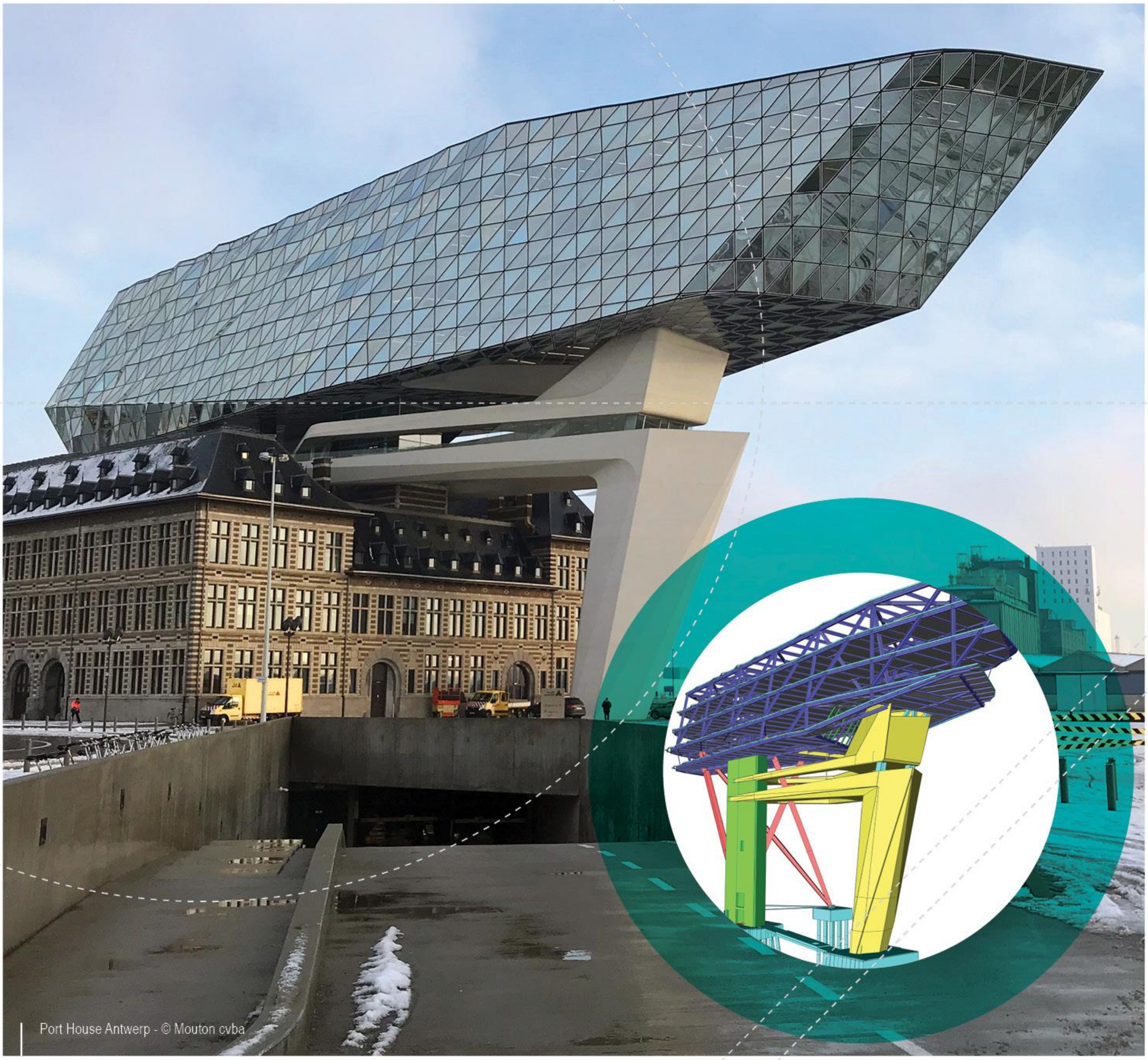
SCIA ENGINEER



BASIC CONCEPT TRAINING

STEEL CODE CHECK

Table of Contents

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.

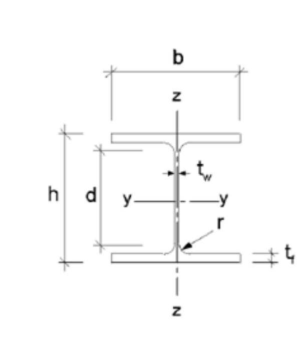
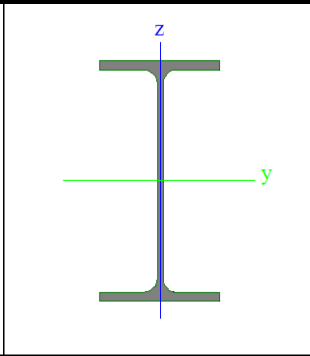
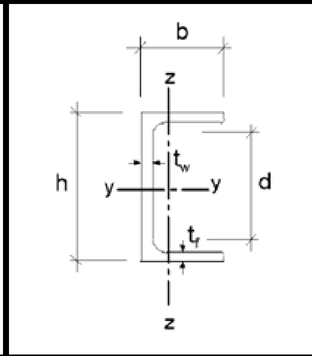
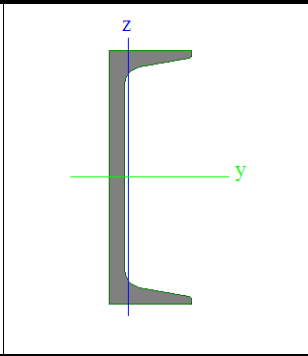
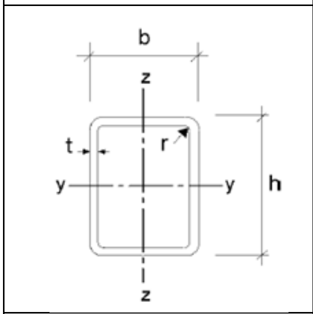
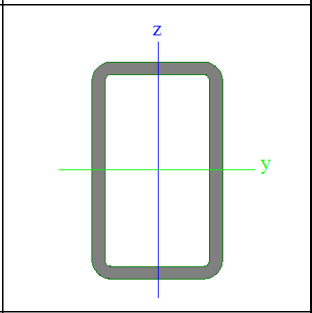
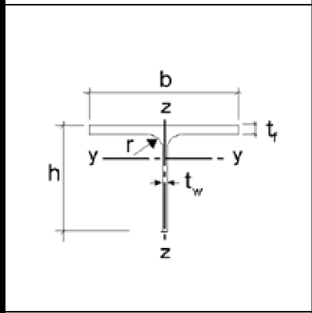
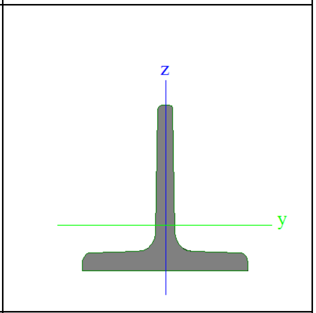
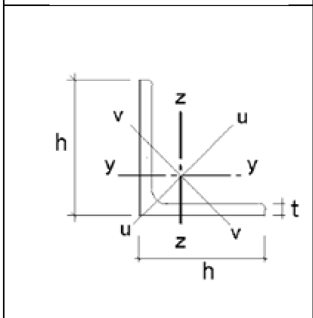
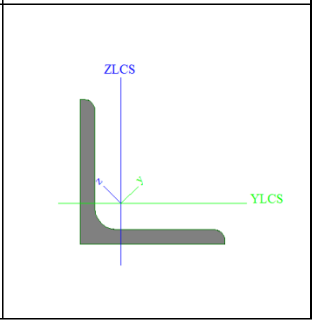
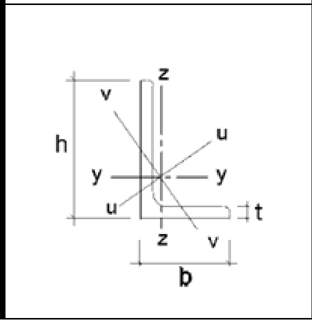
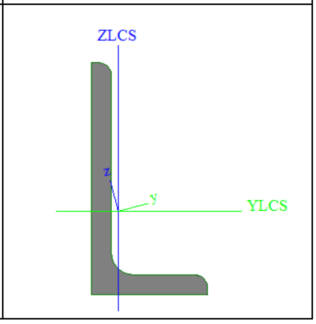
Most of the options in the course can be calculated/checked in SCIA Engineer with the **Concept edition**.

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.

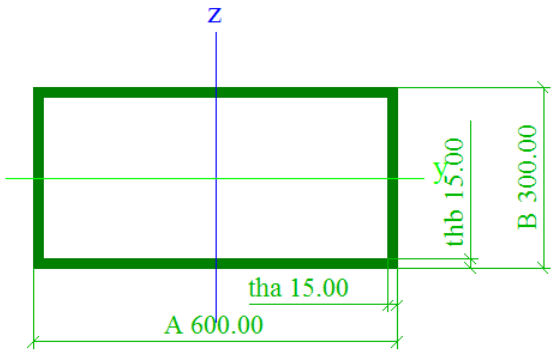
If the principal axes not coincide with the y-y and z-z axes following the EN 1993-1-1, also those axes are indicated:

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
$V_{y,Ed}$	300.00	kN
$V_{z,Ed}$	0.00	kN
T_{Ed}	0.00	kNm
$M_{y,Ed}$	0.00	kNm
$M_{z,Ed}$	-300.00	kNm

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

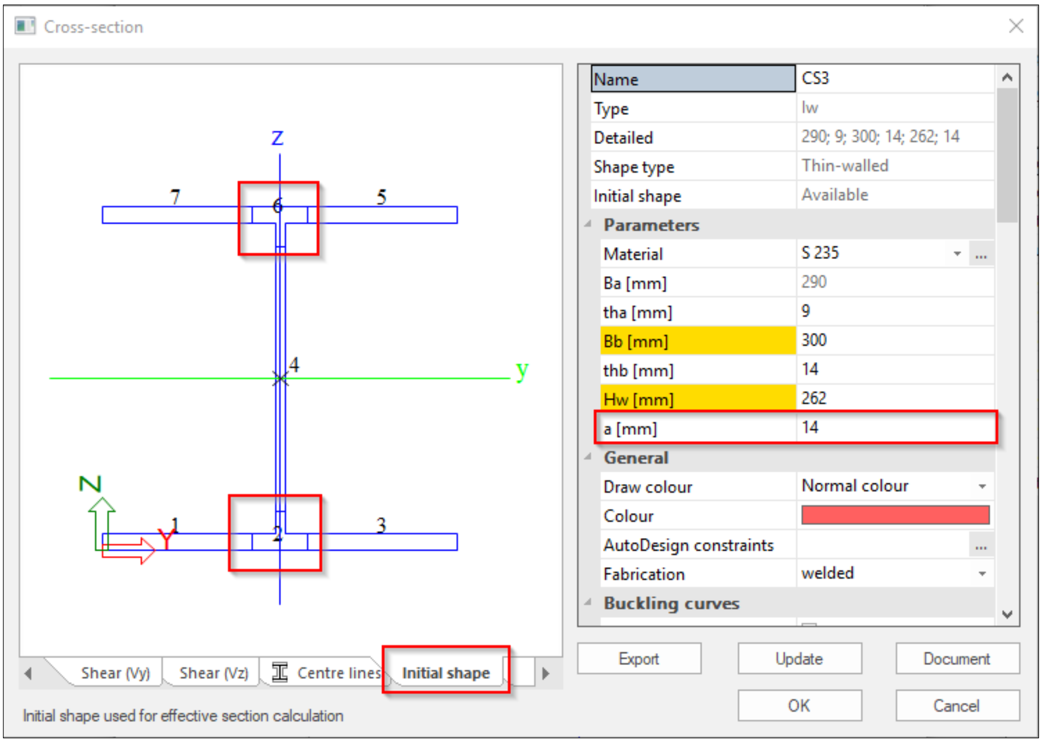
Starting from SCIA Engineer 17.0, all sections containing an initial shape can be classified. The sections without an initial shape cannot be classified and will be automatically checked as being class 3.

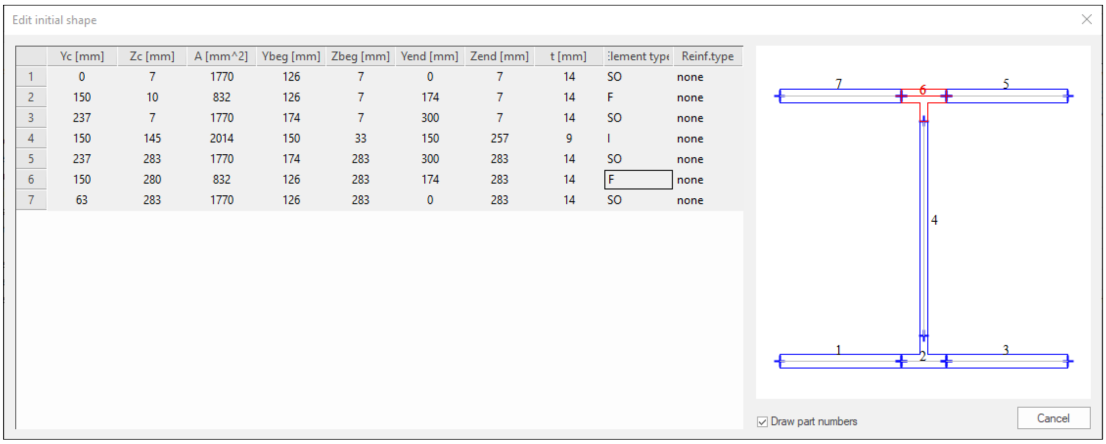
Remark: For the classification applications of sections in SCIA Engineer versions 16.1 or older, see Annex A.

Example: Cross Section.esa

- 3 cross sections:
  - HEA 300 from library
  - HEA 300 inputted as a general cross section (imported from .dwg)
  - HEA 300 inputted as a sheet welded Iw section

Remark: The third section is a sheet welded Iw profile. By creating this section, it is important that the correct value for parameter a is filled in mm. This parameter will be recognized as the fixed part in the calculation of the initial shape:

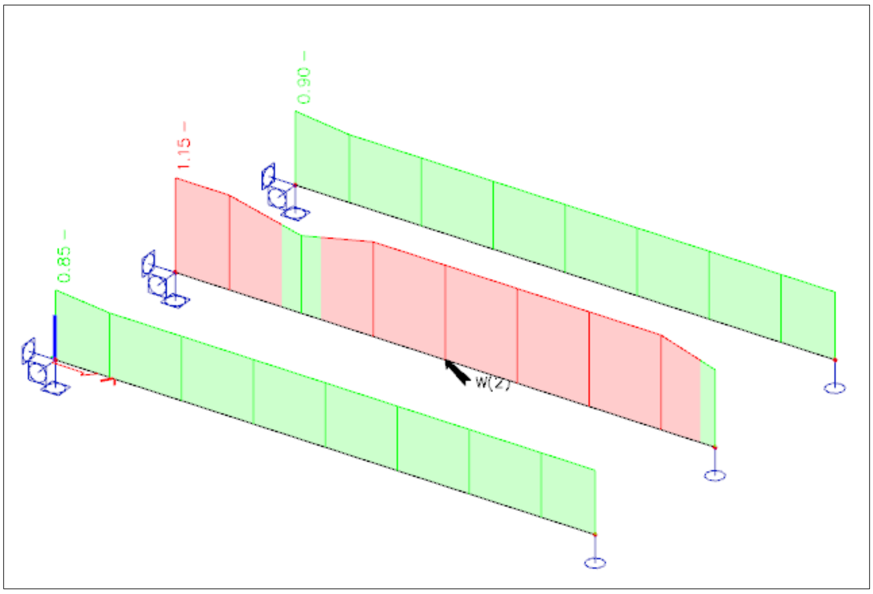




	Yc [mm]	Zc [mm]	A [mm^2]	Ybeg [mm]	Zbeg [mm]	Yend [mm]	Zend [mm]	t [mm]	Element type	Reinf.type
1	0	7	1770	126	7	0	7	14	SO	none
2	150	10	832	126	7	174	7	14	F	none
3	237	7	1770	174	7	300	7	14	SO	none
4	150	145	2014	150	33	150	257	9	I	none
5	237	283	1770	174	283	300	283	14	SO	none
6	150	280	832	126	283	174	283	14	F	none
7	63	283	1770	126	283	0	283	14	SO	none

- Only the first and the third profile are recognized as a symmetric I-shape containing an initial shape. For this reason the classification calculation can be performed.
- The second profile is not recognized as a symmetric I-shape and there is no initial shape available.
- The first and the third profile will be classified as an I-profile and a plastic check will be performed.

