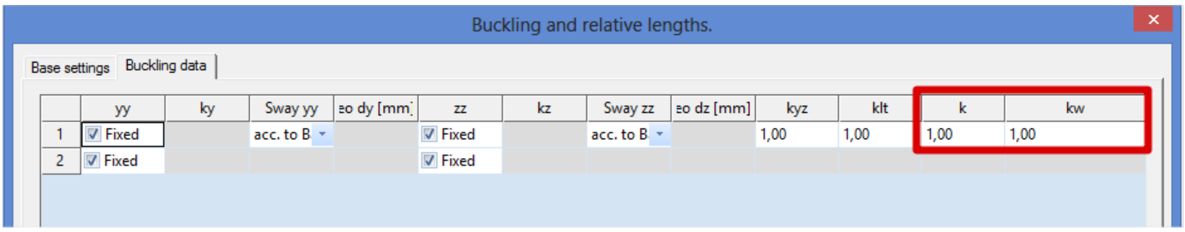
k and kw factors

It is generally assumed that k = kw = 1, which means that the ends are not fixed. If the ends are fixed, values lower than one can be used and this would lead to bigger values of Mcr. You can adapt the values of k and kw from the member buckling data.

Select the member then open the ‘Buckling and relative lengths’ menu from the properties window:



Click on Edit button and go to Buckling data tab, here you can modify the values for k and kw:

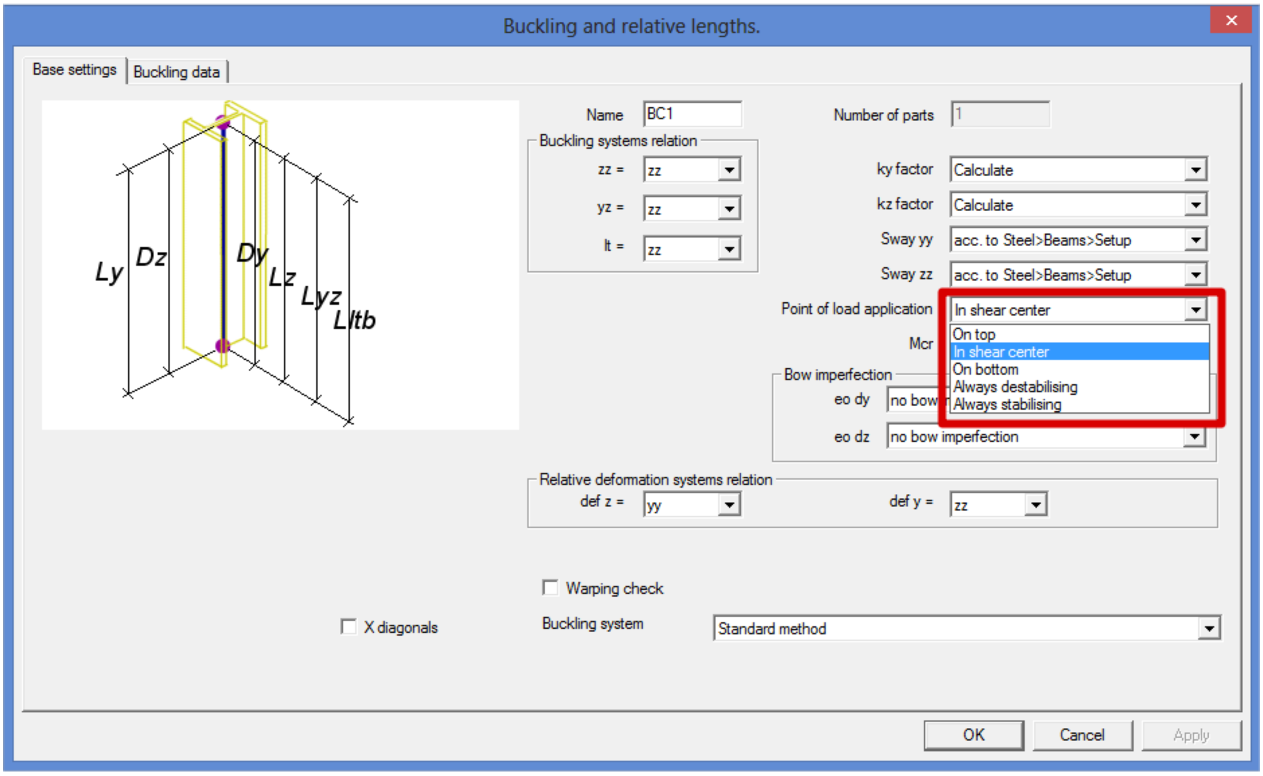


Load position

The load position has an influence on calculated Mcr through the value of zg. The user can chose among five load positions.

If you open the Buckling and relative lengths menu, you can see an option called ‘Point of load application’. The five possibilities are:

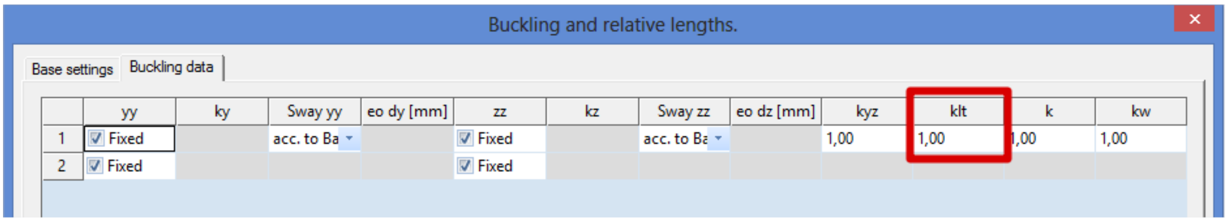
- On top: the load is applied on the top flange
- In shear center: the load is applied in the shear center
- On bottom: the load is applied at the bottom flange
- Always destabilising: the load is applied on the destabilising flange
- Always stabilising: the load is applied on the stabilising flange



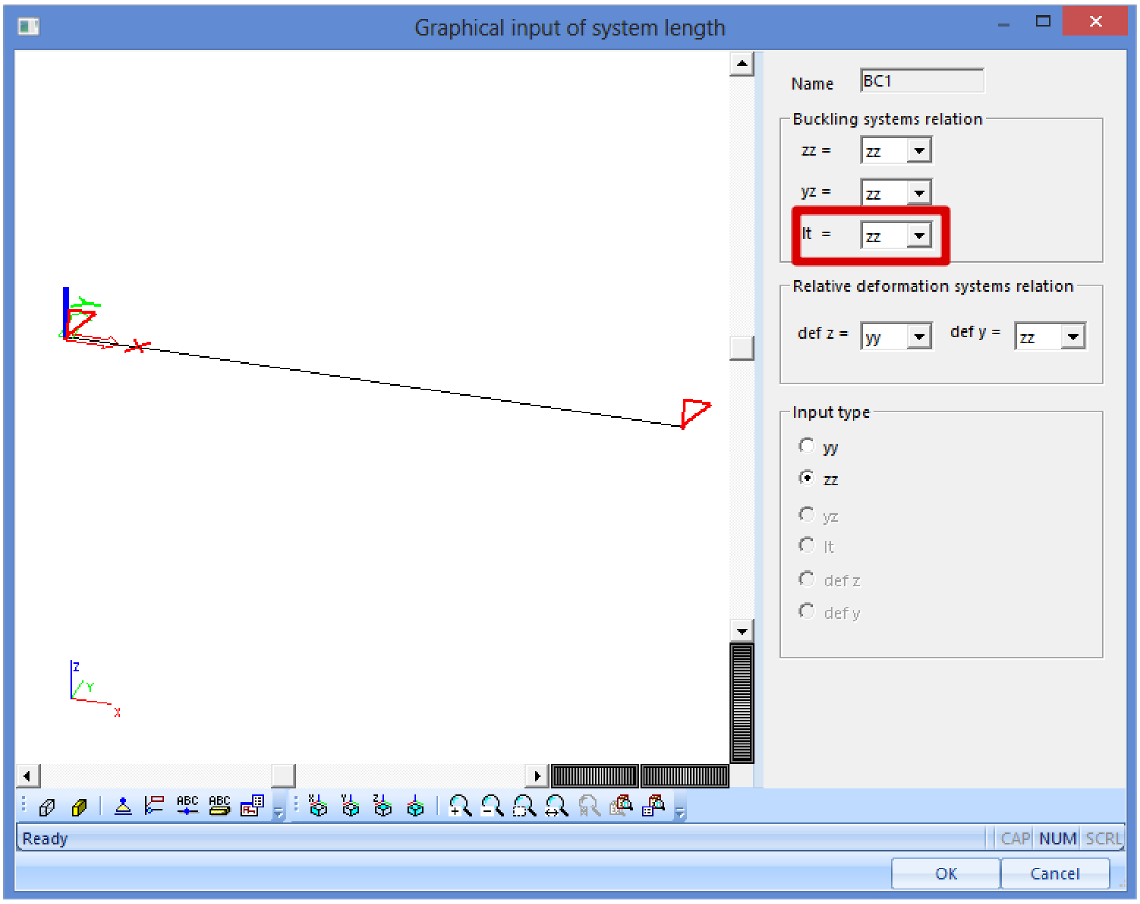
This value is set to ‘In shear center’ by default and can be adapted to influence the value of Mcr.

LTB Length

LTB length is calculated as $L_{lt} = k_{lt} * l_{ref}$
k_{lt} is by default taken equal to 1. A smaller value can be used to reduce the LTB length. You can adapt k_{lt} from the member buckling data:

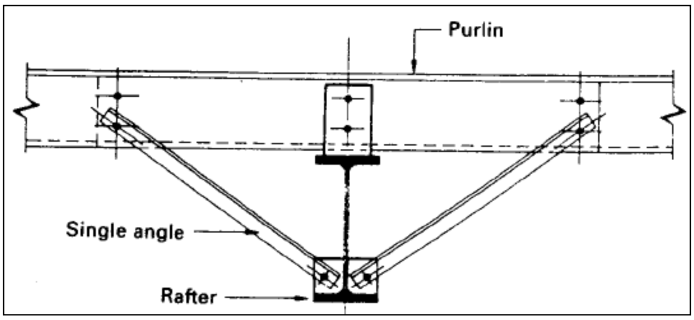


l_{ref} is the reference length. It is by default equal to the reference length around the weak axis (Lz) for both bottom and top flange:



The reference length can be overruled by another value using LTB restraints. These restraints make it possible to define separate conditions for bottom and top flange.

LTB Restraints



In SCIA Engineer Lateral Torsional Buckling restraints can be inputted. Those restraints will change the Lateral Torsional Buckling Length, used for the calculation of M_{cr}:

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(k_{LT} \cdot L)^2} \left\{ \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(k_{LT} \cdot L)^2 G I_t}{\pi^2 E I_z} + \left(C_2 z_g - C_3 z_j\right)^2} - \left[C_2 z_g - C_3 z_j\right] \right\}$$

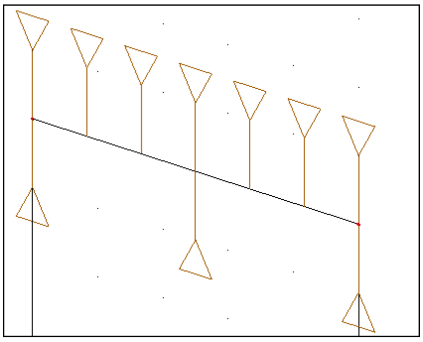
Where

$k_{LT} \cdot L$	the lateral torsional buckling length of the beam between points which have lateral restraint (= l_{LTB}).

This length will be taken as the distance between two LTB restraints.

Fixed LTB restraints are defined on top flange or on bottom flange. The LTB lengths for the compressed flange are taken as distance between these restraints. The LTB moments factors are calculated between these restraints.

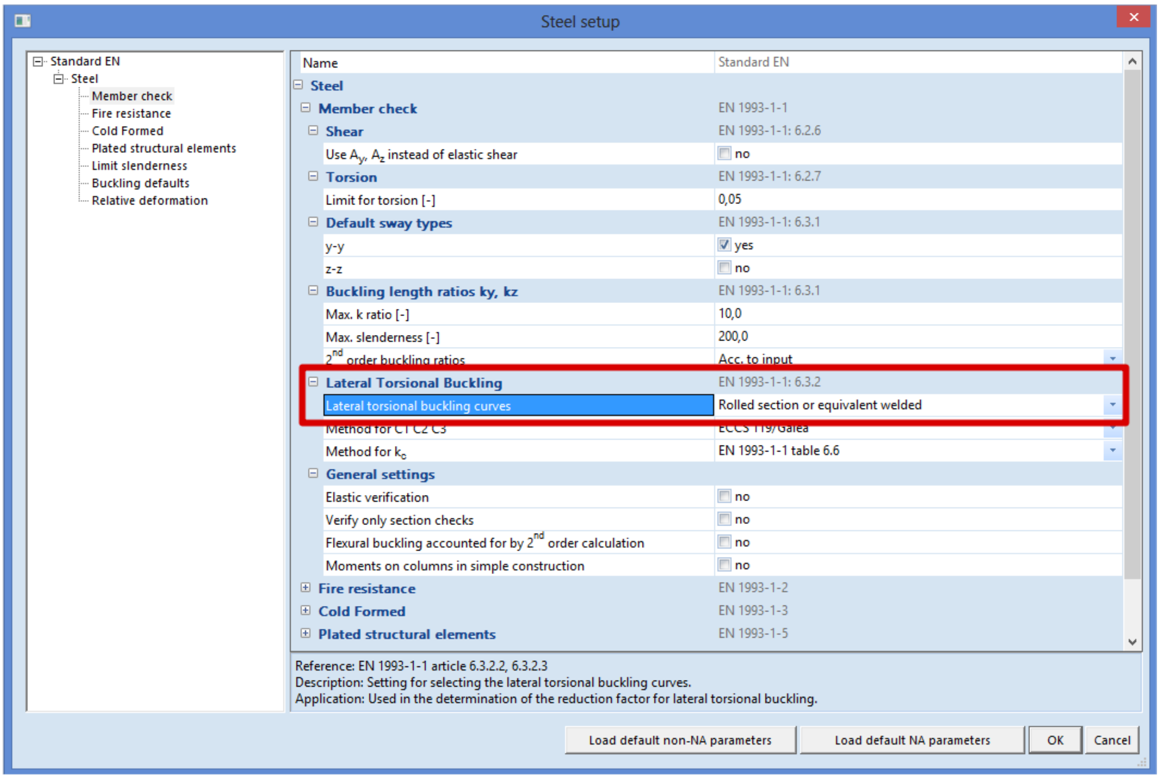
The restraints can be inputted via “Steel -> Beams -> Member Check data -> LTB Restraints”.



And only the restraints on the compressed side are taken into account.

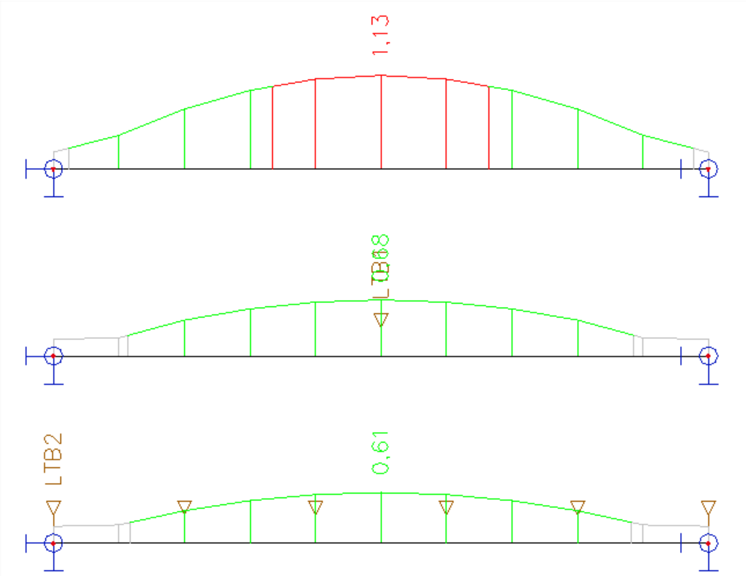
Example: LTB_Restraints.esa

In this example the same beam has been inputted three times. Since in this example an IPE450 is used, the Lateral torsional buckling curves for “Rolled section or equivalent welded” are used in the steel setup:



Note: The default setting for LTB curves is ‘General case’.

The results for LC1 are:



B1

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters		
Method for LTB curve	Alternative case	
Cross-section plastic modulus Wpl,y	1,7020e-03	m^3
Elastic critical moment Mcr	281,38	kNm
Relative slenderness Lambda,rel,LT	1,19	
Limit slenderness Lambda,rel,LT,0	0,40	
LTB curve	c	
Imperfection Alpha,LT	0,49	
LTB factor Beta	0,75	
Reduction factor Chi,LT	0,53	
Correction factor kc	0,94	
Correction factor f	0,98	
Modified reduction factor Chi,LT,mod	0,54	
Design buckling resistance Mb,Rd	216,08	kNm
Unity check	**1,13**	-

Mcr parameters		
LTB length L	7,000	m
Influence of load position	no influence	
Correction factor k	1,00	
Correction factor kw	1,00	
LTB moment factor C1	1,13	
LTB moment factor C2	0,45	
LTB moment factor C3	0,53	
Shear center distance d,z	0	mm
Distance of load application z,g	0	mm
Mono-symmetry constant beta,y	0	mm
Mono-symmetry constant z,j	0	mm

B2

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters		
Method for LTB curve	Alternative case	
Cross-section plastic modulus Wpl,y	1,7020e-03	m^3
Elastic critical moment Mcr	977,38	kNm
Relative slenderness Lambda,rel,LT	0,64	
Limit slenderness Lambda,rel,LT,0	0,40	
LTB curve	c	
Imperfection Alpha,LT	0,49	
LTB factor Beta	0,75	
Reduction factor Chi,LT	0,86	
Correction factor kc	0,91	
Correction factor f	0,96	
Modified reduction factor Chi,LT,mod	0,90	
Design buckling resistance Mb,Rd	360,26	kNm
Unity check	0,68	-

Mcr parameters		
LTB length L	3,500	m
Influence of load position	no influence	
Correction factor k	1,00	
Correction factor kw	1,00	
LTB moment factor C1	1,33	
LTB moment factor C2	0,12	
LTB moment factor C3	1,00	
Shear center distance d,z	0	mm
Distance of load application z,g	0	mm
Mono-symmetry constant beta,y	0	mm
Mono-symmetry constant z,j	0	mm

And for beam B3, 6 LTB restraints are inputted, so no LTB calculation will be executed:

If $\bar{\lambda}_{LT} \leq \bar{\lambda}_{LT,O}$ or for $\frac{M_{Ed}}{M_{cr}} \leq \bar{\lambda}_{LT,O}{}^2$ lateral torsional buckling effects may be ignored. With: $\bar{\lambda}_{LT,O} = 0,4$

So for beam B3 the following LTB check is displayed:

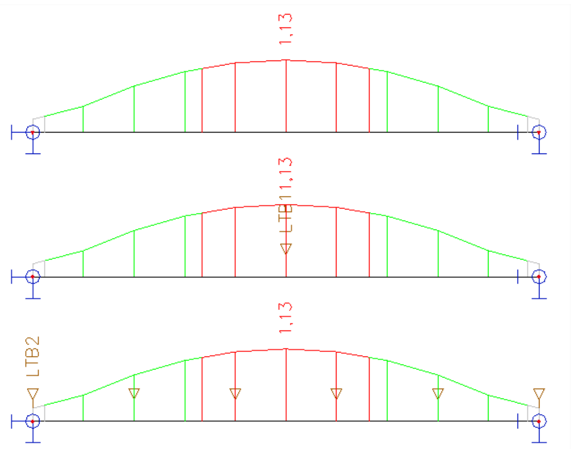
Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters		
Method for LTB curve	Alternative case	
Cross-section plastic modulus Wpl,y	1,7020e-03	m^3
Elastic critical moment Mcr	4043,43	kNm
Relative slenderness Lambda,rel,LT	0,31	
Limit slenderness Lambda,rel,LT,0	0,40	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters		
LTB length L	1,400	m
Influence of load position	no influence	
Correction factor k	1,00	
Correction factor kw	1,00	
LTB moment factor C1	1,01	
LTB moment factor C2	0,02	
LTB moment factor C3	1,00	
Shear center distance d,z	0	mm
Distance of load application z,g	0	mm
Mono-symmetry constant beta,y	0	mm
Mono-symmetry constant z,j	0	mm

When looking at load case LC2, the top side of the beam will be under tension, so SCIA Engineer will not take into account the effects of the LTB Restraints:



Calulation of Mcr for general sections

For the **other supported sections** as defined above, the elastic critical moment for Lateral Torsional Buckling M_{cr} is given by:

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(k_{LT} \cdot L)^2} \sqrt{\frac{I_w}{I_z} + \frac{(k_{LT} \cdot L)^2 G I_t}{\pi^2 E I_z}}$$

with	E	the modulus of elasticity
	G	the shear modulus
	$k_{LT}.L$	the lateral torsional buckling length of the beam between points which have lateral restraint (= l_{LTB}).
	Iw	the warping constant
	It	the torsional constant
	Iz	the moment of inertia about the minor axis

Calulation of Mcr for channel sections

When a channel section is loaded, additional torsion appears due to the eccentricity of the shear centre relative to the centroid of the cross section. For that reason, the value of Mcr has to be adapted.

In order to account for this additional torsion effect, the following procedure can be used where λ_{EXTRA} is calculated.

Modified design rule for LTB of Channel sections

In case this setting is activated within the Steel Setup, the reduction factor for Lateral-Torsional Buckling of Channel sections is determined according to Ref.[22].

More specifically the calculation is done as follows:

Reduction factor: $$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \lambda_{EXTRA}^2}}$$

Where: $$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}(\lambda_{EXTRA} - 0.2) + \lambda_{EXTRA}^2\right] \rightarrow \text{curve A for channel sections, therefore } \alpha_{LT} = 0.21$$

$$\lambda_{EXTRA} = \lambda_{LT} + \lambda_T$$

$$\lambda_{LT} = \sqrt{\frac{M_{pl;y}}{M_{cr;y}}}$$

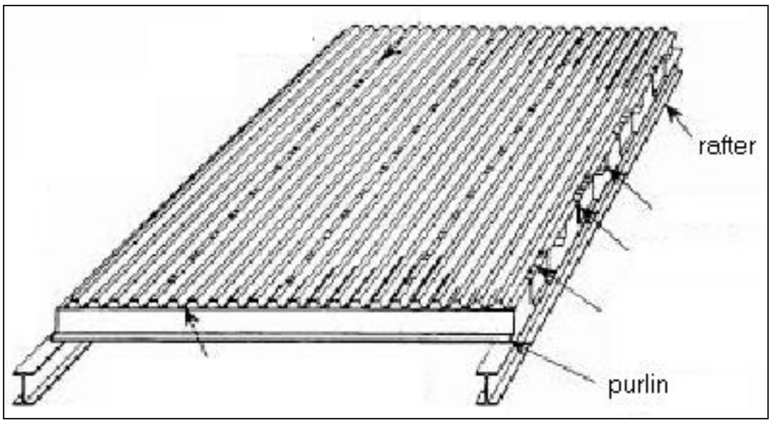
$\lambda_T = 1.0 - \lambda_{LT}$	*if*	$0.5 \le \lambda_{LT} < 0.80$
$\lambda_T = 0.43 - 0.29\lambda_{LT}$	*if*	$0.80 \le \lambda_{LT} < 1.5$
$\lambda_T = 0$	*if*	$\lambda_{LT} > 1.5$

This Modified design rule is applied only in case the following conditions are met:

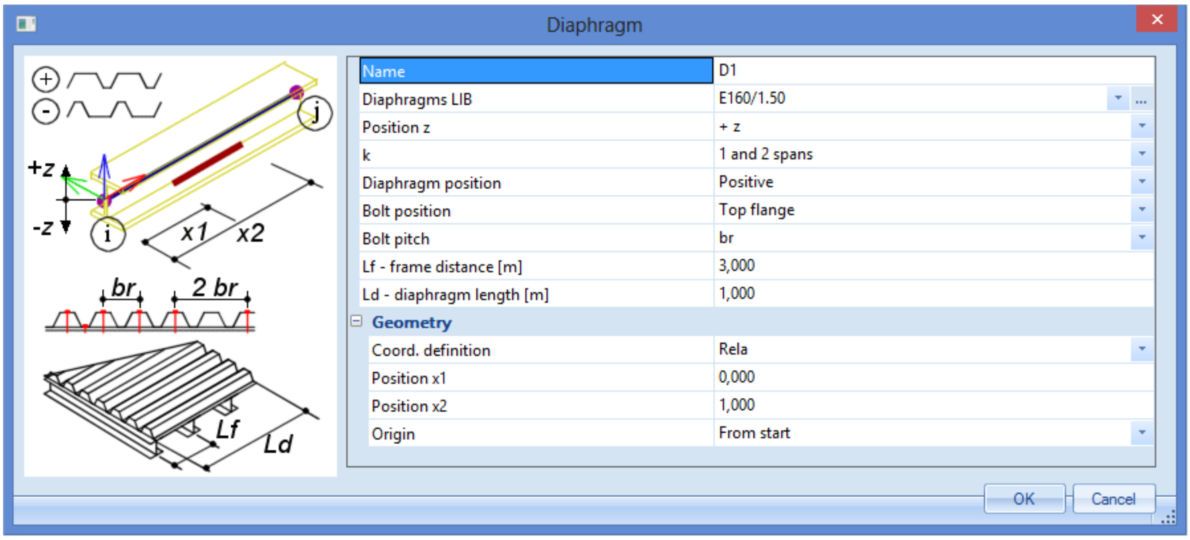
- The section concerns a Channel section (**Form Code 5**)
- The *General Case* is used for LTB (Not the ***Rolled and Equivalent Welded Case***)
- **15 <= Lltb/h <= 40** (with **Lltb** the LTB length and **h** the cross-section height)

Diaphragms

When diaphragms (steel sheeting) are used, the torsional constant It is adapted for symmetric or asymmetric I sections, channel sections, Z sections, cold formed U, C and Z sections.



A diaphragm can be inputted in SCIA Engineer via “Steel -> Beams -> Member Check data -> Diaphragms”:



The settings for the diaphragm are:

Diaphragms LIB	With this option the user can choose between diaphragms of the library of SCIA Engineer or input his own diaphragm.
Position z	**A diaphragm is only taken into account at the compression side of the beam for the LTB check.**
k	The value of coefficient k depends on the number of spans of the diaphragm: k = 2 for 1 or 2 spans, k = 4 for 3 or more spans.
Diaphragm position	The position of the diaphragm may be either positive or negative. Positive means that the diaphragm is assembled in a way so that the width is greater at the top side. Negative means that the diaphragm is assembled in a way so that the width is greater at the bottom side.

Bolt position	Bolts may be located either at the top or bottom side of the diaphragm.
Bold pitch	Bolts may be either: - in every rib (i.e. "br"), - in each second rib (i.e. "2 br").
Frame distance	The distance of frames
Length	The length of the diaphragm (shear field.)

The diaphragm will only have an influence on the torsional constant I_t and will be taken into account in the calculation for M_{cr} for the Lateral Torsional Buckling check.

The torsional constant It is adapted with the stiffness of the diaphragms.

The value for I_t equals the previous value for I_t (thus I_t of the beam) plus a supplementary stiffness calculated with the values of the diaphragm:

$$I_{t,id} = I_t + vorh\ C_\vartheta \frac{l^2}{\pi^2 G}$$

Where:

$$\frac{1}{vorh\ C_\vartheta} = \frac{1}{C_{\vartheta M,k}} + \frac{1}{C_{\vartheta A,k}} + \frac{1}{C_{\vartheta P,k}}$$

$$C_{\vartheta M,k} = k \frac{E\ I_{eff}}{s}$$

$$C_{\vartheta A,k} = C_{100} \left[\frac{b_a}{100}\right]^2 \quad if\ b_a \leq 125$$

$$C_{\vartheta A,k} = 1.25\ C_{100} \left[\frac{b_a}{100}\right] \quad if\ 125 < b_a < 200$$

$$C_{\vartheta P,k} \approx \frac{3 \cdot E \cdot I_s}{(h-t)}$$

$$I_s = \frac{s^3}{12}$$

with	l	the LTB length
	G	the shear modulus
	vorh C_θ	the actual rotational stiffness of diaphragm
	$C_{\theta M,k}$	the rotational stiffness of the diaphragm
	$C_{\theta A,k}$	the rotational stiffness of the connection between the diaphragm and the beam
	$C_{\theta P,k}$	the rotational stiffness due to the distortion of the beam
	k	numerical coefficient = 2 for single or two spans of the diaphragm = 4 for 3 or more spans of the diaphragm
	EI_{eff}	bending stiffness of per unit width of the diaphragm
	s	spacing of the beam
	b_a	the width of the beam flange (in mm)
	C_{100}	rotation coefficient - see table
	h	beam height
	t	thickness beam flange
	s	thickness beam web

Example: Diaphragm.esa

Consider member B1:

- LTB Length = 7 m
- C1 = 1.13, C2= 0.45, C3 = 0.53
- Mcr = 281 kNm

Consider member B2

- It,id = 1840207 mm^4
- LTB Length = 7 m
- C1 = 1.13, C2= 0.45, C3 = 0.53
- Mcr = 405 kNm

The results for the calculation of the diaphragm are shown in the preview of the steel check just before the Section check:

Diaphragm
According to EN 1993-1-1 article BB.2.1 and formula (BB.2)

Parameters		
Diaphragm name	E160/1.50	
Moment of inertia per length I	0,00	m^4/m
Position z	+ z	
Diaphragm position	negative	
Bolt position	top flange	
Bolt pitch	br	
Frame distance Lf	3,000	m
Diaphragm length Ld	3,000	m
Diaphragm factor K1-	0,167	m/kN
Diaphragm factor K2-	15,700	m^2/kN
Numerical coefficient k	4,00	

Stiffness		
Actual stiffness S	5555,21	kN
Required stiffness Serf	42802,77	kN
S < Serf	inadequately braced	
Diaphragm on side	compression side	
Rotational stiffness (Diaphragm) C0M,k	2637,60	kNm/m
Rotational stiffness (Beam distortion) C0P,k	100,15	kNm/m
Rotational stiffness (Connection) C0A,k	23,75	kNm/m
Rotational coefficient C100	10,00	kNm/m
Rotational stiffness vorhC,k	19,06	kNm/m
LTB length L	7,000	m
Cross-section torsional constant It	6,6870e-07	m^4
Additional torsional constant It,add	1,1715e-06	m^4
Adapted torsional constant It,id	1,8402e-06	m^4

The results for the Lateral Torsional Buckling check are:

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters		
Method for LTB curve	General case	
Cross-section plastic modulus Wpl,y	1,7020e-03	m^3
Elastic critical moment Mcr	405,43	kNm
Relative slenderness Lambda,rel,LT	0,99	
Limit slenderness Lambda,rel,LT,0	0,20	
LTB curve	b	
Imperfection Alpha,LT	0,34	
Reduction factor Chi,LT	0,60	
Design buckling resistance Mb,Rd	240,51	kNm
Unity check	**1,02**	-

Mcr parameters		
LTB length L	7,000	m
Influence of load position	no influence	
Correction factor k	1,00	
Correction factor kw	1,00	
LTB moment factor C1	1,13	
LTB moment factor C2	0,45	
LTB moment factor C3	0,53	
Shear center distance d,z	0	mm
Distance of load application z,g	0	mm
Mono-symmetry constant beta,y	0	mm
Mono-symmetry constant z,j	0	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Lateral Torsional Buckling using LTBII – Not in Concept Edition

This option has been inputted in SCIA Engineer as a separate module and it is not included in an edition. The necessary module for this option is **esasd.14**.

For a detailed Lateral Torsional Buckling analysis, a link was made to the Friedrich + Lochner LTBII calculation program.

LTBII is the abbreviation of “Lateral Torsional Buckling with 2nd order calculation”.

The Frilo LTBII solver can be used in 2 separate ways:

1. Calculation of Mcr through eigenvalue solution
2. 2nd Order calculation including torsional and warping effects

For both methods, the member under consideration is sent to the Frilo LTBII solver and the respective results are sent back to SCIA Engineer.

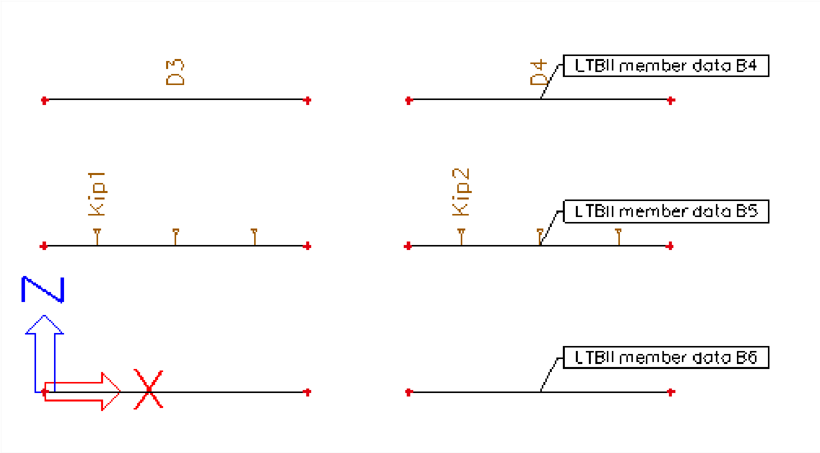
The single element is taken out of the structure and considered as a single beam, with:

- Appropriate end conditions for torsion and warping
- End and begin forces
- Loadings
- Intermediate restraints (diaphragms, LTB restraints)

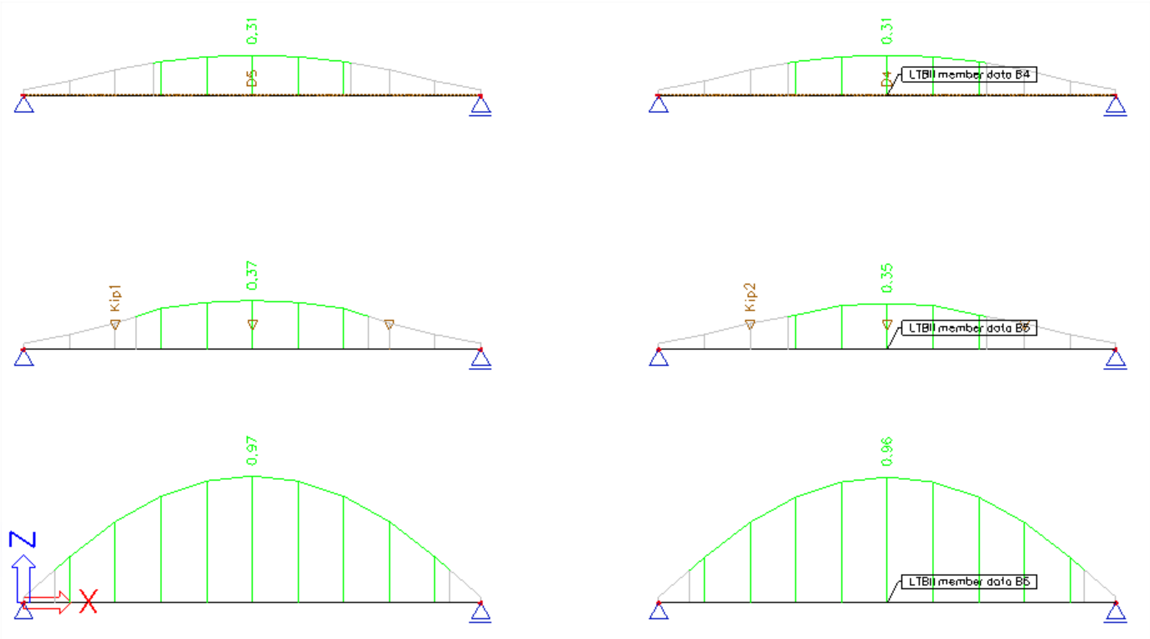
To use this option in SCIA Engineer the functionality “LTB 2nd order” should be activated.
And afterward with the option “Steel -> Beams -> Member check data -> LTB II member data” the user can input LTB-data on a beam.