- The second profile cannot be classified, so an elastic check will be performed. This will result 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:

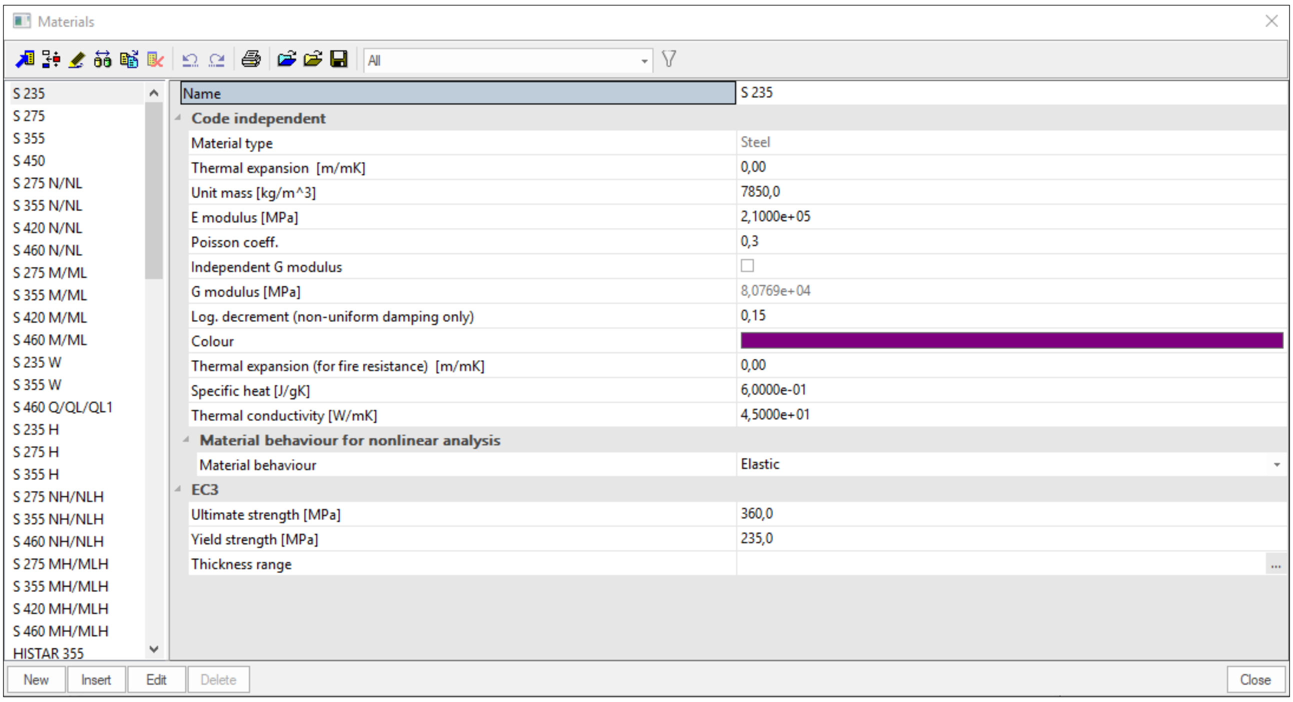
Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 80$ mm	
	f_y [N/mm²]	f_u [N/mm²]	f_y [N/mm²]	f_u [N/mm²]
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

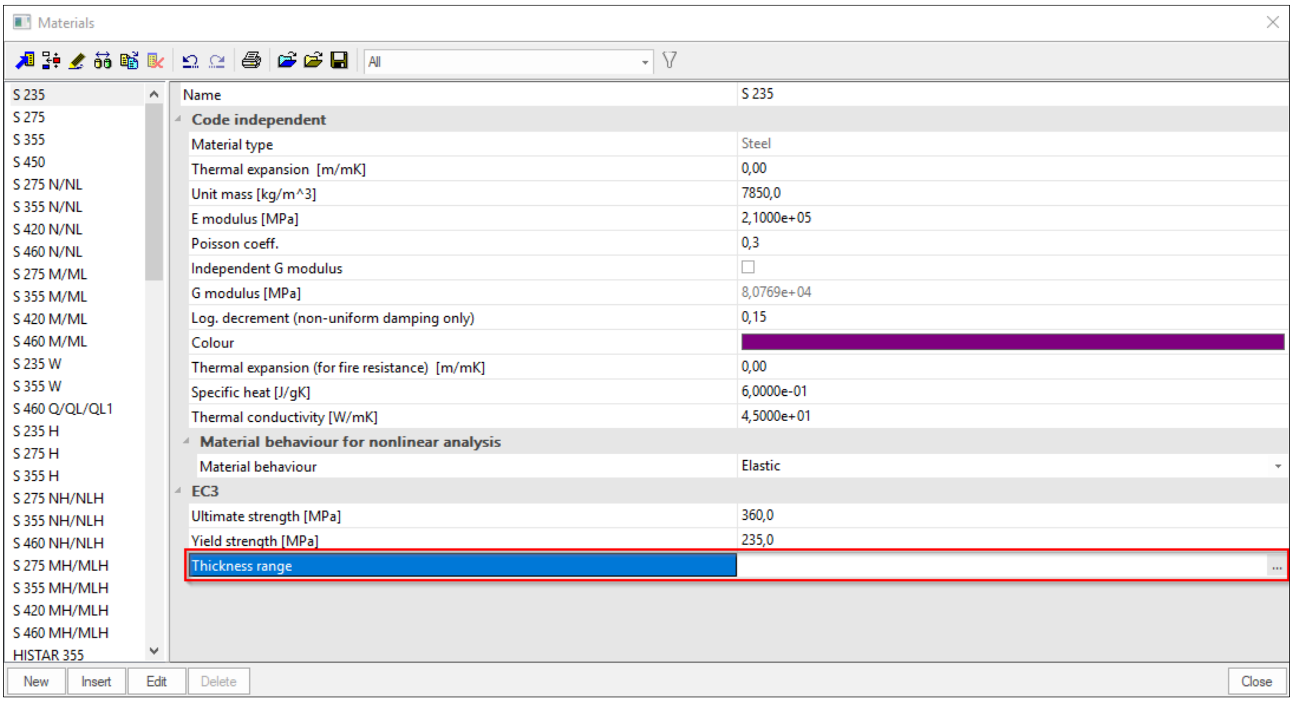
Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 80$ mm	
	f_y [N/mm²]	f_u [N/mm²]	f_y [N/mm²]	f_u [N/mm²]
EN 10210-1				
S 235 H	235	360	215	340
S 275 H	275	430	255	410
S 355 H	355	510	335	490
S 275 NH/NLH	275	390	255	370
S 355 NH/NLH	355	490	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		

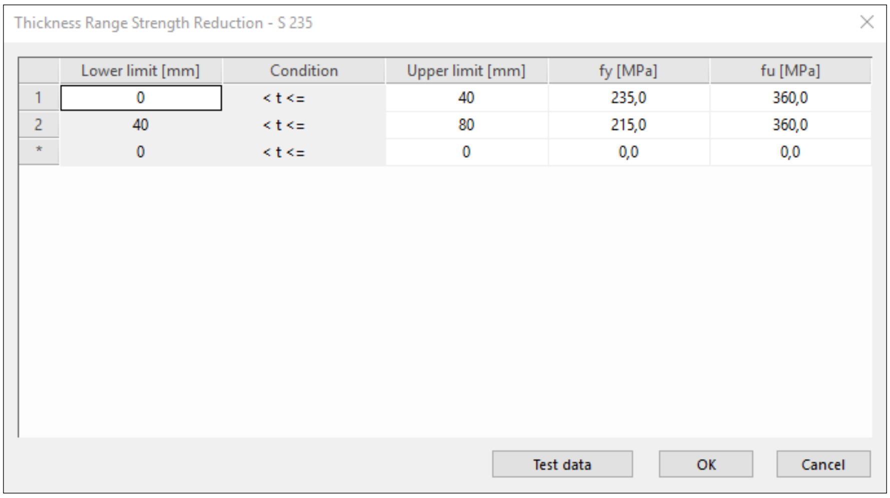
Those materials are included in SCIA Engineer:



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:



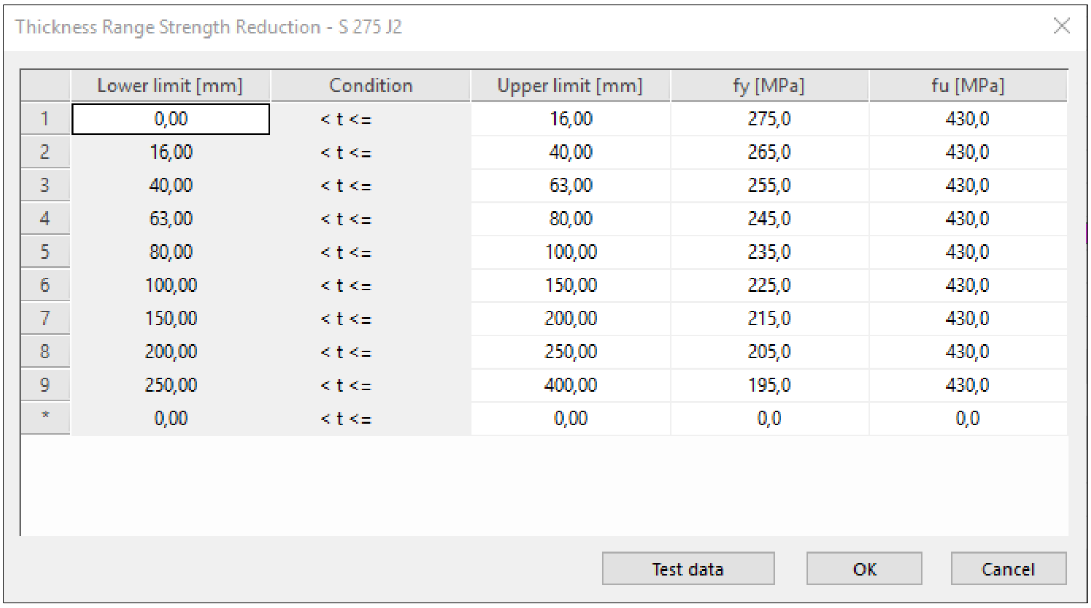

Thickness Range Strength Reduction - S 235

	Lower limit [mm]	Condition	Upper limit [mm]	fy [MPa]	fu [MPa]
1	0	< t <=	40	235,0	360,0
2	40	< t <=	80	215,0	360,0
*	0	< t <=	0	0,0	0,0

Test data OK Cancel

Example: NA_Material_Strength_Application.esa

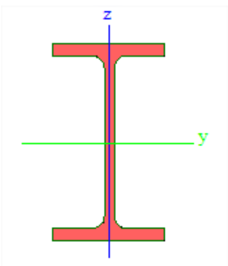
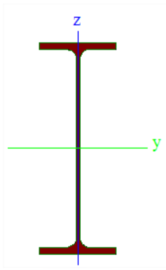
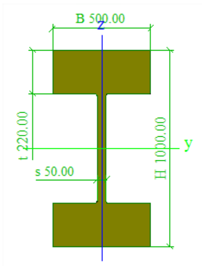
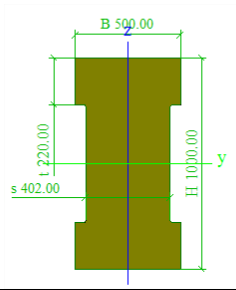
In this example a material of the National Annex of Belgium has been inputted manually: S 275 J2. This material has a lot of thickness range strength reductions:


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**
			

Formcode	1 - I sections
h [mm]	80,00
b [mm]	46,00
t [mm]	5,20
s [mm]	3,80
r [mm]	5,00
r1 [mm]	0,00
a [%]	0
W [mm]	0,00
wm [mm^2]	0,00

Material		
Yield strength fy	275,0	MPa
Ultimate strength fu	430,0	MPa
Fabrication	Rolled	

Formcode	1 - I sections
h [mm]	600,00
b [mm]	220,00
t [mm]	19,00
s [mm]	12,00
r [mm]	24,00
r1 [mm]	0,00
a [%]	0
W [mm]	116,00
wm [mm^2]	0,00

Material		
Yield strength fy	265,0	MPa
Ultimate strength fu	430,0	MPa
Fabrication	Rolled	

Formcode	1 - I sections
h [mm]	1000,00
b [mm]	500,00
t [mm]	220,00
s [mm]	50,00
r [mm]	12,00
r1 [mm]	0,00
a [%]	0
W [mm]	0,00
wm [mm^2]	0,00

Material		
Yield strength fy	205,0	MPa
Ultimate strength fu	430,0	MPa
Fabrication	Rolled	

Formcode	1 - I sections
h [mm]	1000,00
b [mm]	500,00
t [mm]	220,00
s [mm]	402,00
r [mm]	12,00
r1 [mm]	0,00
a [%]	0
W [mm]	0,00
wm [mm^2]	0,00

Material		
Yield strength fy	195,0	MPa
Ultimate strength fu	430,0	MPa
Fabrication	Rolled	

Section classification

The classification of cross-sections is executed according to EN 1993-1-1, art. 5.5.

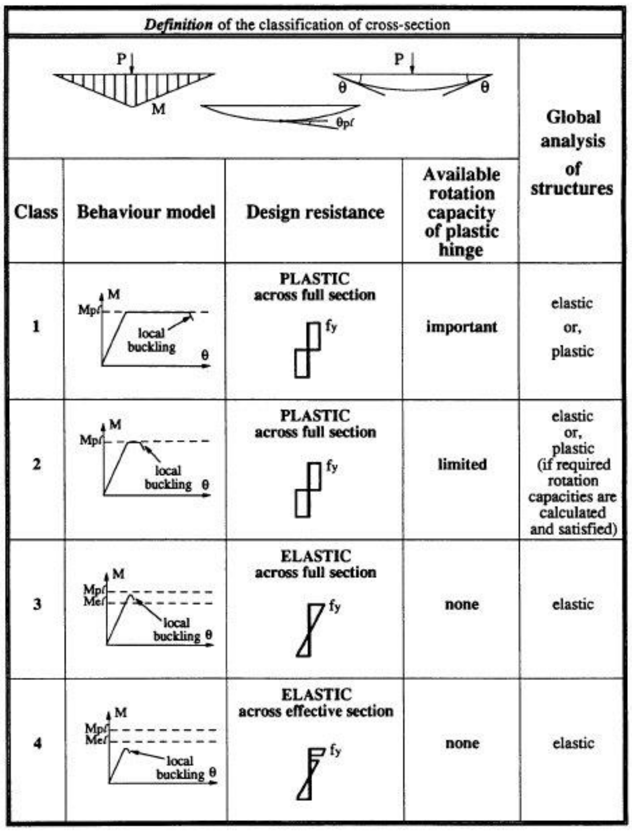
Four classes of cross sections are defined:

- **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.

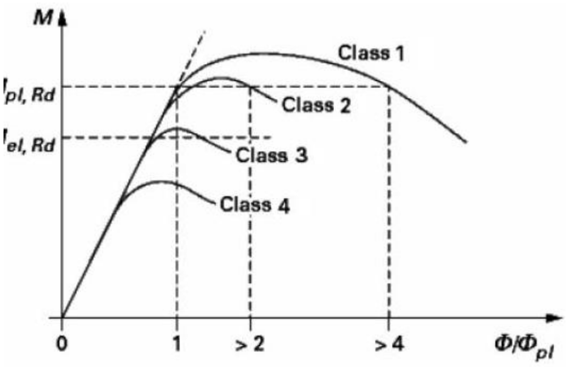
- **Class 3** (EC3, NEN) or EL-EL (DIN) section
  Cross sections in which the calculated stress in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.

- **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		PLASTIC across full section	important	elastic or, plastic
2		PLASTIC across full section	limited	elastic or, plastic (if required rotation capacities are calculated and satisfied)
3		ELASTIC across full section	none	elastic
4		ELASTIC across effective section	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.

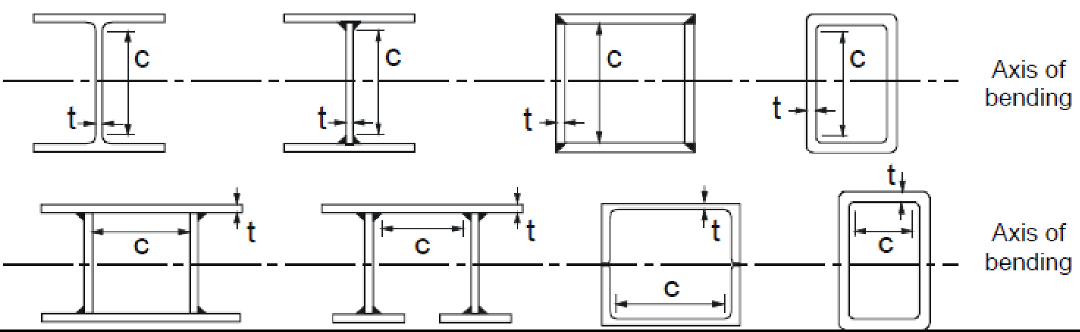
The classification check in SCIA Engineer will be executed following tables 5.2 of the EN 1993-1-1. For standard sections, the classification is done according to the parts of the **Initial Shape**.

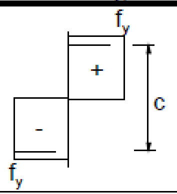
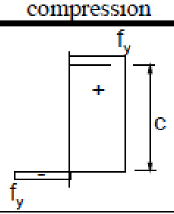
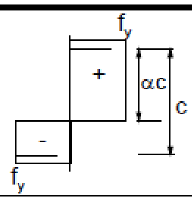
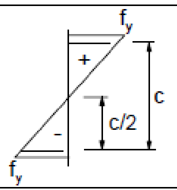
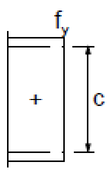
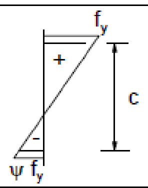
- Internal compression elements (I) are classified according to Table 5.2 Sheet 1.
- Outstand compression elements (SO & UO) are classified according to Table 5.2 Sheet 2.
- CHS sections are classified according to Table 5.2 Sheet 3.
- Angle sections are classified according to Table 5.2 Sheet 2 and in case of uniform compression also Sheet 3.

Remark: Cross-sections without an Initial Shape are classified as elastic class 3.

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

Internal compression parts



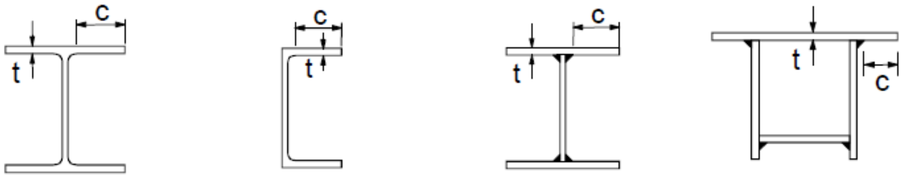
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
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

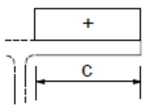
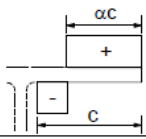
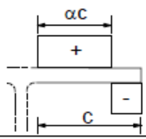
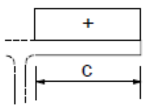
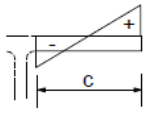
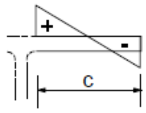
*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

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

Outstand flanges



Rolled sections — Welded sections

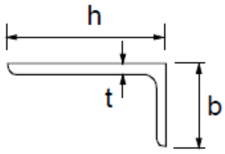
Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \leq 9\varepsilon$	$c/t \leq \frac{9\varepsilon}{\alpha}$	$c/t \leq \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$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

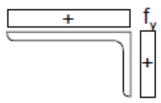
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

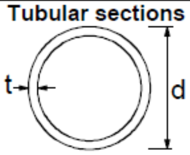
Refer also to "Outstand flanges" (see sheet 2 of 3)



Does not apply to angles in continuous contact with other components

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

Tubular sections



Class	Section in bending and/or compression
1	$d/t \leq 50\varepsilon^2$
2	$d/t \leq 70\varepsilon^2$
3	$d/t \leq 90\varepsilon^2$ 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

As mentioned before, starting from SCIA Engineer 17.0, a new classification tool is used based on the initial shape of the cross-section.

The user can choose between 3 methods to determine the plastic stress distribution in the cross-section:

- Elastic stresses
- Yield surface intersection
- Iterative approach

Elastic stresses

The elastic stresses method is a fast approach using fixed formulas. In this method, the plastic stress distribution is based on the elastic stresses f1 and f2 at the ends of the parts.

<u>Standard calculation of α</u>

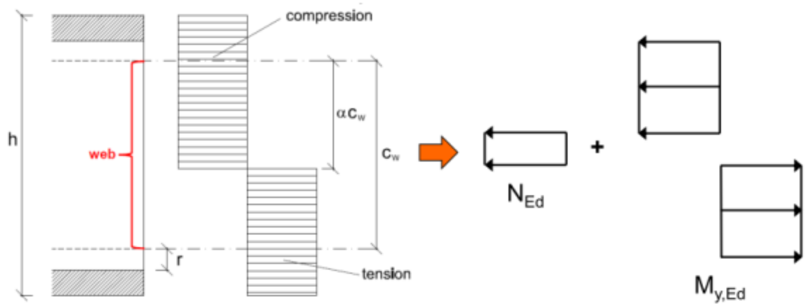
In case one stress is positive (compression) and the other is negative (tension), the following calculation is used:

$$\alpha = \frac{|\sigma_{Compression}|}{|\sigma_{Compression}| + |\sigma_{Tension}|}$$

In all other cases α is taken as 1,00 for the given part.

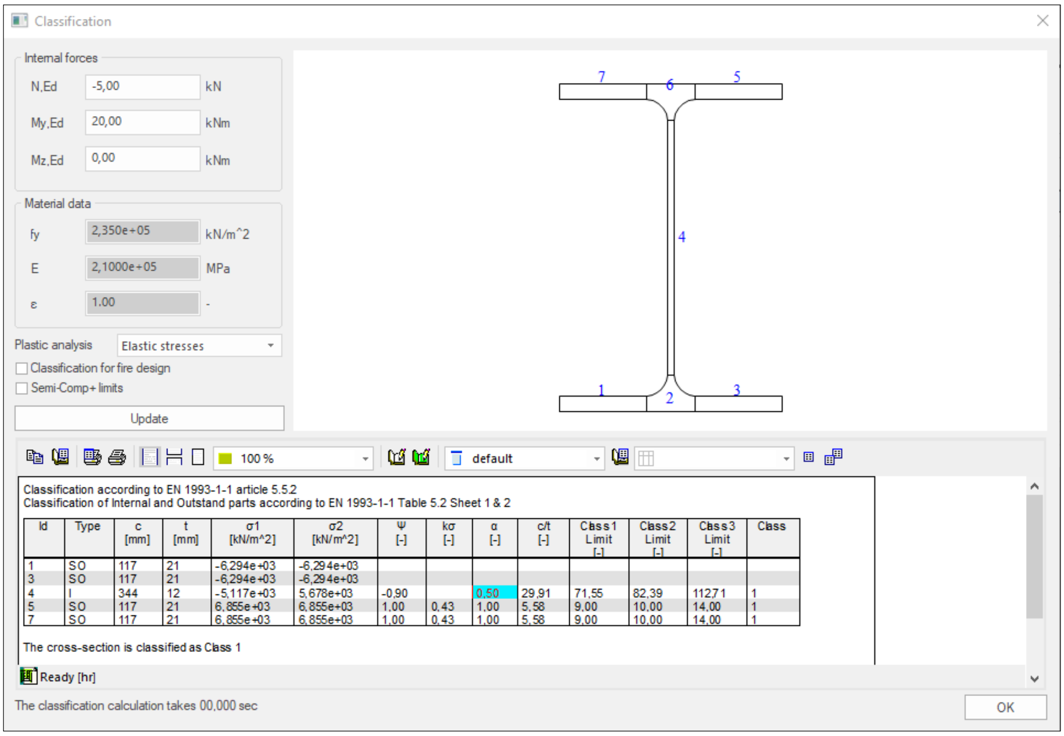
<u>Double-symmetric I-section</u>

Specially for a double-symmetric I-section, the α value of the web element is overruled by the following formula (Ref. [36]):



Only increase M_y: $\alpha = \frac{1}{c_w}\left(\frac{h}{2} + \frac{N_{Ed}}{2t_w f_y} - (t_f + r)\right)$

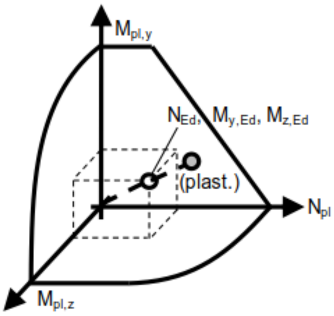
Within this formula the N_{Ed} is taken as positive for compression and negative for tension.



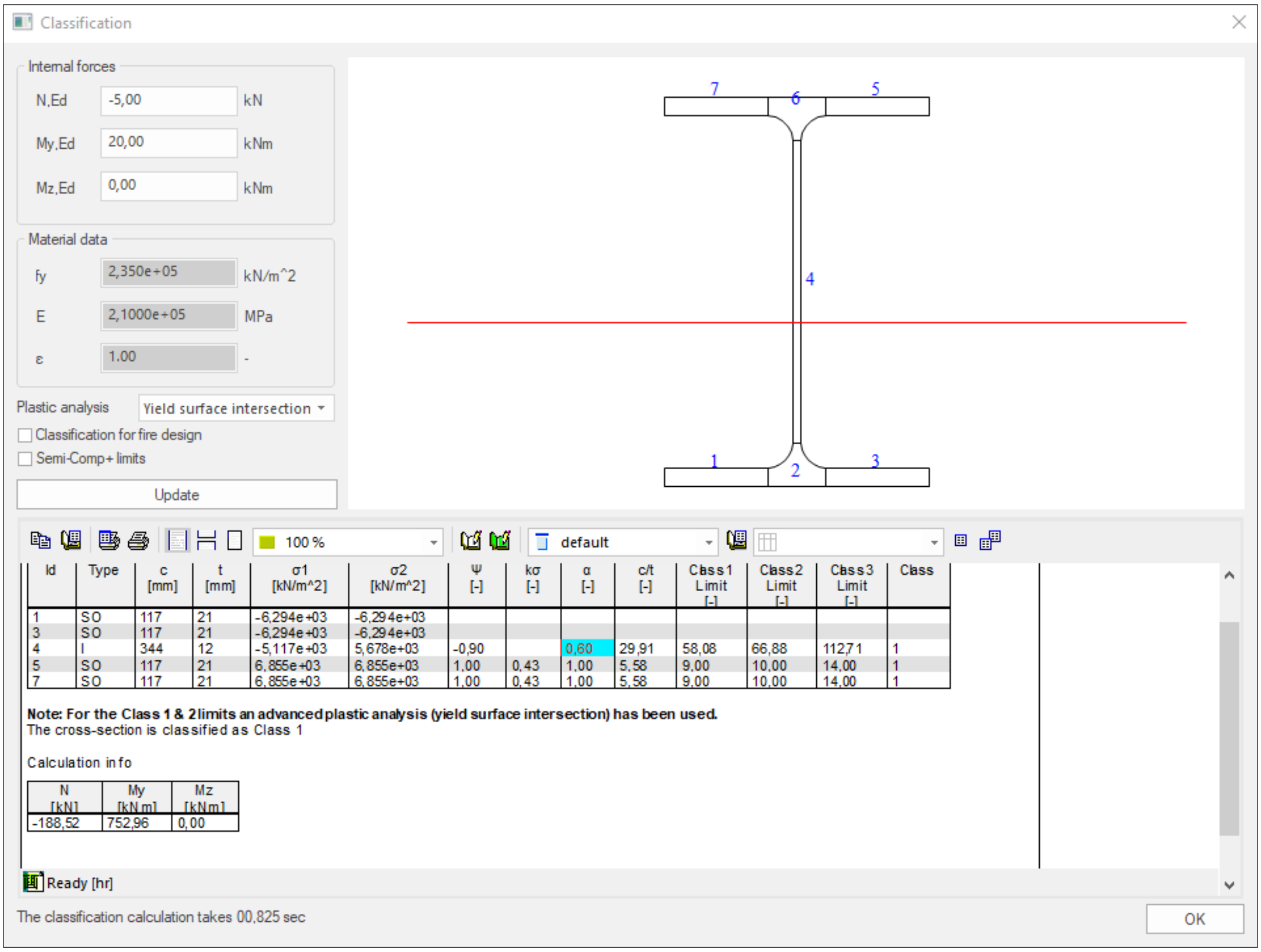
Id	Type	c [mm]	t [mm]	σ1 [kN/m^2]	σ2 [kN/m^2]	Ψ [-]	kσ [-]	α [-]	c/t [-]	Class1 Limit [-]	Class2 Limit [-]	Class3 Limit [-]	Class
1	SO	117	21	-6,294e+03	-6,294e+03								
3	SO	117	21	-6,294e+03	-6,294e+03								
4	I	344	12	-5,117e+03	5,678e+03	-0,90		0,50	29,91	71,55	82,39	112,71	1
5	SO	117	21	6,855e+03	6,855e+03	1,00	0,43	1,00	5,58	9,00	10,00	14,00	1
7	SO	117	21	6,855e+03	6,855e+03	1,00	0,43	1,00	5,58	9,00	10,00	14,00	1

Yield surface intersection

For this method, a full plastic analysis is performed. This plastic analysis is based on the initial shape and uses a stress-strain diagram with the yielding plateau.



The yield surface is generated for the given section and the intersection of the actual forces is determined with this surface.
The actual intersection point (grey) does not always collide with a predetermined point of the surface, so small deviations can occur. From the location of the plastic neutral axis, which results of this analysis, the α value for the different parts can be determined.

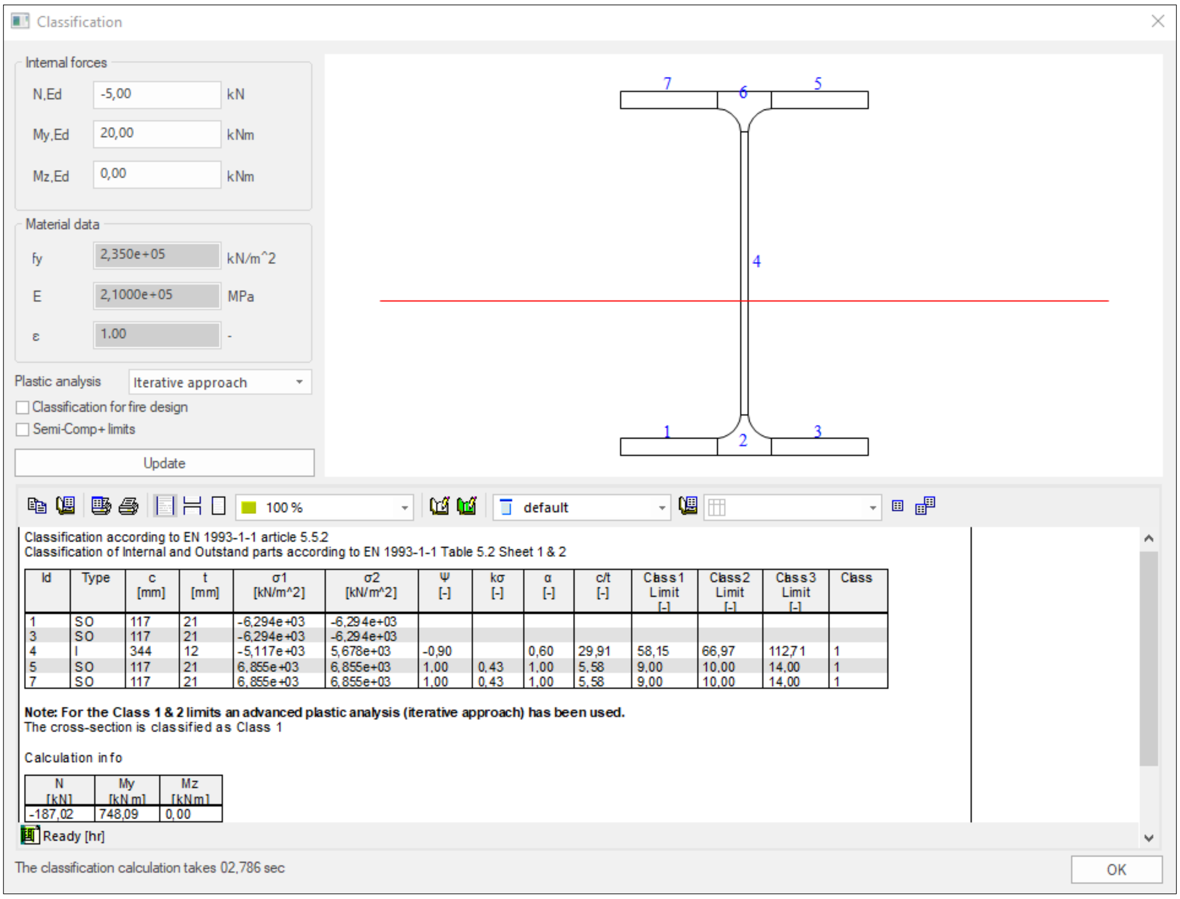


Remark: It can happen that the plastic neutral axis is rotated, but this has a negligible effect on α. This happens when the closest point on the yield surface has a minor M_z moment.

Iterative approach

Also for this method, a full plastic analysis is performed. This plastic analysis is based on the initial shape and uses a stress-strain diagram with the yielding plateau.

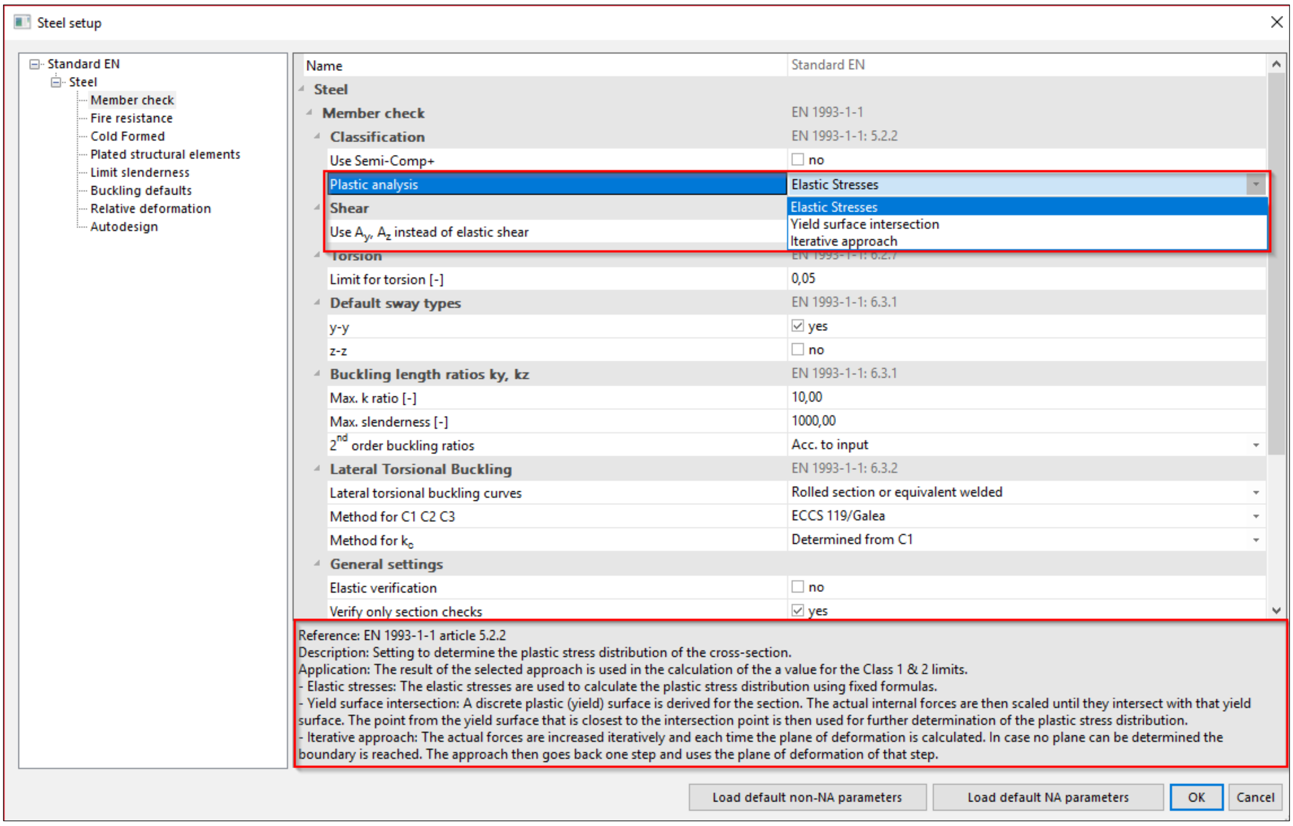
The actual plane of deformation for the given internal forces in determined iteratively which provides an exact solution.