Example: LTBII.esa

In this example the same beam will be calculated with LTBII data on it and without in three configurations:



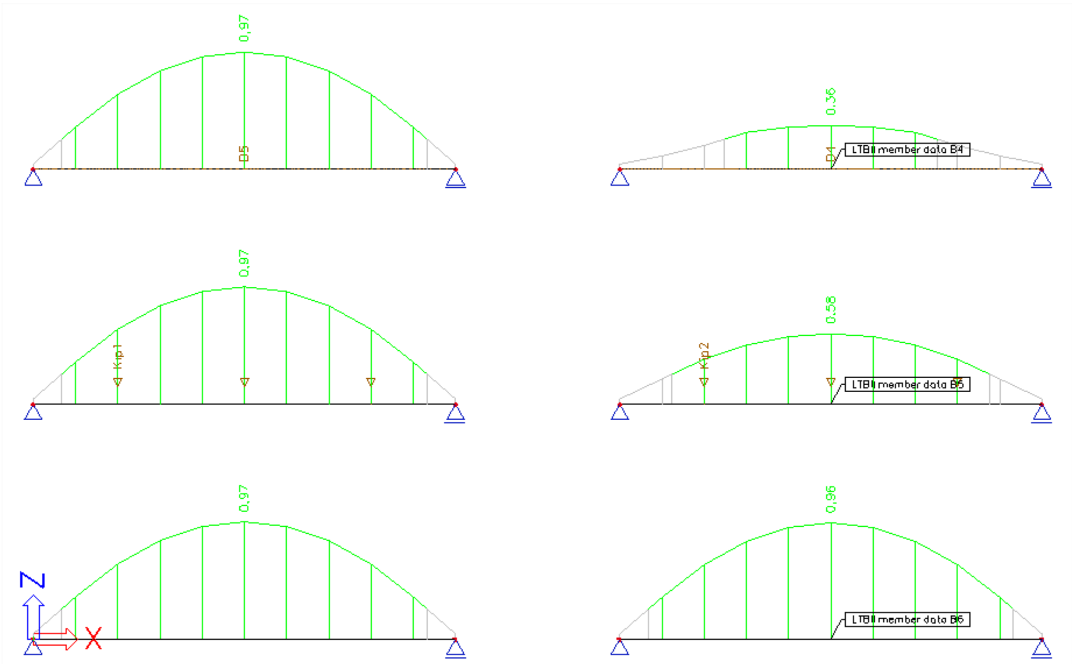
When looking at the check for LC1 there is not a big difference between the beams with or without the LTBII data:



In SCIA Engineer the calculation of a diaphragm is only calculation in one direction.

In Frilo the rigidity of the diaphragm against rotation and translation is taken into account, which results in a higher stiffness of the diaphragm and thus a better unity check.

When looking at the results for Load Case LC2, the direction of the inputted line load is changed. The upper side of the beams is under tension for this load case. SCIA Engineer does not take into account LTB restraints or a diaphragm on the tension side. But Frilo can take this also into account. There will be an augmentation of the rigidity:



Also in the detailed output this calculation is indicated:

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters		
Method for LTB curve	Alternative case	
Cross-section plastic modulus Wpl,y	1,6640e-04	m^3
Elastic critical moment Mcr	27,05	kNm
Relative slenderness Lambda,rel,LT	1,30	
Limit slenderness Lambda,rel,LT,0	0,40	
LTB curve	b	
Imperfection Alpha,LT	0,34	
LTB factor Beta	0,75	
Reduction factor Chi,LT	0,52	
Correction factor kc	0,94	
Correction factor f	0,99	
Modified reduction factor Chi,LT,mod	0,53	
Design buckling resistance Mb,Rd	24,31	kNm
Unity check	0,58	-

Note: The elastic critical moment Mcr is calculated using the FriLo BTII Solver.

Mcr parameters		
LTB length L	7,200	m

Compression and bending check

The compression and bending check for a member will be executed following EN 1993-1-1 **art. 6.3.2**.

Unless second order analysis is carried out, members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

N_{Ed}, $M_{y,Ed}$, $M_{z,Ed}$ are the design values of the compression force and the **maximum** moments about the y-y and z-z axis along the member respectively.

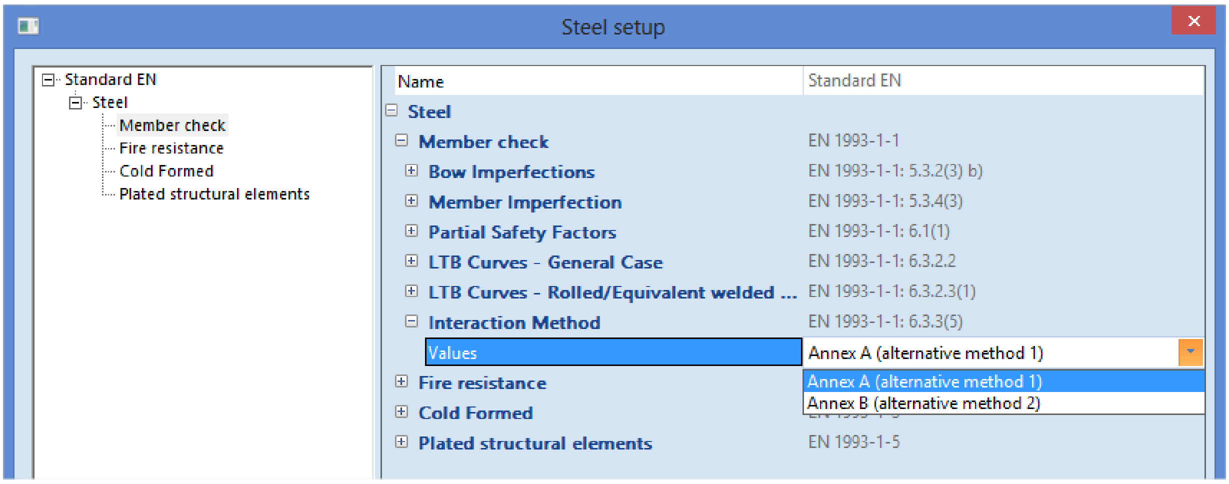
χ_y and χ_z are the reduction factors due to flexural buckling

χ_{LT} is the reduction factor due to lateral torsional buckling

Class	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{eff,z}$
$\Delta M_{y,Ed}$	0	0	0	$e_{N,y}\,N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z}\,N_{Ed}$

The interaction factors **k_{yy}, k_{yz}, k_{zy} and k_{zz}** have been derived from EN 1993-1-1 Annex A (alternative method1) or from Annex B (alternative method 2).

The choice between Interaction Method 1 or 2 can be made in SCIA Engineer in the National Annex parameters:



Example: Industrial Hall.esa

Consider column B28 (for combination CO1-ULS).

In SCIA Engineer first all calculated formulas, in this example following EN 1993 Annex 1, are given:

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters		
Interaction method	alternative method 1	
Cross-section effective area Aeff	1,5269e+04	mm^2
Cross-section effective modulus Weff,y	4,1630e+06	mm^3
Cross-section effective modulus Weff,z	3,9265e+05	mm^3
Design compression force N,Ed	159,86	kN
Design bending moment (maximum) My,Ed	-697,64	kNm
Design bending moment (maximum) Mz,Ed	-0,24	kNm
Additional moment Delta My,Ed	0,00	kNm
Additional moment Delta Mz,Ed	0,00	kNm
Characteristic compression resistance N,Rk	3588,20	kN
Characteristic moment resistance My,Rk	978,31	kNm
Characteristic moment resistance Mz,Rk	92,27	kNm
Reduction factor Chi,y	0,83	
Reduction factor Chi,z	0,45	
Reduction factor Chi,LT	0,75	
Interaction factor k,yy	0,98	
Interaction factor k,yz	0,84	
Interaction factor k,zy	0,94	
Interaction factor k,zz	0,81	

Maximum moment My,Ed is derived from beam B28 position 6,900 m.
Maximum moment Mz,Ed is derived from beam B28 position 6,900 m.

Interaction method 1 parameters		
Critical Euler load N,cr,y	6596,08	kN
Critical Euler load N,cr,z	2293,84	kN
Elastic critical load N,cr,T	4395,48	kN
Cross-section effective modulus Weff,y	4,1630e+06	mm^3
Second moment of area Iy	1,5990e+09	mm^4
Second moment of area Iz	5,1660e+07	mm^4
Torsional constant It	1,3710e+06	mm^4
Method for equivalent moment factor C,my,0	Table A.2 Line 1 (Linear)	
Ratio of end moments Psi,y	0,00	
Equivalent moment factor C,my,0	0,79	
Method for equivalent moment factor C,mz,0	Table A.2 Line 1 (Linear)	
Ratio of end moments Psi,z	0,00	
Equivalent moment factor C,mz,0	0,78	
Factor mu,y	1,00	
Factor mu,z	0,96	
Factor epsilon,y	16,01	
Factor a,LT	1,00	
Critical moment for uniform bending Mcr,0	965,61	kNm
Relative slenderness Lambda,rel,0	1,01	
Limit relative slenderness Lambda,rel,0,lim	0,26	
Equivalent moment factor C,my	0,96	
Equivalent moment factor C,mz	0,78	
Equivalent moment factor C,mLT	1,00	

And afterwards the check is given as two times a sum of three values as in formulas (6.61) and (6.62) of the EN 1993-1-1:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

In SCIA Engineer:

Unity check (6.61) = 0,05 + 0,93 + 0,00 = 0,98 -
Unity check (6.62) = 0,10 + 0,90 + 0,00 = 1,00 -

Shear buckling check – EN 1993-1-5

General

The shear buckling check will check if the web of the cross section can locally buckle.
This check is not included in EN 1993-1-1, but in EN 1993-1-5: Design of steel structures – Part 1-5: Plated structural elements.

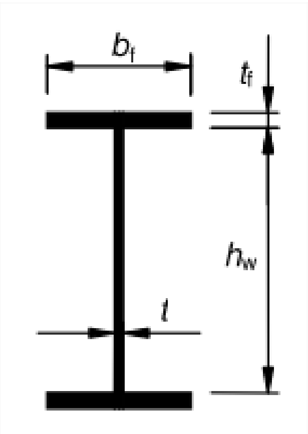
First it will be checked if the slenderness is lesser than a certain value, because for webs with a small slenderness, this check should not be executed.

Plates with h_w/t greater than $72\varepsilon/\eta$ for an unstiffened web should be checked for resistance to shear.

With:

$$\epsilon = \sqrt{\frac{235}{fy[N/mm^2]}}$$

η will be defined in the National Annex. The value $\eta = 1.20$ is recommended for steel grades up to and including S460. For higher steel grades $\eta = 1.00$ is recommended.



If the slenderness is higher than the minimum one, the shear buckling check is executed following EN 1993-1-5, formula (5.10) (this is a check on the shear force) and formula (7.1) (which is the check on the interaction between shear force, bending moment and axial force). Both formulas are checked in SCIA Engineer.

Example: Industrial Hall.esa

Consider Beam B50

This is an IPE 160 profile: $h_w = 160 – 2 \times 7.40 = 145.2$ mm. And t = 5 mm

⇨ $h_w / t = 29.04$

This should be checked with the value: $72\varepsilon/\eta = 72 \times 1.00 / 1.2 = 60$

⇨ $h_w / t < 72\varepsilon/\eta$

⇨ The shear buckling check does not have to be executed.

This is also indicated in SCIA Engineer:

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters		
Buckling field length a	6,000	m
Web	unstiffened	
Web height hw	145,20	mm
Web thickness t	5,00	mm
Material coefficient epsilon	1,00	
Shear correction factor Eta	1,20	

Shear Buckling verification	
Web slenderness hw/t	29,04
Web slenderness limit	60,00

Note: The web slenderness is such that Shear Buckling effects may be ignored according to EN 1993-1-5 article 5.1(2).

Consider Beam B28 (for combination CO1-ULS):

This is an IPE750x137 profile: h_w = 753 – 2 x 17 = 719 mm. And t = 11.5 mm

⇨ h_w / t = 62.52

This should be checked with the value: $72\varepsilon/\eta$ = 72 x 1.00 / 1.2 = 60

⇨ h_w / t > $72\varepsilon/\eta$

⇨ The shear buckling check has to be executed.

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

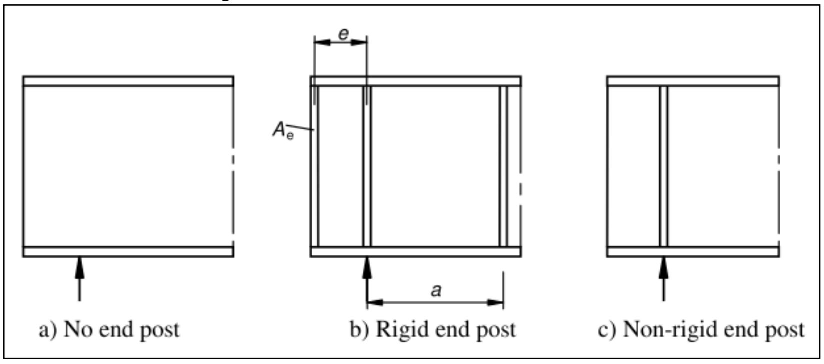
Shear Buckling parameters		
Buckling field length a	6,900	m
Web	unstiffened	
End post	non-rigid	
Web height hw	719,00	mm
Web thickness t	11,50	mm
Yield strength fyw	235,0	MPa
Flange width bf	263,00	mm
Flange thickness tf	17,00	mm
Yield strength fyf	235,0	MPa
Material coefficient epsilon	1,00	
Shear correction factor Eta	1,20	

Shear Buckling verification		
Web slenderness hw/t	62,52	
Web slenderness limit	60,00	
Plate slenderness lambda,w	0,72	
Reduction factor chi,w	1,15	
Contribution of the web Vbw,Rd	1286,75	kN
Capacity of the flange Mf,Rd	714,47	kNm
Flange factor c	1,866	m
Contribution of the flange Vbf,Rd	9,32	kN
Maximum resistance Vb,Rd,limit	1346,22	kN
Resistance Vb,Rd	1296,07	kN
Plastic resistance Mpl,Rd	1143,28	kNm
Shear ratio eta,3,bar	0,08	

Unity check (5.10) = 0,08 -
Note: The interaction between Bending and Shear Buckling does not need to be verified because the shear ratio does not exceed 0.5.

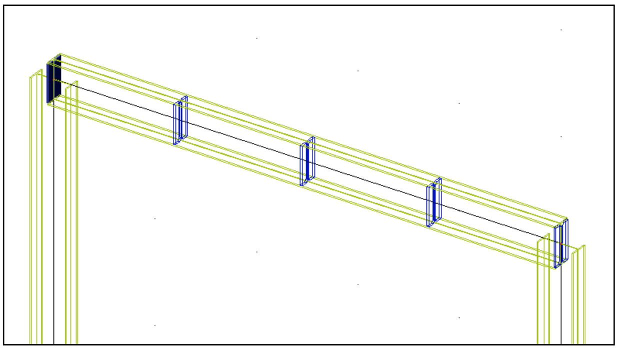
Stiffners

In EN 1993-1-5, Figure 5.1 the definitions of stiffeners on a member are given:



Those stiffeners will influence the total length for shear buckling. Also in SCIA Engineer those stiffeners can be inputted on a beam and will only have influence on the shear buckling check.

Those stiffeners can be inputted via “Steel -> Beams -> Member check data -> Stiffeners”.

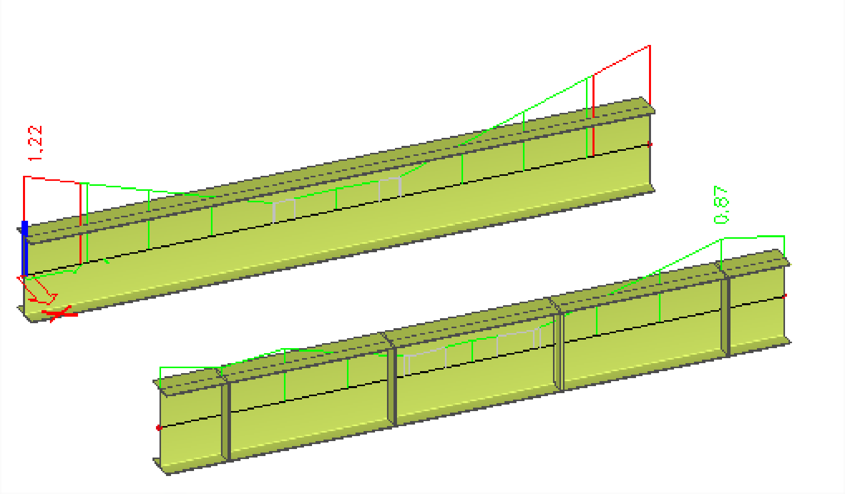


The stiffeners define the field dimensions (a,d) which are only relevant for the shear buckling check. When no stiffeners are defined, the value for ‘a’ is taken equal to the member length.

Example: Stiffeners.esa

In this example 2 identical beams are inputted. B1 without stiffeners and B2 with stiffeners:

Let’s now compare the shear buckling check with and without stiffeners:



Without stiffeners

Shear buckling check:

Shear Buckling parameters		
Buckling field length a	6,000	m
Web	unstiffened	
End post	non-rigid	
Web height hw	782,00	mm
Web thickness t	5,00	mm
Yield strength fyw	235,0	MPa
Flange width bf	275,00	mm
Flange thickness tf	9,00	mm
Yield strength fyf	235,0	MPa
Material coefficient epsilon	1,00	
Shear correction factor Eta	1,20	

Shear Buckling verification		
Web slenderness hw/t	156,40	
Web slenderness limit	60,00	
Plate slenderness lambda,w	1,81	
Reduction factor chi,w	0,46	
Contribution of the web Vbw,Rd	243,24	kN
Capacity of the flange Mf,Rd	460,07	kNm
Flange factor c	1,570	m
Contribution of the flange Vbf,Rd	1,92	kN
Maximum resistance Vb,Rd,limit	636,60	kN
Resistance Vb,Rd	245,16	kN
Plastic resistance Mpl,Rd	693,57	kNm
Shear ratio eta,3,bar	1,23	
Moment resistance MR,eff	569,96	kNm
Moment ratio eta,1,bar	0,43	
Moment ratio limit eta,1,bar,limit	0,66	

Unity check (5.10) = **1,22** -

With stiffeners

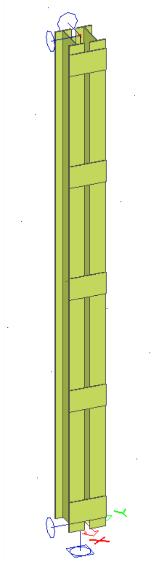
Shear buckling check:

Shear Buckling parameters		
Buckling field length a	1,600	m
Web	stiffened	
End post	non-rigid	
Web height hw	782,00	mm
Web thickness t	5,00	mm
Yield strength fyw	235,0	MPa
Flange width bf	275,00	mm
Flange thickness tf	9,00	mm
Yield strength fyf	235,0	MPa
Material coefficient epsilon	1,00	
Shear correction factor Eta	1,20	

Shear Buckling verification		
Shear buckling coefficient k,tau	6,30	
Web slenderness hw/t	156,40	
Web slenderness limit	64,82	
Plate slenderness lambda,w	1,67	
Reduction factor chi,w	0,50	
Contribution of the web Vbw,Rd	264,19	kN
Capacity of the flange Mf,Rd	460,07	kNm
Flange factor c	0,419	m
Contribution of the flange Vbf,Rd	11,38	kN
Maximum resistance Vb,Rd,limit	636,60	kN
Resistance Vb,Rd	275,57	kN
Plastic resistance Mpl,Rd	693,57	kNm
Shear ratio eta,3,bar	0,91	
Moment resistance MR,eff	569,96	kNm
Moment ratio eta,1,bar	0,20	
Moment ratio limit eta,1,bar,limit	0,66	

Unity check (5.10) = 0,87 -

ULS Check for Battened compression members



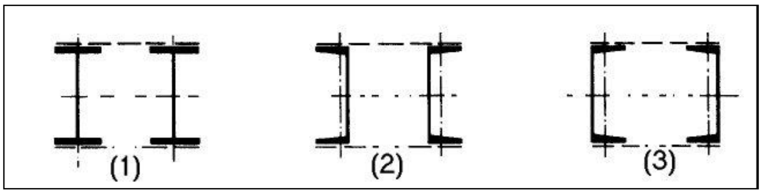
This check can be executed following the EC-ENV and EC-EN 11993-1-1 art.6.4

The following section pairs are supported as battened compression member :

(1) 2I

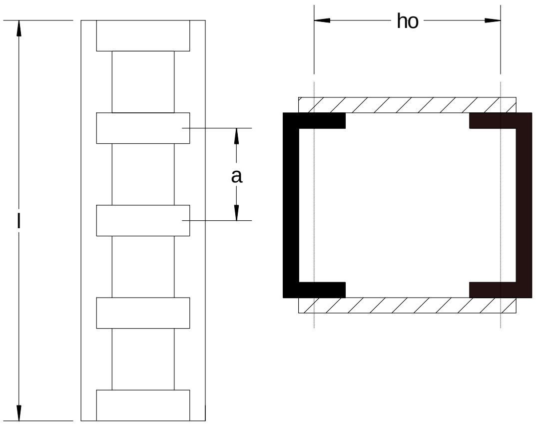
(2) 2Uo

(3) 2Uc



Two links (battens) are used.

The following additional checks are performed:
- buckling resistance check around weak axis of single chord with $N_{f,Sd}$
- section check of single chord, using internal forces
- section check of single batten, using the internal forces



Example – manual calculation of a battened compression member

EC_EN_Battened_Compression_Members_I.esa

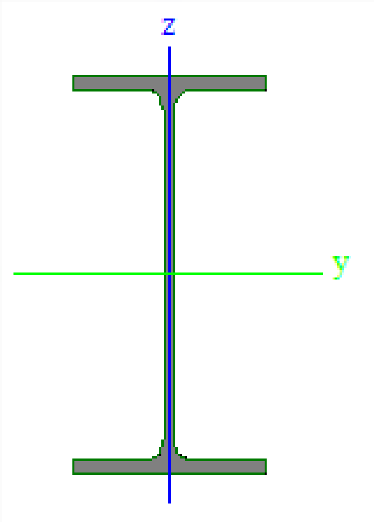
Consider member B1

On this beam, only a compression force of 500kN is applied

Properties of the batten:

Division	4
Distance from begin x [m]	0,200
Distance from end x´ [m]	0,200
Width of link w [m]	0,220
Thickness t [mm]	7,00
On begin	☑
On end	☑

Properties of the IPE 330 profile



A [mm^2]	6,2600e+03
Ay [mm^2]	3,2283e+03
Az [mm^2]	2,3645e+03
AL [m^2/m]	1,2540e+00
AD [m^2/m]	1,2540e+00
cYUCS [mm]	80,00
cZUCS [mm]	165,00
α [deg]	0,00
Iy [mm^4]	1,1770e+08
Iz [mm^4]	7,8810e+06
iy [mm]	137,12
iz [mm]	35,48
Wely [mm^3]	7,1310e+05
Welz [mm^3]	9,8520e+04
Wply [mm^3]	8,0430e+05
Wplz [mm^3]	1,5370e+05
Mply+ [Nmm]	0,00
Mply- [Nmm]	1000,00
Mplz+ [Nmm]	0,00
Mplz- [Nmm]	0,00
dy [mm]	0,00
dz [mm]	0,00
It [mm^4]	2,8150e+05
Iw [mm^6]	1,9910e+11
β y [mm]	0,00
β z [mm]	0,00

Values in SCIA Engineer:

Check for battened compression member

Table of values		
l	4600.00	mm
a	1150.00	mm
h0	210.00	mm
Ich	7.8810e+06	mm^4
Lambda	41.50	
Mu	1.00	
Ach	6.2600e+03	mm^2
Ieff	1.5380e+08	mm^4
Sv	24384.44	kN
e0	9.20	mm
MEd,I	0.00	kNm
MEd	4.86	kNm
VEd	3.32	kN

Manual calculation of those values:

l length = Member length – (distance from begin) – (distance from end)
= 5000mm – 200mm – 200mm
= 4600mm

a distance between battens = l / (number of divisions) = 4600mm / 4 = 1150 mm

h0 distance between centroids of chords = 210mm

Ich Iz of the IPE330 profile

Ach area if one I-profile, so A of IPE 330

Lambda = l / i0 = 4600mm / 110,833 mm = 41.5

with

$$l_1 = 0.5 \cdot h0^2 \cdot Ach + 2 \cdot Ich$$

$$= 0.5 \cdot (210mm)^2 \cdot (6260mm^2) + 2 \cdot (7881000mm^4) = 1.538\ x\ 10^8 mm^4$$

$$i0 = \sqrt{\frac{l_1}{2 \cdot Ach}} = \sqrt{\frac{1.538\ x\ 10^8 mm^4}{2 \cdot 6260mm^2}} = 110{,}833mm$$

Mu μ:

= 0 if Lambda $\geq$ 150

= $2 - \frac{Lambda}{75}$ if 75 < Lambda < 150

= 1.0 if Lambda $\leq$ 75

And Lambda = 41.5 < 75 in this example => $\mu = 1.00$

Ieff $0.5 \cdot h0^2 \cdot Ach + 2 \cdot \mu \cdot Ich$

$$= 0.5 \cdot (210mm)^2 \cdot (6260mm^2) + 2 \cdot (1.00) \cdot (7881000mm^4) = 1.538\ x\ 10^8 mm^4$$

Sv $= \frac{24 \cdot E \cdot Ich}{a^2 \cdot \left(1 + \frac{2 \cdot Ich}{n \cdot Ib} \cdot \frac{h0}{a}\right)}\ but\ Sv\ \leq\ \frac{2 \cdot \pi^2 \cdot E \cdot Ich}{a^2}$

With:

n = number of planes of battens: n=2

Ib = (tickness batten) x (width of batten)³ / 12 = (7 mm) x (220 mm)³ / 12 = 6,211 x 10^6 mm^4

$$\frac{24 \cdot E \cdot Ich}{a^2 \cdot \left(1 + \frac{2 \cdot Ich}{n \cdot Ib} \cdot \frac{h0}{a}\right)} = \frac{24 \cdot \left(\frac{210000N}{mm^2}\right) \cdot (7881000mm^4)}{(1150mm)^2 \cdot \left(1 + \frac{2 \cdot (7881000mm^4)}{2 \cdot (6{,}211\ x\ 10^6\ mm^4)} \cdot \frac{210mm}{1150mm}\right)} = 2.438\ x\ 10^4 kN$$

And

$$\frac{2 \cdot \pi^2 \cdot E \cdot Ich}{a^2} = \frac{2 \cdot \pi^2 \cdot \left(\frac{210000N}{mm^2}\right) \cdot (7881000mm^4)}{(1150)^2} = 2.470\ x\ 10^4 kN$$

⇨ $Sv = 2.438\ x\ 10^4 kN$

e0 = l/500 = 4600mm/500 = 9.2 mm

MEd,I Mz in the cross section under consideration = 0 kNm

MEd will be calculated as follows:

If $\left(1 - \frac{NEd}{Ncr} - \frac{NEd}{Sv}\right) > 0$ $=> MEd = \frac{NEd \cdot e0 + MEd\_I}{1 - \frac{NEd}{Ncr} - \frac{NEd}{Sv}}$

Otherwise: = > *Med* = 1 x 10^6 kNm

With:

$$N_{cr} = \frac{\pi^2 \cdot E \cdot Ieff}{L^2} = \frac{\pi^2 \cdot \left(\frac{210000N}{mm^2}\right) \cdot (1.538\ x\ 10^8 mm^4)}{(4600mm)^2} = 15064\ kN$$

$$1 - \frac{500kN}{15064kN} - \frac{500kN}{24700\ kN} = 0{,}9466 > 0$$

⇨ $MEd = \frac{NEd \cdot e0 + MEd\_I}{1 - \frac{NEd}{Ncr} - \frac{NEd}{Sv}} = \frac{500kN \cdot 9.2mm + 0kNm}{0.9466} = 4860kNmm = 4{,}860kNm$

VEd $= \frac{\pi \cdot MEd}{l} = \frac{\pi \cdot 4860kNmm}{4600mm} 3{,}32kN$

Check of chord as beam in field between battens
According to article EN 1993-1-1: 6.4.3.1 & 6.2.9.1 and formula (6.42)

Table of values		
NG	270.78	kN
VG	1.66	kN
MG	0.95	kNm
Unity check	0.19	-

The check of the chord will be executed with a section check following the EN 1993-1-1 for the chord profile with the following internal forces on one batten:

NG = NChord = $0.5 \cdot NEd + \frac{Med \cdot h0 \cdot Ach}{2 \cdot leff} = 0.5 \cdot 500kN + \frac{(4860kNmm)\cdot(210mm)\cdot(6260mm^2)}{2 \cdot 1.538\, x\, 10^8 mm^4} = 270{,}78\, kN$

VG = VEd /2 = 3,32kN / 2 = 1,66kN

MG = $\frac{Ved \cdot a}{4} = \frac{3{,}32kN \cdot 1150mm}{4} = 0.95kNm$

For an I-section a classification is made, for an U-section always an elastic check is done.

This section check will result in a unity check of **0,19**

Buckling check of chord

Buckling check of chord
According to article EN 1993-1-1: 6.4.3.1 & 6.3.1.1 and formula (6.46)

Table of values		
Nch,Ed	270.78	kN
Buckling length	1150.00	mm
Slenderness	32.41	
Relative slenderness Lambda	0.37	
Buckling curve	b	
Imperfection Alpha	0.34	
Reduction factor Chi	0.94	
Unity check	0.17	-

Nch,Ed = NChord = 170,78 kN (see check of Check of chord as beam in field between battens)

Buckling length = a

Slenderness = $\frac{Buckling\ length}{\sqrt{\frac{Ich}{Ach}}} = \frac{1150mm}{\sqrt{\frac{7881000mm^4}{6260mm^2}}} = 32{,}411$

Relative slenderness Lambda $\sqrt{\frac{Ach \cdot fy}{Ncr_z}} = \sqrt{\frac{6260mm^2 \cdot \frac{275N}{mm^2}}{12351000\, N}} = 0.373$

With:

$$Ncr_z = \frac{\pi^2 \cdot E \cdot Ich}{(Buckling\ length)^2} = \frac{\pi^2 \cdot \left(\frac{210000N}{mm^2}\right) \cdot (7881000mm^4)}{(1150mm)^2} = 12351\, kN$$

Buckling curve = b, this is the buckling curve for the IPE 330 around the z-axis

Imperfection alpha for buckling curve b = 0.34

Reduction factor Chi can be calculated as explained in the buckling check.