Id	Type	c [mm]	t [mm]	σ1 [kN/m^2]	σ2 [kN/m^2]	Ψ [-]	kσ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	117	21	-6,294e+03	-6,294e+03								
3	SO	117	21	-6,294e+03	-6,294e+03								
4	I	344	12	-5,117e+03	5,678e+03	-0,90		0,60	29,91	58,15	66,97	112,71	1
5	SO	117	21	6,855e+03	6,855e+03	1,00	0,43	1,00	5,58	9,00	10,00	14,00	1
7	SO	117	21	6,855e+03	6,855e+03	1,00	0,43	1,00	5,58	9,00	10,00	14,00	1

N [kN]	My [kNm]	Mz [kNm]
-187,02	748,09	0,00

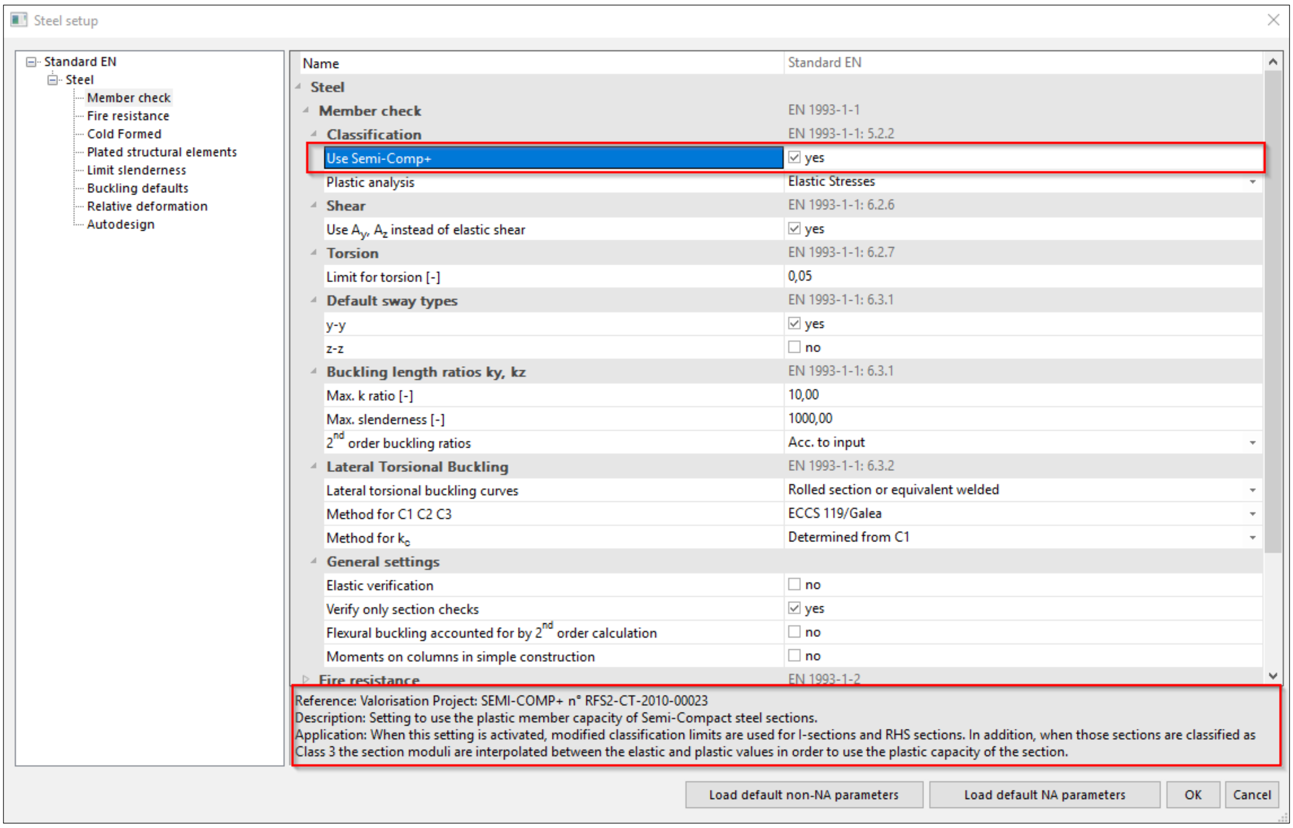
In the cross-section properties, the user can choose the plastic analysis method. However, this is only informative and this choice is not decisive. The method that will be used to perform the steel code check can be chosen in the **Steel setup** window:



Semi-Comp+

Another development that has been available since SCIA Engineer 17.0, is the classification of Semi-compact steel sections. Semi-compact steel sections are sections which are classified as class 3.

This option can be activated in the **Steel setup**:



The use of this option has a dual application:

- Classification limits are modified for I-sections and RHS sections.
- Interpolated section modulus between the elastic and plastic values

Adaption classification limits

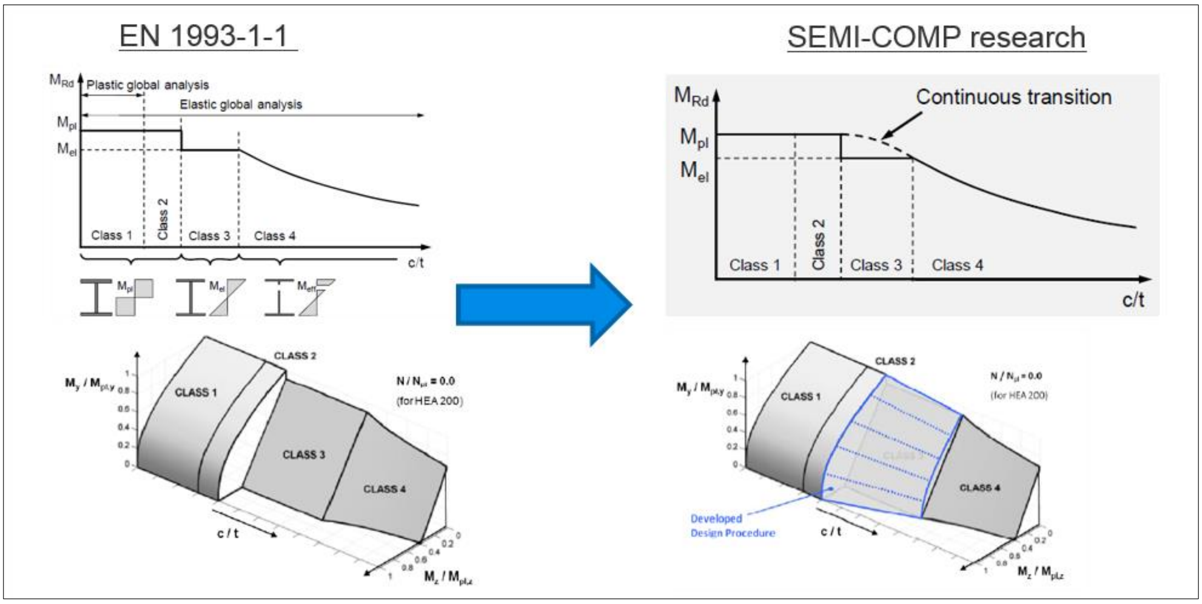
At the moment, there is a discrepancy in the used classification limits between EN1993-1-1 and EN1993-1-5. The Semi-Comp+ publication identified this discrepancy and proposed new classification limits (Ref. [36]):

Classification criterion for compression parts: $\bar{\lambda}_p = \frac{c/t}{28{,}427\varepsilon\sqrt{k_\sigma}} = \bar{\lambda}_{p,min}$	ESDEP-Background		EN 1993-1-1
	$\bar{\lambda}_{p,min}$	c/t-limit ($*\varepsilon$)	
Limit between class 3/4:			
internal part in compression	0,673 *)	38,23	42
in bending	0,874	121,35	124
outstand flange in compression	0,748	13,93	14
Limit between class 2/3:			
internal part in compression	0,6	34,11	38
in bending	0,6	83,38	83
outstand flange in compression	0,6	11,18	10
Limit between class 1/2:			
internal part in compression	0,5	28,43	33
in bending	0,5	69,48	72
outstand flange in compression	0,5	9,32	9
*) previously	0,74	42,07	42

Important remark: The adaptions on the classification limits will be implemented in the next iteration of EN1993-1-1.

Interpolated section modulus

The section moduli will be interpolated between the elastic and plastic values. This results in the advantage that the plastic capacity of the section is taken into account for class 3 sections.



If the 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.

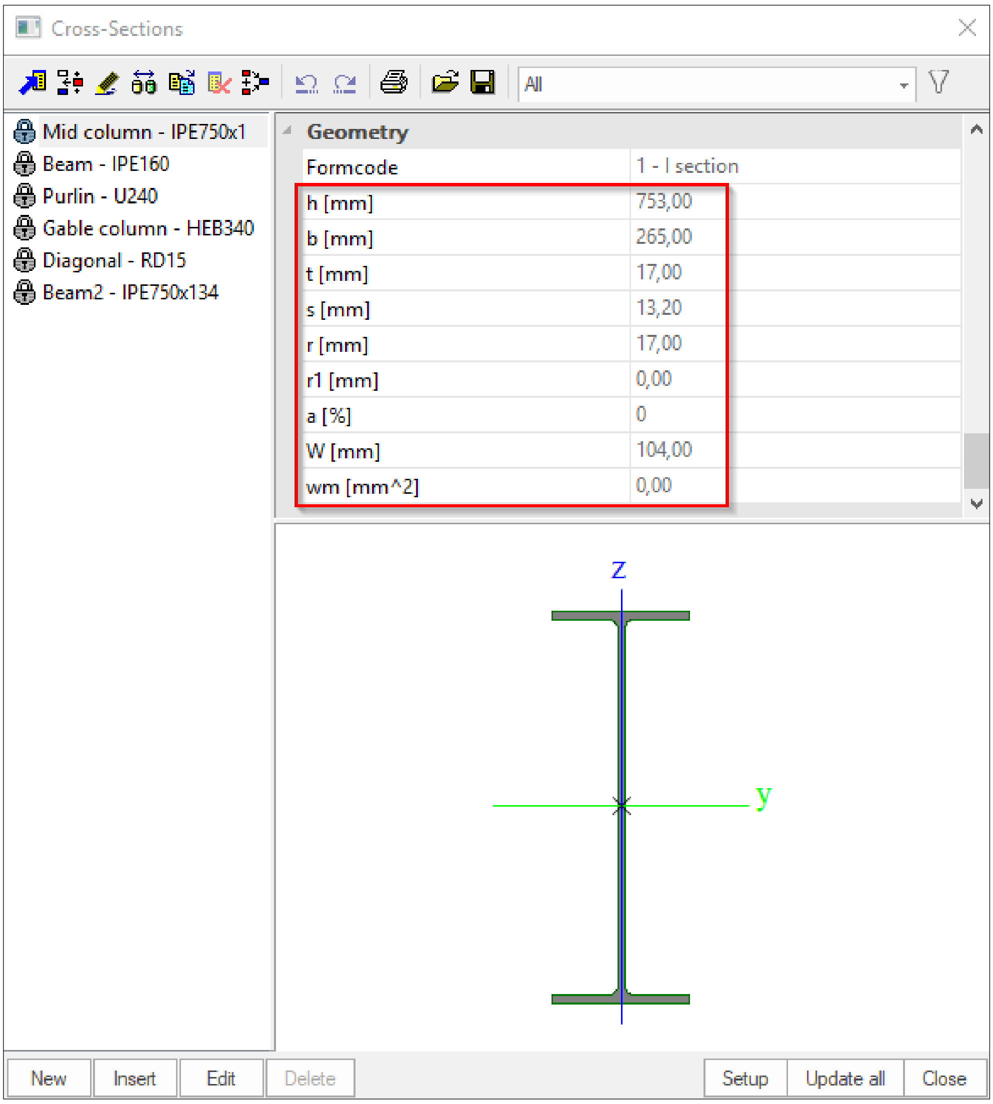
- W_{eff} is the effective section modulus of the cross-section when subject only to moment about the relevant axis.
- e_N is the shift of the relevant centroidal axis when the cross section is subject to uniform compression.

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

Example: Industrial Hall.esa

In this example the classification will be done for an IPE750x134 profile, resulting in a class 4 cross section. And afterwards the calculation of the effective shape following EN 1993-1-5 will be given.

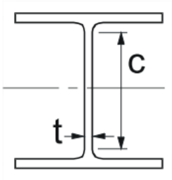
Consider column B28



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):

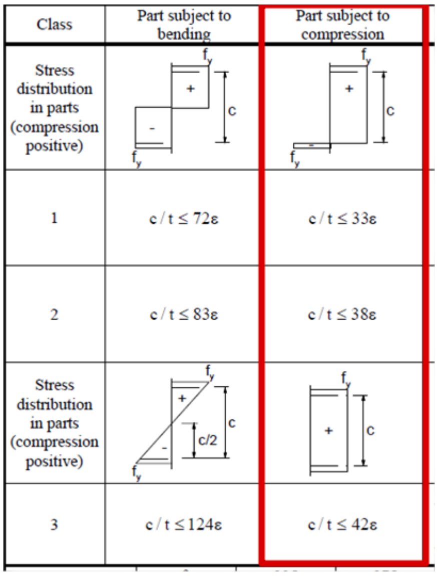


c = H – 2 * thickness flange – 2* radius

c = 753mm – 2x17mm – 2x17mm = 685mm

t = 13,20 mm

c/t = **51,89**



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

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

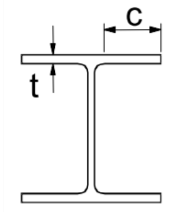
Maximum ratio class 1: 33

Maximum ratio class 2: 38

Maximum ratio class 3: 42

c/t = 51,89 > 42 => Class 4

Table 5.2 (sheet 2):

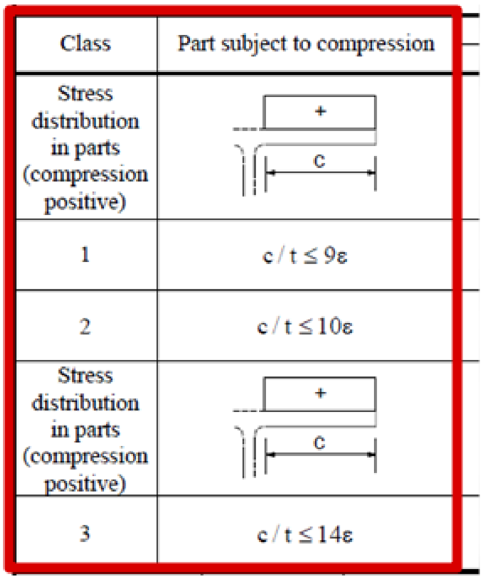


c = B/2 – thickness web/2 – radius

c = 265mm/2 – 13,20mm/2 – 17 = 108,9mm

t = 17 mm

c/t = **6,40**



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

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 according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	108.90	17.00	8653.916	8653.916	1.0	0.4	1.0	6.4	9.0	10.0	14.0	1
3	SO	108.90	17.00	8653.916	8653.916	1.0	0.4	1.0	6.4	9.0	10.0	14.0	1
4	I	685.00	13.20	8653.916	8653.916	1.0		1.0	51.9	33.0	38.0	42.0	4
5	SO	108.90	17.00	8653.916	8653.916	1.0	0.4	1.0	6.4	9.0	10.0	14.0	1
7	SO	108.90	17.00	8653.916	8653.916	1.0	0.4	1.0	6.4	9.0	10.0	14.0	1

The cross-section is classified as Class 4

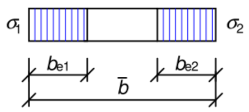
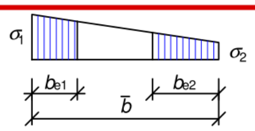
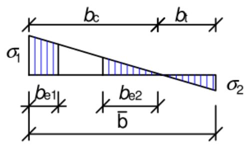
This cross section has a classification class 4 for the stability classification, so effective properties have to be calculated.
Those properties are also given in SCIA Engineer in the preview of the steel code check, just below the classification calculation.