Unity check = $\frac{NchEd}{NbRd} = \frac{270{,}78kN}{1618{,}2\, kN} = 0.17$

With:

$$Nb,Rd = reduction\ factor\ Chi\ \frac{Ach \cdot fy}{\gamma_{M1}} = 0.94\ \frac{6260mm^2 \cdot 275N/mm^2}{1.00} = 1618{,}21\, kN$$

Check of batten:

Check of batten
According to article EN 1993-1-1: 6.4.3.1, 6.2.9.2 & 6.2.6 and formula (6.42), (6.19)

Table of values		
t	7.00	mm
b	220.00	mm
T	9.09	kN
MG	0.95	kNm
Sigma	16.9	MPa
Unity check	0.06	-
Tau	5.9	MPa
Unity check	0.04	-

t thickness of batten

b width of batten

T $= \frac{VEd \cdot a}{h0 \cdot 2} = \frac{3{,}32kN \cdot 1150mm}{210mm \cdot 2} = 9.09kN$

MG = 0.954 kNm (see previous check)

Sigma $\frac{MG \cdot b/2}{Ib} = \frac{954000Nmm \cdot 220mm/2}{6.211 \cdot 10^6 mm^4} = 16{,}9MPa$

(calculation of Ib see calculation of Sv at the properties)

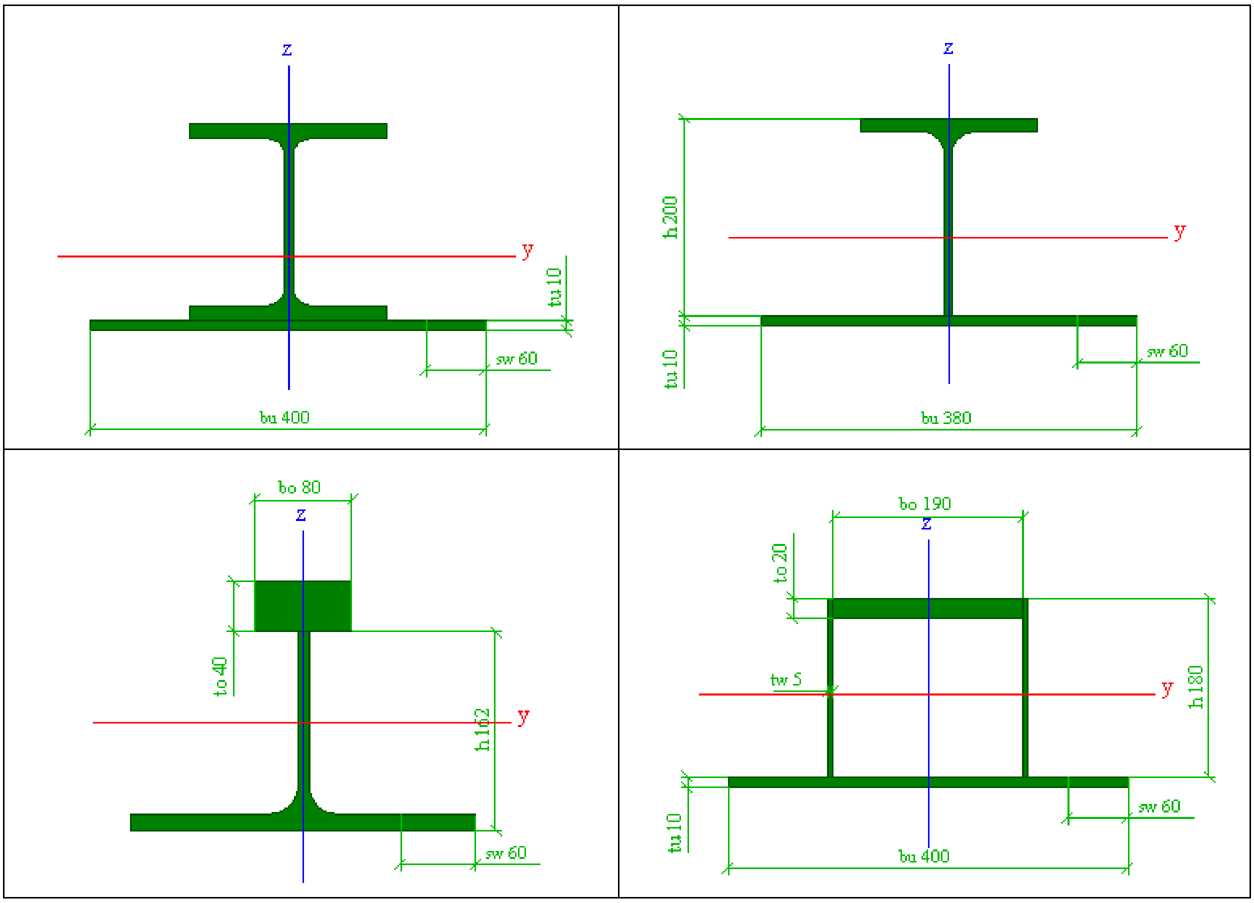
Unity check sigma: $= \frac{Sigma}{\frac{fy}{\gamma_{M0}}} = \frac{16{,}9MPa}{\frac{275N/mm^2}{1.00}} = 0.06$

Tau $= \frac{T}{b \cdot t} = \frac{9090N}{220mm \cdot 7mm} = 5.9MPa$

Unity check Tau: $= \frac{Tau}{\frac{fy/\sqrt{3}}{\gamma_{M0}}} = \frac{5{,}9MPa}{\frac{275N/mm^2/\sqrt{3}}{1.00}} = 0.04$

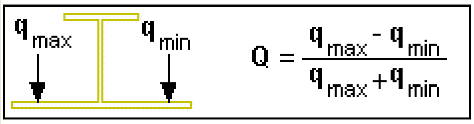
ULS Check for Built-in beams

The built-in beams are of the types SFB (Slim Floor Beam), IFB (Integrated Floor Beam) or THQ (Top Hat Beams).
The relevant section classification according to the proper code is performed. Classification for class 1, 2, 3 and 4 is supported. For the ULS section check, the local plate bending is taken into account for the plastic moment capacity and the bending stresses in the section. Out-of-balance loading is checked.



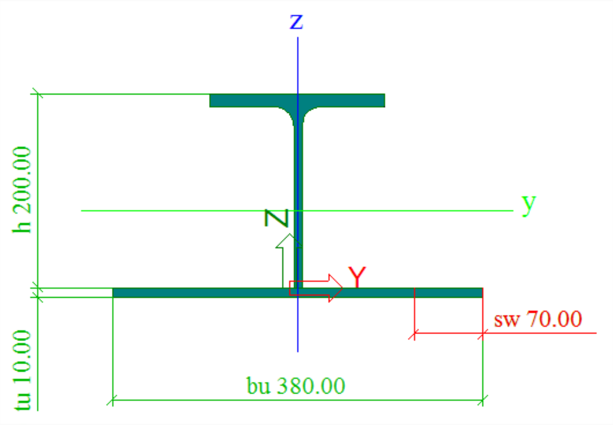
This out-of-balance check has been inputted by a factor “Q”.

When inputting a line load on such a section, the option “Bottom flange” can be checked and afterwards a Q factor (an out-of-balance factor) can be inputted in the properties window of this line load. This Q factor indicates the distribution of the force:



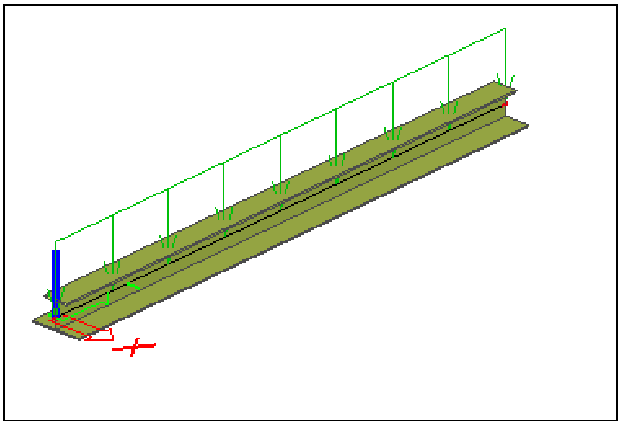
If Q = 0, both sides are loaded equally, Q=1: one side is loaded the other side isn’t.

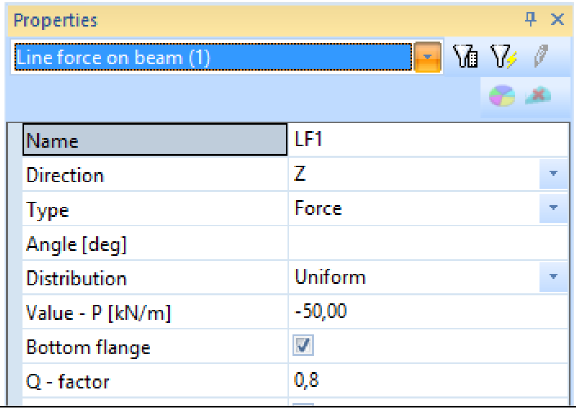
The position of the load is indicated in the cross section library by the value s_w:



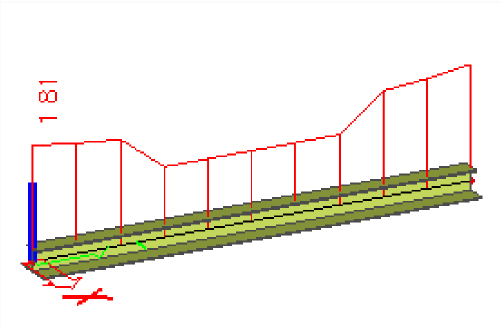
Example: Built-in beams.esa

In this example a beam with an IFB profile has been inputted with a line load of 100kN/m and an out-of-balance factor Q=0.8:





In the steel code check, the out-of-balance check is given:



Section check inclusive local q loading
according to ECCS No.83 "Design Guide for Slim Floors with Built-in Beams"

Table of values		
q max	45.00	kN/m
q min	5.00	kN/m
e1	286.67	mm
e2	0.00	mm
bu	380.00	mm
tu	10.00	mm
bo	180.00	mm
to	13.50	mm
hf	198.25	mm
bf	0.00	mm

Normal stress check according to formula (5.15)

Table of values		
Sigma x	-70.25	MPa
Sigma y	387.00	MPa
unity check	1.81	

Shear stress check

Table of values		
Sigma x	-75.33	MPa
Tau	6.75	MPa
unity check	0.32	

Torsion check according to formula (4.15)

Table of values		
e	143.33	mm
Q	40.00	kN/m
L	4.00	m
Lk	2.91	m
Io	6561000.00	mm^4
Mt,max	4.13	kNm
Mw,max	7.34	kNm
Tau t max	176.05	MPa
Tau w max	22.86	MPa
unity check	1.47	

Optimisation

In SCIA Engineer there are two ways of performing an optimization:

1. Cross section optimisation: Optimisation of a chosen cross section.
2. Overall optimisation: Optimisation of one or more (or all) cross sections at the same time.

Both options are explained in the next two paragraphs.

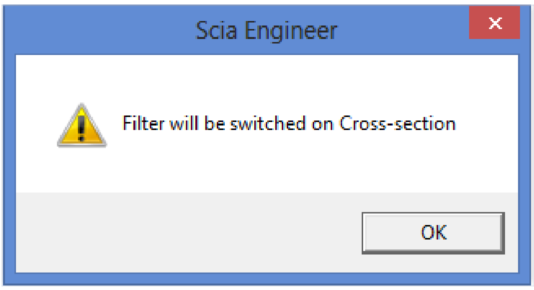
Cross section optimisation

With this option the user can optimise a cross section for the steel code check. This will be explained in the example below.

Example: Optimisation.esa

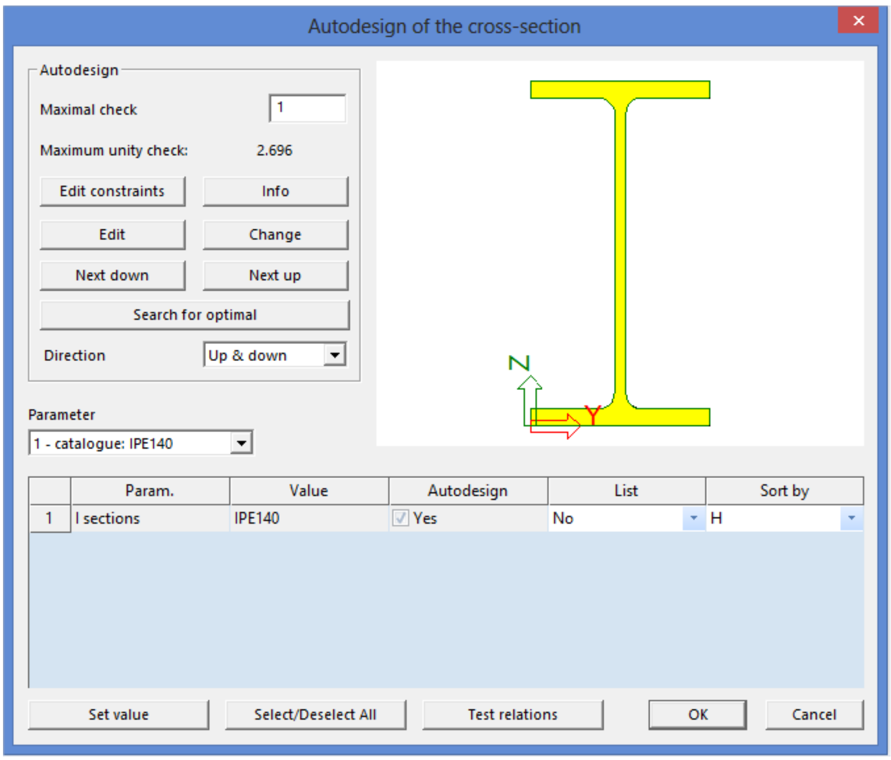
The optimisation can be found via “Steel -> Beams -> ULS Checks -> Check”. First the steel code check has to be calculated, so first click on “Refresh”.

When clicking now on “Autodesign”, SCIA Engineer will give the following message:



And in the properties window the option for “Filter” has been changed into “Cross-section”. Now choose here for the profile IPE 140 and click again on “Refresh” and afterwards on “Autodesign”.

Now the “Autodesign of the cross-section” window will open:



First you can fill in the maximal check. Normally this is a unity check of 1.00.
The Maximum unity check displayed below is the maximum unity check for the IPE140, found in this project.

When clicking on “Search for optimal”, SCIA Engineer will suggest the smallest IPE profile which will resist the calculated internal forces.

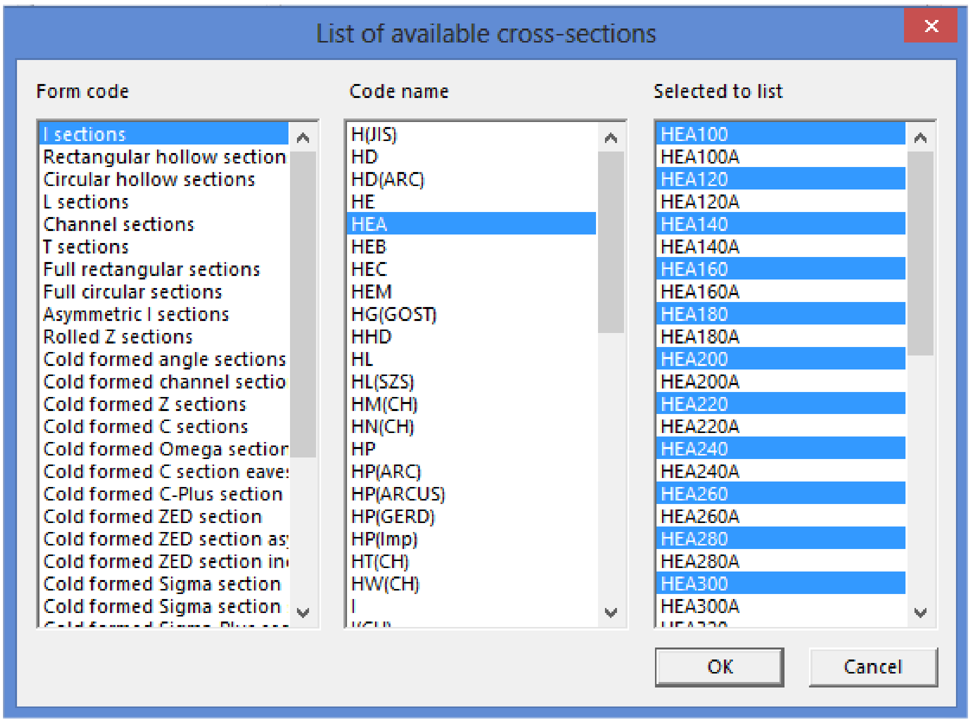
In this example an IPE 200 will be found. When clicking on “OK” now, SCIA Engineer will replace this IPE140 profile by an IPE200 profile automatically.

Now the steel code check of this IPE200 profile is executed with the internal forces calculated with the properties of the IPE140 profile. For example the self weight will not be taken into account correctly. So the project should be recalculated, before accepting those new unity check results.

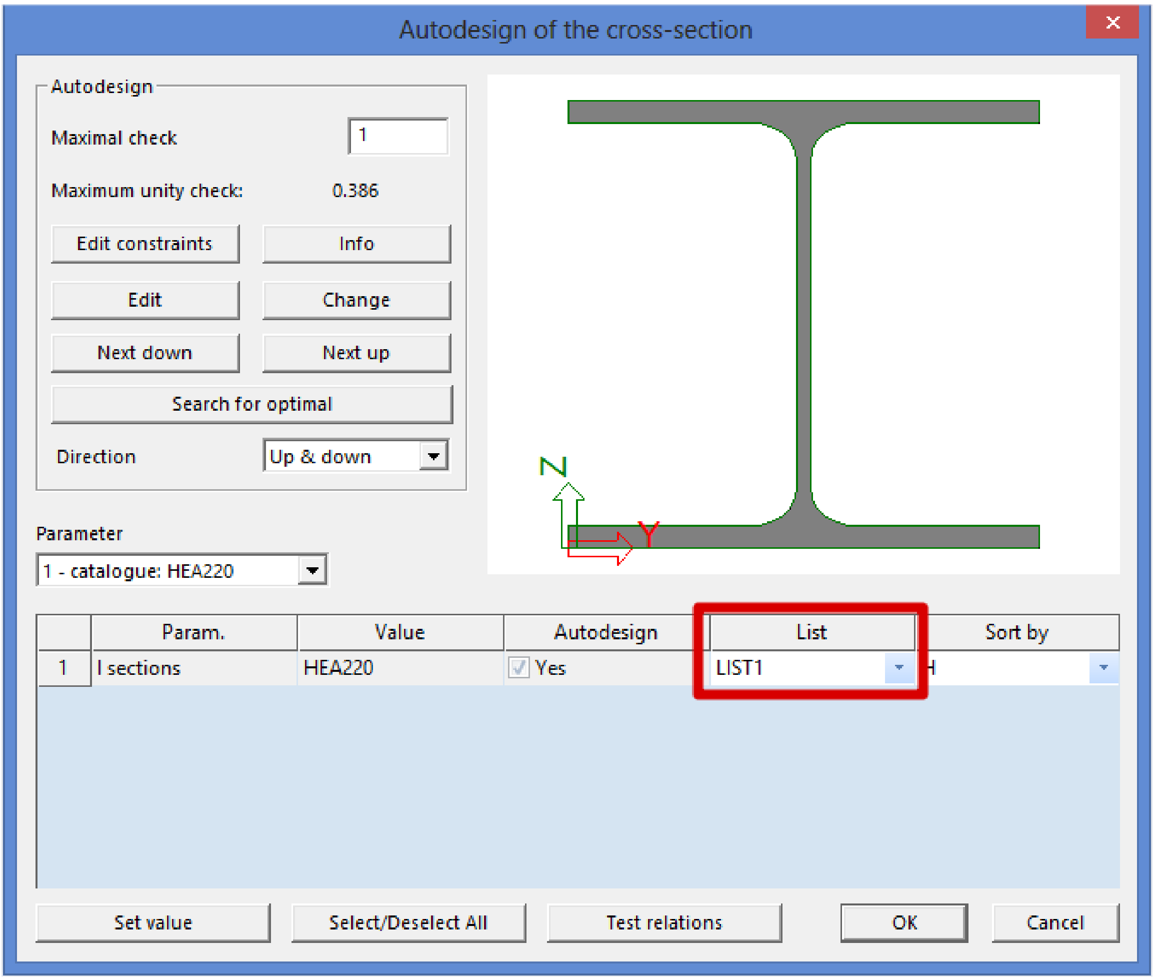
It is also possible to use a **list of sections**.
With this list it is possible to indicate which sections can be used or not. For example the section IPE140A can be filtered out the possibilities for the autodesign.

Recalculate the project and let’s make a list for the columns. This can be done through “Library -> Structure, analysis, Cross-section list” and choose for “Library cross-section of one type”.

Now select the profiles that should be added to this list:



When performing an Autodesign in the steel menu now for the columns HEA220, this list can be chosen:



For the moment this profile has a maximum unity check of 0.386, so SCIA Engineer will search for the smallest profile from this "LIST1" which will pass the unity check

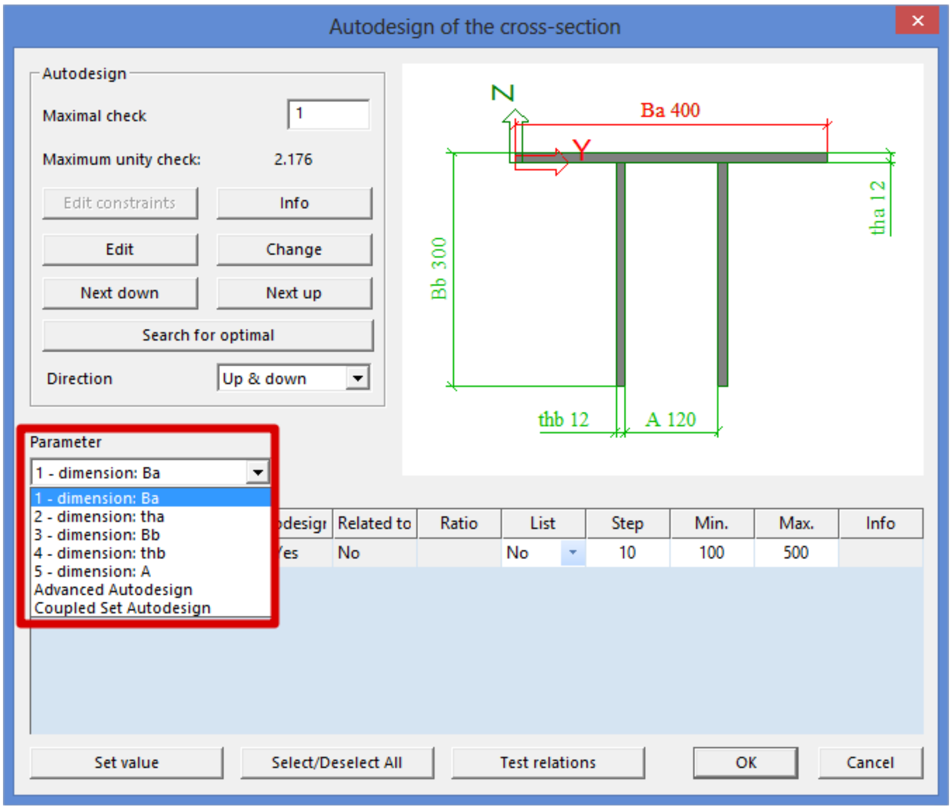
Close this example without saving! It will be used in the next paragraph again!

In the next example the different options of the cross-section optimisation are explained.

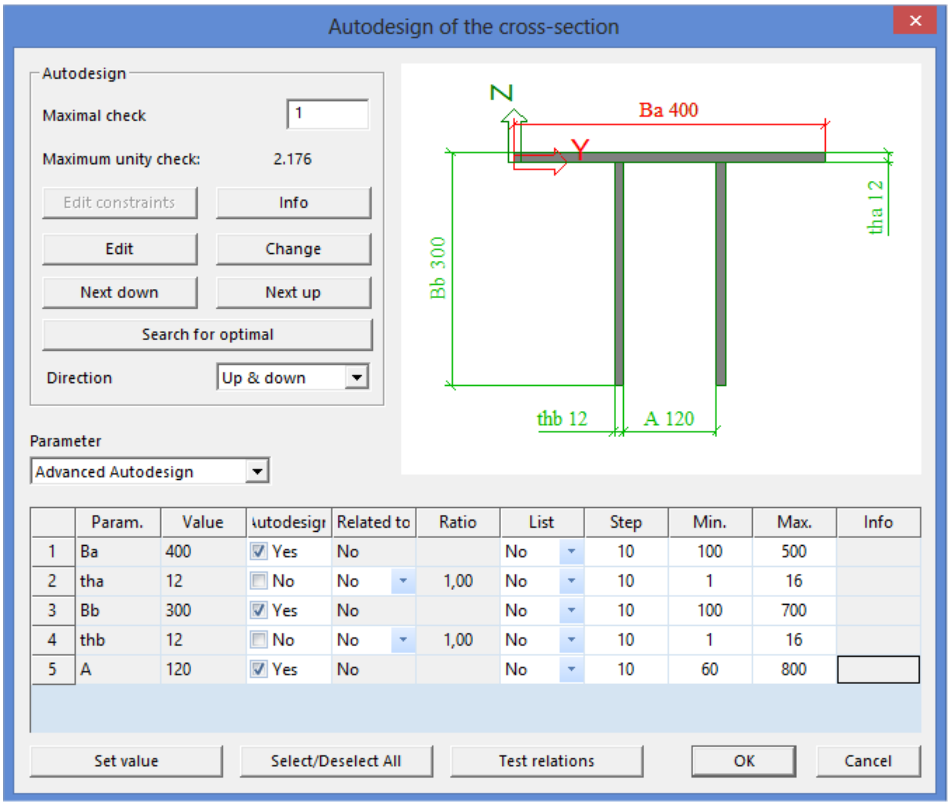
Example: Optimisation2.esa

Go here to "Steel -> Beams -> ULS checks -> Check" and go to the Autodesign menu for this beam.

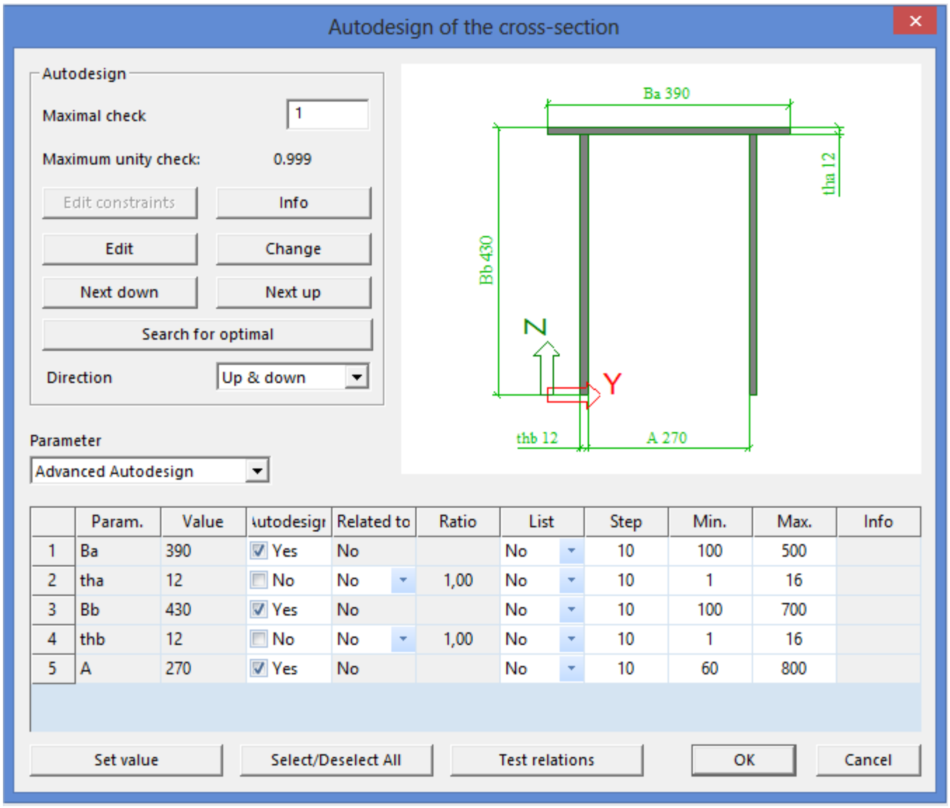
This profile has a lot of parameters, so the user can choose which parameters should be updated:



Also the option “**Advanced Autodesign**” will appear. With this option several parameters can be optimized at the same time and restriction for each parameter can be given. So input the following options:



So the thicknesses thb and tha are not autodesigned. All other options can be adapted by SCIA Engineer. When clicking on “Search for an optimal”, the following profile is suggested by SCIA Engineer:



With the option “Set value” a value for a certain parameter can be set. So select the parameter “Ba” and click on “Set value”. Now a value of 500mm can be inputted. And the Maximum unity check for this profile will be adapted automatically.

Overall optimisation

It is also possible to perform an overall optimisation in SCIA Engineer. With this option, one or more profiles can be optimised at the same time. Afterwards the calculation will be restarted and the internal forces are recalculated with the new cross-sections, followed again with a new optimisation.

This iterative process can

- Or stop because all profiles does not have to be autodesigned and the same profile was found as in the previous step
- Or stop because the maximum number of iteration steps has been reached if this is inputted by the user.

It is advised to input a number of iteration steps, otherwise this optimisation process can become a loop and will stop after 99 iteration steps. This will cost a lot of calculation time.

The principle of the overall optimisation process is explained by the following example.

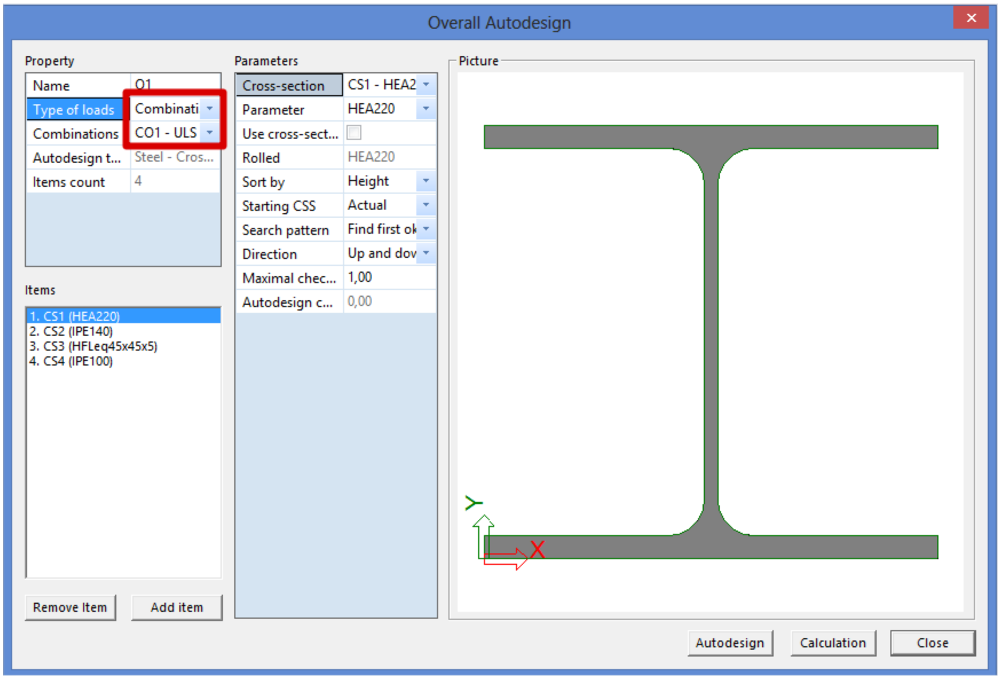
Example: Optimisation.esa

If this example is still open from the previous chapter, please close it and reopen it without saving.

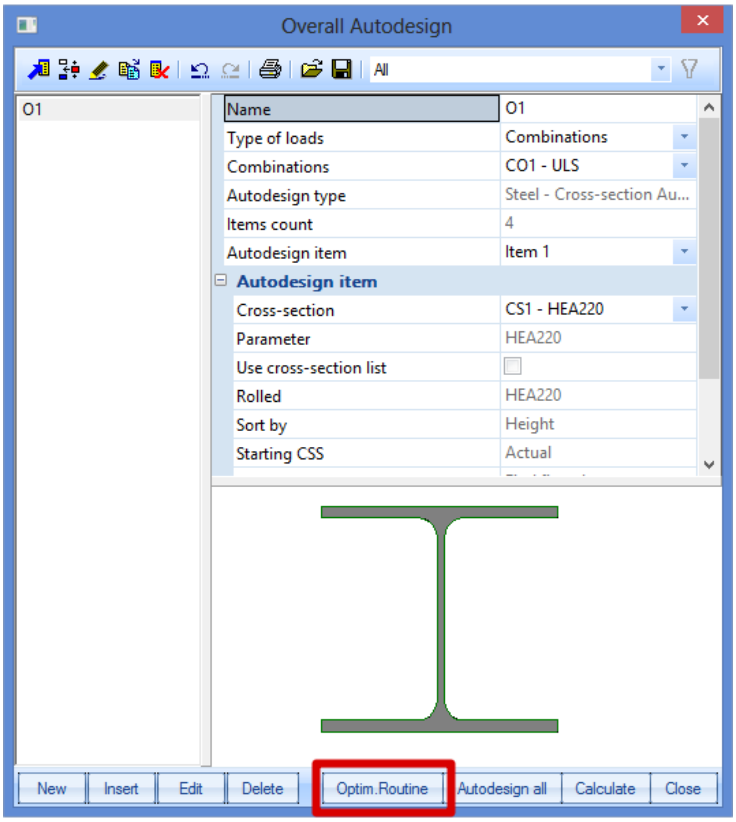
Calculate the project and go to “Main -> Calculation, Mesh -> **Autodesign**”.
Click here on **Add item** and choose for “Steel > Cross-section AutoDesign” and add all cross-sections in this Autodesign process.

The combination, for which the optimisation calculation has to be calculated, can be chosen.

Also for each profile, the user can indicate if this profile has to be autodesigned only in the "Up" direction (so only become bigger) or in the 'Up and down" direction (and also can become a smaller profile as result). With this last option, there is a chance that the iterative process will become a loop.



Now **Close** this window. And click in the next window on **Optimisation Routine**:



Now the user can choose for a maximum number of iteration steps:

- Determine automatically: the iteration process will stop if all the profiles are autodesigned and no different result will be found in a certain iteration step. So no maximum of number of iteration steps will be inputted.
- Limit number of iteration: maximum of number of iteration steps.

Input **5** iteration steps as limit number and click on “Start”.

Now the iteration process will start. The iteration process will stop after no difference will be found or after 5 iteration steps. In this example SCIA Engineer will stop after 4 iteration steps, because all profiles remain the same after the fourth step.

After this process an information window about this iteration will be displayed.

ITERATION STEP 1:

1. Routine step: 1
1.1. O1

Cross-section	Parameter	Sort by	Original cross-section	Autodesign of cross-section	Autodesign check [-]
CS1 - HEA160	HEA160	Height	CS1 - HEA220	CS1 - HEA200	0,85
CS2 - IPE200	IPE200	Height	CS2 - IPE140	CS2 - IPE200	0,94
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq45x45x5	CS3 - HFLeq60x60x6	0,95
CS4 - IPE140A	IPE140A	Height	CS4 - IPE100	CS4 - IPE140AA	0,91

Changed profiles:

- CS1: HEA220 - > HEA200
- CS2: IPE140 -> IPE200
- CS3: HFLeq45x45x5 -> HFLeq60x60x6
- CS4: IPE100 -> IPE140AA

ITERATION STEP 2:

2. Routine step: 2

Cross-section	Parameter	Sort by	Original cross-section	Autodesign of cross-section	Autodesign check [-]
CS1 - HEA160	HEA160	Height	CS1 - HEA200	CS1 - HEA180A	0,93
CS2 - IPE200	IPE200	Height	CS2 - IPE200	CS2 - IPE200	0,91
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq60x60x6	CS3 - HFLeq80x80x10	0,81
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140AA	CS4 - IPE140A	0,94

Changed profiles:

- CS1: HEA200 - > HEA180A
- CS2: IPE200 -> IPE200
- CS3: HFLeq60x60x6 -> HFLeq80x80x10
- CS4: IPE140AA -> IPE140A

ITERATION STEP 3:

3. Routine step: 3

Cross-section	Parameter	Sort by	Original cross-section	Autodesign of cross-section	Autodesign check [-]
CS1 - HEA160	HEA160	Height	CS1 - HEA180A	CS1 - HEA160	0,95
CS2 - IPE200	IPE200	Height	CS2 - IPE200	CS2 - IPE200	0,93
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq80x80x10	CS3 - HFLeq90x90x9	0,97
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140A	CS4 - IPE140A	0,93

Changed profiles:

- CS1: HEA180Z - > HEA160
- CS3: HFLeq80x80x10 -> HFLeq90x90x9
- CS4: IPE140A -> IPE140A

ITERATION STEP 4:

4. Routine step: 4

Cross-section	Parameter	Sort by	Original cross-section	Autodesign of cross-section	Autodesign check [-]
CS1 - HEA160	HEA160	Height	CS1 - HEA160	CS1 - HEA160	0,92
CS2 - IPE200	IPE200	Height	CS2 - IPE200	CS2 - IPE200	0,95
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq90x90x9	CS3 - HFLeq90x90x9	0,99
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140A	CS4 - IPE140A	0,93

No profile is adapted. So the iteration process is stopped.

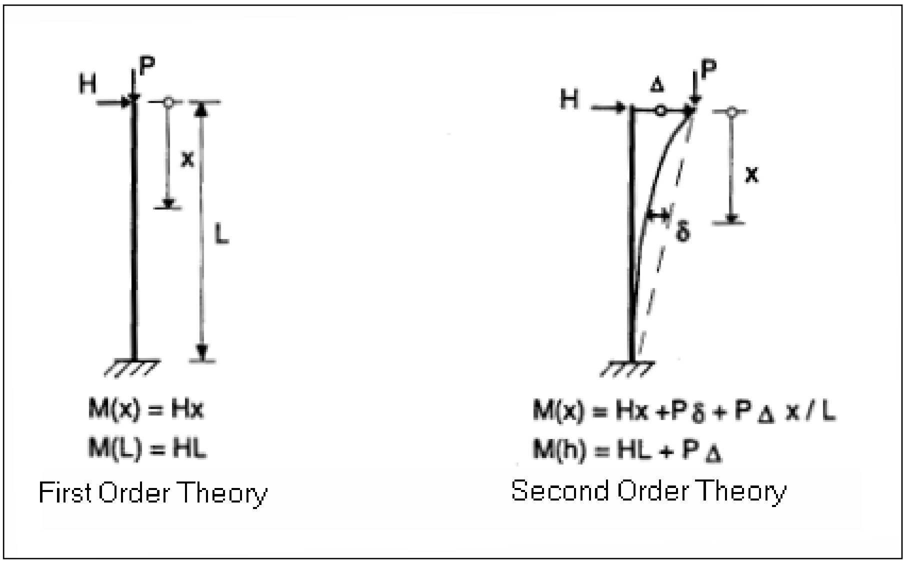
2^{nd} order calculation and imperfections

Overview

Global analysis aims at determining the distribution of the internal forces and moments and the corresponding displacements in a structure subjected to a specified loading.
The first important distinction that can be made between the methods of analysis is the one that separates elastic and plastic methods. Plastic analysis is subjected to some restrictions.
Another important distinction is between the methods, which make allowance for, and those, which neglect the effects of the actual, displaced configuration of the structure. They are referred to respectively as **second-order theory and first-order theory** based methods.

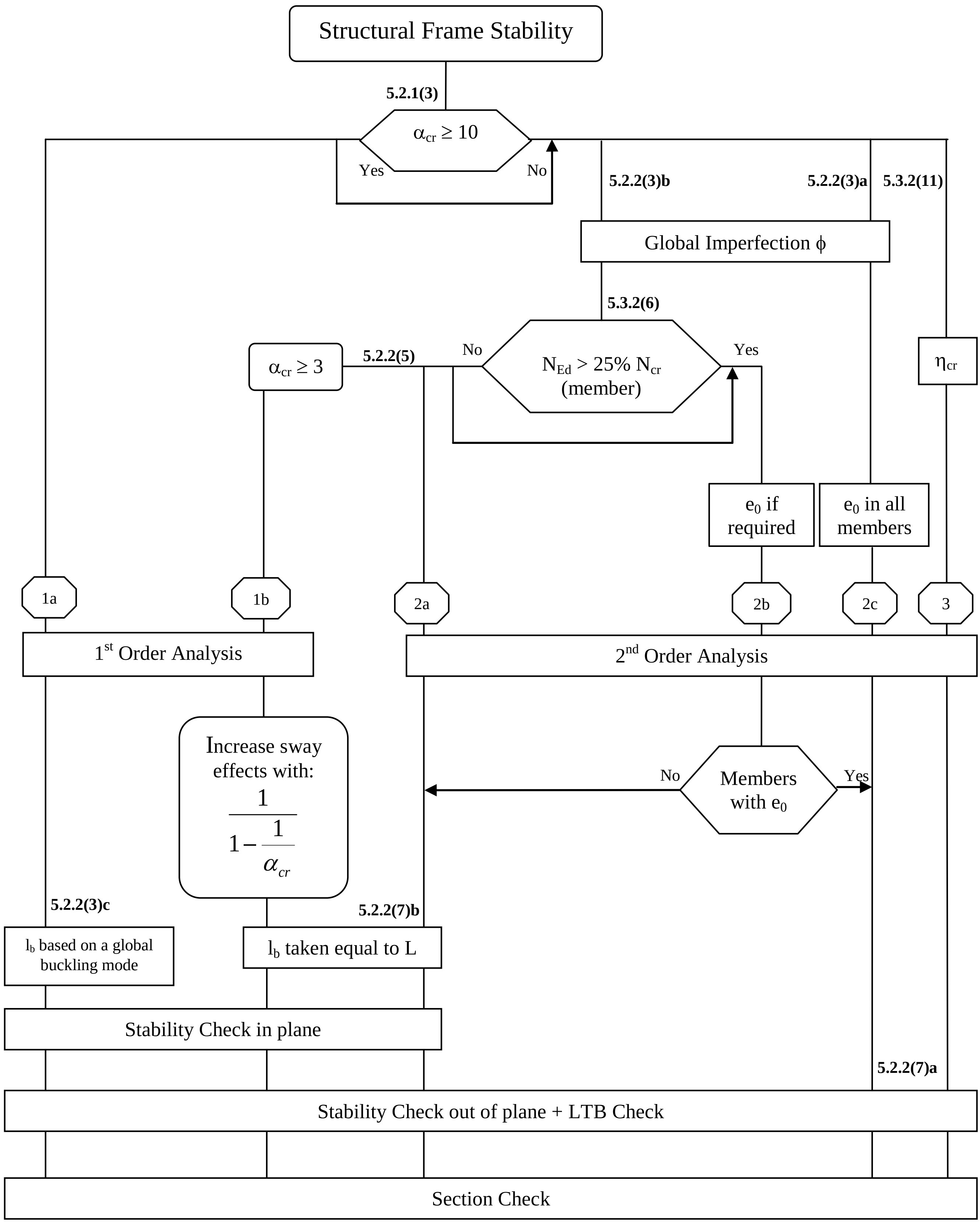
The second-order theory can be adopted in all cases, while first-order theory may be used only when the displacement effects on the structural behavior are negligible.

The second-order effects are made up of a local or member second-order effects, referred to as the P-δ effect, and a global second-order effect, referred to as the P-Δ effect.



On the next page an overview of the global analysis following the EN 1993-1-1, chapter 5, will be given:

- All the rules in this overview are given in the EN 1993-1-1 art. 5. For each step the rule will be indicated. The first rule ($\alpha_{cr} \geq 10$) will be explained in EN 1993-1-1 **art. 5.2.1(3)**.
- In this overview 3 paths are defined:
  - Path 1: In this path a first order calculation will be executed
  - Path 2: In this path a second order calculation will be executed with global (and bow) imperfections.
  - Path 3: In this path a second order calculation will be executed with the buckling shape of the construction as imperfection.
- The calculation will become more precise when choosing for a higher path.
- The lower paths will result in a faster calculation, because a first order calculation can be executed without iterations, but this first-order theory may be used only when the displacement effects on the structural behavior are negligible.
- In the next paragraphs the rules in this overview will be explained.



With: η_{cr} Elastic critical buckling mode.

L Member system length

l_b Buckling Length

Alpha critical – Not in concept edition

The calculation of alpha critical is done by a stability calculation in SCIA Engineer. For this calculation a Professional or an Expert edition is necessary. The stability calculation has been inputted in module esas.13.

According to the EN 1993-1-1, 1st Order analysis may be used for a structure, if the increase of the relevant internal forces or moments or any other change of structural behaviour caused by deformations can be neglected. This condition may be assumed to be fulfilled, if the following criterion is satisfied:

$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10$ for elastic analysis

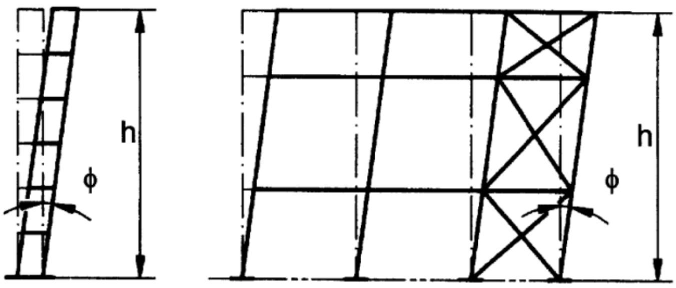
With: α_{cr}: the factor by which the design loading has to be increased to cause elastic instability in a global mode.
F_{Ed}: the design loading on the structure.
F_{cr}: the elastic critical buckling load for global instability, based on initial elastic stiffnesses.

If α_{cr} has a value lower then 10, a 2nd Order calculation needs to be executed. Depending on the type of analysis, both Global and Local imperfections need to be considered.

EN1993-1-1 prescribes that 2nd Order effects and imperfections may be accounted for both by the global analysis or partially by the global analysis and partially through individual stability checks of members.

Global frame imperfection φ

The global frame imperfection will be inputted for the whole structure by an imperfection value ϕ. This value can be calculated with the following formula (EN 1993-1-1 art. 5.3.2(3)a):



$$\varphi = \frac{1}{200} \cdot \alpha_h \cdot \alpha_m$$

$\alpha_h = \frac{2}{\sqrt{h}}$ but $\frac{2}{3} \leq \alpha_h \leq 1{,}0$

$$\alpha_m = \sqrt{0{,}5\left(1 + \frac{1}{m}\right)}$$

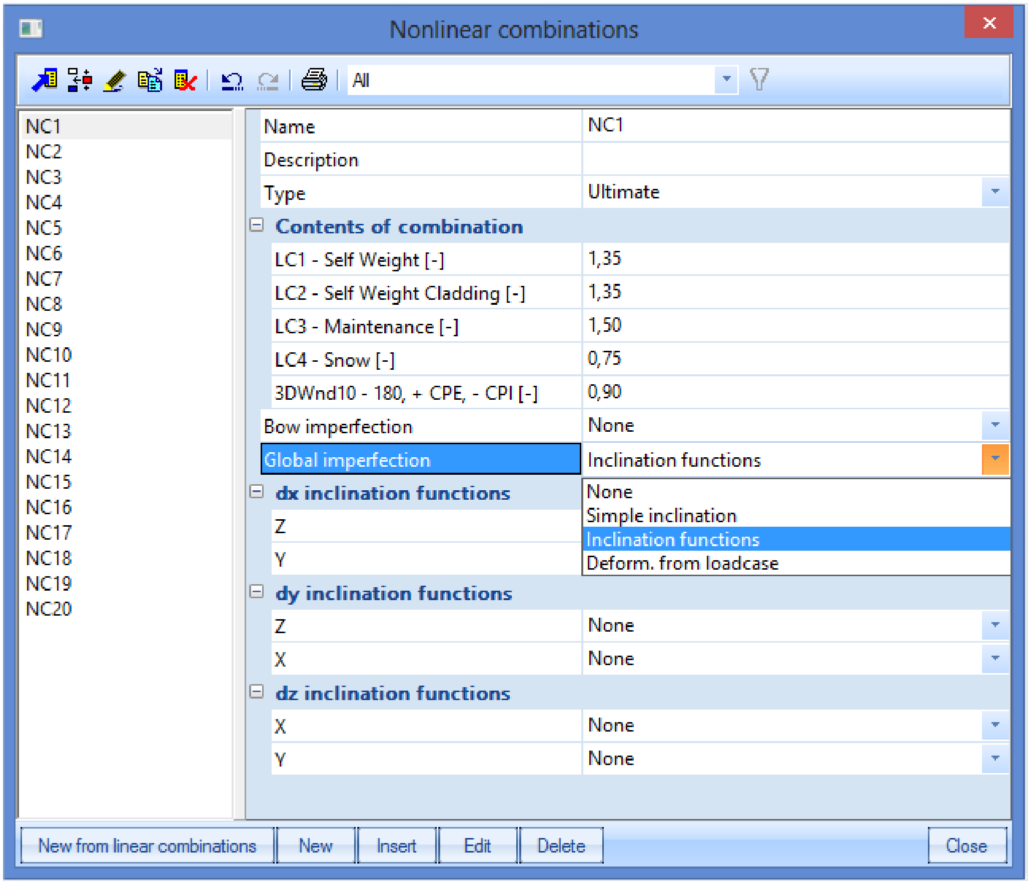
With: h The height of the structure in meters
m The number of columns in a row including only those columns which carry a vertical load N_{Ed} not less than 50% of the average value of the vertical load per column in the plane considered.

The global imperfection can be inputted in SCIA Engineer in the non linear combinations as explained by the example below.

Example: Industrial Hall.esa

In this example, two global imperfections functions will be used: one according to global X-direction and one according to the global Y direction. It is not necessary to combine both imperfections in the same combination.

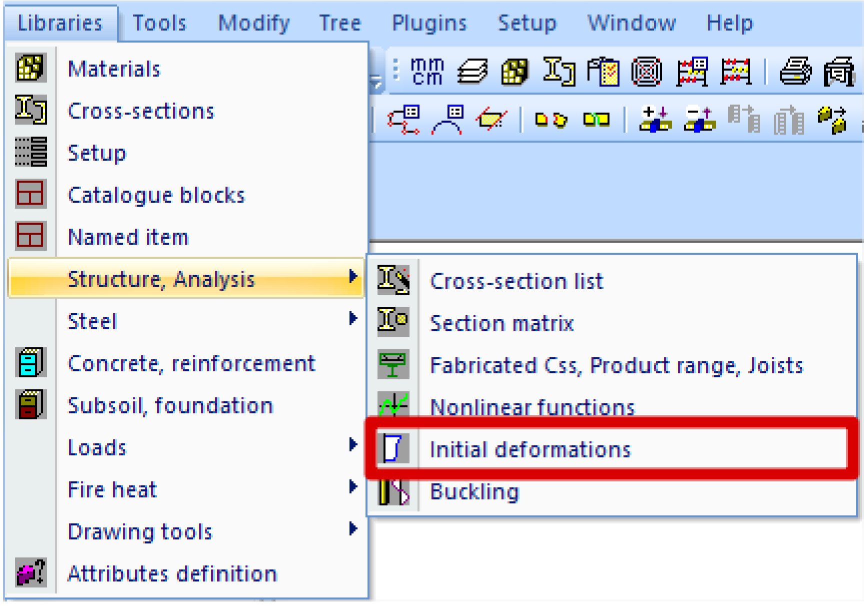
In SCIA Engineer the user can choose between four options for the **global imperfection** in the non linear combinations:



- **None**: No global imperfection will be taken into account
- **Simple inclination**: the user will input a deformation in the global X- or/and Y-direction as the imperfection in mm per meter height in the global Z-direction [mm/m].
- **Inclination functions**: input a user inclination function or the inclination function of the EN 1993-1-1 (this option is explained below).
- **Deformation from loadcase**: with this option the user can choose for a calculated loadcase and the deformation of that loadcase will be used as initial global imperfection.

In this example the option “inclination functions” has been chosen.

These inclination functions are entered through “Main -> Library -> Structure, Analysis -> Initial deformations”:



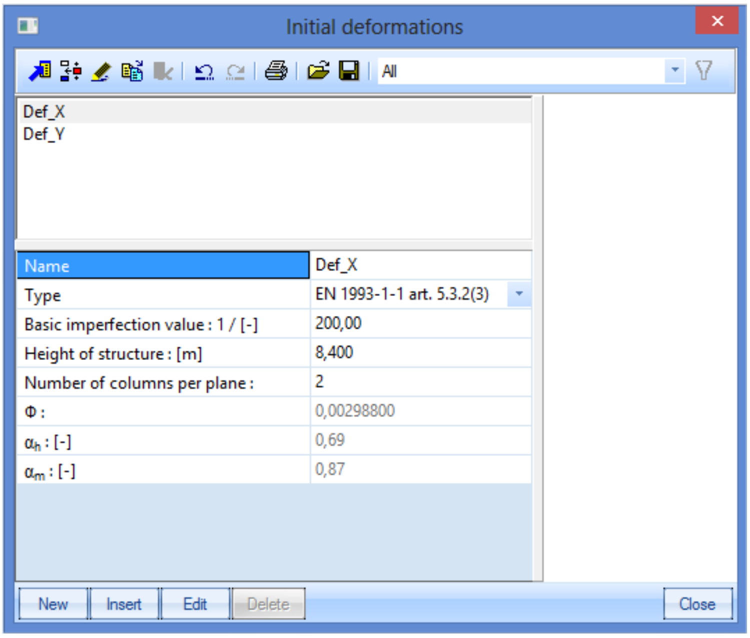
The Type is chosen as “EN 1993-1-1 art. 5.3.2(3)”, with a standard imperfection of 1/200.

The height of the construction is 8.4m for both inclination functions.

There are 6 columns in the X-direction, but in the middle only 2 columns are inputted. Because a long part of structure only has 2 columns in the X-direction, in this example the number of columns in this direction has been inputted as “2”.

There are 11 columns in the Y-direction. But the columns at the end are smaller than the middle ones. So in this example it is decided to input “9” columns in the Y-direction.

The inclination function for the x-direction (Def_X) in SCIA Engineer is displayed below:



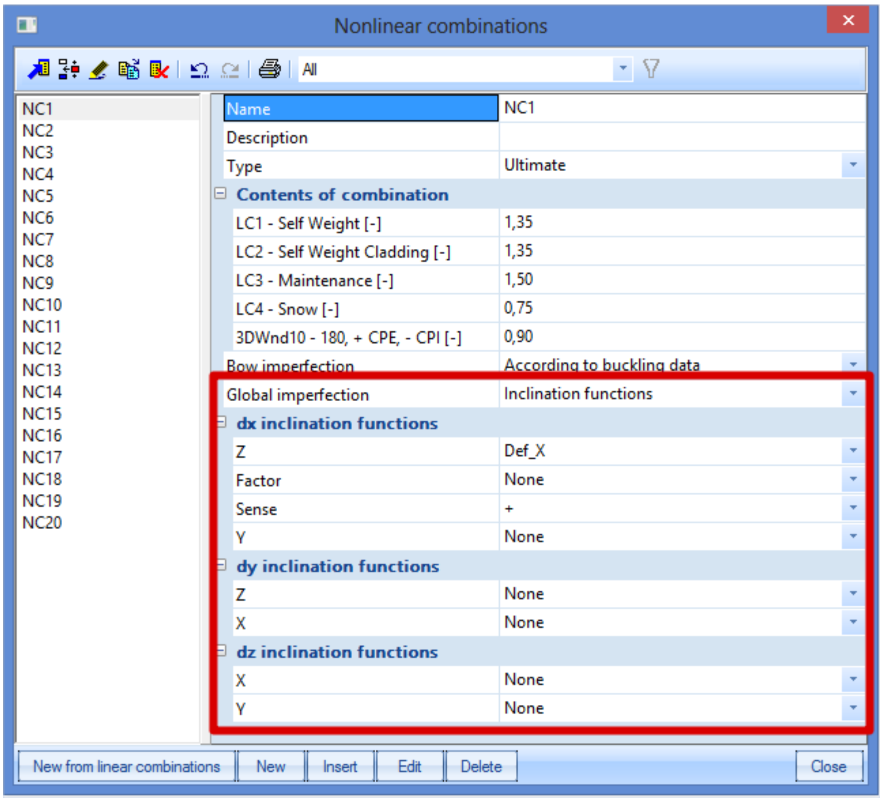
In this example 5 non linear combinations are inputted:

- 1,35 x Self Weight + 1,35 x Self Weight Cladding + 1,5 x Maintenance
  + 0,75 x Snow + 0,9 x 3DWind10
- 1,35 x Self Weight + 1,35 x Self Weight Cladding + 0,75 x Snow + 1,5 x 3DWind13
- 1,35 x Self Weight + 1,35 x Self Weight Cladding + 0,75 x Snow + 1,5 x 3DWind14
- 1,35 x Self Weight + 1,35 x Self Weight Cladding + 0,75 x Snow + 1,5 x 3DWind15

- 1,35 x Self Weight + 1,35 x Self Weight Cladding + 0,75 x Snow + 1,5 x 3DWind16

All combinations are entered four times:

- NC1-NC5: inclination in the positive X- direction
- NC6-NC10: inclination in the negative X- direction
- NC11-NC15: inclination in the positive Y- direction
- NC16-NC20: inclination in the negative Y- direction



Bow imperfection

N_{Ed} > 25% N_{cr}

The relative initial local bow imperfections of members for flexural buckling is given as the value: e_0/L.

This bow imperfection does not have to be applied on each member as given in EN 1993-1-1 art. 5.3.2(6):

The bow imperfection has to be applied when the normal force N_{Ed} in a member is higher than 25% of the member's critical load N_{cr}:

When performing the global analysis for determining end forces and end moments to be used in member checks according to 6.3 local imperfections may be neglected. However for frames sensitive to second order effects local bow imperfections of members additionally to global sway imperfections should be introduced in the structural analysis of the frame of each compressed member where the following conditions are met:

- At least one moment resistant joint at one member end
- $\bar{\lambda} > 0,5\sqrt{\frac{A \cdot f_y}{N_{Ed}}}$

And $\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$

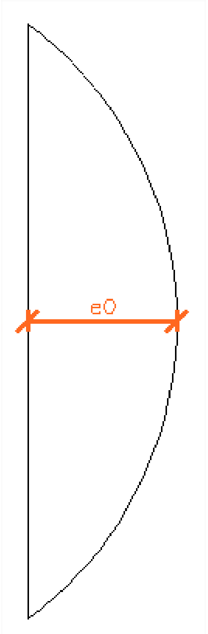
Thus: $\sqrt{\frac{A \cdot f_y}{N_{cr}}} > 0{,}5 \sqrt{\frac{A \cdot f_y}{N_{Ed}}}$

$\frac{1}{N_{cr}} > 0{,}25 \frac{1}{N_{Ed}}$

$\boldsymbol{N_{Ed}} > 0{,}25\, \boldsymbol{N_{cr}}$

Bow imperfection e_0

The initial bow imperfection is given by:



Buckling curve acc. to Table 6.1	elastic analysis e_0 / L	plastic analysis e_0 / L
a_0	1 / 350	1 / 300
a	1 / 300	1 / 250
b	1 / 250	1 / 200
c	1 / 200	1 / 150
d	1 / 150	1 / 100

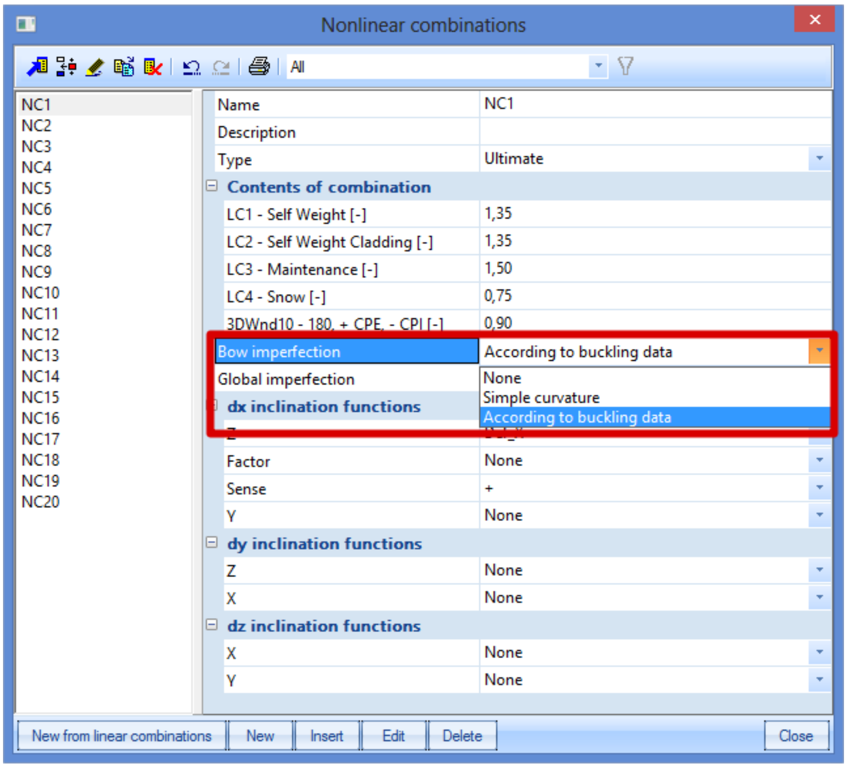
Where L is the member length.

As explained in the previous chapter, the bow imperfection has to be applied when the normal force N_{Ed} in a member is higher than 25% of the member's critical buckling load N_{cr}. If N_{Ed} < 25% N_{cr}, the user can choose to apply this bow imperfection or not.

The buckling curve used for calculation of the imperfection is the curve inputted in the cross-section library. For standard sections, the curve according to the code is automatically used, for non-standard cross sections (as general cross sections) the user needs to input the buckling curve manually.

Example: Industrial hall.esa

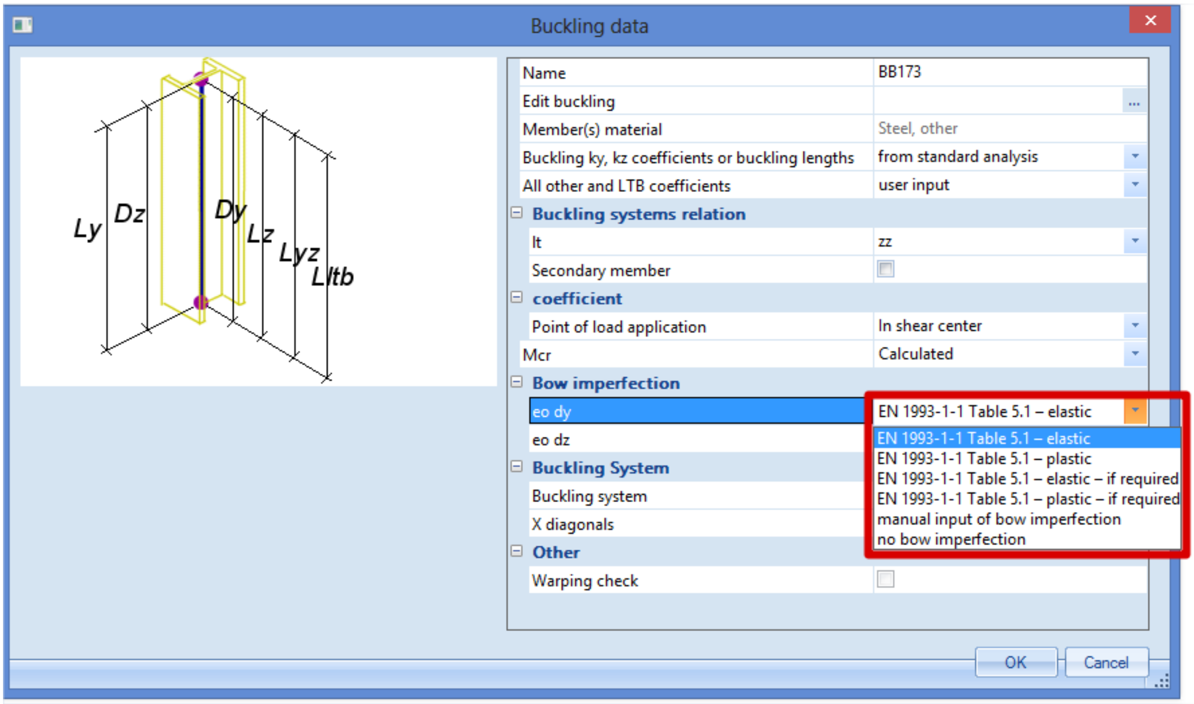
In SCIA Engineer the user can choose between three options for the **bow imperfection** in the non linear combinations:



- **None**: No bow imperfection will be taken into account
- **Simple curvature**: the user will input a deformation “f” or “1/f” for all the members, where “1/f” corresponds with the value “e0/L” as explained earlier.
- **According to buckling data**: with this option the user can choose for a local imperfection as defined in the buckling data, so for each member following his own buckling curve (this option is explained below).

In this example the option “According to buckling data” has been chosen.

The buckling data can be inputted via “Steel -> Beams -> Member Check data -> Member buckling data”



The user can choose between 6 options:

- **EN 1993-1-1 Table 5.1 – elastic**: the elastic value following the buckling curve of the cross section will be used.
- **EN 1993-1-1 Table 5.1 – plastic**: the plastic value following the buckling curve of the cross section will be used.
- **EN 1993-1-1 Table 5.1 – elastic – if required**: the elastic value following the buckling curve of the cross section will be used if Ned > 25% Ncr.
- **EN 1993-1-1 Table 5.1 – plastic – if required**: the plastic value following the buckling curve of the cross section will be used if Ned > 25% Ncr
- **Manual input of bow imperfection**: the user can input manually a value for this bow imperfection.
- **No bow imperfection**: no bow imperfection is taken into account for the member.

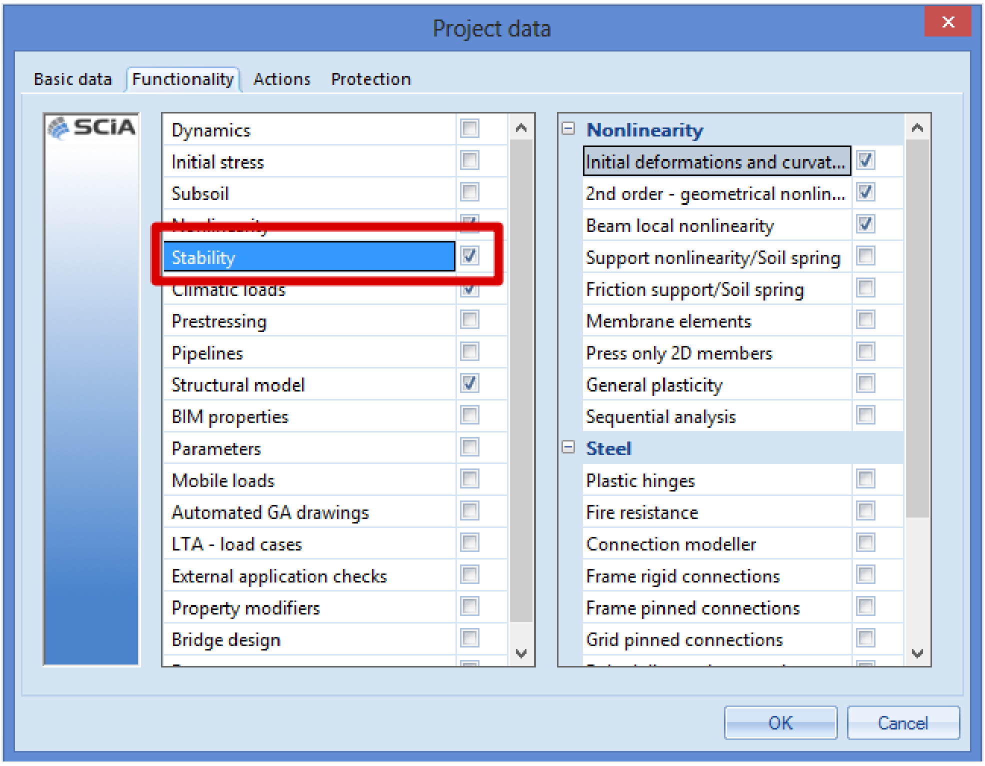
In this example on all beams the bow imperfection “According to code – elastic” has been inputted. Except for the diagonals no bow imperfection has been inputted in this example.

Buckling shape as imperfection - η_{cr} – Not in concept edition

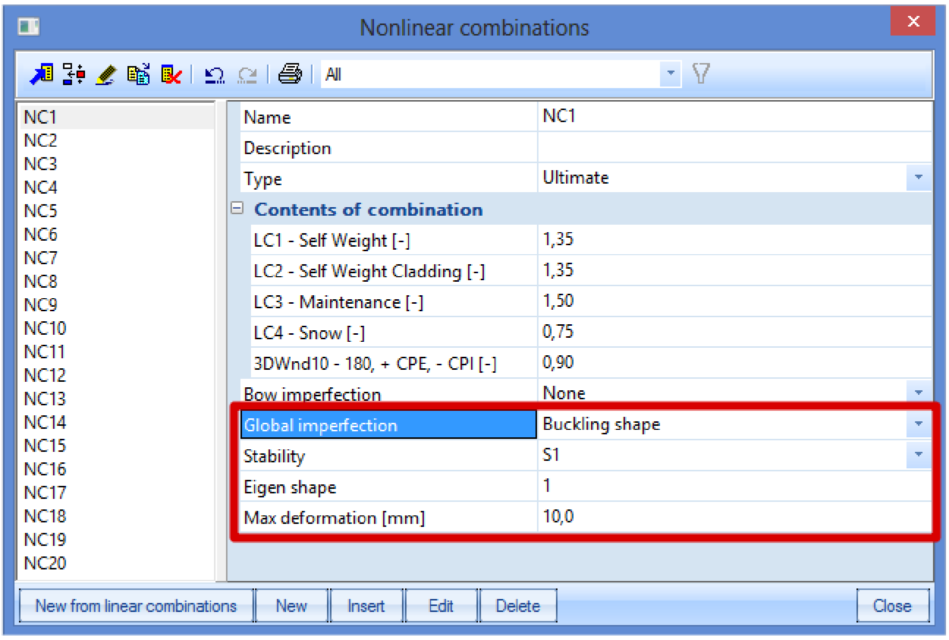
The calculation of alpha critical is done by a stability calculation in SCIA Engineer. For this calculation a Professional or an Expert edition is necessary. The stability calculation has been inputted in module esas.13.

As an alternative the global and bow imperfection can be replaced by the buckling shape as imperfection (path 3 from the global diagram).

To input geometrical imperfections, the functionality **Nonlinearity > Initial deformations and curvature** and **Stability** must be activated.



So first the stability calculation (linear or non linear) is calculated. Afterwards in the non linear combinations, the user can choose the buckling shape he wants to take into account. So first he can choose for the stability combination and just below for the calculated mode. If for example the first and second mode give a negative value for the alpha critical, it could be possible that the user wants to use the 3rd calculated buckling mode:



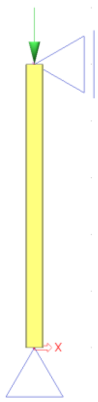
The last option that has to be inputted is a value for the maximum deformation. This is the deformation of the node which has the biggest deformation of the construction. SCIA Engineer will recalculate all the displacement of the other nodes using this maximal deformation.

Since the buckling shape is dimensionless, EN 1993-1-1 gives the formula to calculate the amplitude η_{init} of the imperfection. In below an example of this calculation is given for a simple example.

Example: Buckling Shape.esa

In this example, the procedure to calculate a buckling shape is illustrated for a column.

The column has a cross-section of type **IPE 300**, is fabricated from **S235** and has the following relevant properties:



$E = 210.000\ N/mm^2$

$f_y = 235\ N/mm^2$

$\gamma_{M1} = 1.00$

$L = 5000\ mm$

$A = 5380\ mm^2$

$I_y = 83560000 mm^4$

$W_{pl,y} = 628400\ mm^3$

Calculation of the buckling shape

First a **Stability calculation** is done using a load of 1kN. This way, the elastic critical buckling load N_{cr} is obtained.
In order to obtain precise results, the **Number of 1D elements** is set to **10**. This can be done in “Setup -> Mesh”. In addition, the **Shear Force Deformation** is neglected so the result can be checked by a manual calculation.