Effective properties

Effective area	A_{eff}	1.7222e+04	mm²			
Effective second moment of area	$I_{eff,y}$	1.6608e+09	mm⁴	$I_{eff,z}$	5.2895e+07	mm⁴
Effective section modulus	$W_{eff,y}$	4.4111e+06	mm³	$W_{eff,z}$	3.9920e+05	mm³
Shift of the centroid	$e_{N,y}$	0.00	mm	$e_{N,z}$	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}
	$\psi = 1$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = 0{,}5\ b_{eff}$ $\quad b_{e2} = 0{,}5\ b_{eff}$
	$1 > \psi \geq 0$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = \dfrac{2}{5-\psi} b_{eff}$ $\quad b_{e2} = b_{eff} - b_{e1}$
	$\psi < 0$: $b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$ $b_{e1} = 0{,}4\ b_{eff}$ $\quad b_{e2} = 0{,}6\ b_{eff}$

$\psi = \sigma_2/\sigma_1$	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	$5{,}98\ (1 - \psi)^2$

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

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

ε = 1,00 (S235)

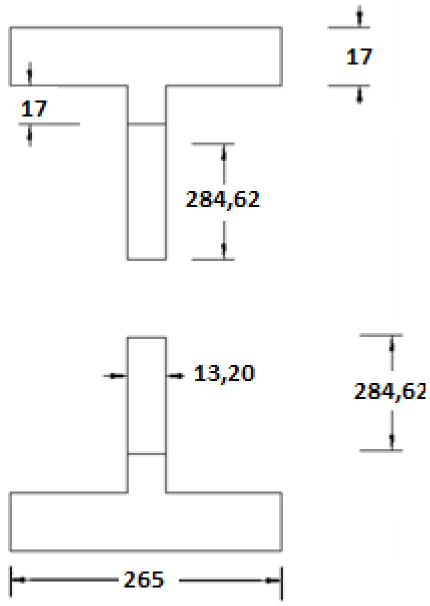
k_σ = 4,0 (Tabel 4.1 of EN 1993-1-5:2006)

$$\overline{\lambda_p} = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{51.89}{28.4 \cdot 1.00 \cdot \sqrt{4.0}} = 0.91$$

$$\rho = \frac{0.91 - 0.22}{(0.91)^2} = 0{,}83$$

$$\mathrm{b_{e1}} = \mathrm{b_{e2}} = 0.5 \cdot \mathrm{b_{eff}} = 0{,}5 \cdot \rho \cdot \bar{\mathrm{b}} = 0{,}5 \cdot 0{,}83 \cdot 685\ \mathrm{mm} = 284{,}62\ \mathrm{mm}$$

$$A_{eff} = [265 \cdot 17 + 284{,}62 \cdot 13{,}20 + 17 \cdot 13{,}20] \cdot 2 = 16972{,}77\ \mathrm{mm}^2$$

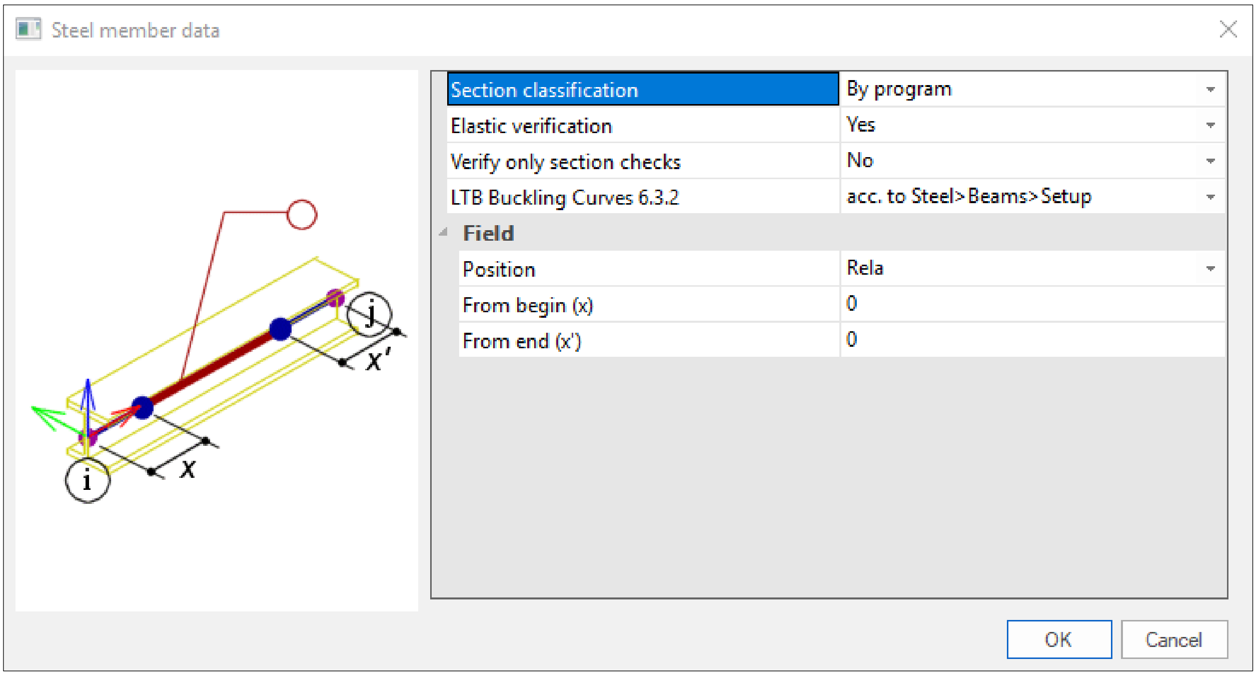


In this calculation the roundings at the corners between the flanges and the web are not taken into account. Therefore the result in SCIA Engineer will be a bit higher:

Effective properties						
Effective area	A_{eff}	1.7222e+04	mm^2			
Effective second moment of area	$I_{eff,y}$	1.0608e+09	mm^4	$I_{eff,z}$	5.2895e+07	mm^4
Effective section modulus	$W_{eff,y}$	4.4111e+06	mm^3	$W_{eff,z}$	3.9920e+05	mm^3
Shift of the centroid	$e_{N,y}$	0.00	mm	$e_{N,z}$	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.

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 Checks - EC-EN 1993 Steel Check ULS”. In this menu the user can choose to look at the “Brief, Summary or Detailed” output.

With the brief output the results are shown in one line.
With the summary output, the results of all unity checks are shown.
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.

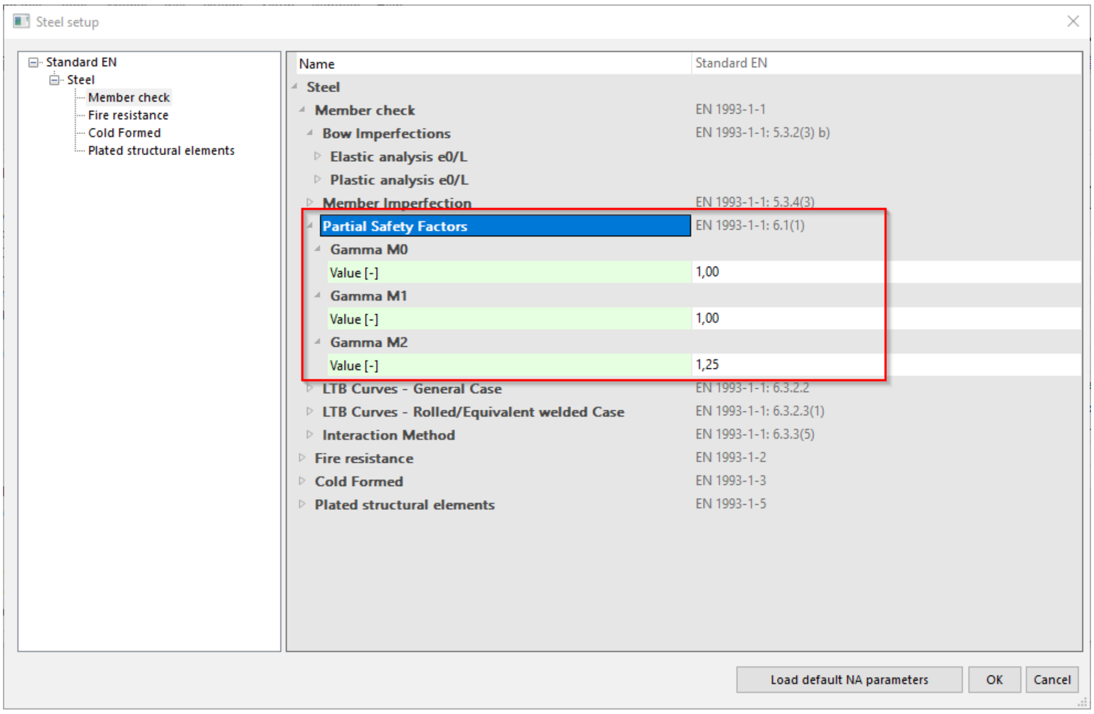
Partial Safety factors

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 B23 (for load case 3DWind1). In this load case the bracing is under tension:

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

$$Unity\ Check = \frac{30{,}13kN}{41{,}51kN} = 0{,}73$$

Internal forces	Calculated	Unit
N_{Ed}	30.13	kN
$V_{y,Ed}$	0.00	kN
$V_{z,Ed}$	0.00	kN
T_{Ed}	0.00	kNm
$M_{y,Ed}$	0.00	kNm
$M_{z,Ed}$	0.00	kNm

Classification for cross-section design
Warning: Classification is not supported for this type of cross-section.
The section is checked as elastic, class 3.

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

A	1.7663e+02	mm^2
$N_{pl,Rd}$	41.51	kN
$N_{u,Rd}$	45.78	kN
$N_{t,Rd}$	41.51	kN
Unity check	0.73	-

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}	-160.01	kN
$V_{y,Ed}$	-0.03	kN
$V_{z,Ed}$	-101.68	kN
T_{Ed}	0.00	kNm
$M_{y,Ed}$	-116.93	kNm
$M_{z,Ed}$	-0.04	kNm

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

$$N_{c,Rd} = \frac{1{,}8800 \cdot 10^4 mm^2 \cdot 235MPa}{1.0} = 4418{,}0\ kN$$

$$Unity\ Check = \frac{160.01kN}{4418.0kN} = 0{,}04$$

Compression check

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

A	1.8800e+04	mm²
$N_{c,Rd}$	4418.00	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

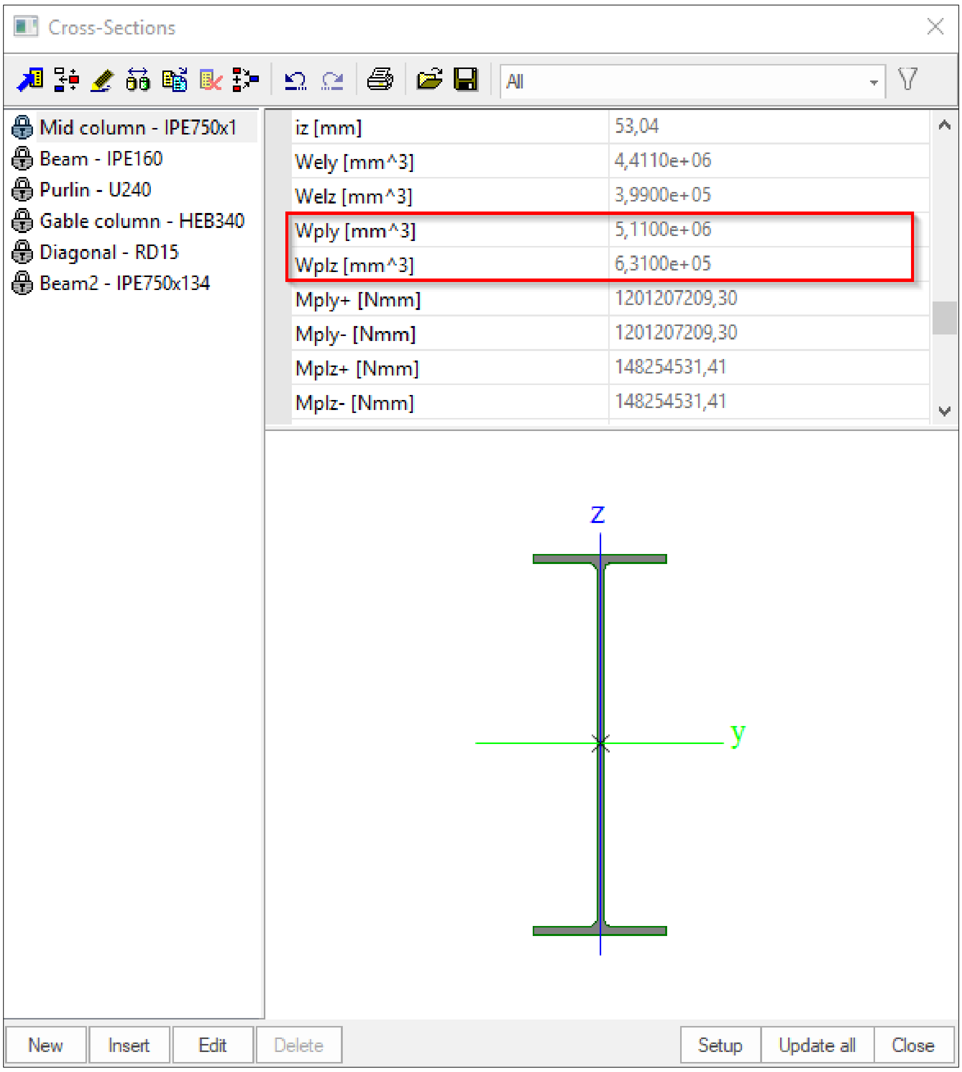
Example: Industrial Hall.esa

Consider column B28 (for combination CO1-ULS).

The critical check is on position 1.150 m

Internal forces	Calculated	Unit
N_{Ed}	-160.01	kN
$V_{y,Ed}$	-0.03	kN
$V_{z,Ed}$	-101.68	kN
T_{Ed}	0.00	kNm
$M_{y,Ed}$	-116.93	kNm
$M_{z,Ed}$	-0.04	kNm

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



$$M_{y,c,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = \frac{5.1100 \cdot 10^6 mm^3 \cdot 235MPa}{1.0} = 1{,}20085 \cdot 10^9\ Nmm = 1200{,}85\ kNm$$

$$M_{z,c,Rd} = \frac{W_{pl,z} \cdot f_y}{\gamma_{M0}} = \frac{6.3100 \cdot 10^5 mm^3 \cdot 235MPa}{1.0} = 1{,}4828\ \cdot 10^8\ Nmm = 148{,}28\ kNm$$

$$Unity\ Check\ My = \frac{116{,}93\ kNm}{1200{,}85\ kNm} = 0{,}10$$

$$Unity\ Check\ Mz = \frac{0{,}04\ kNm}{148{,}28\ kNm} = 0{,}00$$

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

$W_{pl,y}$	5.1100e+06	mm^3
$M_{pl,y,Rd}$	1200.85	kNm
Unity check	0.10	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

$W_{pl,z}$	6.3100e+05	mm^3
$M_{pl,z,Rd}$	148.28	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}	-160.01	kN
$V_{y,Ed}$	-0.03	kN
$V_{z,Ed}$	-101.68	kN
T_{Ed}	0.00	kNm
$M_{y,Ed}$	-116.93	kNm
$M_{z,Ed}$	-0.04	kNm

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

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

η	1.20	
A_v	9.4086e+03	mm²
$V_{pl,y,Rd}$	1276.54	kN
Unity check	0.00	-

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

η	1.20	
A_v	1.1389e+04	mm²
$V_{pl,z,Rd}$	1545.22	kN
Unity check	0.07	-

Torsion

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

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

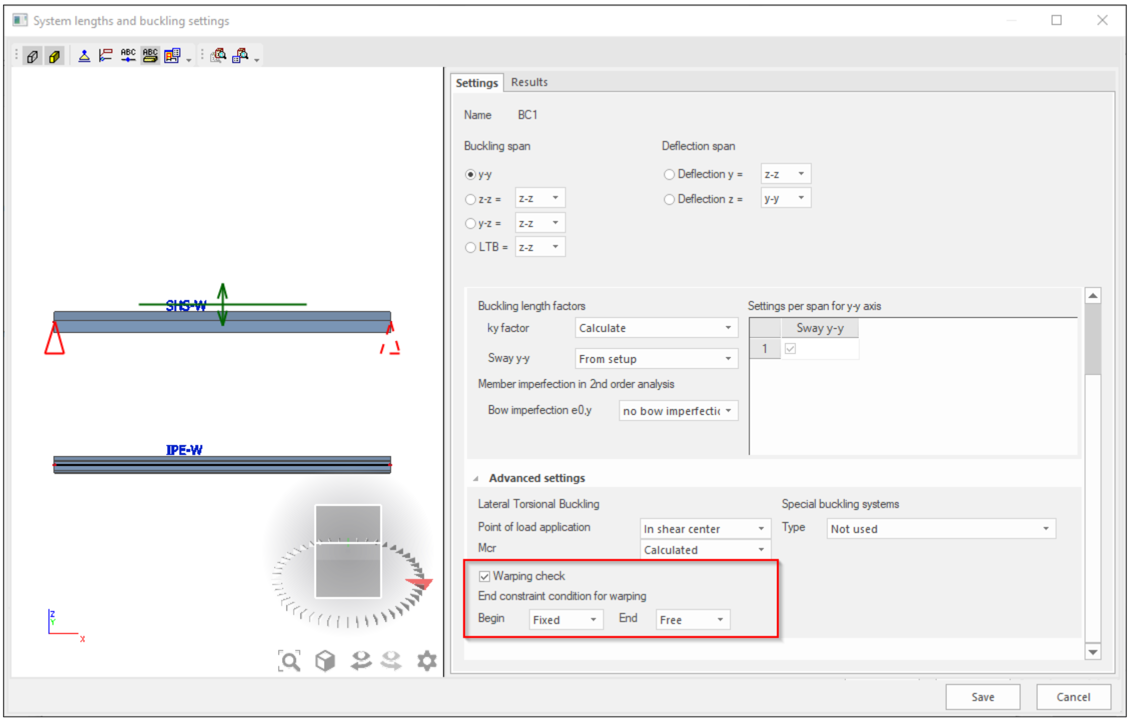
Where T_{Rd} is the design torsional resistance of the cross-section.

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 within the “System lengths and buckling settings” of this member:



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

Example: Warping.esa

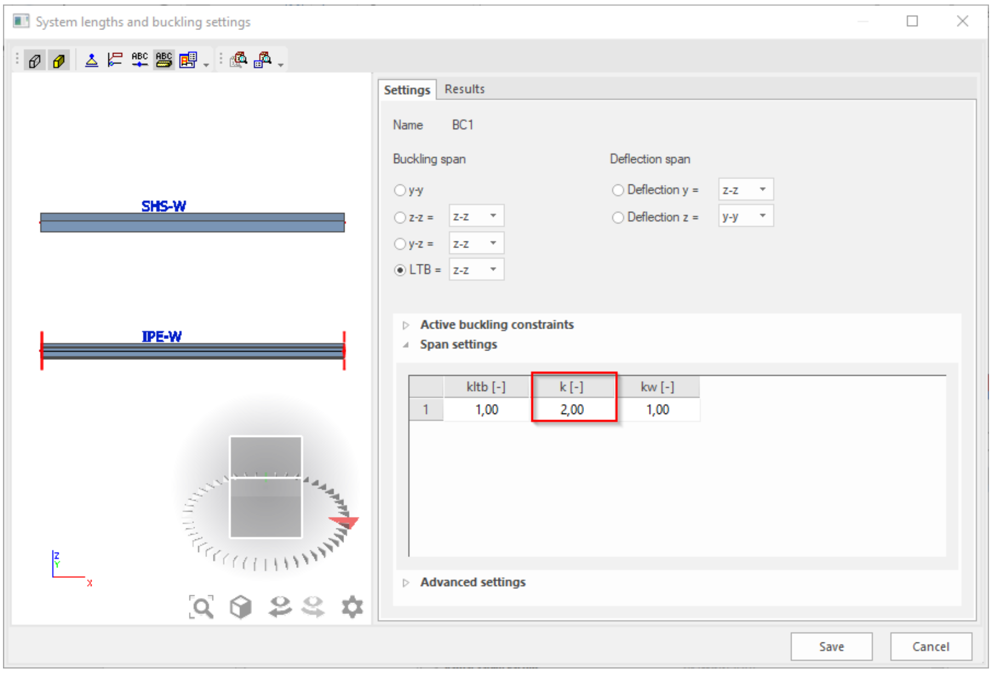
As quoted in the EN 1993-1-1, art. 6.2.7(7), for closed hollow sections the effects of torsional warping can be neglected and in the case of a member with an open cross section, such as I of H, it may be assumed that the effects of St.Venant torsion can be neglected. This article is a simplification and is valid for SCIA Engineer version up to and until SCIA Engineer 16.1.

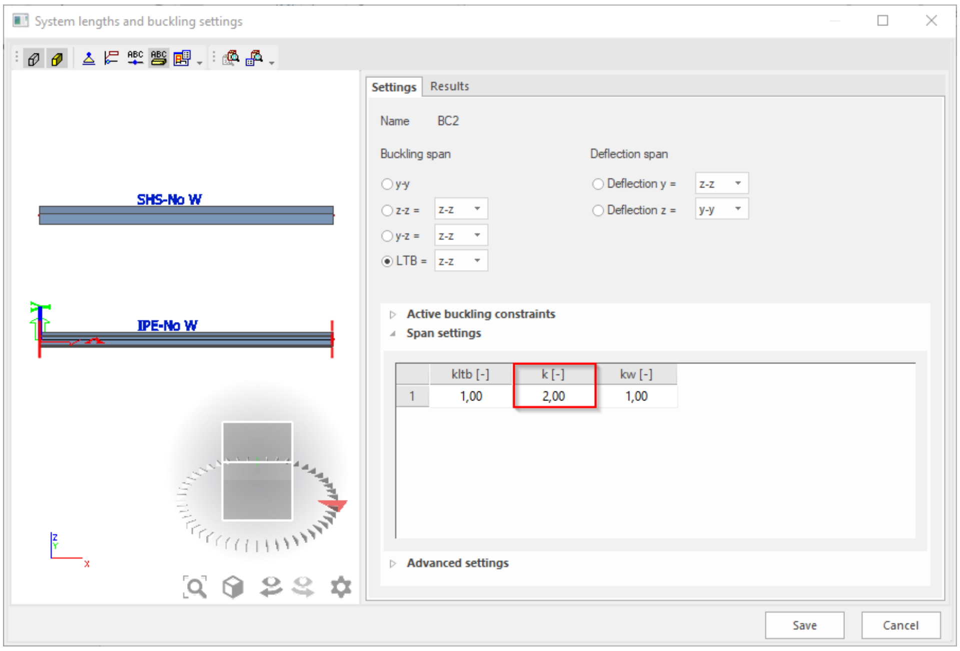
Since SCIA Engineer 17.0, the warping check will also be performed for closed hollow sections, even though the fact that this will not have the same influence as for open profiles.

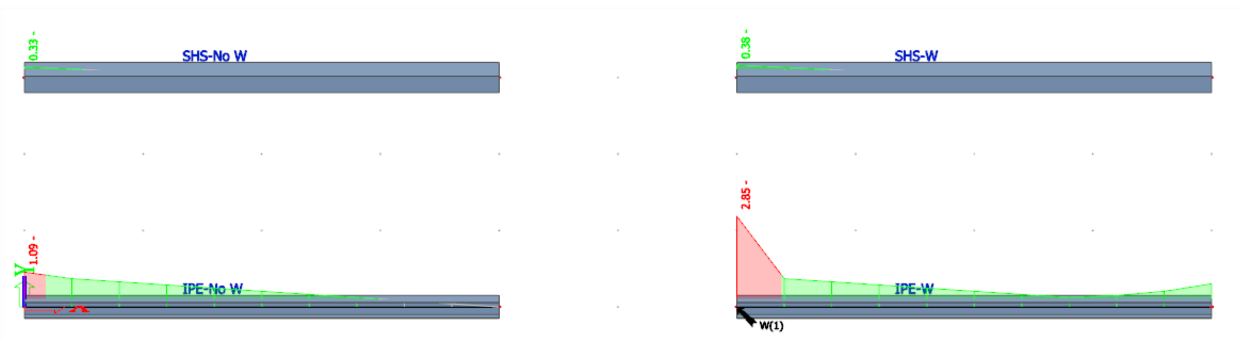
In this example 4 beams are inputted:

- 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 2,00 within the buckling data for each buckling group.







For the **SHS profiles**, there is a small difference between the option "Warping" activated or not. This was expected as warping is not important 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)

Fibre	2	
τ_{Ed}	133.6	MPa
τ_{Rd}	135.7	MPa
Unity check	0.98	-

Combined Shear and Torsion check for V_z and $T_{t,Ed}$

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

$V_{pl,T,z,Rd}$	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)

Fibre	2	
τ_{Ed}	133.6	MPa
τ_{Rd}	135.7	MPa
Unity check	0.98	-

Combined Shear and Torsion check for V_z and $\tau_{t,Ed}$
According to EN 1993-1-1 article 6.2.6 & 6.2.7 and formula (6.25),(6.26)

Fibre	1	
$\tau_{t,Ed}$	0.0	MPa
$V_{pl,T,z,Rd}$	152.01	kN
Unity check	0.11	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.7 & 6.2.1(5) and formula (6.1)

Warping conditions at the extremities of the buckling system

Extremity	Condition
Begin	Free
End	Fixed

Decomposition of torsional moment

x [m]	$M_{xp,Ed}$ [kNm]	$M_{xs,Ed}$ [kNm]	$M_{w,Ed}$ [kNm²]
0.000	0.00	1.60	-0.86
0.400	-0.21	0.85	-0.38
0.800	0.03	0.45	-0.13
1.200	0.09	0.23	0.01
1.600	0.06	0.10	0.09
2.000	-0.02	0.02	0.13
2.000	-0.02	0.02	0.13
2.400	-0.11	-0.05	0.17
2.800	-0.18	-0.14	0.20
3.200	-0.19	-0.29	0.25
3.600	-0.09	-0.55	0.34
4.000	0.25	-1.05	0.51

Internal forces			
St. Venant torsion	$M_{xp,Ed}$	0.00	kNm
Warping torsion	$M_{xs,Ed}$	1.60	kNm
Bimoment	$M_{w,Ed}$	-0.86	kNm²

Elastic verification		
Fibre	1	
$\sigma_{N,Ed}$	0.0	MPa
$\sigma_{My,Ed}$	218.7	MPa
$\sigma_{Mz,Ed}$	0.0	MPa
$\sigma_{w,Ed}$	450.4	MPa
$\sigma_{tot,Ed}$	669.1	MPa
$\tau_{Vy,Ed}$	0.0	MPa
$\tau_{Vz,Ed}$	0.0	MPa
$\tau_{t,Ed}$	0.0	MPa
$\tau_{w,Ed}$	0.0	MPa
$\tau_{tot,Ed}$	0.0	MPa
$\sigma_{von\ Mises,Ed}$	669.1	MPa
Unity check	**2.85**	-

Warning: Due to extreme internal forces the plastic warping check according to I. Vayas cannot be executed. Therefore the elastic yield criterion according to EN 1993-1-1 article 6.2.1(5) is verified.

As stated before, since SCIA Engineer 17.0, the same warping check will be executed for the closed SHS profile.

The **torsional properties of the cross sections** are not calculated correctly in a lot of cases. The next example gives 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 be taken as 0.00 mm⁶.

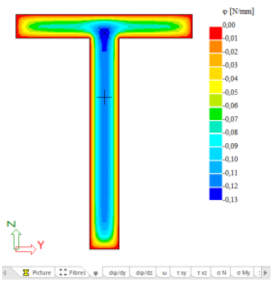
Property	
A [mm^2]	7,5000e+03
Ay [mm^2]	3,8894e+03
Az [mm^2]	4,3810e+03
AL [m^2/m]	7,0000e-01
AD [m^2/m]	7,0000e-01
cYUCS [mm]	75
cZUCS [mm]	130
α [deg]	0,00
Iy [mm^4]	3,0250e+07
Iz [mm^4]	5,8594e+06
iy [mm]	64
iz [mm]	28
Wely [mm^3]	2,3269e+05
Welz [mm^3]	7,8125e+04
Wply [mm^3]	4,1250e+05
Wplz [mm^3]	1,4062e+05
Mply+ [Nmm]	96937500,00
Mply- [Nmm]	96937500,00
Mplz+ [Nmm]	33046875,00
Mplz- [Nmm]	33046875,00
dy [mm]	0
dz [mm]	0
It [mm^4]	2,5672e+06
Iw [mm^6]	0,0000e+00
β y [mm]	-25
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
Show mesh	☐

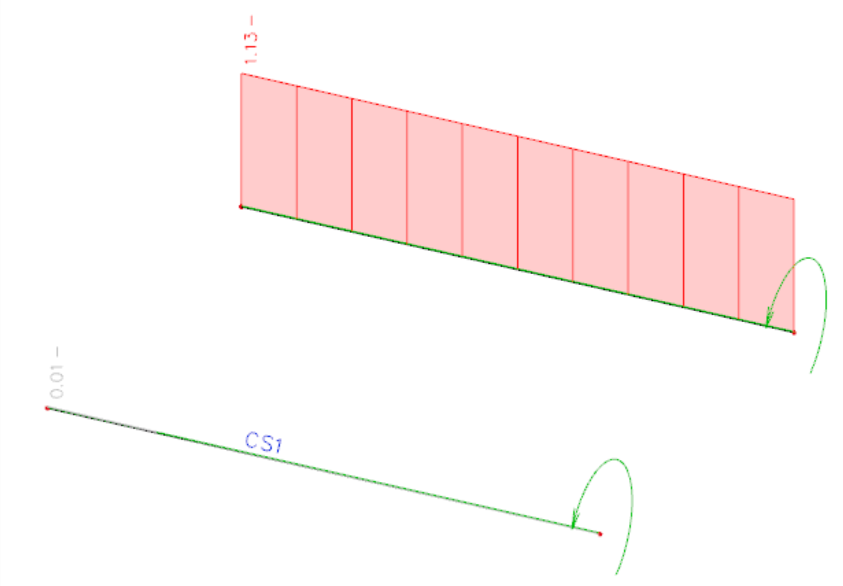
So the value for It and Iw will be calculated with a FEM analysis:



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]	1,4062e+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 results 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 stiffness' 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
$V_{y,Ed}$	0.00	kN
$V_{z,Ed}$	0.40	kN
T_{Ed}	-8.00	kNm
$M_{y,Ed}$	-0.80	kNm
$M_{z,Ed}$	0.00	kNm

Torsion check

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

Fibre	10	
τ_{Ed}	153.0	MPa
τ_{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:

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

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	1.8800e+04	mm²
$N_{c,Rd}$	4418.00	kN
Unity check	0.04	-

⇨ n = 0,04

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

η	1.20	
A_v	9.4086e+03	mm²
$V_{pl,y,Rd}$	1276.54	kN
Unity check	0.00	-

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

η	1.20	
A_v	1.1389e+04	mm²
$V_{pl,z,Rd}$	1545.22	kN
Unity check	0.07	-

⇨ 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)

$M_{pl,y,Rd}$	1200.85	kNm
α	2.00	
$M_{pl,z,Rd}$	148.28	kNm
β	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 Checks -> EC-EN 1993 Steel Check ULS. In this menu the user can choose to look at the “Brief, Summary or Detailed” output.

With the brief output the results are shown in one line.
With the summary output, the results of all unity checks are shown.
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’

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)

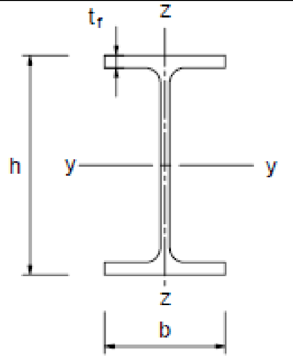
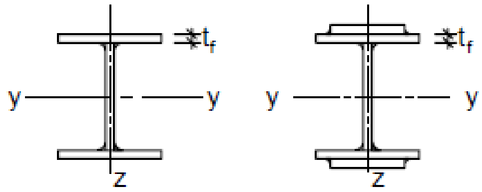
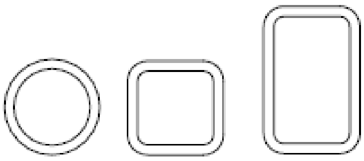
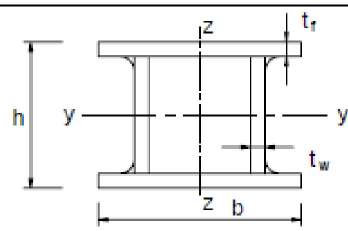
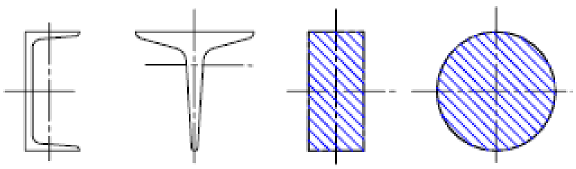
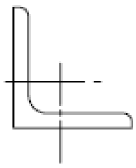
$$N_{cr} = \frac{\pi^2 EI}{k^2 L^2}$$

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		$h/b > 1,2$	$t_f \leq 40$ mm	y – y z – z	a b	a_0 a_0
			40 mm < $t_f \leq 100$	y – y z – z	b c	a a
		$h/b \leq 1,2$	$t_f \leq 100$ mm	y – y z – z	b c	a a
			$t_f > 100$ mm	y – y z – z	d d	c c
Welded I-sections			$t_f \leq 40$ 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 manually inputted by the user.

Since SCIA Engineer 18.0 a new dialog is introduced for applying buckling settings on a specific buckling system called **System lengths and buckling settings**. Prior to SCIA Engineer 18.0 there was a dialog for the buckling settings called "Buckling and relative lengths" which offered similar settings but without the graphical window and even without the results.

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

Example: Buckling Curves.esa

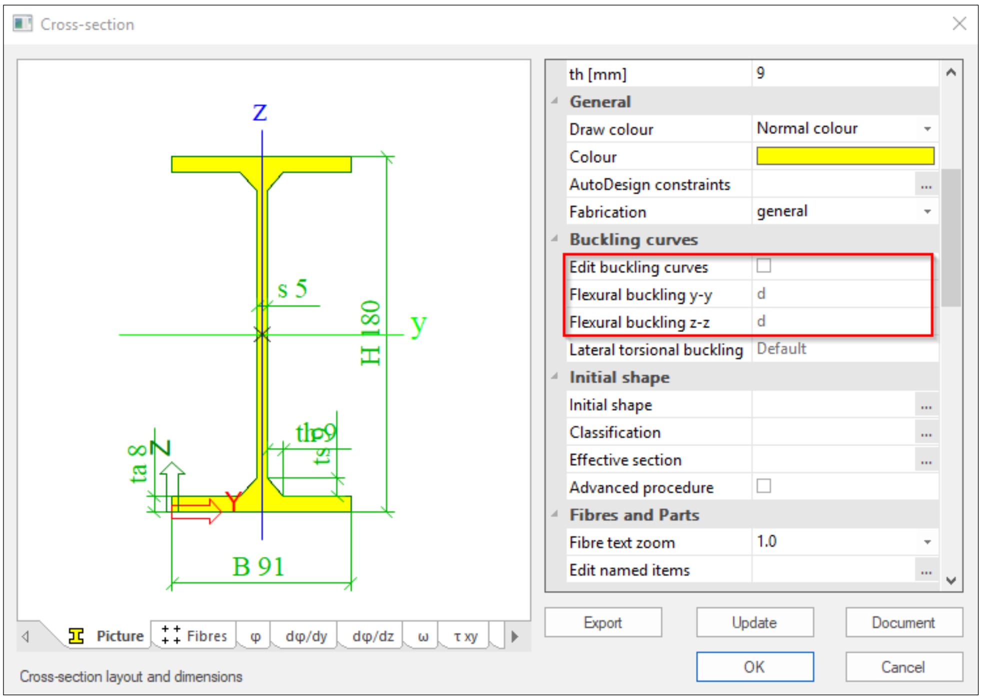
In Table 6.2 general buckling curves are given for the most common profiles. For some cross-section types, SCIA Engineer will automatically use these curves. The cross-section types that are supported are:

- 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



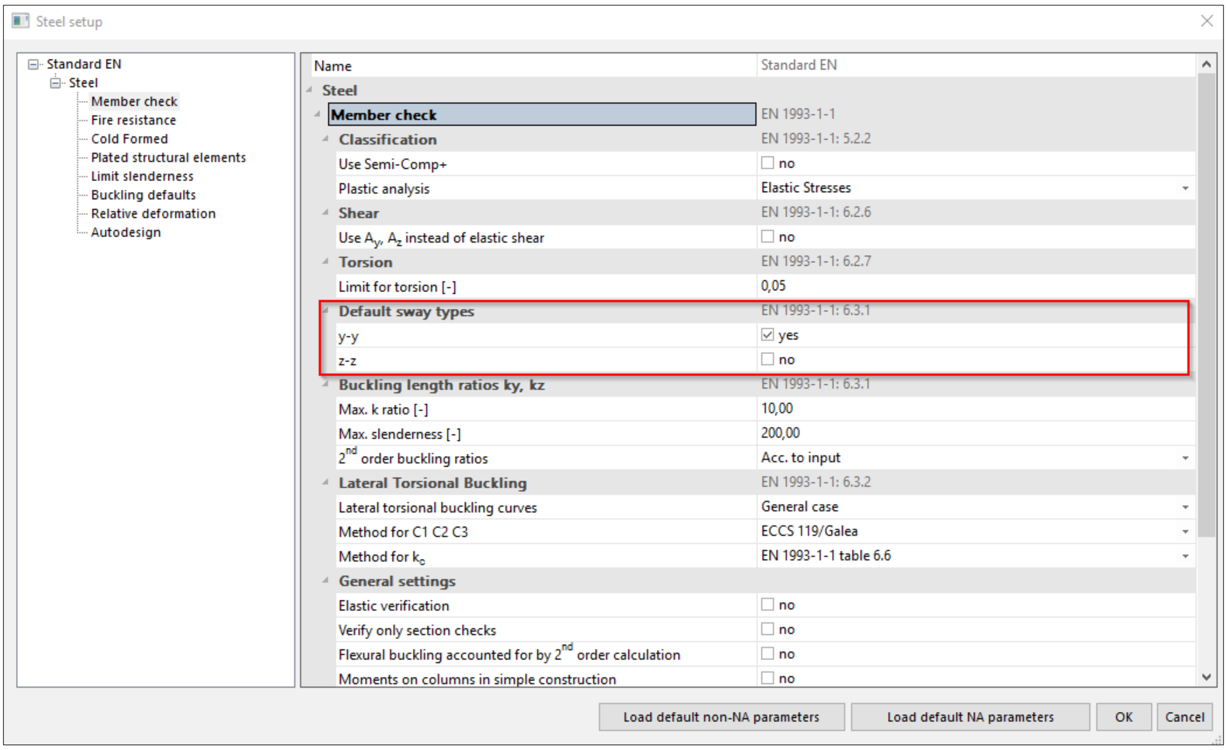
Buckling factors

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.