The stability calculation gives the following result:

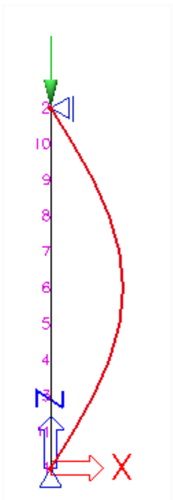
Critical load coefficients

N -	f []
Stability combination : S1	
1	6927,50
2	27714,23
3	62404,64
4	111170,55

This can be verified with Euler's formula using the member length as the buckling length:

$$N_{cr} = \frac{\pi^2 EI}{l^2} = \frac{\pi^2 \cdot 210000N/mm^2 \cdot 8{,}3560 \cdot 10^7 mm^4}{(5000mm)^2} = 6927{,}51\ kN$$

The following picture shows the mesh nodes of the column and the corresponding buckling shape:

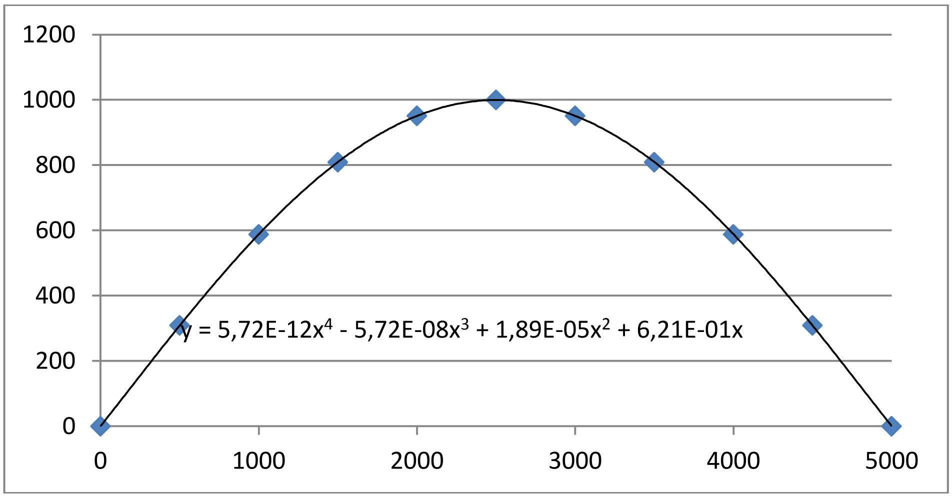


Displacement of nodes

Stability calculation, Extreme : No
Selection : All
Stability combinations : S1/1 - 6927,50
Buckling shapes are dimensionless, units are printed for consistency purposes.

Node of mesh	Case	Ux [mm]	Uz [mm]	Fiy [mrad]
1	S1/1 - 6927,50	0,0	0,0	628,3
11	S1/1 - 6927,50	309,0	0,0	597,6
3	S1/1 - 6927,50	587,8	0,0	508,3
4	S1/1 - 6927,50	809,0	0,0	369,3
5	S1/1 - 6927,50	951,1	0,0	194,2
6	S1/1 - 6927,50	1000,0	0,0	0,0
7	S1/1 - 6927,50	951,1	0,0	-194,2
8	S1/1 - 6927,50	809,0	0,0	-369,3
9	S1/1 - 6927,50	587,8	0,0	-508,3
10	S1/1 - 6927,50	309,0	0,0	-597,6
2	S1/1 - 6927,50	0,0	0,0	-628,3

Using for example an Excel worksheet, the buckling shape can be approximated by a 4th grade polynomial.



A polynomial has the advantage that the second derivative can easily be calculated.

$$\Rightarrow \quad \eta_{cr} = 5{,}72 \cdot 10^{-12} \cdot x^4 - 5{,}72 \cdot 10^{-8} \cdot x^3 + 1{,}89 \cdot 10^{-5} \cdot x^2 + 6{,}21 \cdot 10^{-1} \cdot x$$

$$\Rightarrow \quad \eta''_{cr,max} = 6{,}86 \cdot 10^{-11} \cdot x^2 - 3{,}43 \cdot 10^{-7} \cdot x + 3{,}78 \cdot 10^{-5}$$

Calculation of e_0

$$N_{Rk} = f_y \cdot A = 235 \frac{N}{mm^2} \cdot 5380\ mm^2 = 1264300\ N$$

$$M_{Rk} = f_y \cdot W_{pl} = 235 \frac{N}{mm^2} \cdot 628400\ mm^3 = 147674000\ Nmm\ (class2)$$

$$\bar{\lambda} = \sqrt{N_{Rk}/N_{cr}} = \sqrt{1264300N/6885280N} = 0{,}43$$

$\alpha = 0{,}21$ for buckling curve a

$$\chi = \frac{1}{0{,}5\left[1+\alpha(\bar{\lambda}-0{,}2)+(\bar{\lambda})^2\right]^2+\sqrt{\left(0{,}5\left[1+\alpha(\bar{\lambda}-0{,}2)+(\bar{\lambda})^2\right]\right)^2-(\bar{\lambda})^2}} = 0{,}945$$

These intermediate results can be verified through SCIA ENGINEER when performing a Steel Code Check on the column:

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5,000	5,000	m
Buckling factor k	1,00	1,00	
Buckling length Lcr	5,000	5,000	m
Critical Euler load Ncr	6927,51	500,58	kN
Slenderness Lambda	40,12	149,25	
Relative slenderness Lambda,rel	0,43	1,59	
Limit slenderness Lambda,rel,0	0,20	0,20	

$$\Rightarrow \quad e_0 = \alpha \cdot (\bar{\lambda} - 0{,}2) \cdot \frac{M_{Rk}}{N_{Rk}} \cdot \frac{1-\frac{\chi \cdot (\bar{\lambda})^2}{\gamma_{M1}}}{1-\chi \cdot (\bar{\lambda})^2} = \alpha \cdot (\bar{\lambda} - 0{,}2) \cdot \frac{M_{Rk}}{N_{Rk}}$$

$$\Rightarrow \quad e_0 = 0.21(0.43 - 0.2) \cdot \frac{147674000\ Nmm}{1264300\ N} = \mathbf{5.573\ mm}$$

The required parameters have now been calculated so in the final step the amplitude of the imperfection can be determined.

Calculation of η_{init}

The mid section of the column is decisive ⇒ x = 2500

η_{cr} at mid section = 1000,31

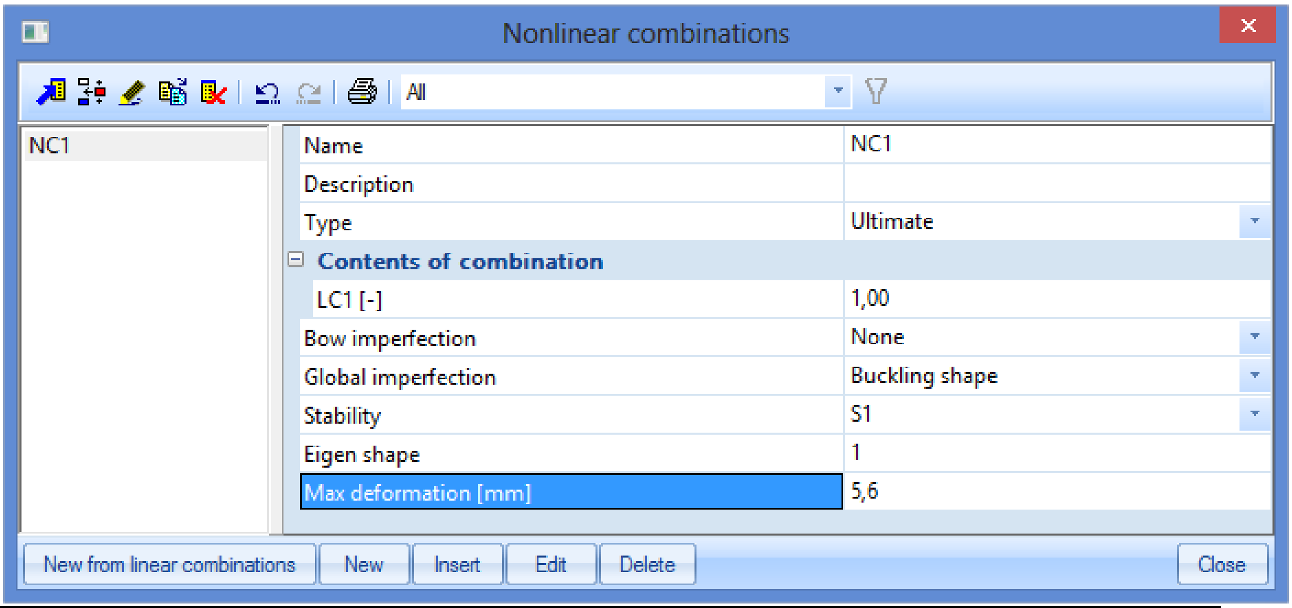
$\eta''_{cr,max}$ at mid section = -3,912 $\cdot 10^{-4} \cdot 1/mm^2$

$$\Rightarrow \quad \eta_{init} = e_0 \cdot \frac{N_{cr}}{E \cdot I_y \cdot \eta''_{cr,max}} \cdot \eta_{cr}$$

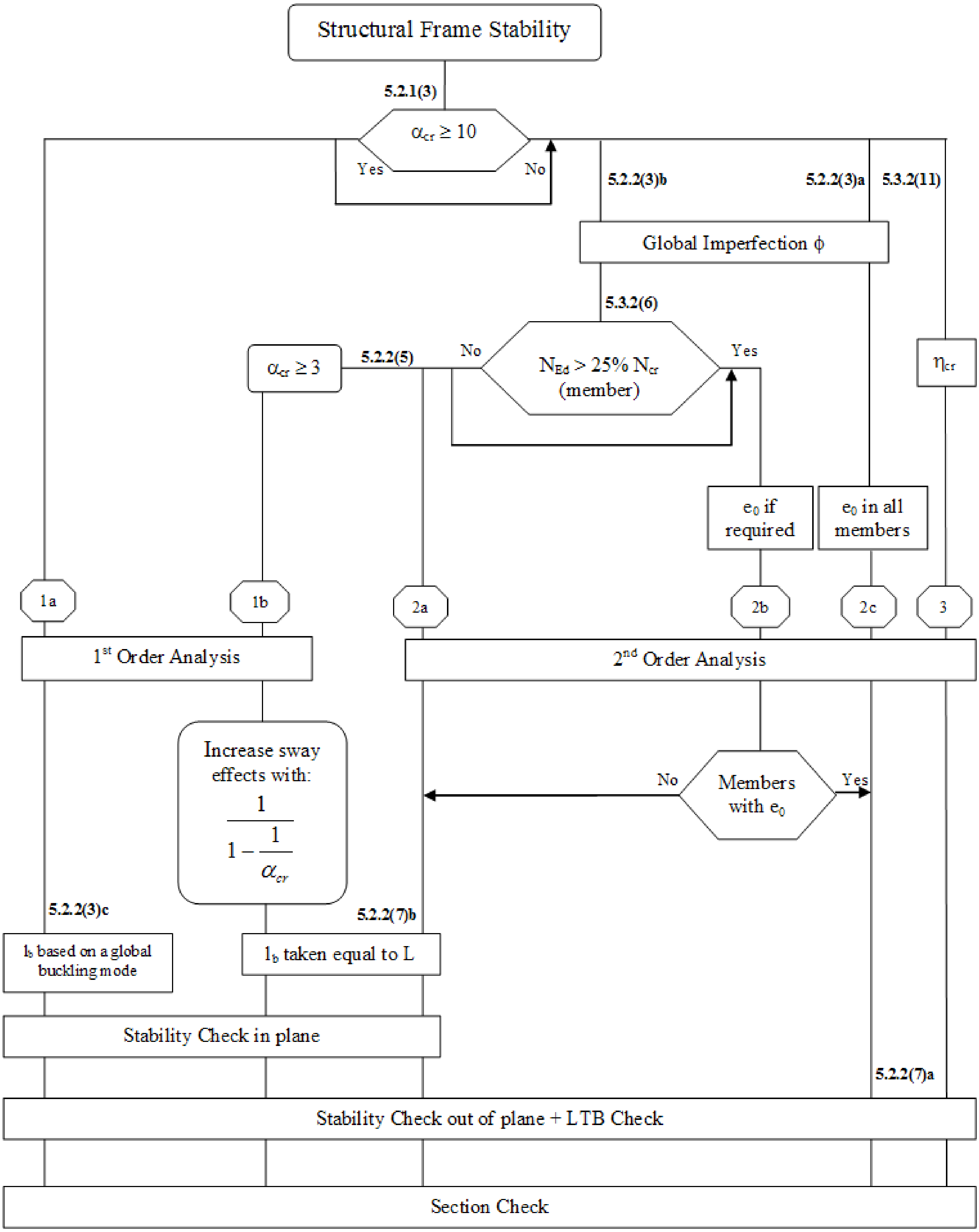
$$\Rightarrow \quad \eta_{init} = 5{,}642\ mm \frac{6885280\ N}{210000 N/mm^2 \cdot 83560000\ mm^4 \cdot 3{,}912 \cdot 10^{-4} \cdot 1/mm^2} \cdot 1000$$

$$\Rightarrow \quad \eta_{init} = \mathbf{5{,}629\ mm}$$

This value can now be inputted as amplitude of the buckling shape for imperfection.



1st or 2nd order analysis – Overview paths acc. to EN1993: overview



Examples:

Example Overview Paths.esa

Example Overview Paths_path 1a.esa

Example Overview Paths_path 1b.esa

Example Overview Paths_path 2a.esa

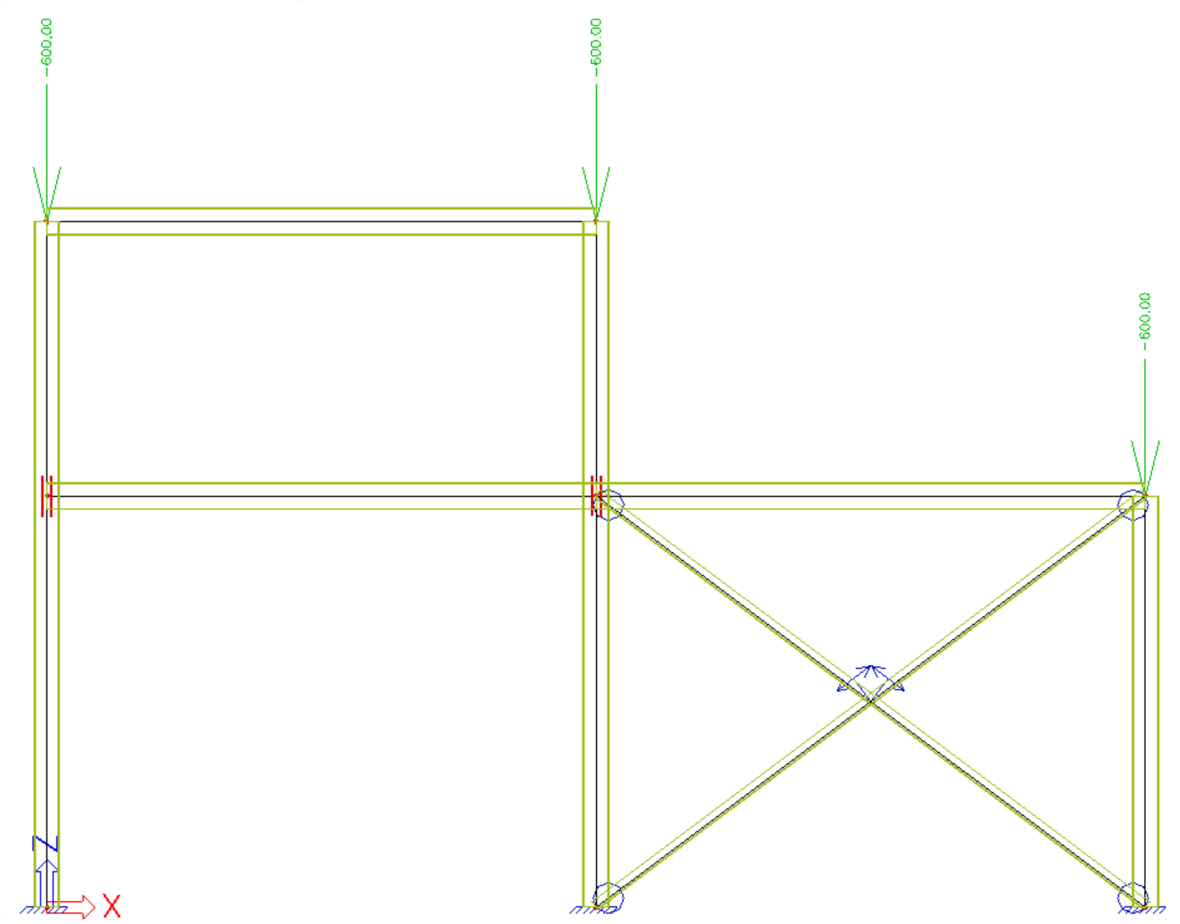
Example Overview Paths_path 2b.esa

Example Overview Paths_path 2c.esa

Example Overview Paths_path 3.esa

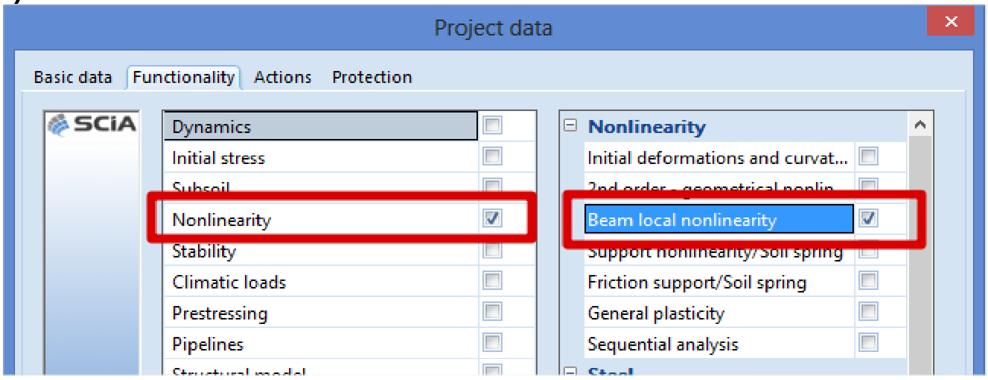
Example Overview

1 load case, containing 3 point loads of 600 kN.



Path 1a 1st order analysis

1) Functionalities

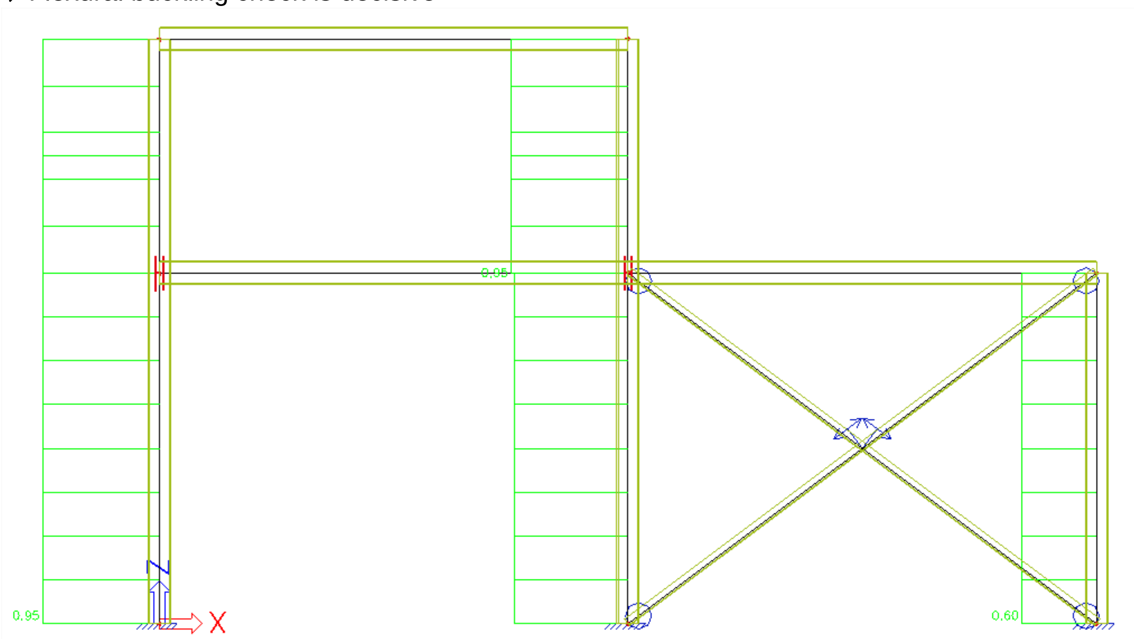


2) Linear calculation

Setup of buckling properties
- buckling factor k: calculated by default by SCIA Engineer (only valid for simple structures!)
- sway/non-sway property: proposition of SCIA Engineer to be checked by user
- system length of member: proposition of SCIA Engineer to be checked by user

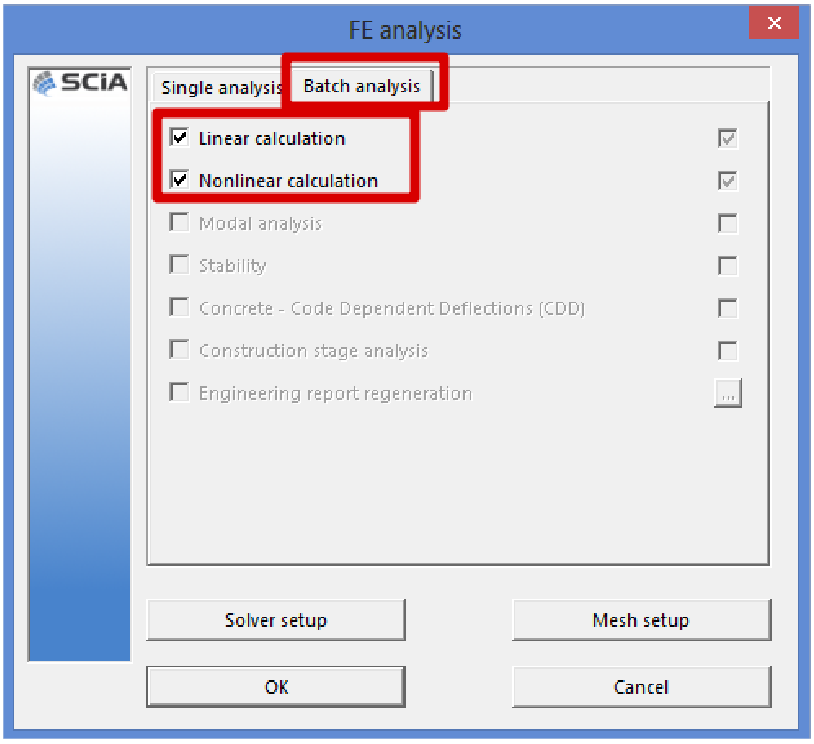
Alternative: input of buckling factor k or buckling length l by user

Unity check for ULS
→ Flexural buckling check is decisive

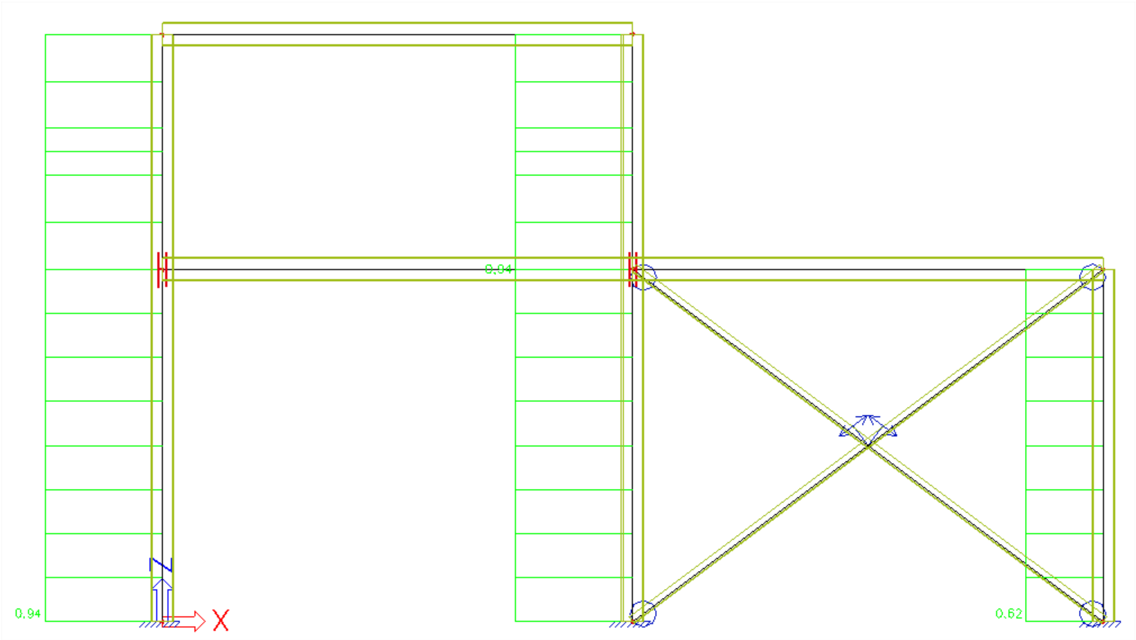


3) Nonlinear calculation

Same setup of buckling properties as for linear calculation.
The only difference is the 'tension only' property that has been added to the wind bracings. This means we are still dealing with a 1st order calculation, but a nonlinear calculation has to be executed to take the local nonlinearity 'tension only' into account.
Attention: It is necessary to execute a BATCH calculation (linear & nonlinear), because the buckling properties are calculated only during the linear calculation!

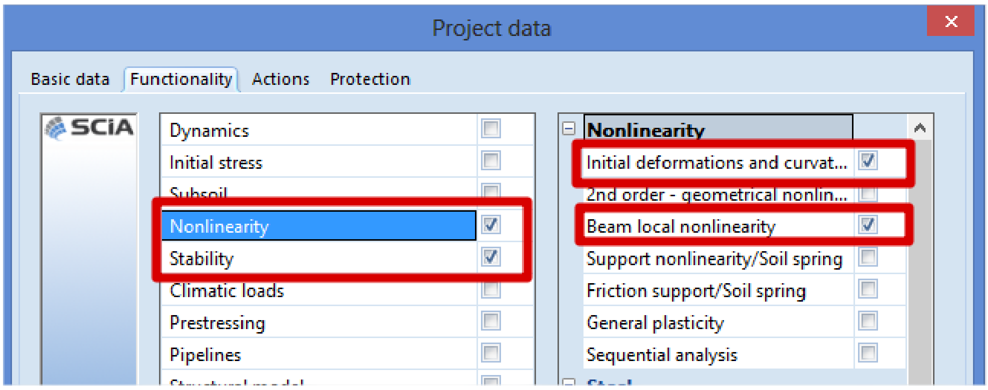


Unity check for ULS
→ Flexural buckling check is decisive



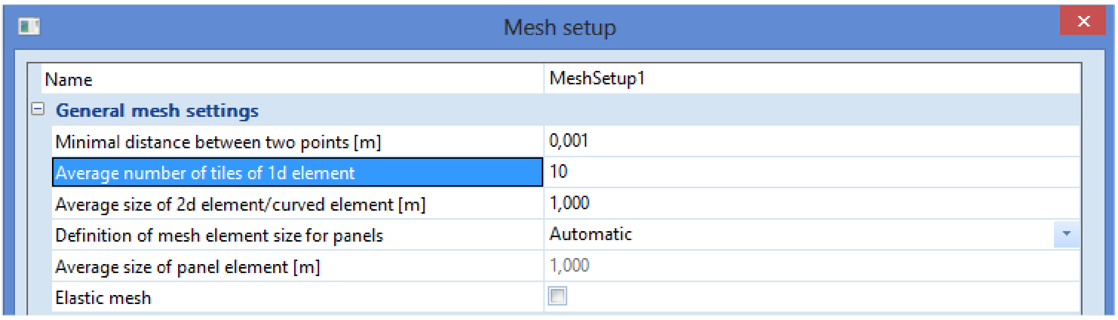
Path 1b 1st order analysis

1) Functionalities



2) Stability calculation

The stability calculation will be performed to determine the critical alpha coefficient. As mentioned in the chapter “Buckling shape as imperfection”, the **Average number of tiles of 1d element** is set to **10**.



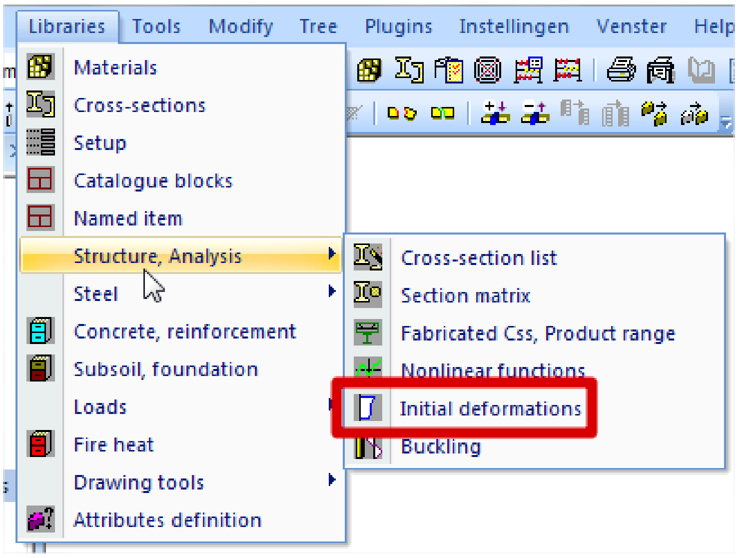
The critical alpha value has to be higher than 3 to be able to use path b of the structural frame stability scheme. In this example, the critical load coefficient is equal to 7,03 (which is in between 3 and 10).

Critical load coefficients

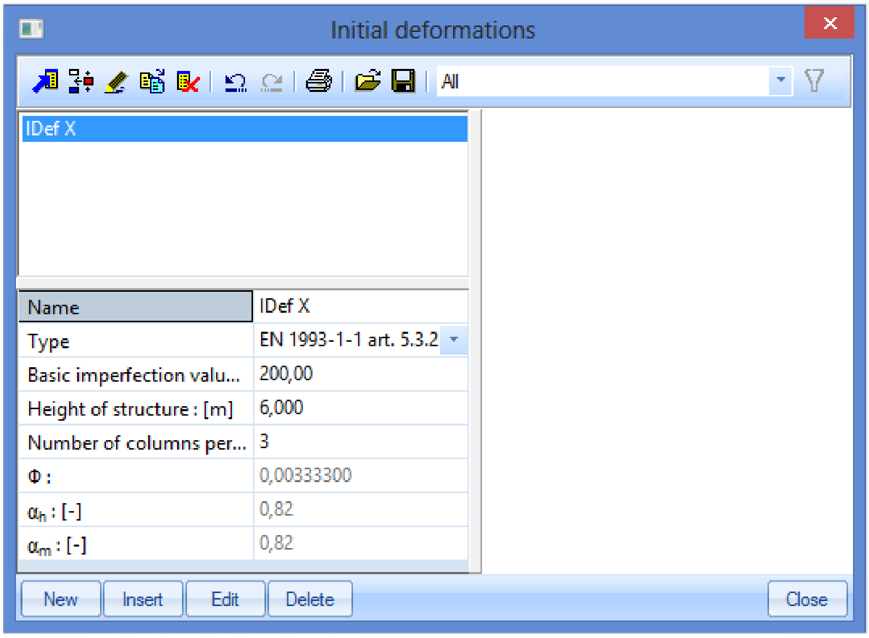
N -	f []
Stability combination : S1	
1	7,03
2	22,78
3	27,07
4	35,01

3) Global imperfection

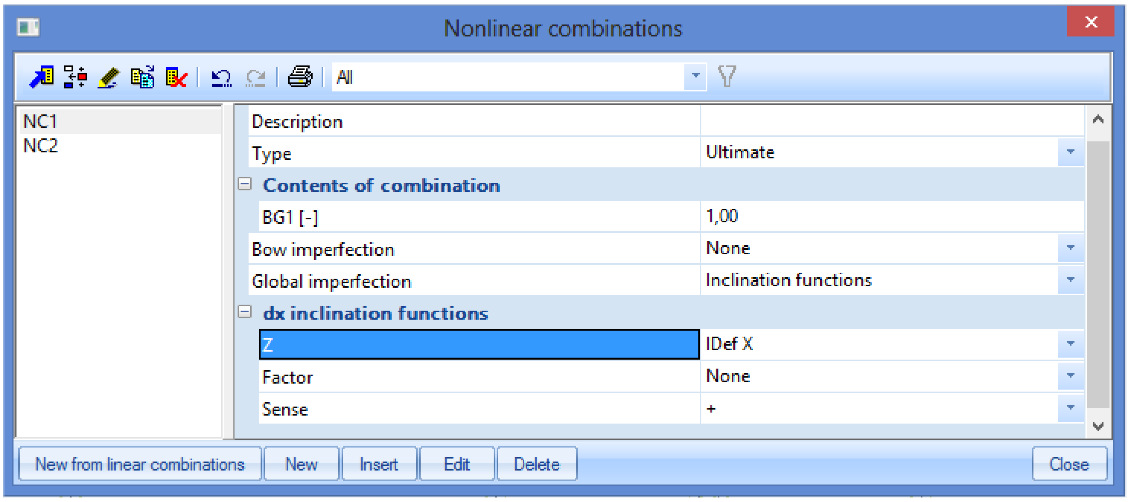
The next step is to insert the global imperfection. This will be done using “Libraries > Structure, Analysis > Initial deformations”.



Since this example is made in a 2D-environment, one initial deformation for the x-direction is sufficient. If the project was made in a 3D-environment, it would be needed to create two initial deformations (one in x-direction and one in y-direction).

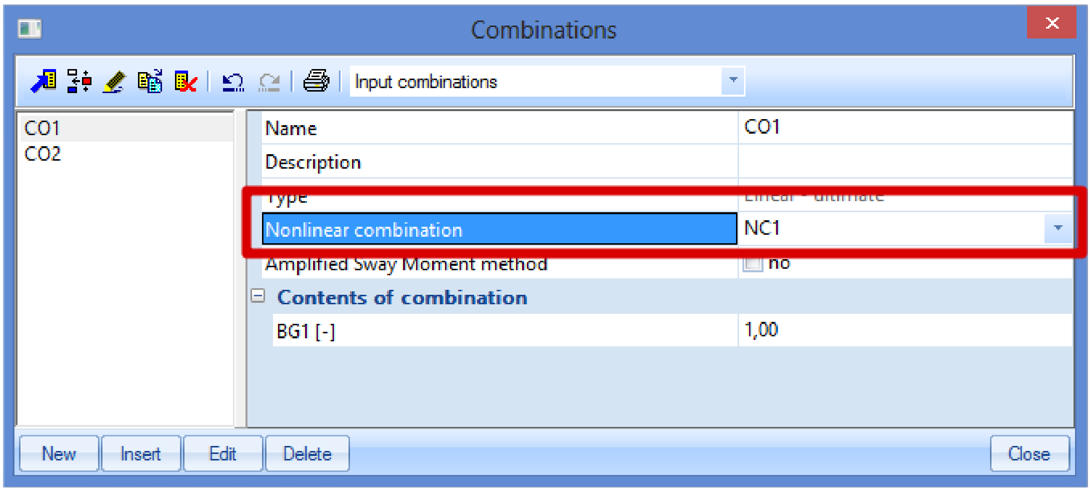


This global imperfection can now be inserted into the nonlinear combination. Once in the positive x-direction and once in the negative x-direction.