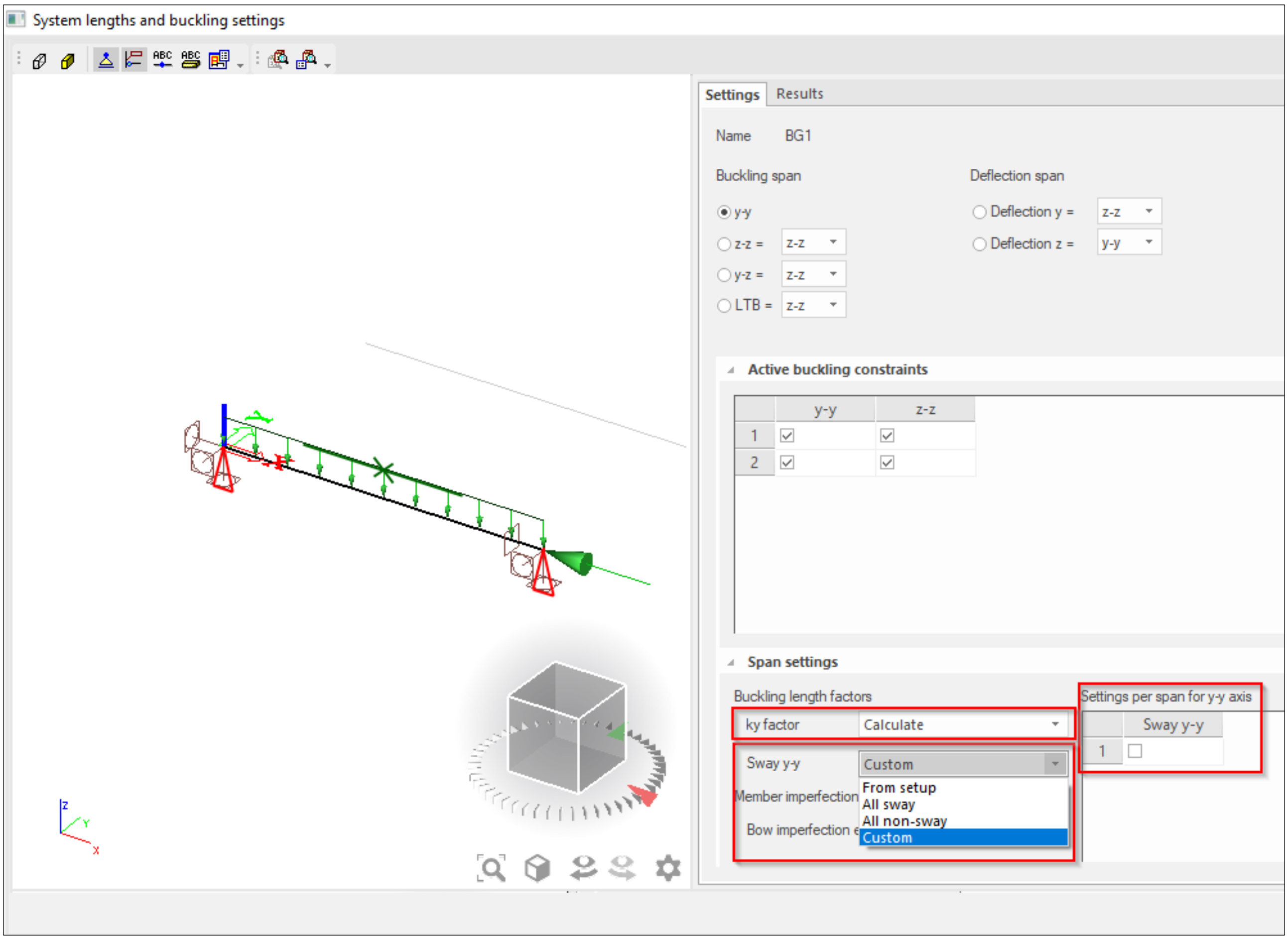
So it is important for this method that the user chooses the correct option sway or non-sway for the two local directions:
yy: buckling around the local y-axis (so deformation in the direction of the local z-axis)
zz: buckling around the local z-axis (so deformation in the direction of the local y-axis)

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



If 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 System lengths and buckling settings. This can be found for each beam in Property window -> Buckling -> System lengths and buckling settings -> Settings -> Span settings. These properties can be inputted in the graphical window as well.



For Sway y-y and Sway z-z there are 4 options:

- According to Steel>Beams>Setup: The same option will be taken as in the steel setup as shown above.
- All sway: sets all spans of the axis system as sway.
- All non-sway: sets all spans of the axis system as non-sway.
- Custom: allows editing the sway settings per span.

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} = 79883 kNm
  - ϕ_1 = fiy = 1523.3 mrad
  - $C_1 = M_{y1}/\phi_1$ = 79883 kNm / 1523.3 mrad = 52.44 kNm/mrad
    = 5,44 x 10^{10} Nmm/rad
  - E = 210 000 N/mm²

- Iy = 162700000 mm^4
- $\rho_1 = \frac{C_i L}{EI} = \frac{5.44\cdot\frac{10^{10}Nmm}{rad}\cdot 4000mm}{210000\frac{N}{mm^2}\cdot 162700000\,mm^4} = 6.369$
- $x = \frac{4\rho_1\rho_2+\pi^2\rho_1}{\pi^2(\rho_1+\rho_2)+8\rho_1\rho_2} = \frac{4\cdot 6.369\cdot 0.0+\pi^2\cdot 6.369}{\pi^2(6.369+0.0)+8\cdot 6.369\cdot 0.00} = 1.0$
- $k = x\sqrt{\frac{\pi^2}{\rho_1 x}+4} = 1.0\sqrt{\frac{\pi^2}{6.369\cdot 1.00}+4} = 2.36$
- $N_{cr} = \frac{\pi^2 EI}{k^2L^2} = \frac{\pi^2\cdot 210000N/mm^2\cdot 162700000\,mm^4}{(2.36)^2(4000)^2} = 3797668.777N = 3797.7\,kN$

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]	lLTB [m]
B1	CS1	1	Yes	4.000	2.36	9.459	63.23	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 - EC-EN 1993 Steel Check ULS” under the buckling parameters. And here also the critical normal force Ncr can be found:

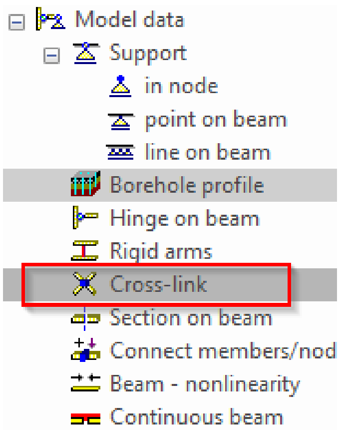
Buckling parameters	yy	zz	
Sway type	sway	sway	
System length L	4.000	4.000	m
Buckling factor k	2.36	1.00	
Buckling length L_{cr}	9.459	4.000	m
Critical Euler load N_{cr}	3768.62	1351.09	kN
Slenderness λ	63.23	105.61	
Relative slenderness λ_{rel}	0.67	1.12	
Limit slenderness $\lambda_{rel,0}$	0.20	0.20	

Calculation of the buckling factors for crossing diagonals

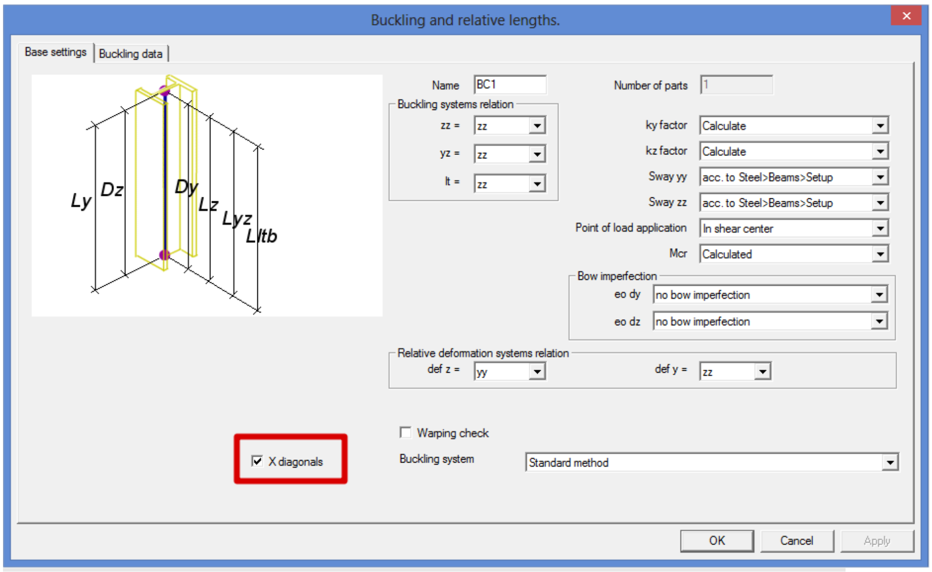
As the previous default method is only valid for perpendicular connection, this can’t be used for the calculation of the buckling factors for diagonals.

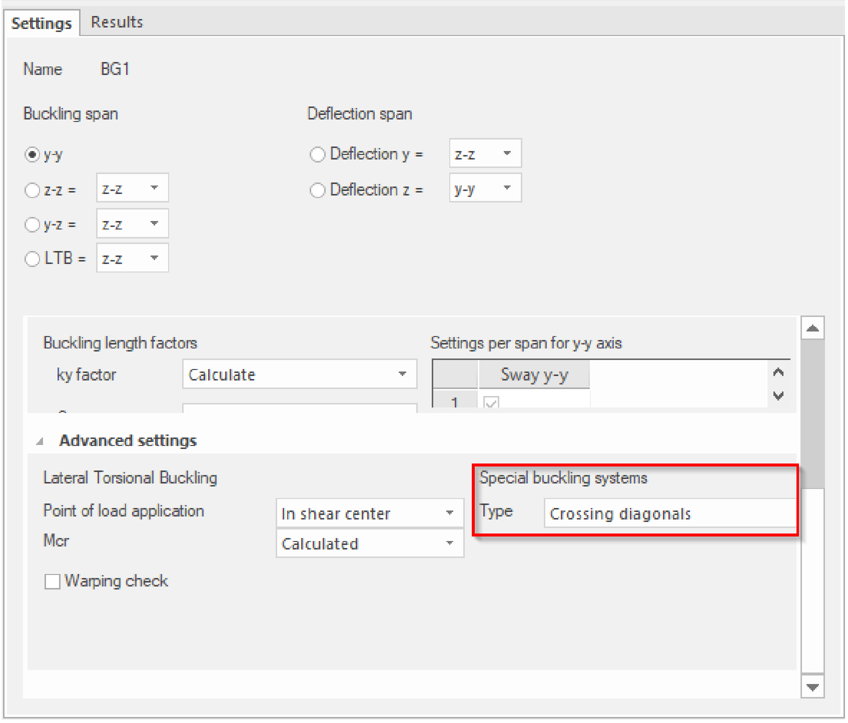
In the DIN 1880 Teil 2, Table 15 a method is given for the calculation of the buckling factor for crossing diagonals. In SCIA Engineer this option is also implemented. In this method the buckling length s_K is calculated in function of the load distribution in the element and s_K is not a purely geometrical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

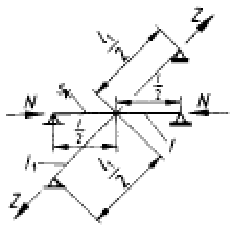
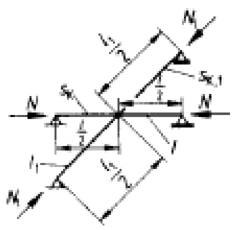
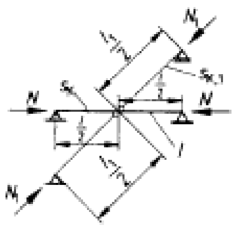
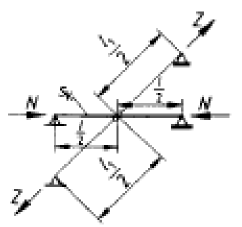
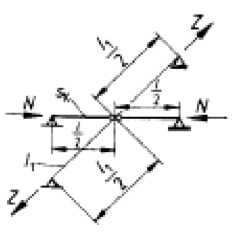
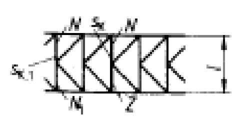


When connecting two beams with this option, in the buckling and relative lengths properties of the two beams the option **X-diagonals** can be checked (in the System lengths and buckling settings properties of the two beams the option **Crossing diagonals** can be checked)





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

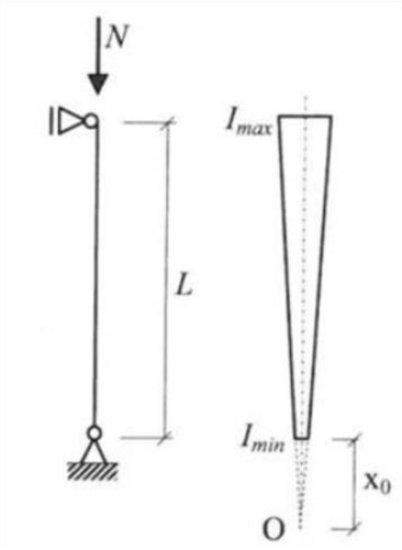
Z	tension force in supporting diagonal
E	elastic modulus

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 we can define

k_y	buckling coefficient around the yy-axis
L_y	system length around the yy-axis
$I_{y,max}$	Maximum moment of inertia around the y-axis
$I_{y,min}$	Minimum moment of inertia around the y-axis
$I_{y,eq}$	Equivalent moment of inertia around the y-axis
E	Modulus of Young
$N_{cr,y}$	Critical Euler force around the y-axis



Hirt and Crisinel Ref[xx], present expressions for the elastic critical load of axially loaded non-prismatic members of double symmetric cross-sections (i.e. I sections formcode 1). Flexural buckling around the strong axis of the cross-sections occurs for:

$$N_{cr,y} = \frac{\pi^2 \cdot E \cdot I_{y,eq}}{\left(k_y \cdot L_y\right)^2}$$

Where

$$I_{y,eq} = C \cdot I_{y,\max}$$

And C is a coefficient that depends on the parameter r, defined as the ratio between the minimum and the maximum moments of inertia.