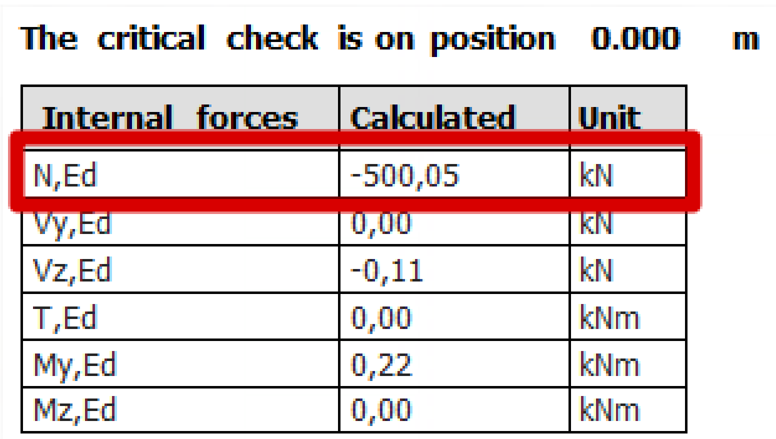


4) Linear combination

The inserted imperfections have to be used during the linear calculation. This can be done by referring to the correct nonlinear combination in the linear combination window.



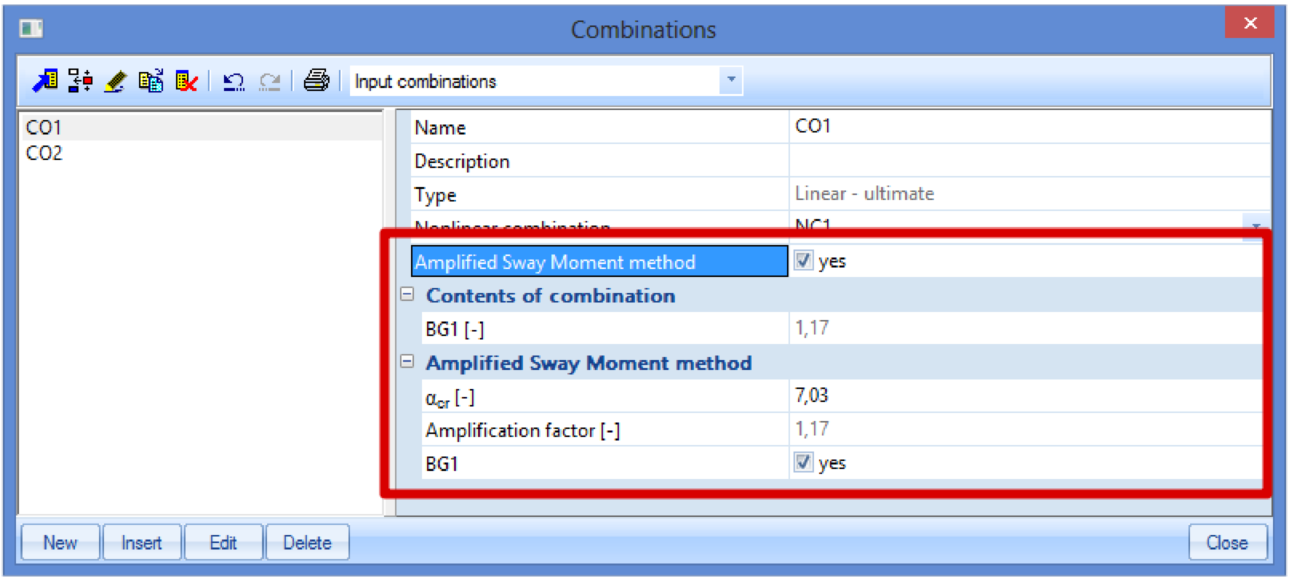
The **Amplified Sway Moment method** may only be used, if the actual normal force N_{Ed} is smaller than 25% of the critical Euler force N_{cr}. Both values are printed in the detailed output of the steel code check. For column S1:


The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-500,05	kN
Vy,Ed	0,00	kN
Vz,Ed	-0,11	kN
T,Ed	0,00	kNm
My,Ed	0,22	kNm
Mz,Ed	0,00	kNm

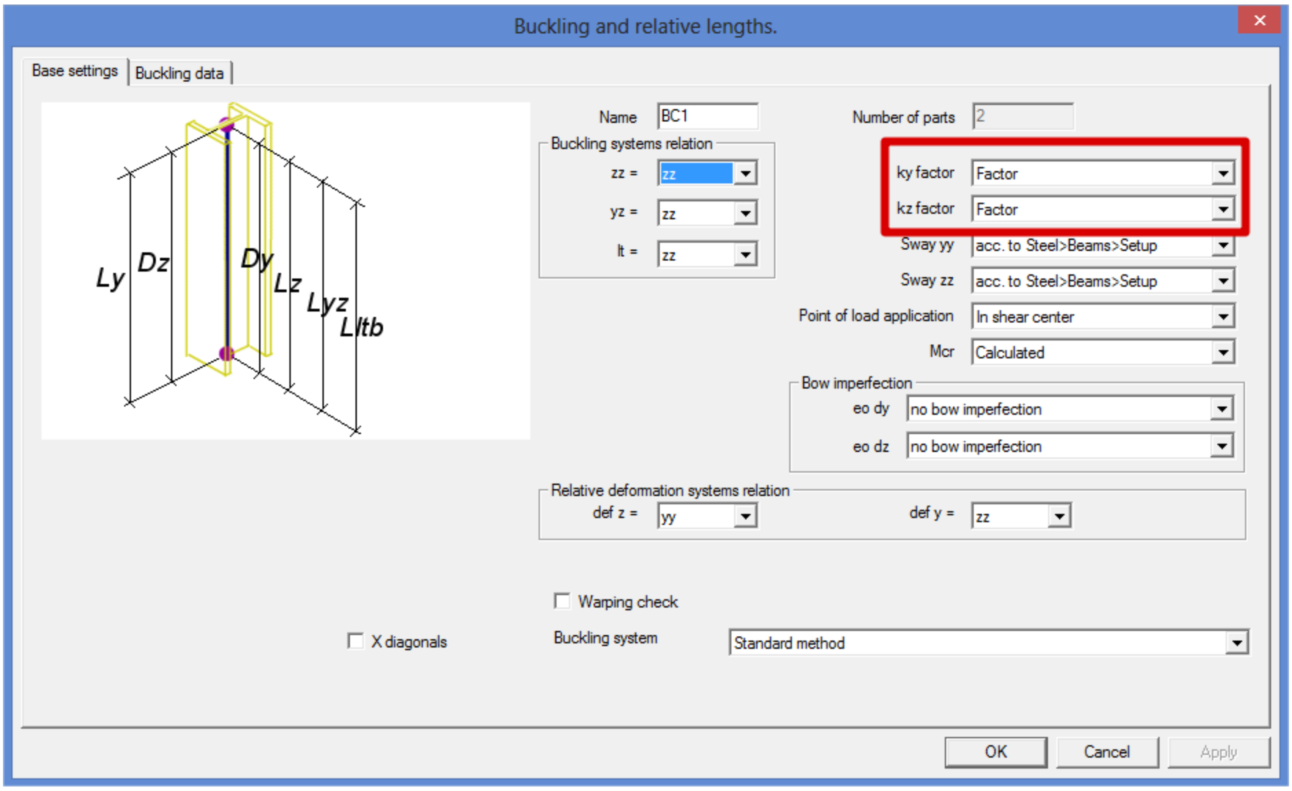
Buckling parameters	yy
Sway type	non-sway
System length L	3,000
Buckling factor k	0,60
Buckling length Lcr	1,802
Critical Euler load Ncr	23558,43
Slenderness Lambda	21,76
Relative slenderness Lambda,rel	0,23
Limit slenderness Lambda,rel,0	0,20
Buckling curve	b
Imperfection Alpha	0,34
Reduction factor Chi	0,99
Buckling resistance Nb,Rd	1250,09

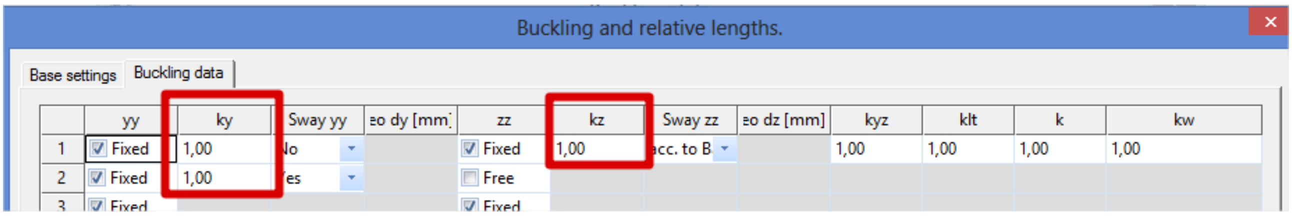
Next the **Amplified Sway Moment Method** can be activated in the linear combinations and the critical alpha value has to be inserted:



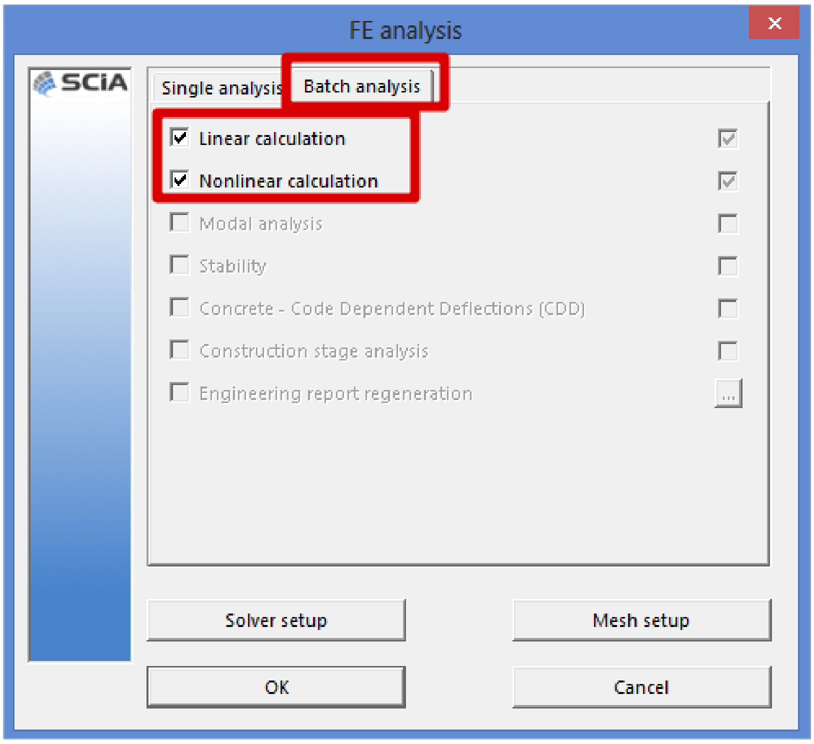
5) Linear calculation

The buckling lengths are taken equal to the system lengths. This can be done by inserting buckling ratios equal to 1 manually:

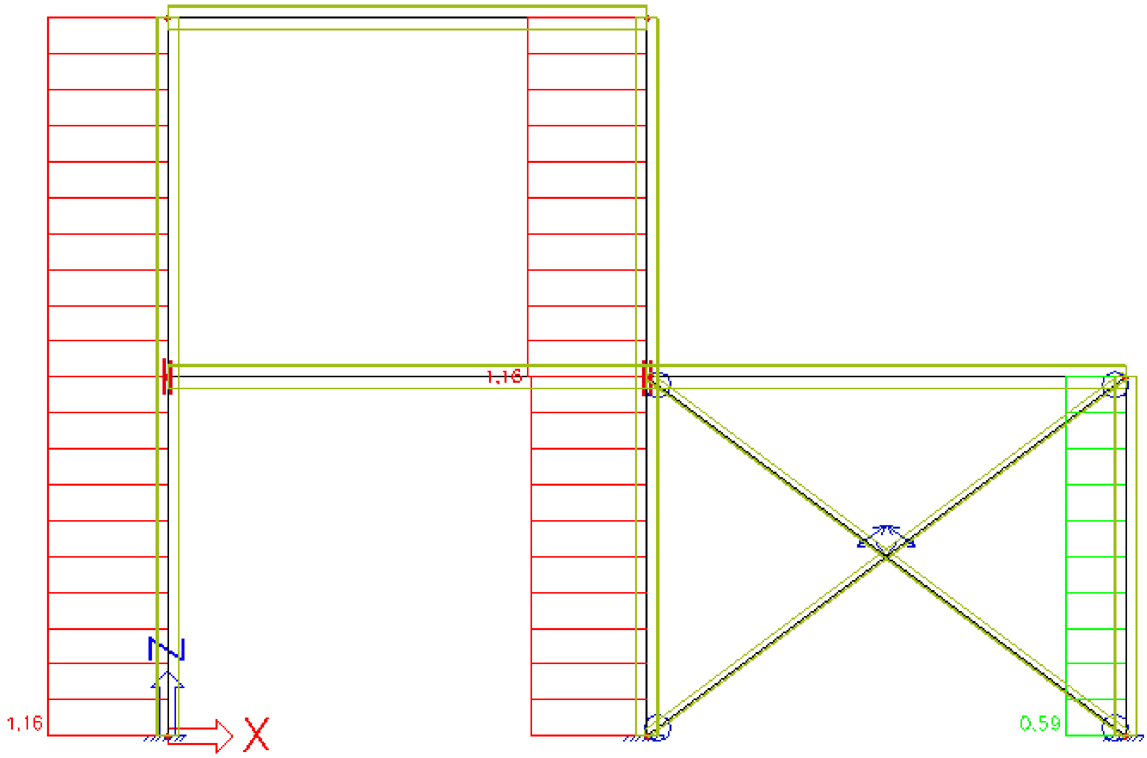




In the next step, the calculation can be performed. Since the global imperfections are inserted in the non-linear combination, the linear as well as the nonlinear calculation need to be performed using the Batch analysis.

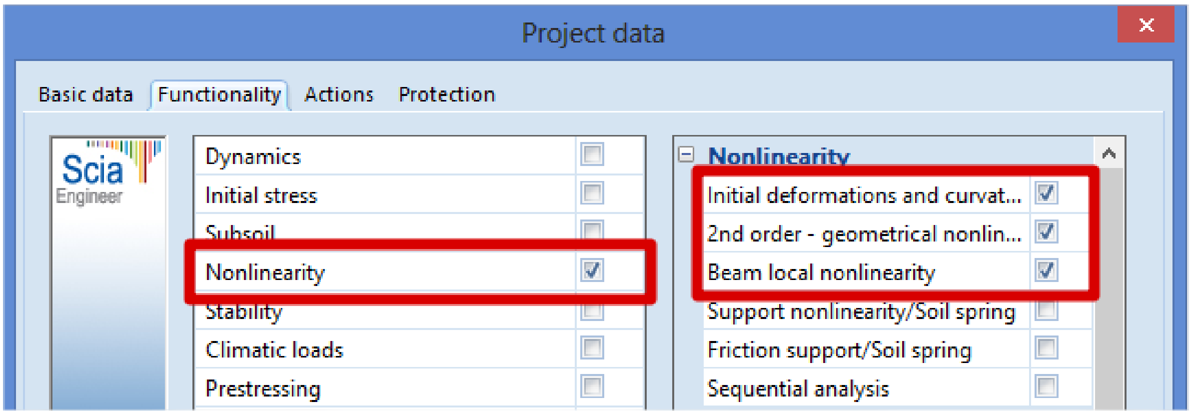


Unity check for ULS
→ Flexural buckling check is decisive



Path 2a 2nd order analysis – Global imperfection (initial inclination)

1) Functionalities

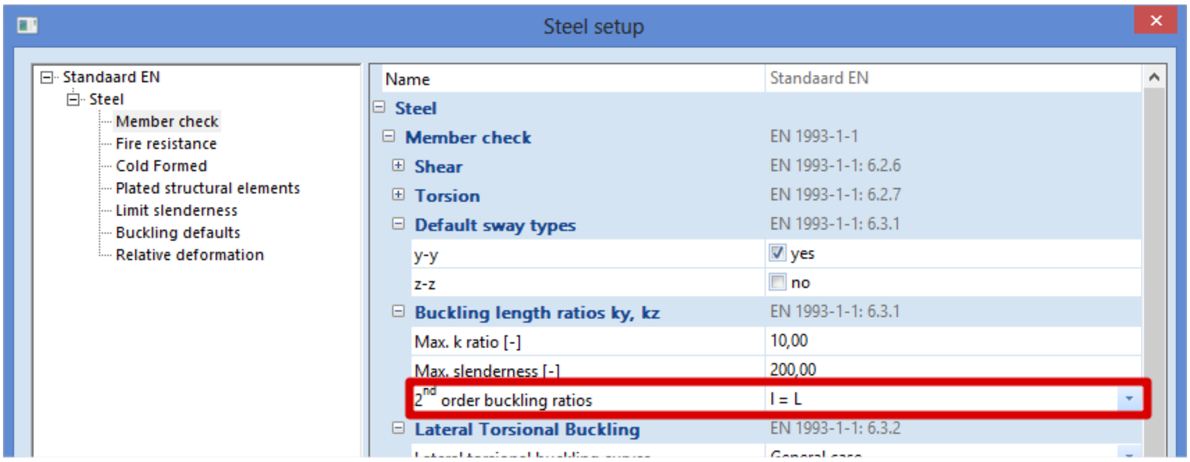


2) Global imperfection

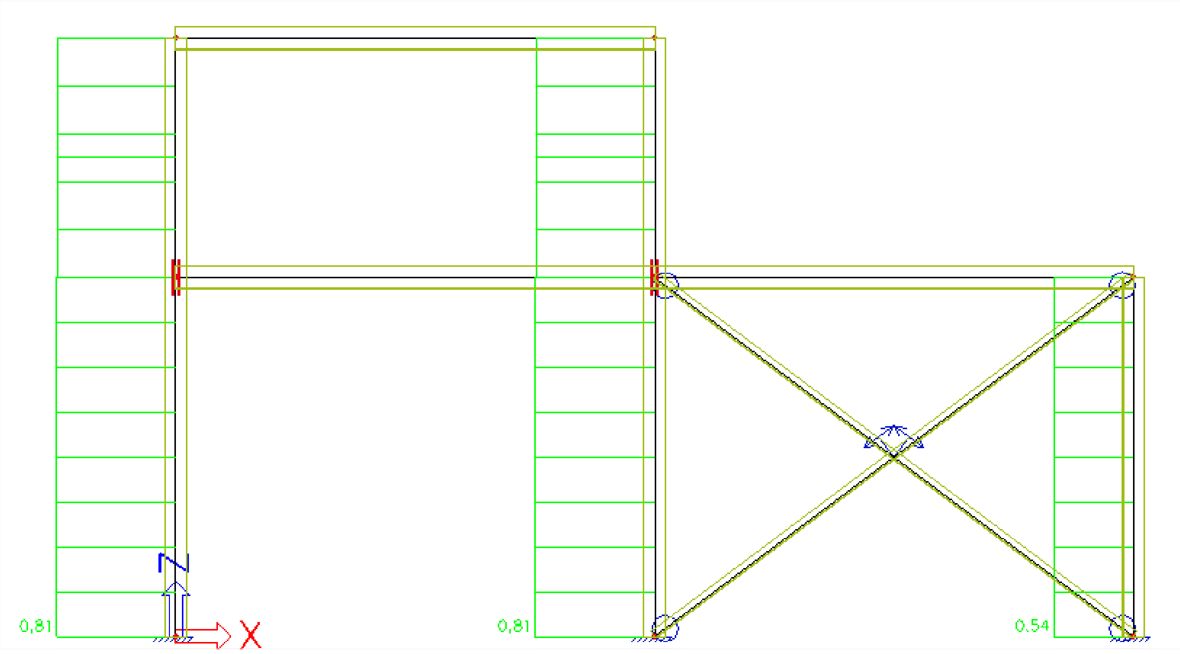
See previous chapter.

3) Steel setup

The whole structure can be considered as non-sway, which means that l ≤ L (or conservatively l = L). SCIA Engineer will execute the flexural buckling check with k = 1.

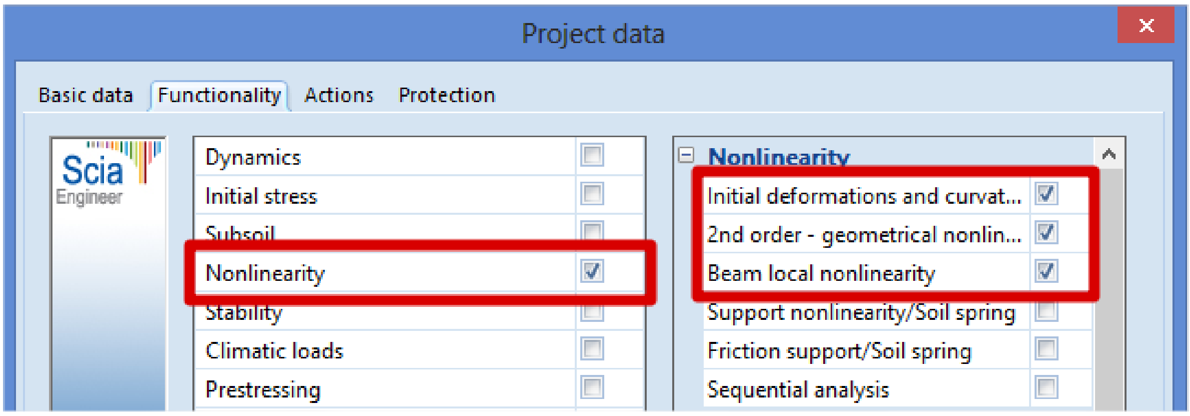


Unity check for ULS
→ Combined compression and bending check is decisive


0,81
0,81
0,54
Z
X

Path 2c 2nd order analysis – Global imperfection (initial inclination) + local imperfection (curvature)

1) Functionalities

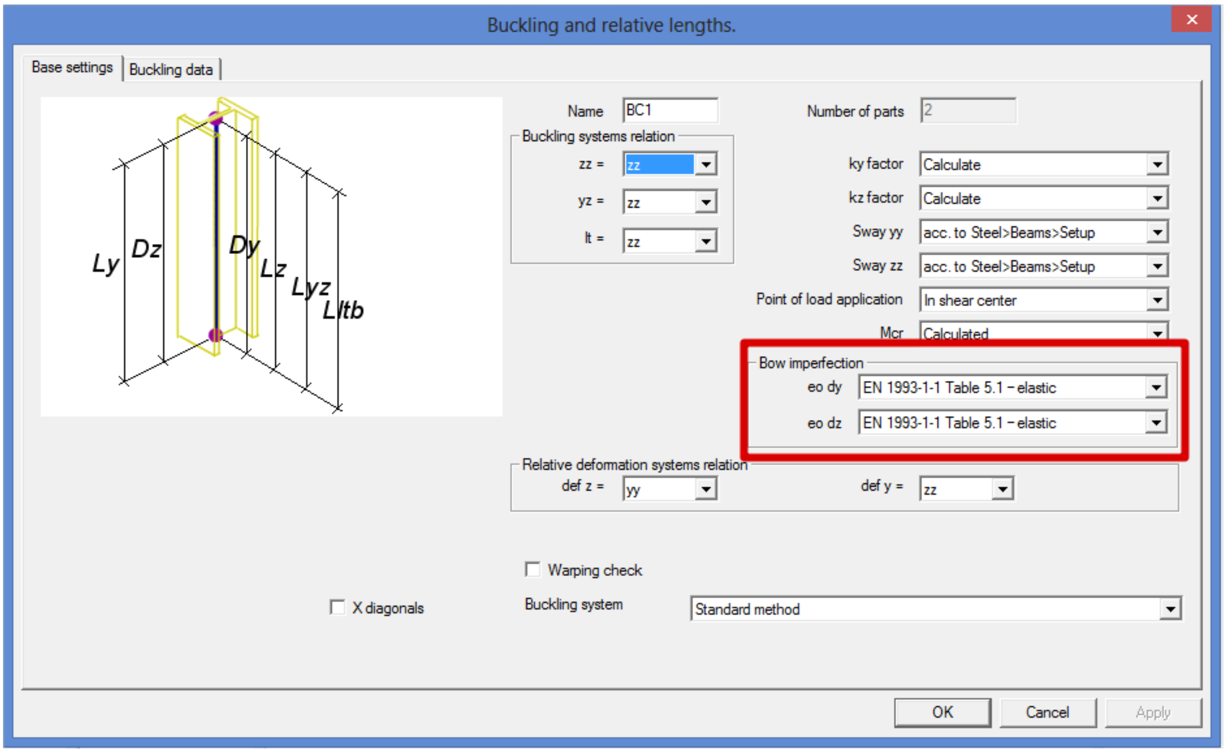


2) Global imperfection

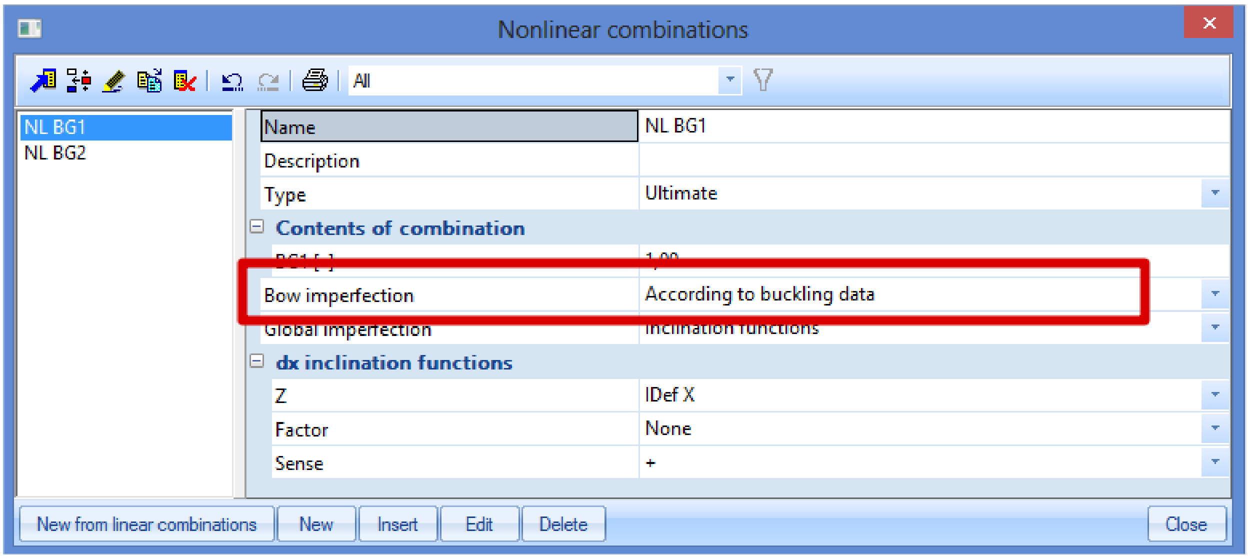
See previous chapters.

3) Bow imperfection

The bow imperfection e0 is inserted in the Buckling parameters:

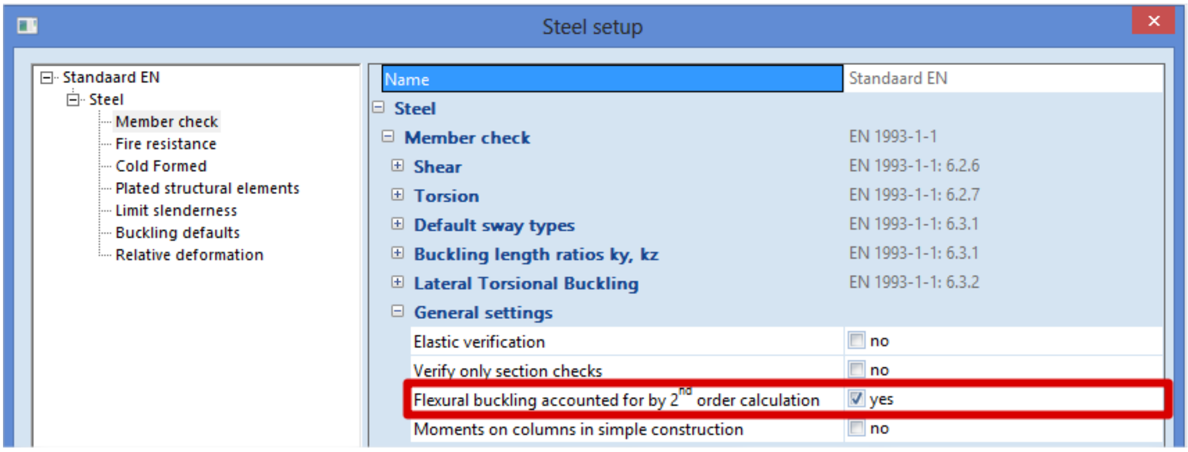


And in the nonlinear combination, a reference is made to this inserted buckling data:

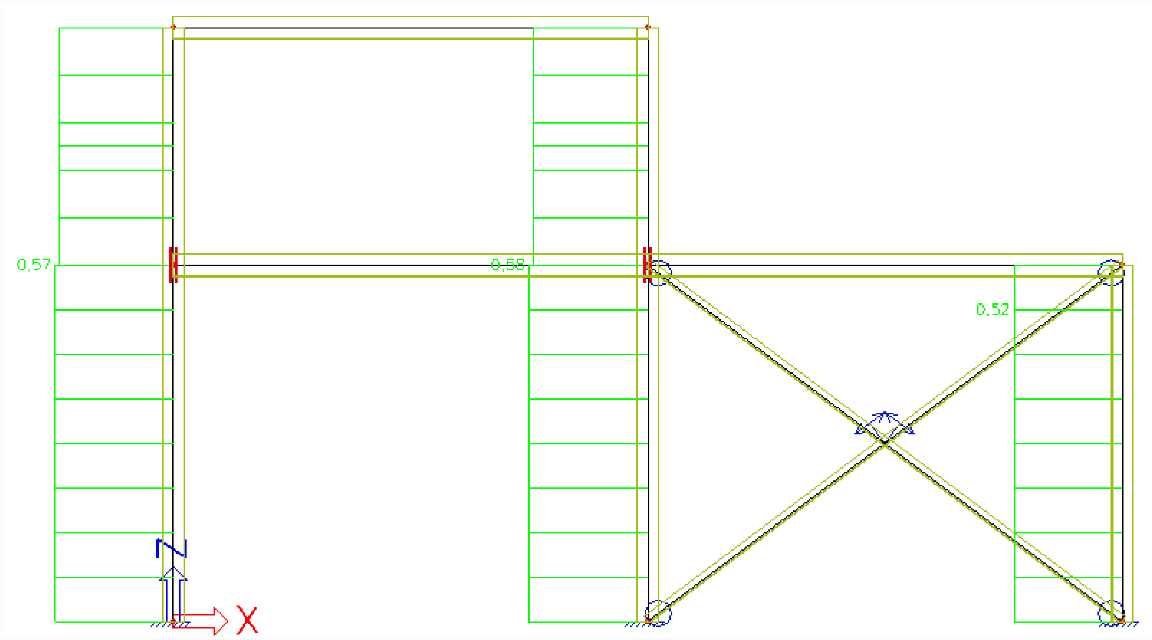


4) Steel setup

According to the Eurocode, it is not necessary anymore to execute the flexural buckling check. SCIA Engineer will execute the flexural buckling check with k = 0,001, so that it will not be decisive.

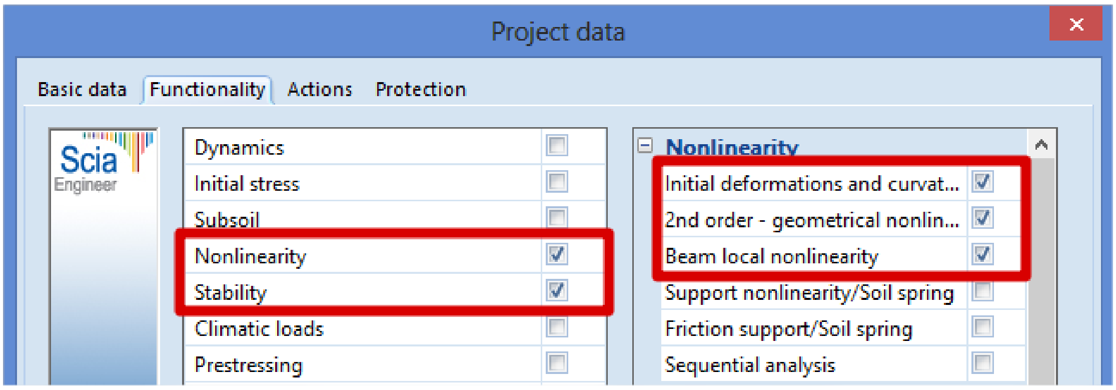


Unity check for ULS
→ Combined compression and bending check is decisive


0,57
0,58
0,52
Z
X

Path 3 2nd order analysis – Buckling form replacing both global + local imperfection

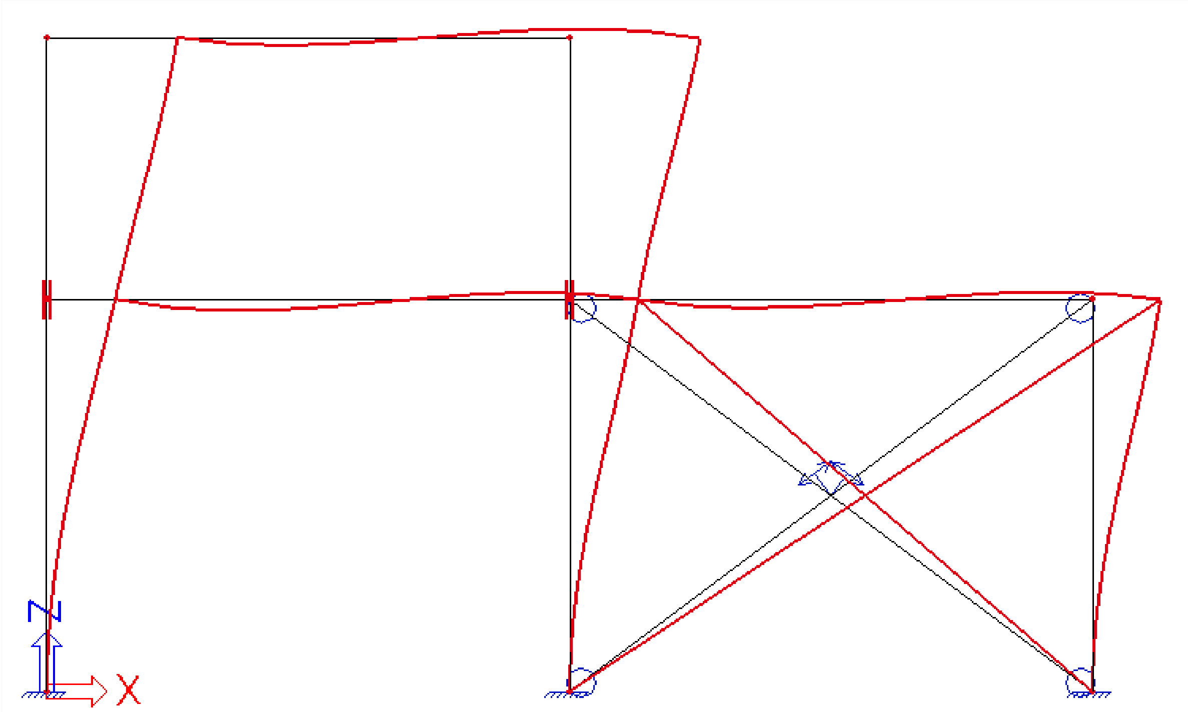
1) Functionalities



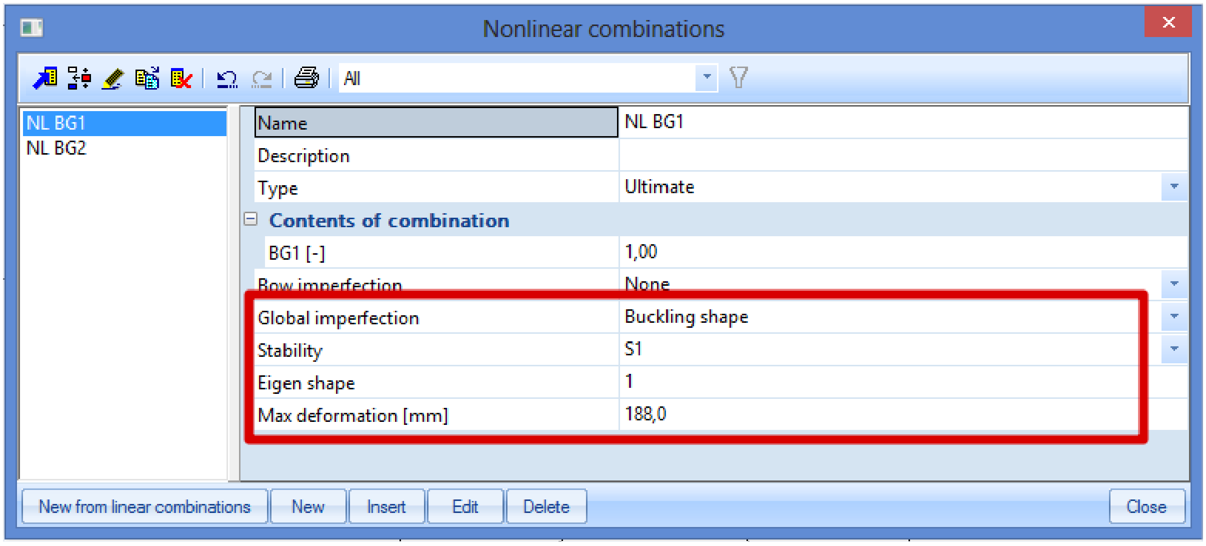
2) Stability calculation

See previous chapters.

The first buckling mode looks like this:

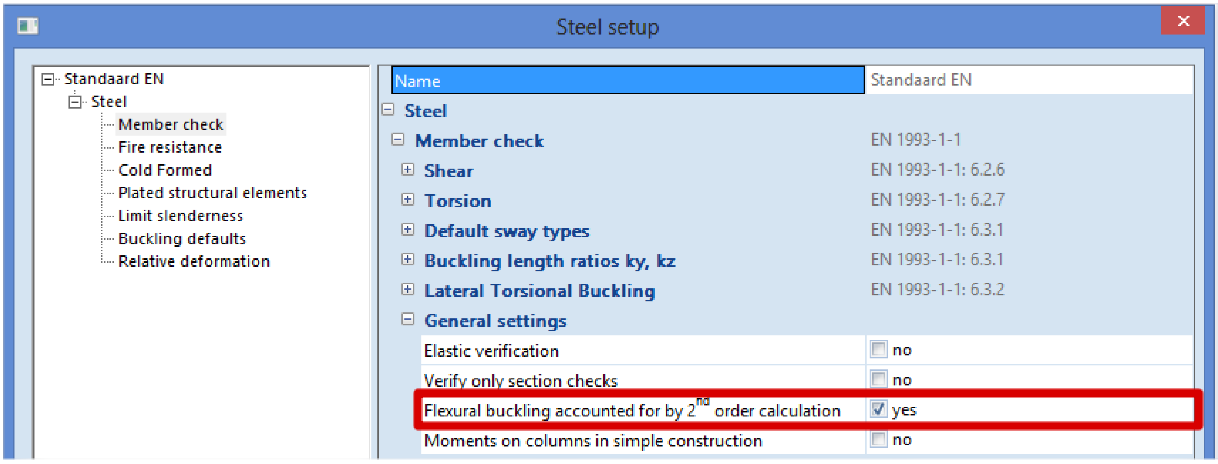


Calculation of η_{init} → To be filled in as 'max. deformation' (see 'Nonlinear combinations' window)



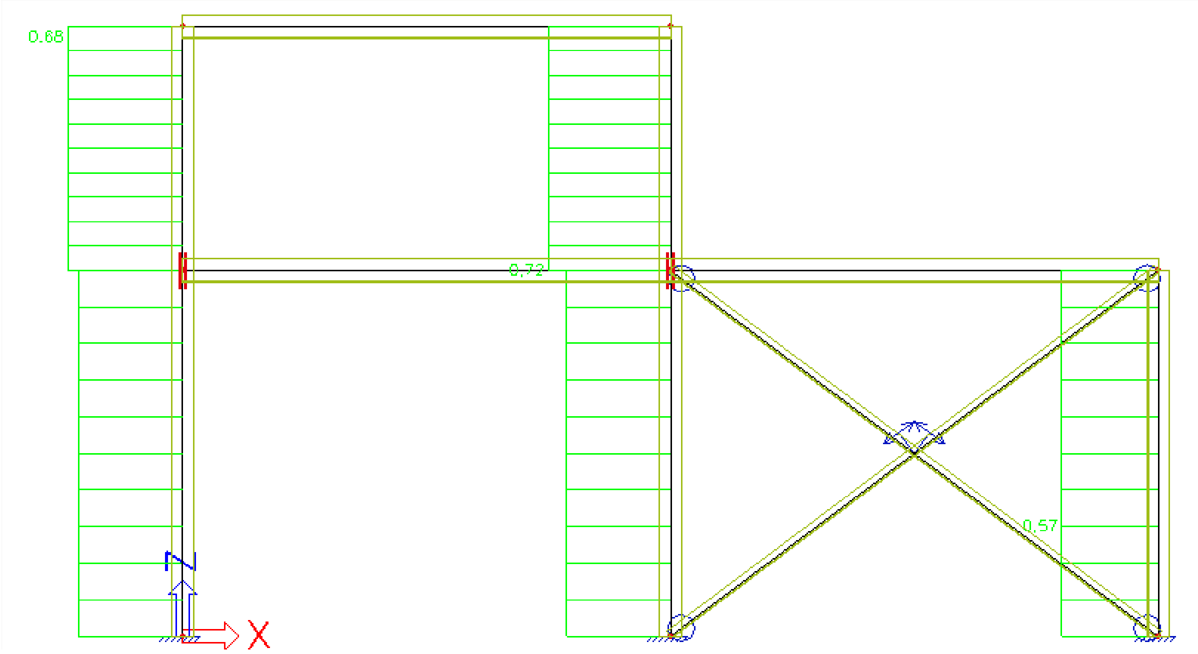
3) Steel setup

According to the Eurocode, it is not necessary anymore to execute the flexural buckling check. SCIA Engineer will execute the flexural buckling check with k = 0,001, so that it will not be decisive.



Unity check for ULS

→ Combined compression and bending check is decisive



SLS Check

Nodal displacement

The nodal displacement defines the maximum global deflections in the vertical and horizontal directions.
The following values are controlled in the example below:

- Limit for horizontal deflections δ is h/150
- Limit for vertical deflection δ_{max} is L/200

Example: Industrial Hall.esa

Look at “Results -> Displacement of nodes” and look at the combination CO2 – SLS.

<u>Horizontal deformation</u>
The maximum displacement in the X direction is 25.3mm on a height of 6.9m.
And in the Y-direction 27.4mm on a height of 8.1m.

Limit for Limit for horizontal deflection δ is h/150

⇨ 6900/150 = 46 mm → 25.3mm < 46 mm → OK

⇨ 8100/150 = 54 mm → 27.4mm < 54mm → OK

<u>Vertical deformation</u>
The maximum displacement in the Z direction is 57.8 mm

Limit for Limit for vertical deflection δ is L/200

⇨ 30000/200 = 150 mm → 57.8 mm < 150 mm → OK

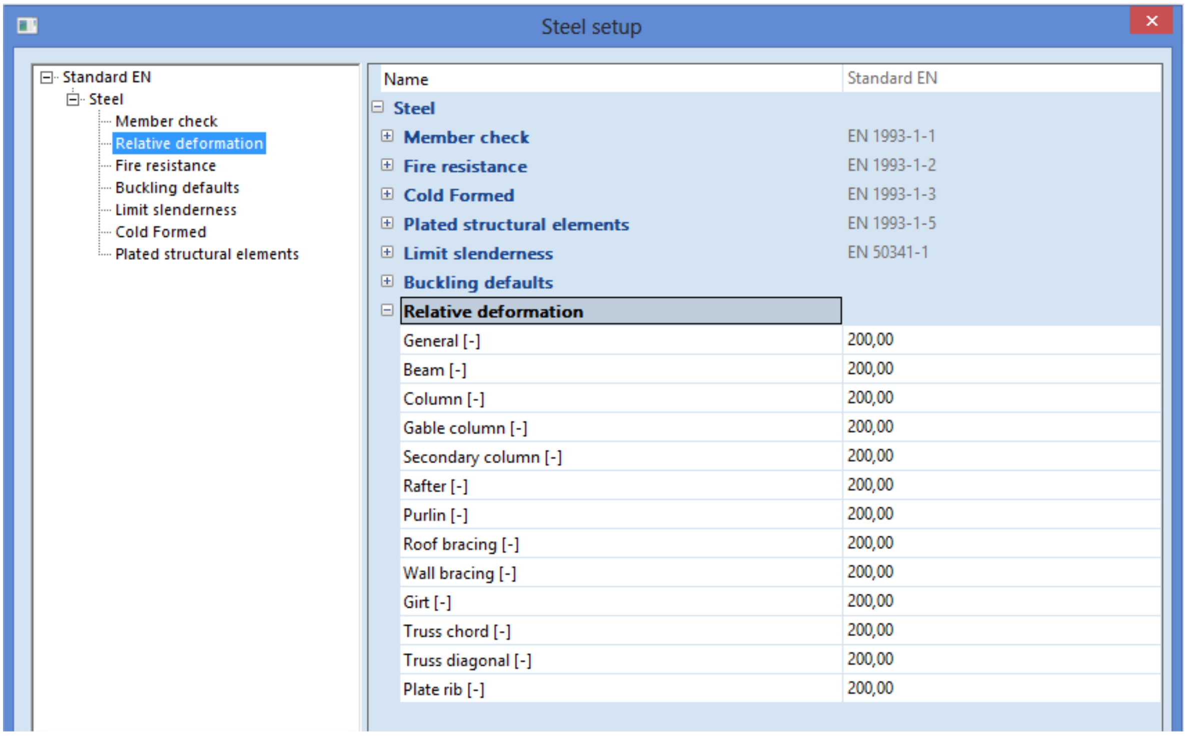
Displacement of nodes

Linear calculation, Extreme : Global
Selection : All
Combinations : CO2

Node	Case	Ux [mm]	Uy [mm]	Uz [mm]	Fix [mrad]	Fiy [mrad]	Fiz [mrad]
N113	CO2/1	**-25,3**	-0,5	-0,2	-1,3	0,4	0,0
N114	CO2/2	**25,3**	-0,5	-0,2	-1,3	-0,4	0,0
N70	CO2/3	0,0	**-27,4**	0,0	1,5	0,0	1,2
N60	CO2/4	0,0	**27,4**	0,0	-1,5	0,0	1,2
N109	CO2/5	12,7	0,1	**-57,8**	-0,1	-0,2	0,0
N82	CO2/6	9,8	-8,6	**13,8**	-24,8	-0,2	1,7
N80	CO2/6	9,5	-9,6	0,0	**-27,6**	0,0	1,2
N179	CO2/7	-9,5	9,6	0,0	**27,6**	0,0	1,2
N116	CO2/8	-9,2	-0,1	-21,3	-0,4	**-5,8**	0,0
N112	CO2/5	9,2	-0,1	-21,3	-0,4	**5,8**	0,0
N95	CO2/7	-9,6	-9,0	0,0	-26,0	0,0	**-4,0**
N87	CO2/6	9,6	-9,0	0,0	-26,0	0,0	**4,0**

Relative deformation

For each beam type, limiting values for the relative deflections are set, using the menu “Steel -> Beams -> Setup -> Relative deformations”:



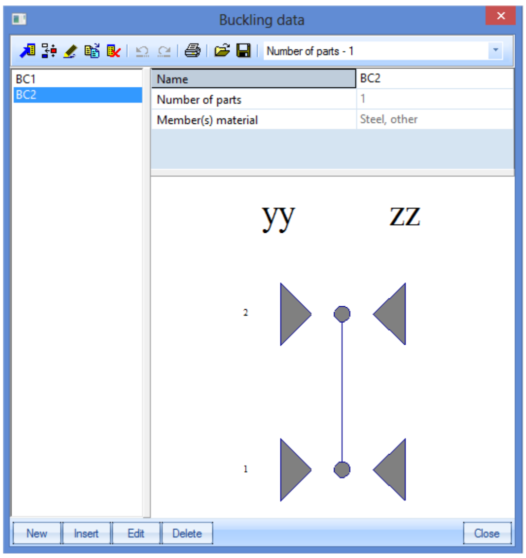
With the option “Steel -> Beams -> SLS Checks - Relative deformation”, the relative deformations can be checked. The relative deformations are given as absolute value, relative value related to the span, or as unity check related to the limit for the relative value to the span.

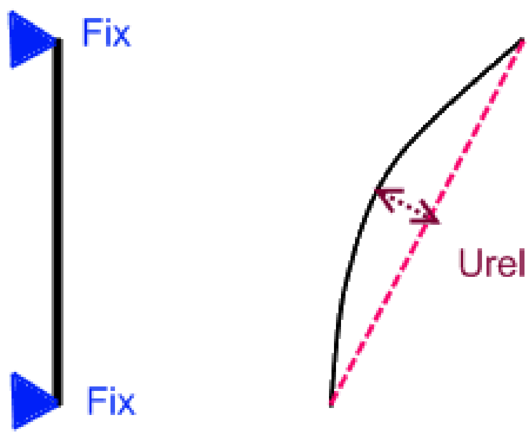
The span is defined in the menu “Buckling and relative lengths”.

Two options for this span are possible and explained below:

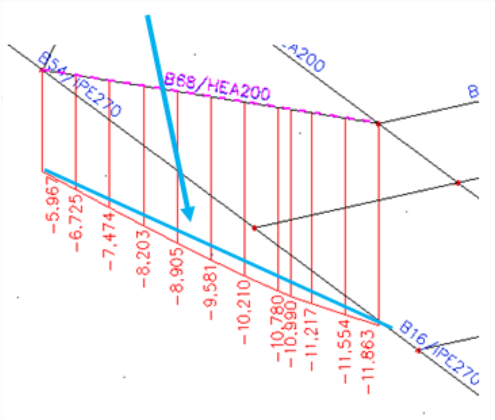
2 nodes supported

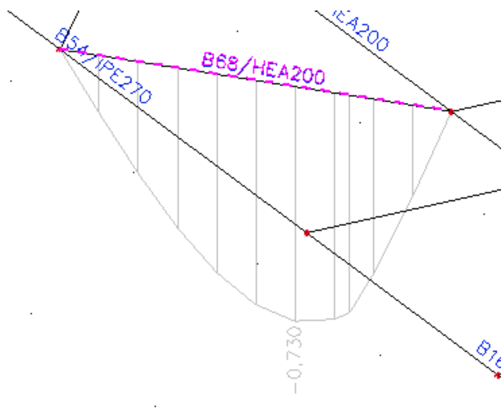
When two nodes are supported in this menu as shown below, the deformation is taken as the maximum deflection of the beam in comparison with a line connecting the two end nodes:





In below an example on this principle:





Calculation for this relative deformation:

Deformation in the beginning of the beam = 5,967mm and at the end = 11,863mm.

Maximum deformation is at 0,979m from the beginning at the beam.
So this point has already displaced 9,5mm (see blue line at the picture):

$$uz_{blue\ line;0{,}979m} = 5{,}967mm + \frac{0{,}979m}{1{,}632m} * (11{,}863mm - 5{,}967mm) = 9{,}504mm$$

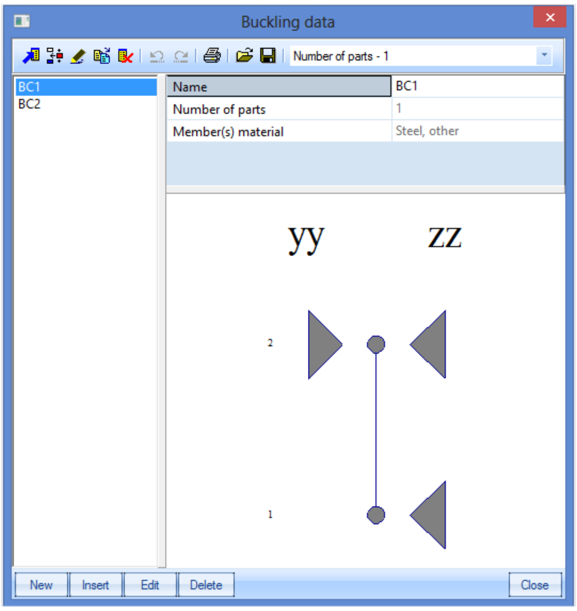
And $u_{z,\ relative}$ = 10,2mm – 9,5mm = 0,7mm

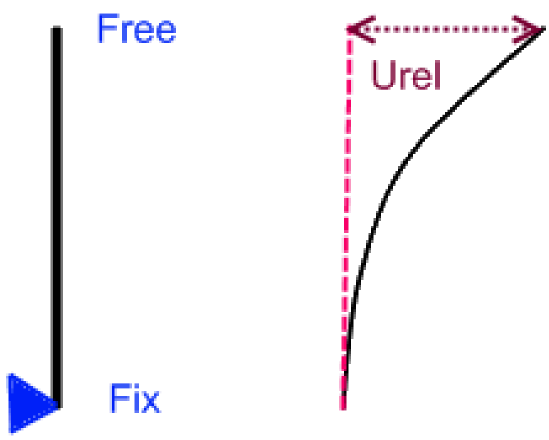
And suppose the length of this beam = 1632mm:

$$\Rightarrow \quad rel\ uz = \frac{0{,}73mm}{1632mm} = 1/2236$$

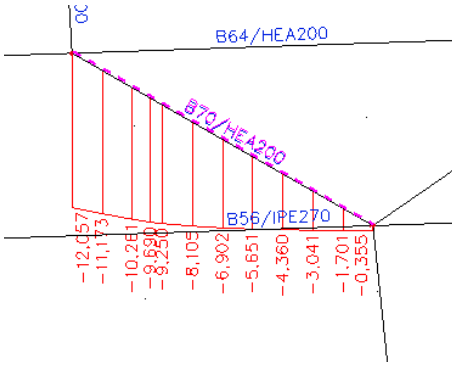
1 node supported and the other free

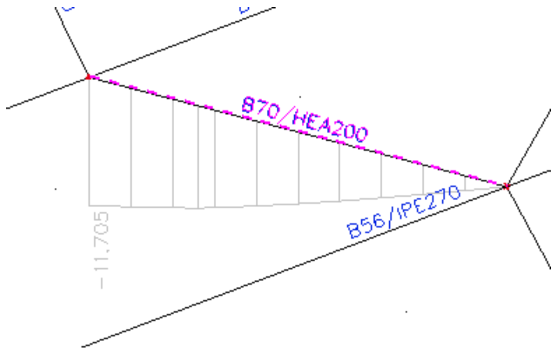
This is the case if one end node is free:





In below an example on this principle:





No the maximal relative deformation is taken as the deformation minus the deformation of the fixed node.

And $u_{z,\ relative}$ = 12,057mm – 0,355mm = 11,7mm

And suppose the length of this beam = 1632mm:

$$\Rightarrow \quad rel\ uz = \frac{11{,}7mm}{1632mm} = 1/139$$

Now this principle is shown at the example of the industrial hall.

Example: Industrial hall

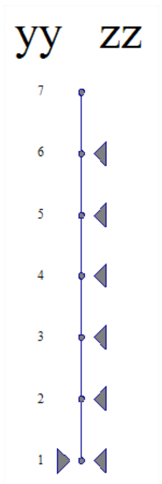
Consider beam B10.

Relative deformation

Linear calculation, Extreme : Global, System : Principal
Selection : B10
Combinations : CO2

Member	dx [m]	Case - combination	uy [mm]	Rel uy [1/xx]	uz [mm]	Rel uz [1/xx]	Check uy [-]	Check uz [-]
B10	15,075	CO2/4	**-0,2**	**1/3103**	-28,0	1/539	**0,06**	0,37
B10	15,075	CO2/9	**0,2**	**1/3170**	9,5	1/1589	**0,06**	0,13
B10	14,325	CO2/8	0,0	0	**-57,8**	**1/261**	0,00	**0,77**
B10	12,029	CO2/6	0,0	1/10000	**13,2**	**1/1142**	0,00	**0,18**

Around the y-y axis (in the z-direction), only the first node has been supported:



Length = 15,075 m

Deformation uz = 57.8 mm (and 0 mm at the beginning of the beam)

$uz_{relative}$ = 57.8 mm – 0.0 mm = 57.8 mm

$uz_{relative}$ / Length = 57.8 mm / 15075 mm = 1/261

Check = $\frac{1/261}{1/200}$ = 0,77

Fire resistance Check - Not in concept edition

For the fire resistance calculation a Professional or an Expert edition is necessary. The fire resistance check has been inputted in module esasd.05.xx (esasd.05.01 for the EC-EN).

General

The fire resistance check in SCIA Engineer has been inputted following the EN 1993-1-2 – simple calculation model.

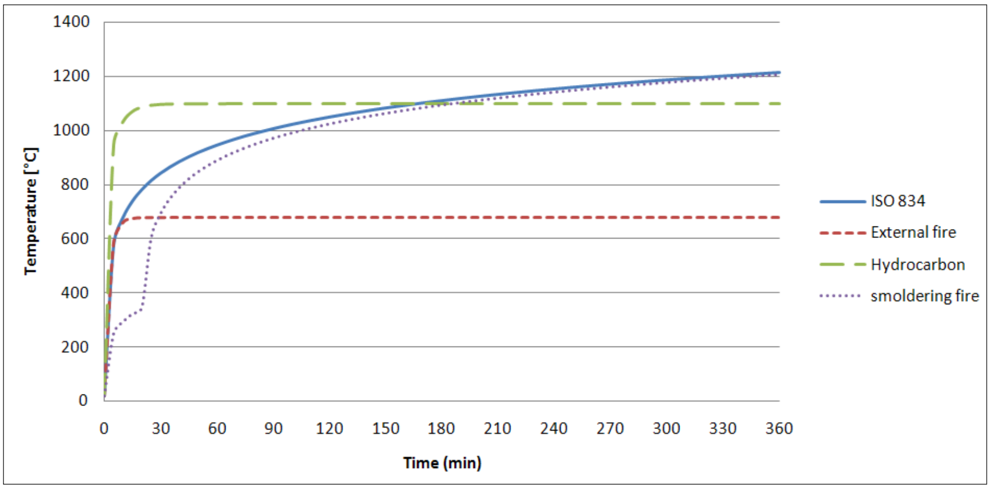
For a selected temperature curve, the temperature in the material after a required period is calculated. And with this material temperature, the material characteristics are adapted.

The required levels of fire safety depend on factors such as:

- type of occupancy
- height and size of the building
- effectiveness of fire brigade action
- active measures such as vents and sprinklers

Temperature time curves

The user can choose between 4 nominal temperature-time curves in SCIA Engineer:



Standard temperature-time curve: $\Theta_g = 20 + 345\, log_{10}(8\,t + 1)$ [°C]

$\alpha_c = 25\ \mathrm{W/m^2K}$

External fire curve: $\Theta_g = 660\,(1 - 0{,}687\, e^{-0{,}32\,t} - 0{,}313\, e^{-3{,}8\,t}) + 20$ [°C]

$\alpha_c = 25\ \mathrm{W/m^2K}$

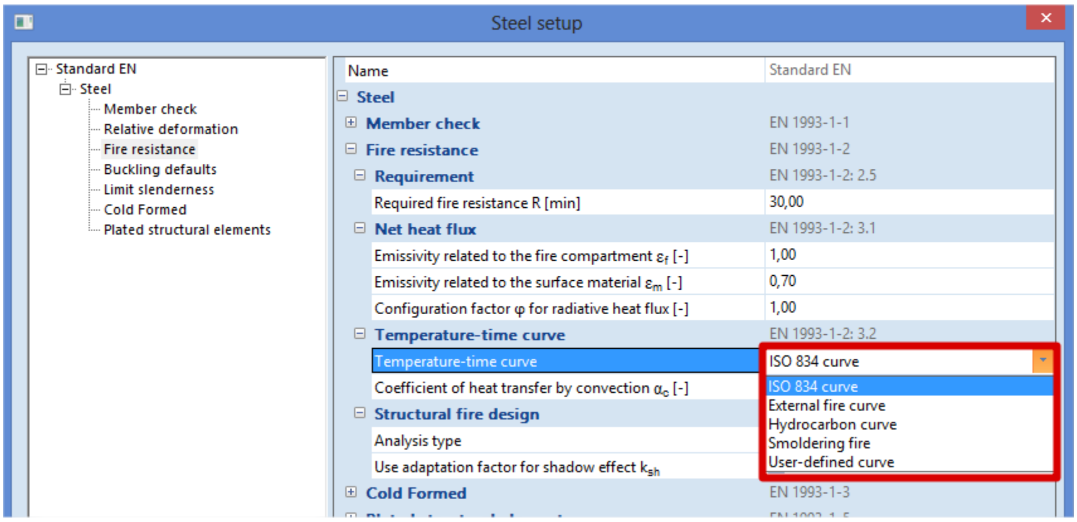
Hydrocarbon curve : $\Theta_g = 1080\,(1 - 0{,}325\, e^{-0{,}167\,t} - 0{,}675\, e^{-2{,}5\,t}) + 20$ [°C]

$\alpha_c = 50\ \mathrm{W/m^2K}$

Smoldering fire: $\Theta_g = 154\sqrt[4]{t} + 20$ [°C]

During 20 minutes followed by the standard ISO 834 curve

This can be set in “Steel -> Beams -> Steel setup”:



This is the temperature of the air during the time.

Steel temperature

Afterwards the steel temperature will be calculated after a certain time with the following formulas. This steel temperature will be assumed as a uniform temperature in the whole section:

Unprotected steel member:

$$\Delta\theta_{a,t} = k_{sh}\frac{A_m/V}{c_a\rho_a}\dot{h}_{net}\Delta t$$

Protected steel member:

$$\Delta\theta_{a,t} = \frac{\lambda_p A_p/V}{d_p c_a \rho_a}\cdot\frac{\theta_{g,t}-\theta_{a,t}}{(1+\phi/3)}\Delta t - \left(e^{\frac{\phi}{10}} - 1\right)\Delta\theta_{g,t}$$

Where:

k_{sh} Correction factor for shadow effect

For I-sections under nominal fire actions: $k_{sh} = 0.9\,[A_m/V]_b/[A_m/V]$

All other cases: $k_{sh} = [A_m/V]_b/[A_m/V]$

A_m/V Section factor for unprotected steel members [1/m]
In table 4.2 (EN 1993-1-2) some section factors are calculated for unprotected steel members.

$[A_m/V]_b$ the box value for the section factor

$\dot{h}_{net}$ the design value on the net heat flux per unit area [W/m²]
This value should be obtained from EN 1991-1-2 with ε_f = 1,0 and ε_m = 0,7.

ϕ $= \frac{c_p\rho_p}{c_a\rho_a}\mathrm{d_p}\cdot A_p/V$

A_p/V section factor – see also table 4.3. En 1993-1-2

And the netto heat flux can be calculated according EN 1991-1-2 article 3.1:

$$\dot{h}_{net} = \dot{h}_{net,r} + \dot{h}_{net,c}$$

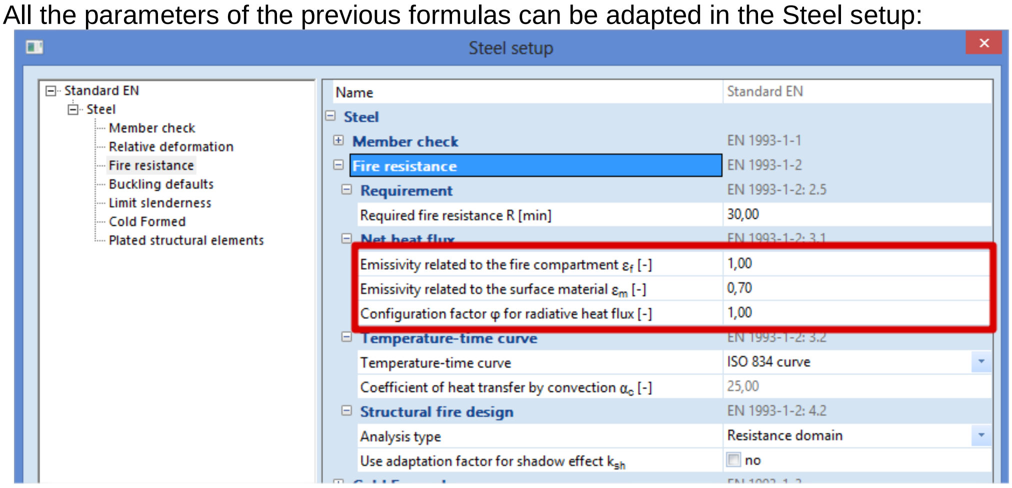
$\dot{h}_{net,c}$ = heat transfer by convection
$= \alpha_c(\Theta_g - \Theta_m)$ [W/m²]

$\dot{h}_{net,r}$ = heat transfer by radiation
$= \Phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma \cdot [(\Theta_r + 273)^4 - (\Theta_m + 273)^4]$ [W/m²]

With

α_c	Coefficient of heat transfer by convection [W/m²K]
Θ_g	The gas temperature in the vicinity of the fire exposed member [°C] This temperature may be adopted as nominal temperature-time curves as given below
Θ_m	The surface temperature of the member [°C]
σ	Stephan Boltzmann constant (=5,67 $\cdot 10^{-8}$ W/m²K⁴)
ε_m	Surface emissivity of fire = 0,7 (EN 1993-1-2)
ε_f	Emissivity of fire = 1
Φ	Configuration factor - Φ = 1,0. A lower value may be chosen to take account of so called position and shadow effects (calculation is given in EN 1991-1-2 – Annex G).
Θ_r	Is the effective radiation temperature of the fire environment [°C] In case of fully fire engulfed members, the radiation temperature Θ_r may be represented by the gas temperature Θ_g around that member.

All the parameters of the previous formulas can be adapted in the Steel setup:



Steel properties

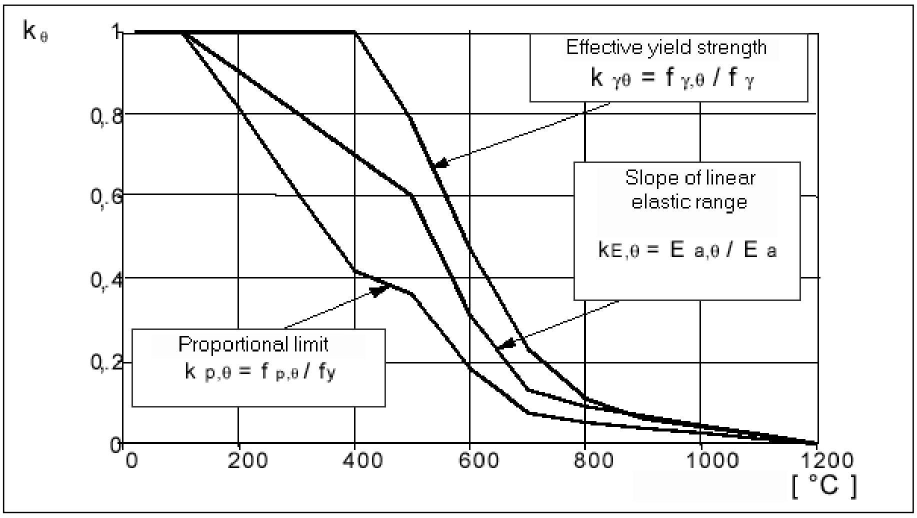
Most of the steel properties will change by a different temperature, so once the steel temperature is known, the steel properties can be calculated. In below the properties for carbon steel from EN 1993-1-2, art.3 are used. The properties for stainless steel can be found in EN 1993-1-2: 2005: Annex C.

Reduction factors

Effective yield strength, relative to yield strength at 20°C: $k_{y,\theta} = f_{y,\theta}/f_y$

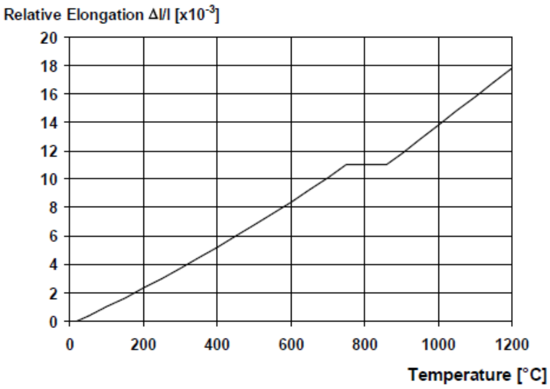
Proportional limit, relative to yield strength at 20°C: $k_{P,\theta} = f_{P,\theta}/f_y$

Slope of linear elastic range, relative to slope at 20°C: $k_{E,\theta} = E_{a,\theta}/E_a$



Thermal elongation

The relative thermal elongation of steel $\Delta l/l$ should be determined from the following:



For $20°C \leq \theta_a < 750°C$:

$$\frac{\Delta l}{l} = 1{,}2\ x\ 10^{-5}\theta_a + 0{,}4\ x\ 10^{-8}\theta_a^2 - 2{,}416\ x\ 10^{-4}$$

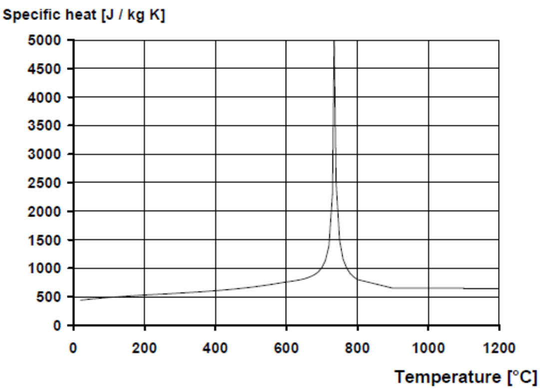
For $750°C \leq \theta_a < 860°C$:

$$\frac{\Delta l}{l} = 1{,}1\ x\ 10^{-2}$$

For $860°C \leq \theta_a < 1200°C$:

$$\frac{\Delta l}{l} = 2\ x\ 10^{-5}\ \theta_a - 6{,}2\ x\ 10^{-3}$$

Specific heat



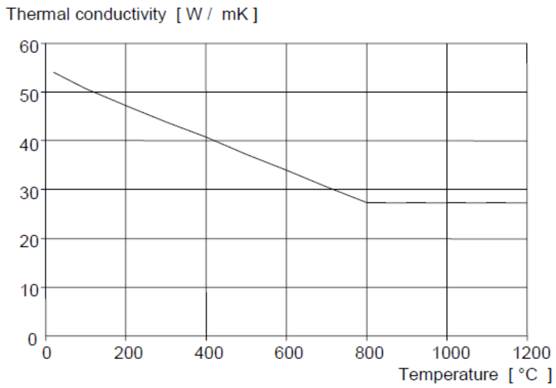
For $20°C \leq \theta_a < 600°C$: $c_a = 425 + 7{,}73\ x\ 10^{-1}\ \theta_a - 1{,}69\ x\ 10^{-3}\theta_a^2 + 2{,}22\ x\ 10^{-6}\theta_a^3\quad J/kgK$

For $600°C \leq \theta_a < 735°C$: $c_a = 666 + \frac{13002}{738-\theta_a}\ J/kgK$

For $735°C \leq \theta_a < 900°C$: $c_a = 545 + \frac{17820}{\theta_{a-731}}\ J/kgK$

For $900°C \leq \theta_a < 1200°C$: $c_a = 650\ J/kgK$

Thermal conductivity



For $20°C \leq \theta_a < 800°C$:

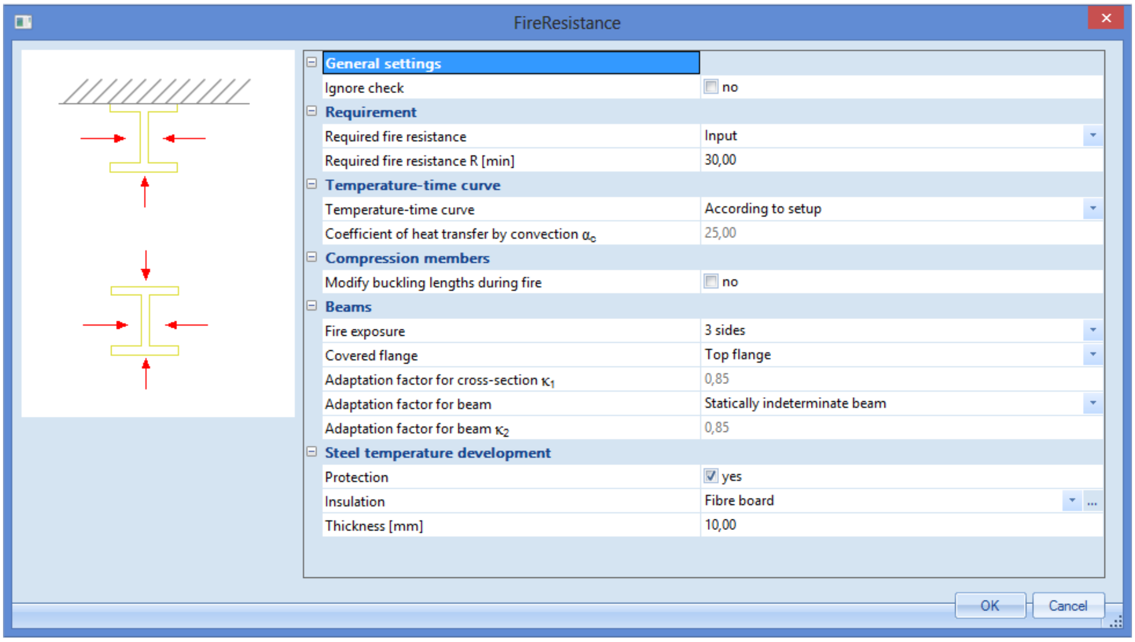
$$\lambda_a = 54 - 3{,}33 \; x \; 10^{-2} \, \theta_a \;\; W/mK$$

For $800°C \leq \theta_a < 1200°C$:

$$\lambda_a = 27{,}3 \;\; W/mK$$

Fire resistance properties in SCIA Engineer

In SCIA Engineer the user can input fire resistance properties on a steel member by “Steel -> Beams -> Member Check data -> Fire resistance”:



Where:

Required fire resistance:	Inputted by the user or according to the steel setup
Required fire resistance R:	Specifies the required resistance (input).
Temperature-time curve:	According to the steel setup, or overruled for the selected member.
Coefficient of heat transfer by convection αc:	Can only be changed by a user defined temperature time curve.
Modified buckling lengths during fire:	The buckling ratios can be inputted manually for the fire resistance check.
Fire exposure:	The section may be exposed to fire on all or only three sides.
Covered flange:	When a section is exposed to fire on only three sides, the covered flange can be chosen here.

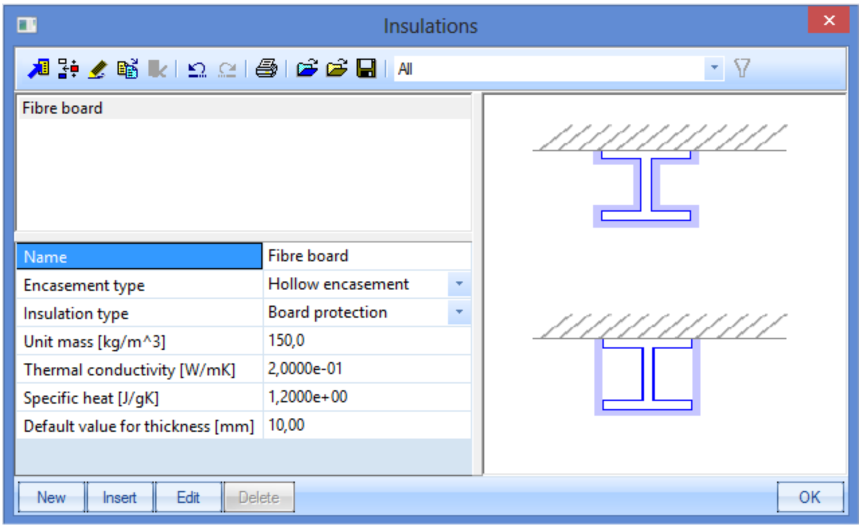
Adaption factor for cross-section κ1: This parameters is the adaptation factor for non-uniform temperature distribution across a cross-section. This factor κ_1 is used for the check on the design moment resistance $M_{fi,\theta,Rd}$.

Adaption factor for beam: At the supports of a statically indeterminate beam or all other cases.

Adaption factor for beam κ2: This parameters is the adaptation factor for non-uniform temperature distribution along the beam. This factor κ_2 is used for the check on the design moment resistance $M_{fi,\theta,Rd}$.

Protection: Yes or no.

Insulation: Here the insulation properties can be inputted.



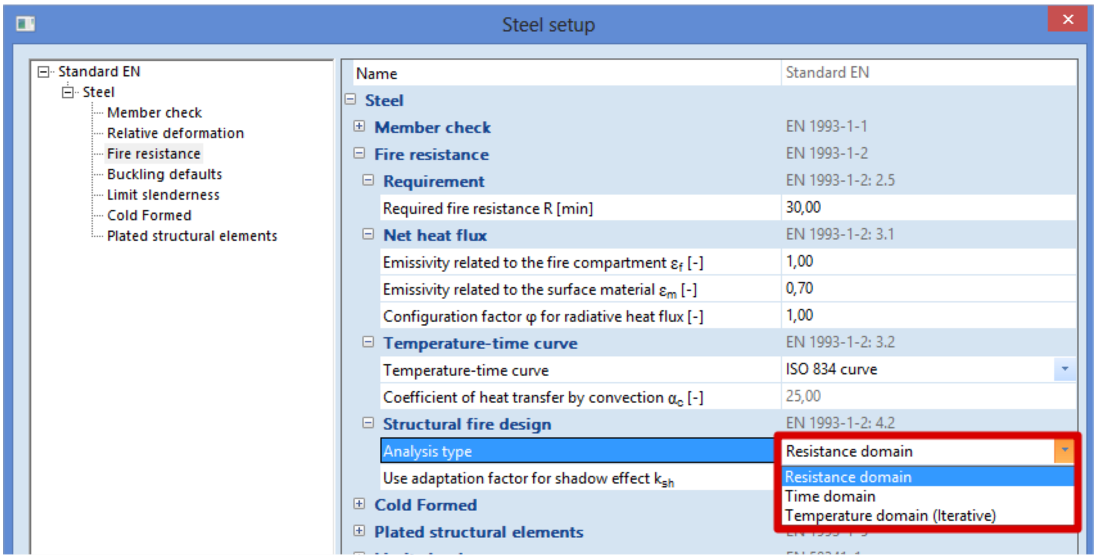
Thickness [mm]: Thickness of the insulation.

Resistance domain

In SCIA Engineer 3 calculation methods are implemented:

- Resistance domain
- Time domain.
- Temperature domain (iterative).

The choice between those analysis types can be made in “Steel -> Beams -> Steel setup”:



In this chapter the fire resistance check following the Resistance domain will be explained.

Principle

The user will choose the used temperature time curve and will input a required fire resistance time. After this time the temperature of the gas and afterwards of the steel will be calculated.

With this steel temperature the reduced properties will be calculated and a fire resistance check according to EN 1993-1-2, art. 4 will be executed with those adapted steel properties. This check will result in a unity check, which is the fire resistance check for the resistance domain.

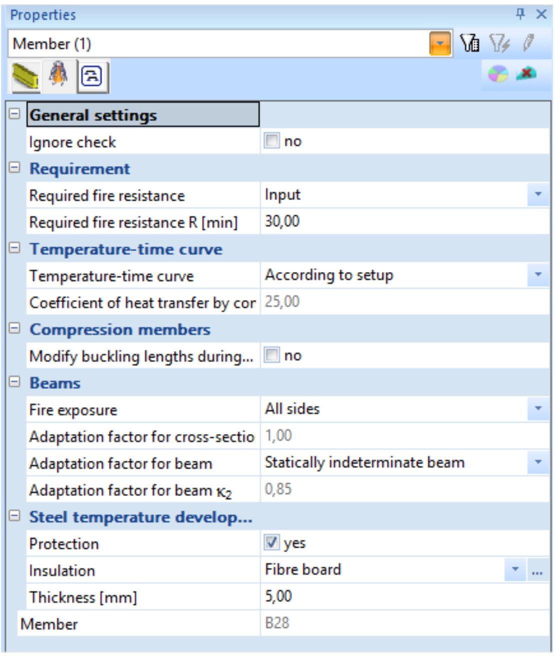
Example in SCIA Engineer

This principle is explained with an example in SCIA Engineer.

Example: Industrial hall.esa

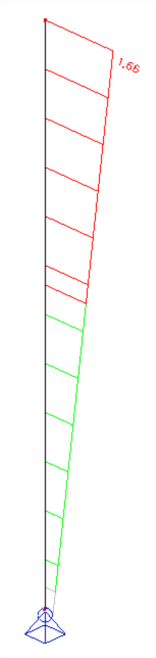
Consider member B28:

Following fire resistance properties are inputted on this column:



So the fire resistance will be checked after 30 minutes (= 1800 seconds) with a fibre board protection and the buckling factors are taken equal as the buckling factors of the normal steel code check.

The fire resistance check is executed via “Steel -> Beams -> ULS check -> Check - fire resistance” for this column and for combination “CO3 - Fire”, resulting in a unity check of 0,95:



When looking at the detailed output, this calculation is given by SCIA Engineer:

First the partial safety factors are given:

Partial safety factors	
Gamma M0 for resistance of cross-sections	1,00
Gamma M1 for resistance to instability	1,00
Gamma M2 for resistance of net sections	1,25
Gamma M,fi for resistance to fire	1,00

Afterwards the material properties (not adapted by the temperature) are given:

Material		
Yield strength fy	235,0	MPa
Ultimate strength fu	360,0	MPa
Fabrication	Rolled	

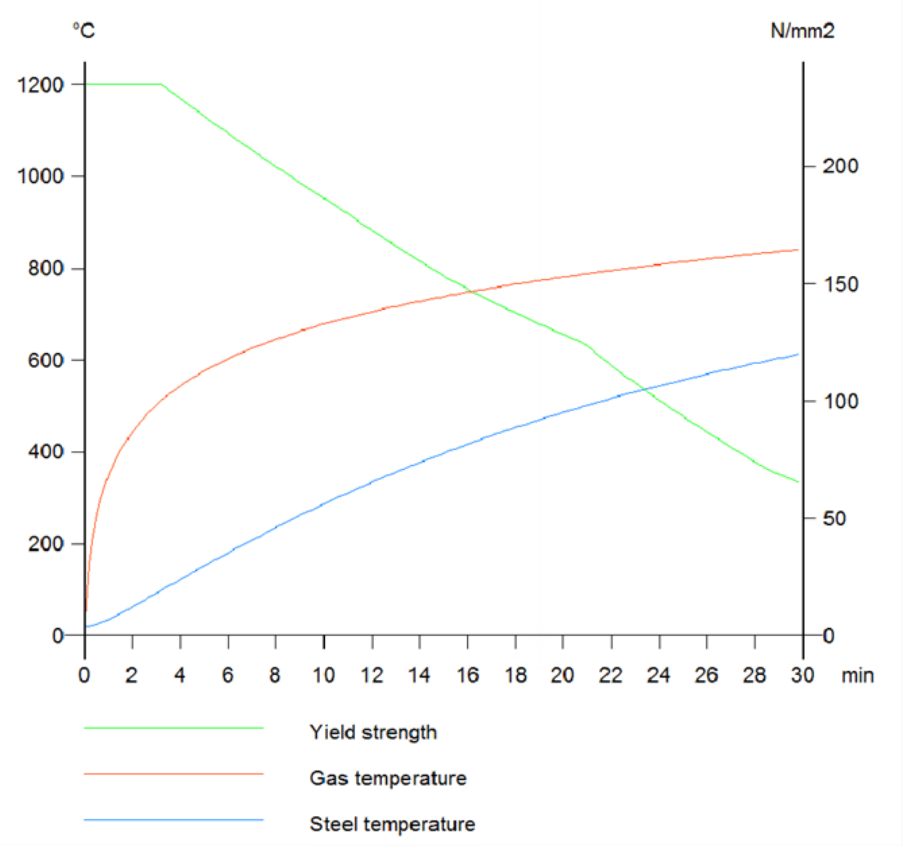
And the fire resistance properties as inputted in SCIA Engineer. Here is also indicated that the fire resistance check has been executed after 30 minutes of fire.

Fire resistance
Verification in Resistance domain according to EN 1993-1-2 article 4.2.3

Fire resistance		
Temperature-time curve	ISO 834 curve	
Coefficient of heat transfer by convection alpha,c	25,00	W/m²K
Emissivity related to fire compartment epsilon,f	1,00	
Emissivity related to surface material epsilon,m	0,70	
Configuration factor for radiation heat flux phi	1,00	
Required fire resistance R	30,00	min
Gas temperature theta,g	841,80	°C
Material temperature theta,a,t	614,72	°C
Beam exposure	All sides	
Adaptation factor for cross-section kappa,1	1,00	
Adaptation factor for beam kappa,2	0,85	
Reduction factor for the 0.2% proof strength k,0.2p,theta	0,27	
Reduction factor for the E modulus k,E,theta	0,28	

Insulation properties		
Name	Fibre board	
Encasement type	Hollow encasement	
Insulation type	Board	
Thickness d,p	5,00	mm
Unit mass rho,p	150,0	kg/m³
Thermal conductivity lambda,p	2,0000e-01	W/mK
Specific heat c,p	1,2000e+00	J/gK
Section factor for insulated steel members Ap/V	1,1611e-01	1/mm

And now a graph is shown with the gas temperature (in this example follow the ISO 834 curve), the steel temperature calculated with a protection and the reduction of the yield strength.



And after all the unity check is shown with the reduced properties:

....::SECTION CHECK::....

Classification for cross-section design
According to EN 1993-1-2 article 4.2.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	59,57
Class 1 Limit	58,91
Class 2 Limit	67,83
Class 3 Limit	93,50

=> Internal Compression parts Class 2
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6,40
Class 1 Limit	7,65
Class 2 Limit	8,50
Class 3 Limit	11,71

=> Outstand Flanges Class 1
=> Section classified as Class 2 for cross-section design

The critical check is on position 6.900 m

Internal forces	Calculated	Unit
N,fi,Ed	-61,02	kN
Vy,fi,Ed	-0,01	kN
Vz,fi,Ed	-34,87	kN
T,fi,Ed	0,00	kNm
My,fi,Ed	-240,62	kNm
Mz,fi,Ed	-0,05	kNm

Compression check
According to EN 1993-1-2 article 4.2.3.2 and formula (4.5)

A	1,7500e+04	mm^2
N,fi,t,Rd	1130,81	kN
Unity check	0,05	-

Bending moment check for My
According to EN 1993-1-2 article 4.2.3.3 and formula (4.10)

Wpl,y	4,8650e+06	mm^3
Mpl,y,Rd	1143,28	kNm
My,fi,theta,Rd	314,37	kNm
My,fi,t,Rd	369,84	kNm
Unity check	0,65	-

Bending moment check for Mz
According to EN 1993-1-2 article 4.2.3.3 and formula (4.10)

Wpl,z	6,1410e+05	mm^3
Mpl,z,Rd	144,31	kNm
Mz,fi,theta,Rd	39,68	kNm
Mz,fi,t,Rd	46,68	kNm
Unity check	0,00	-

Shear check for Vy
According to EN 1993-1-2 article 4.2.3.3 and formula (4.16)

Eta	1,20	
Av	9,2698e+03	mm^2
Vpl,y,Rd	1257,69	kN
Vy,fi,t,Rd	345,83	kN
Unity check	0,00	-

Shear check for Vz
According to EN 1993-1-2 article 4.2.3.3 and formula (4.16)

Eta	1,20	
Av	9,9222e+03	mm^2
Vpl,z,Rd	1346,22	kN
Vz,fi,t,Rd	370,17	kN
Unity check	0,09	-

Combined bending, axial force and shear force check
According to EN 1993-1-2 article 4.2.3
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

My,fi,t,Rd	369,84	kNm
Alpha	2,00	
Mz,fi,t,Rd	46,68	kNm
Beta	1,00	

Unity check (4.9) = 0,42 + 0,00 = 0,42 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.
Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	59,57
Class 1 Limit	28,05
Class 2 Limit	32,30
Class 3 Limit	35,70

=> Internal Compression parts Class 4
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6,40
Class 1 Limit	7,65
Class 2 Limit	8,50
Class 3 Limit	11,90

=> Outstand Flanges Class 1
=> Section classified as Class 4 for member buckling design

Properties					
sectional area A eff	1.5269e+04	mm^2			
Shear area Vy eff	8.9420e+03	mm^2	Vz eff	6.3269e+03	mm^2
radius of gyration iy eff	320.39	mm	iz eff	58.15	mm
moment of inertia Iy eff	1.5674e+09	mm^4	Iz eff	5.1634e+07	mm^4
elastic section modulus Wy eff	4.1630e+06	mm^3	Wz eff	3.9265e+05	mm^3
Eccentricity eny	0.00	mm	enz	0.00	mm

Flexural Buckling check
According to EN 1993-1-2 article 4.2.3.2 & Annex E and formula (4.5)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	6,900	6,900	m
Buckling factor k	3,25	0,99	
Buckling length Lcr	22,415	6,832	m
Critical Euler load Ncr	6596,08	2293,84	kN
Slenderness Lambda	74,15	125,75	
Relative slenderness Lambda,rel	0,74	1,25	
Relative slenderness Lambda,rel,theta	0,73	1,23	
Imperfection Alpha	0,65	0,65	
Reduction factor Chi,fi	0,59	0,36	
Buckling resistance Nb,fi,t,Rd	584,85	356,17	kN

Flexural Buckling verification		
Cross-section effective area Aeff	1,5269e+04	mm^2
Buckling resistance Nb,fi,t,Rd	356,17	kN
Unity check	0,17	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-2 article 4.2.3.2 & Annex E and formula (4.5)
Note: For this I-section the Torsional(-Flexural) buckling resistance is higher than the resistance for Flexural buckling. Therefore Torsional(-Flexural) buckling is not printed on the output.

Lateral Torsional Buckling check
According to EN 1993-1-2 article 4.2.3.4 & Annex E and formula (4.19)

LTB parameters		
Cross-section effective modulus Weff,y	4,1630e+06	mm^3
Elastic critical moment Mcr	1709,12	kNm
Relative slenderness Lambda,rel,LT	0,76	
Relative slenderness Lambda,rel,LT,theta	0,75	
Imperfection Alpha,LT	0,65	
Reduction factor Chi,LT,fi	0,58	
Design buckling resistance Mb,fi,t,Rd	156,77	kNm
Unity check	**1,53**	-

Mcr parameters		
LTB length L	6,900	m
Influence of load position	no influence	
Correction factor k	1,00	
Correction factor kw	1,00	
LTB moment factor C1	1,77	
LTB moment factor C2	0,00	
LTB moment factor C3	1,00	
Shear center distance d,z	0,00	mm
Distance of load application z,g	0,00	mm
Mono-symmetry constant beta,y	0,00	mm
Mono-symmetry constant z,j	0,00	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-2 article 4.2.3.5 & Annex E and formula (4.21c),(4.21d)

Bending and axial compression check parameters		
Cross-section effective area Aeff	1,5269e+04	mm^2
Cross-section effective modulus Weff,y	4,1630e+06	mm^3
Cross-section effective modulus Weff,z	3,9265e+05	mm^3

Design compression force N,fi,Ed	61,02	kN
Design bending moment My,fi,Ed	-240,62	kNm
Design bending moment Mz,fi,Ed	-0,05	kNm
Reduction factor Chi,min,fi	0,36	
Reduction factor Chi,z,fi	0,36	
Reduction factor Chi,LT,fi	0,58	
Equivalent moment factor beta,M,y	1,80	
Factor mu,y	0,07	
Interaction factor k,y	0,99	
Equivalent moment factor beta,M,z	1,80	
Factor mu,z	-0,05	
Interaction factor k,z	1,01	
Equivalent moment factor beta,M,LT	1,80	
Factor mu,LT	0,18	
Interaction factor k,LT	0,97	

Unity check (4.21c) = 0,17 + 0,89 + 0,00 = **1,06** -
Unity check (4.21d) = 0,17 + 1,49 + 0,00 = **1,66** -

Shear Buckling check
According to EN 1993-1-2 article 4.2.3
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters		
Buckling field length a	6,900	m
Web	unstiffened	
End post	non-rigid	
Web height hw	719,00	mm
Web thickness t	11,50	mm
Yield strength fyw	235,0	MPa
Flange width bf	263,00	mm
Flange thickness tf	17,00	mm
Yield strength fyf	235,0	MPa
Material coefficient epsilon	0,85	
Shear correction factor Eta	1,20	

Shear Buckling verification		
Web slenderness hw/t	62,52	
Web slenderness limit	51,00	
Plate slenderness lambda,w,theta	0,71	
Reduction factor chi,w,fi	1,16	
Contribution of the web Vbw,fi,t,Rd	359,26	kN
Capacity of the flange Mf,fi,t,Rd	190,18	kNm
Flange factor c	0,000	m
Contribution of the flange Vbf,fi,t,Rd	0,00	kN
Maximum resistance Vb,fi,t,Rd,limit	370,17	kN
Resistance Vb,fi,t,Rd	359,26	kN
Plastic resistance Mpl,fi,t,Rd	314,37	kNm
Shear ratio eta,3,bar	0,10	

Unity check (5.10) = 0,10 -
Note: The interaction between Bending and Shear Buckling does not need to be verified because the shear ratio does not exceed 0.5.

The member does NOT satisfy the stability check!

So in this example the **Compression and bending check** will result in a unity check op **1,66**.

Time domain

In SCIA Engineer 3 calculation methods are implemented:

- Resistance domain
- Time domain.
- Temperature domain (iterative).

The choice between those analysis types can be made in “Steel -> Beams -> Steel setup”:
In this chapter the fire resistance check following the Time domain will be explained.

Principle

The user will choose the used temperature time curve and will input a required fire resistance time. After this time the temperature of the gas and afterwards of the steel will be calculated.

Now also the critical steel temperature will be calculated. And the Fire resistance check according to the time domain will be the ratio of the real steel temperature after a chosen time and the critical steel temperature.

This critical steel temperature $\theta_{a,cr}$ will be calculated with a simple formula:

$$\theta_{a,cr} = 39{,}19\ ln\left[\frac{1}{0{,}9674\ \mu_0^{3{,}833}} - 1\right] + 482$$

Where μ_0 is the degree of utilization. This means the unity check following EN 1993-1-2 at time = 0sec, so without augmentation of the temperature.

NOTE
This simple formula is only valid if no stability phenomena or deformation criteria have to be taken into account.

Example in SCIA Engineer

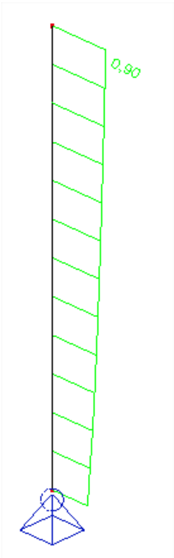
This principle is explained with an example in SCIA Engineer.

Example: Industrial hall.esa

Consider member B28:

And change in “Steel -> Beams -> Steel setup” the analysis type to “Time domain”.

The fire resistance check is executed via “Steel -> Beams -> ULS check -> Check - fire resistance” for this column and for combination “CO3 - Fire”, resulting in a unity check of 1,03:



When looking at the detailed output, this calculation is given by SCIA Engineer:

The check results shown hereafter are given at time t = 0,00 min. These results have been used to determine the degree of utilization for the critical temperature.

So the check following EN 1993-1-2 will be given at t = 0min, so 20°C, without any reduction of the steel properties. This check will result in a low unity check, which is the degree of utilisation μ_0:

In this example again the Compression and bending check will result in the highest unity check = 0.47 = μ_0.

Bending and axial compression check
According to EN 1993-1-2 article 4.2.3.5 & Annex E and formula (4.21c),(4.21d)

Bending and axial compression check parameters		
Cross-section effective area Aeff	1,5269e+04	mm^2
Cross-section effective modulus Weff,y	4,1630e+06	mm^3
Cross-section effective modulus Weff,z	3,9265e+05	mm^3
Design compression force N,fi,Ed	61,02	kN
Design bending moment My,fi,Ed	-240,62	kNm
Design bending moment Mz,fi,Ed	-0,05	kNm
Reduction factor Chi,min,fi	0,35	
Reduction factor Chi,z,fi	0,35	
Reduction factor Chi,LT,fi	0,58	
Equivalent moment factor beta,M,y	1,80	
Factor mu,y	0,05	
Interaction factor k,y	1,00	
Equivalent moment factor beta,M,z	1,80	
Factor mu,z	-0,06	
Interaction factor k,z	1,00	
Equivalent moment factor beta,M,LT	1,80	
Factor mu,LT	0,19	
Interaction factor k,LT	0,99	

Unity check (4.21c) = 0,05 + 0,25 + 0,00 = 0,29 -
Unity check (4.21d) = 0,05 + 0,42 + 0,00 = 0,47 -

This value is used in the simple formula for the critical steel temperature:

$$\theta_{a,cr} = 39,19\ ln\left[\frac{1}{0,9674\ \mu_0^{3,833}} - 1\right] + 482$$

$$\theta_{a,cr} = 39,19\ ln\left[\frac{1}{0,9674\ (0.47)^{3.833}} - 1\right] + 482 = 594\ °C$$

And the steel temperature after 30 minutes is 614.72°C.

The unity check is: $\frac{614.72°C}{594.26°C} = 1.03$

All those values are also given in the overview table in the preview of the fire resistance check:

Fire resistance		
Temperature-time curve	ISO 834 curve	
Coefficient of heat transfer by convection alpha,c	25,00	W/m^2K
Emissivity related to fire compartment epsilon,f	1,00	
Emissivity related to surface material epsilon,m	0,70	
Configuration factor for radiation heat flux phi	1,00	
Required fire resistance R	30,00	min
Gas temperature theta,g	841,80	ºC
Material temperature theta,a,t	614,72	ºC
Degree of utilization mu,0	0,47	
Critical material temperature theta,a,cr	594,26	ºC
Fire resistance t,cr	28,10	min
Beam exposure	All sides	
Adaptation factor for cross-section kappa,1	1,00	
Adaptation factor for beam kappa,2	0,85	
Reduction factor for the 0.2% proof strength k,0.2p,theta	1,00	
Reduction factor for the E modulus k,E,theta	1,00	
Unity check	**1,03**	-

And also the **Fire resistance time** is given in this table:
This member van resist fire for 28,10 minutes.

NOTE
As stated before, this simple calculation method can only be used if no stability phenomena have to be taken into account. In this example the stability causes the highest unity check and is thus taken into account, so **this method is not correct and should not be used**!

Temperature domain (iterative)

In SCIA Engineer 3 calculation methods are implemented:

- Resistance domain
- Time domain.
- Temperature domain (iterative).

The choice between those analysis types can be made in “Steel -> Beams -> Steel setup”:
In this chapter the fire resistance check following the Temperature domain will be explained.

If this method is used, the critical steel temperature will be calculated with an iterative process. So first an estimation of this critical temperature will be chosen and the unity check following EN 1993-1-2 will be executed, if this check is lower than one, a higher critical temperature is chosen or when this check is higher than one, a lower temperature is chosen. Now this unity check is recalculated just until the moment this unity check gives a result for this critical steel temperature between 0,99 and 1.

This is a more accurate procedure to calculate the critical temperature and this method is also valid if stability phenomena or deformation criteria have to be taken into account.

Example in SCIA Engineer

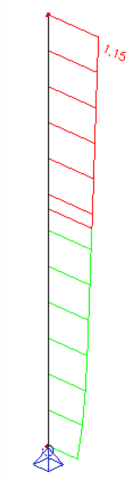
This principle is explained with an example in SCIA Engineer.

Example: Industrial hall.esa

Consider member B28:

And change in “Steel -> Beams -> Steel setup” the analysis type to “Temperature domain (iterative)”.

The fire resistance check is executed via “Steel -> Beams -> ULS check -> Check - fire resistance” for this column and for combination “CO3 - Fire”, resulting in a unity check of 1,15:



When looking at the detailed output, this calculation is given by SCIA Engineer:

The check results shown hereafter are given at the critical material temperature theta,a,cr = 535,10 °C. These results have been used to determine the critical temperature i.e. the temperature at which the unity checks become near to 1,00.

So the check following EN 1993-1-2 will be given at $\theta_{a,cr}$ = 535,10°C. This temperature is calculated iterative resulting in a unity check following the EN 1993-1-2 equal to 1:

Bending and axial compression check
According to EN 1993-1-2 article 4.2.3.5 & Annex E and formula (4.21c),(4.21d)

Bending and axial compression check parameters		
Cross-section effective area Aeff	1,5269e+04	mm^2
Cross-section effective modulus Weff,y	4,1630e+06	mm^3
Cross-section effective modulus Weff,z	3,9265e+05	mm^3
Design compression force N,fi,Ed	61,02	kN
Design bending moment My,fi,Ed	-240,62	kNm
Design bending moment Mz,fi,Ed	-0,05	kNm
Reduction factor Chi,min,fi	0,38	
Reduction factor Chi,z,fi	0,38	
Reduction factor Chi,LT,fi	0,60	
Equivalent moment factor beta,M,y	1,80	
Factor mu,y	0,10	
Interaction factor k,y	0,99	
Equivalent moment factor beta,M,z	1,80	
Factor mu,z	-0,01	
Interaction factor k,z	1,00	
Equivalent moment factor beta,M,LT	1,80	
Factor mu,LT	0,17	
Interaction factor k,LT	0,98	

Unity check (4.21c) = 0,10 + 0,54 + 0,00 = 0,64 -
Unity check (4.21d) = 0,10 + 0,90 + 0,00 = 1,00 -

And indeed the highest unity check will be equal to 1.00 for this critical temperature.

So for this case the unity check is: $\frac{614.72°C}{535.10°C} = 1.15$

All those values are also given in the overview table in the preview of the fire resistance check:

Fire resistance
Verification in Temperature domain according to EN 1993-1-2 article 4.2.4

Fire resistance		
Temperature-time curve	ISO 834 curve	
Coefficient of heat transfer by convection alpha,c	25,00	W/m^2K
Emissivity related to fire compartment epsilon,f	1,00	
Emissivity related to surface material epsilon,m	0,70	
Configuration factor for radiation heat flux phi	1,00	
Required fire resistance R	30,00	min
Gas temperature theta,g	841,80	°C
Material temperature theta,a,t	614,72	°C
Critical material temperature theta,a,cr	535,10	°C
Fire resistance t,cr	23,30	min
Beam exposure	All sides	
Adaptation factor for cross-section kappa,1	1,00	
Adaptation factor for beam kappa,2	0,85	
Reduction factor for the 0.2% proof strength k,0.2p,theta	0,45	
Reduction factor for the E modulus k,E,theta	0,50	
Unity check	**1,15**	-

And also the **Fire resistance time** is given in this table:
This member van resist fire for 23,30 minutes.

NOTE
As stated before, this simple calculation method can only be used if no stability phenomena have to be taken into account. In this example the critical steel temperature was calculated using the iterative process, so **this method is correct and can be used as an alternative for the resistance domain.**

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