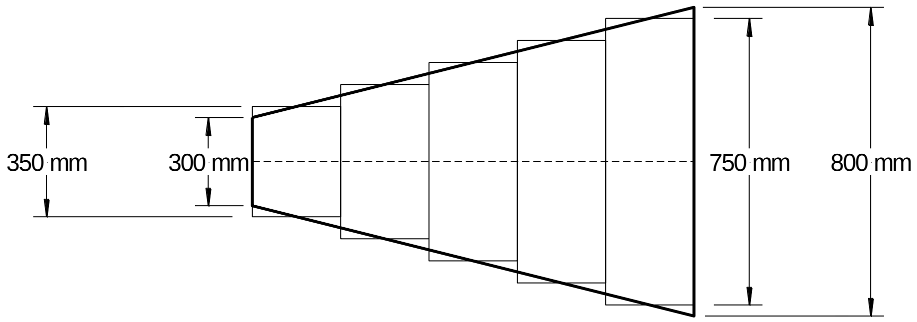
$$r = \sqrt{\frac{I_{y,\min}}{I_{y,\max}}}$$

For a tapered member, C can be calculated as:

$$C = 0,08 + 0,92 \cdot r$$

Example:

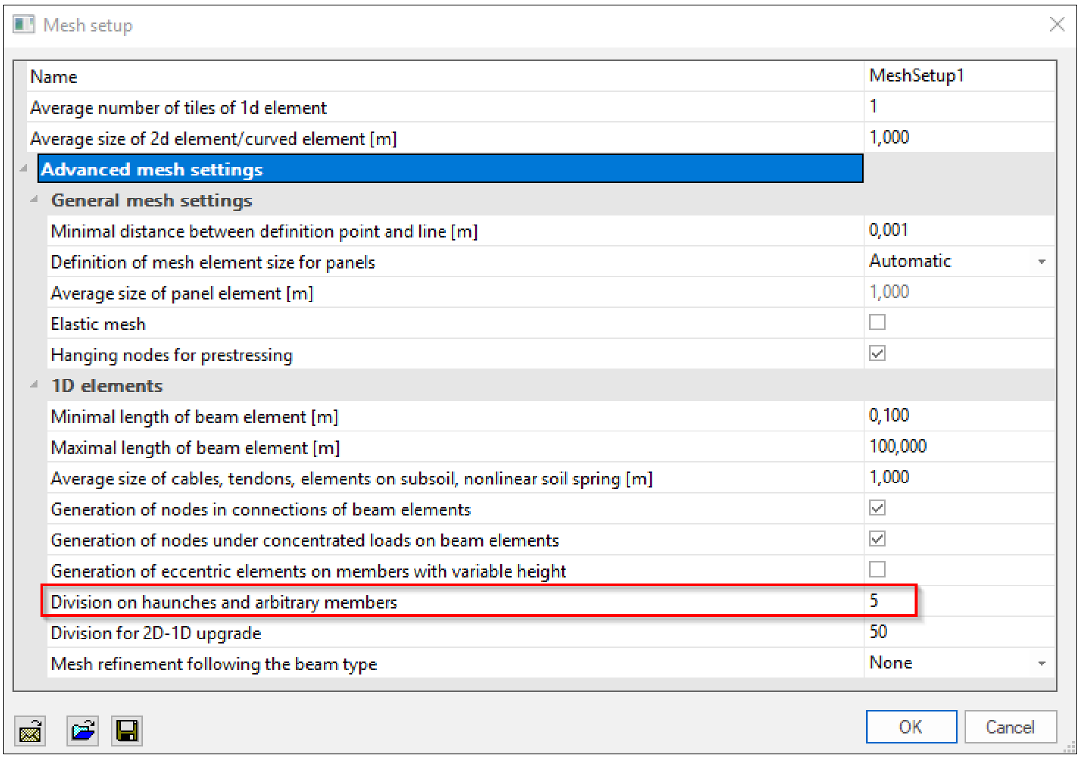
VARH.esa



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:



$$I_{y,min} = 1{,}7041e^8 mm^4$$

$$I_{y,max} = 1{,}6989e^9 mm^4$$

$$r = \sqrt{\frac{I_{y,min}}{I_{y,max}}} = \sqrt{\frac{1{,}7041e^8 mm^4}{1{,}6989e^9 mm^4}} = 0{,}316711$$

$$C = 0{,}08 + 0{,}92 * r = 0{,}3713$$

$$I_{y,eq} = C * I_{y,max} = 0{,}3713 * 1{,}6989e^9 = 6{,}3093e^8 mm^4$$

$$N_{cr,y} = \frac{\pi^2 * E * I_{y,eq}}{k_y^2 * L_y^2} = 4503{,}61\ kN$$

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	sway	
System length L	4,000	4,000	m
Buckling factor k	4,26	1,00	
Buckling length L_{cr}	17,033	4,000	m
Critical Euler load N_{cr}	4507,35	7000,23	kN
Slenderness λ	133,78	55,79	
Relative slenderness λ_{rel}	1,42	0,59	
Limit slenderness $\lambda_{rel,0}$	0,20	0,20	
Buckling curve	b	c	
Imperfection α	0,34	0,49	
Reduction factor χ	0,37	0,79	
Buckling resistance $N_{b,Rd}$	917,79	1948,85	kN

Tapered member data		
Minimum second moment of area $I_{y,min}$	1,7041e+08	mm^4
Maximum second moment of area $I_{y,max}$	1,6989e+09	mm^4
Taper coefficient C	0,37	
Equivalent second moment of area $I_{y,eq}$	6,3093e+08	mm^4

Flexural Buckling verification		
Cross-section area A	1,0512e+04	mm^2
Buckling resistance $N_{b,Rd}$	917,79	kN
Unity check	0,76	-

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:

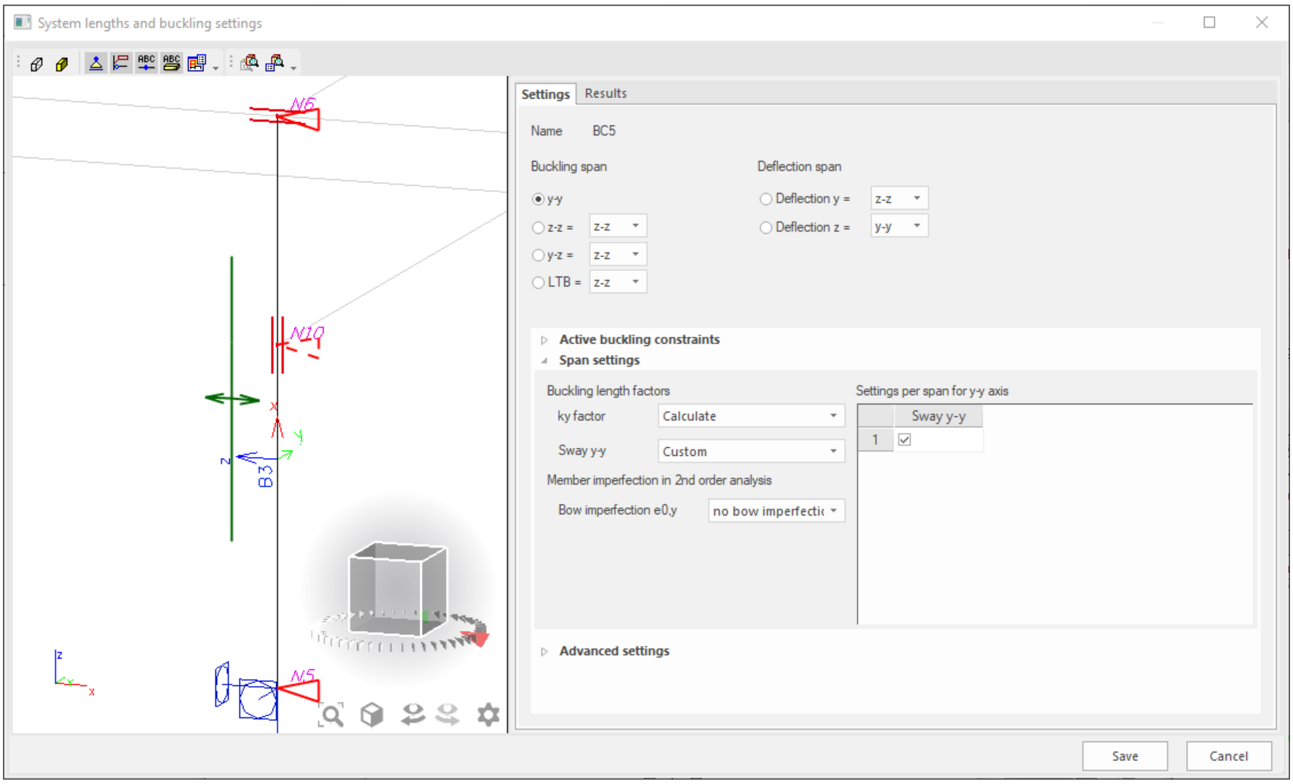
l	the buckling length
k	the buckling factor
L	the system length

The system length of a beam is defined by the property “System lengths and buckling settings” 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 “System lengths and buckling settings”. 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

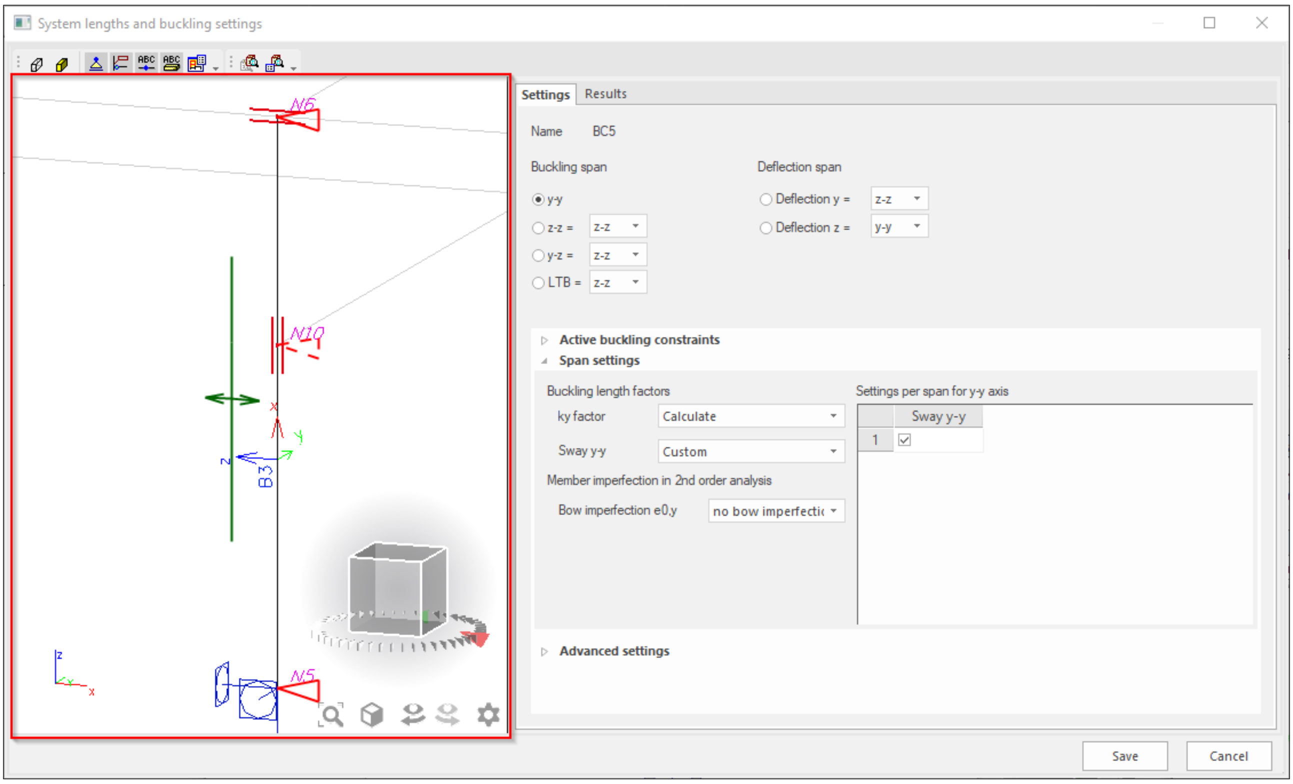
Linear calculation

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	3.270	18.51	1.800	1.800
			No	1.800	0.51	0.919	26.78		
B3	CS3	2	Yes	3.000	1.09	3.270	18.51	1.200	1.200
			No	1.200	0.57	0.680	19.83		

  - 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).

As mentioned before, since SCIA Engineer 18.0 a new dialog is introduced for applying buckling settings on a specific buckling system called System lengths and buckling settings.

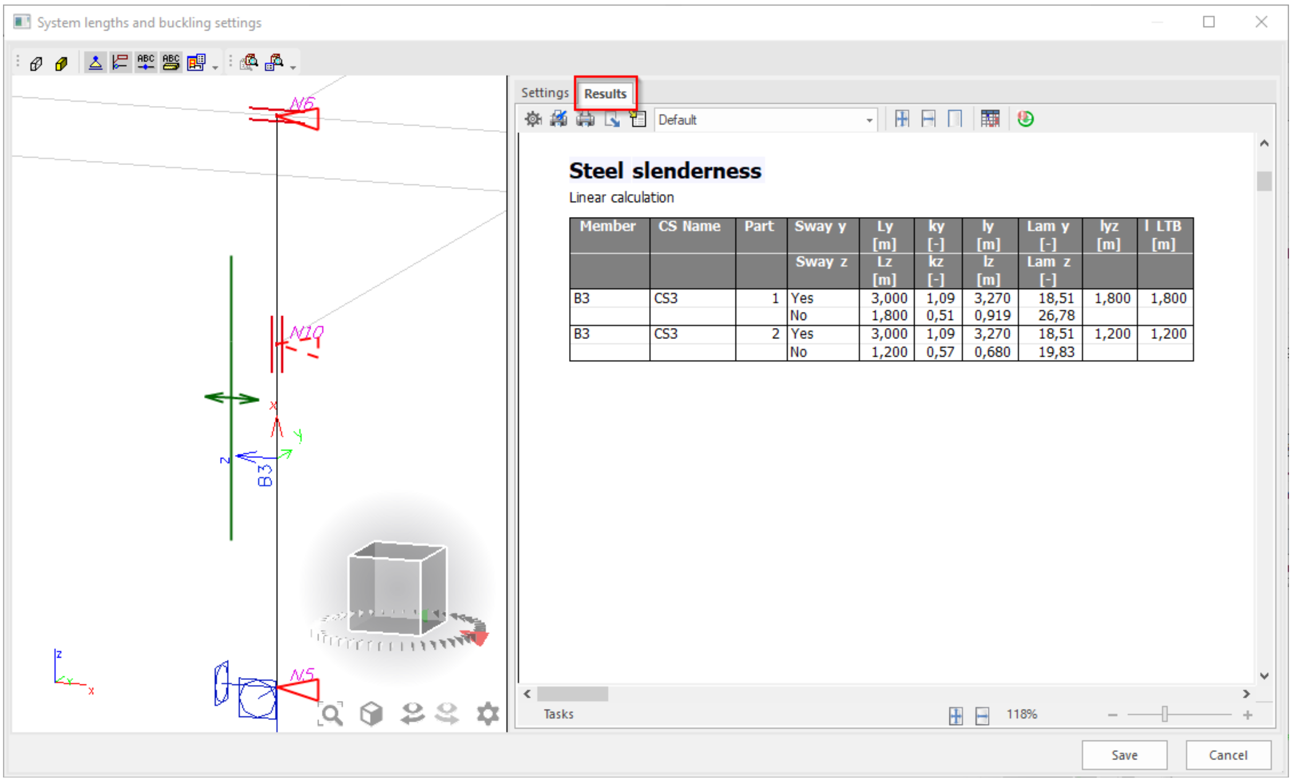
System lengths and buckling settings can be accessed either:
- via Libraries > Structure, analysis > System lengths and buckling groups > click on new for creating a new buckling group or click on edit to modify an existing buckling group.
- via 1D-member property > System lengths and buckling settings.



The left part of the dialog gives the user a graphical representation of the 1D-members in the buckling system with their buckling constraints and information about the sway settings per span. It is not only a representation of the above mentioned settings but it also allows **editing directly in that graphical window** by clicking on the buckling constraints to set them to fixed/free or by clicking on the sway symbols per span to set them to From setup, All sway, All non-sway or Custom. By clicking on the triangles indicating the system lengths of each part of a beam, the user can change the buckling constraints.

For the buckling constraints there are two symbol types depending on the chosen span:
- Triangle symbol (buckling span y-y, z-z and also for the deflection span deflection y, deflection z)
- Rectangular symbol (buckling span y-z, LTB)

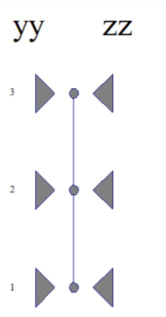
Besides this, with this new improved buckling settings, it is easier to access the results. The user can see them by clicking on the 'Results' tab of the 'System lengths and buckling settings' window.



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	3,270	18,51	1,800	1,800
			No	1,800	0,51	0,919	26,78		
B3	CS3	2	Yes	3,000	1,09	3,270	18,51	1,200	1,200
			No	1,200	0,57	0,680	19,83		

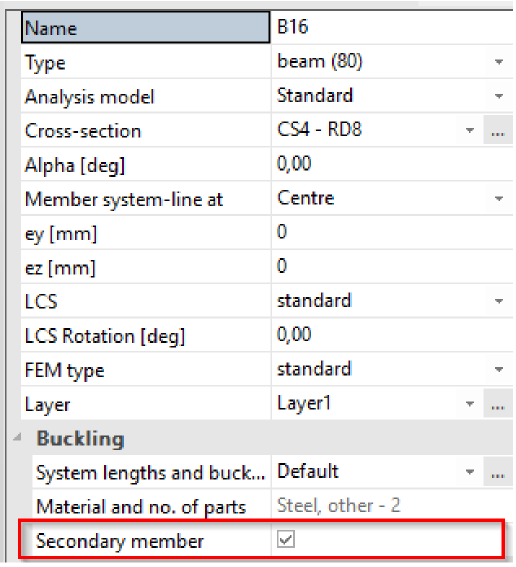
- Compare beam B3 and beams (B2+B30) with each other: they should have exactly the same system lengths. The only difference between those columns is that beam B3 was inputted as a beam of 3m and beams (B2+B30) are divided in two parts. SCIA Engineer will consider those two beams also as one buckling 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.

- 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):

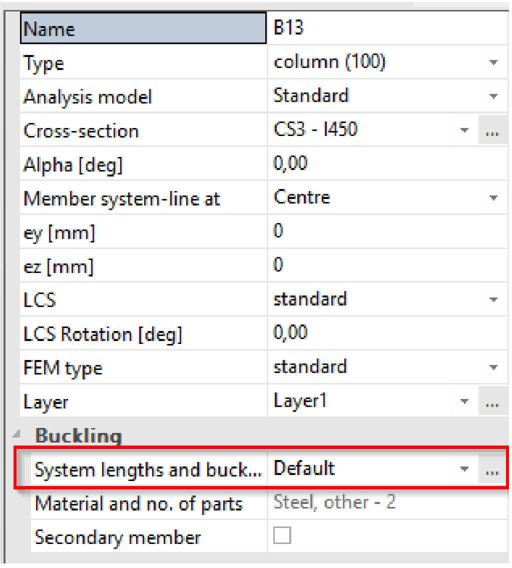


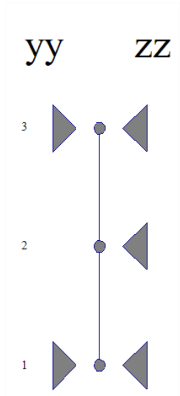
When looking at the rendered view, it will be clear that beam B16 is too weak to have an influence on the system length of beam B13. In SCIA Engineer there is possibility to exclude a beam from a buckling system.

So select beam B16 and go to Buckling properties in its Property window. 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 System lengths and buckling settings back to "Default" the member B16 will not be included in the system lengths:



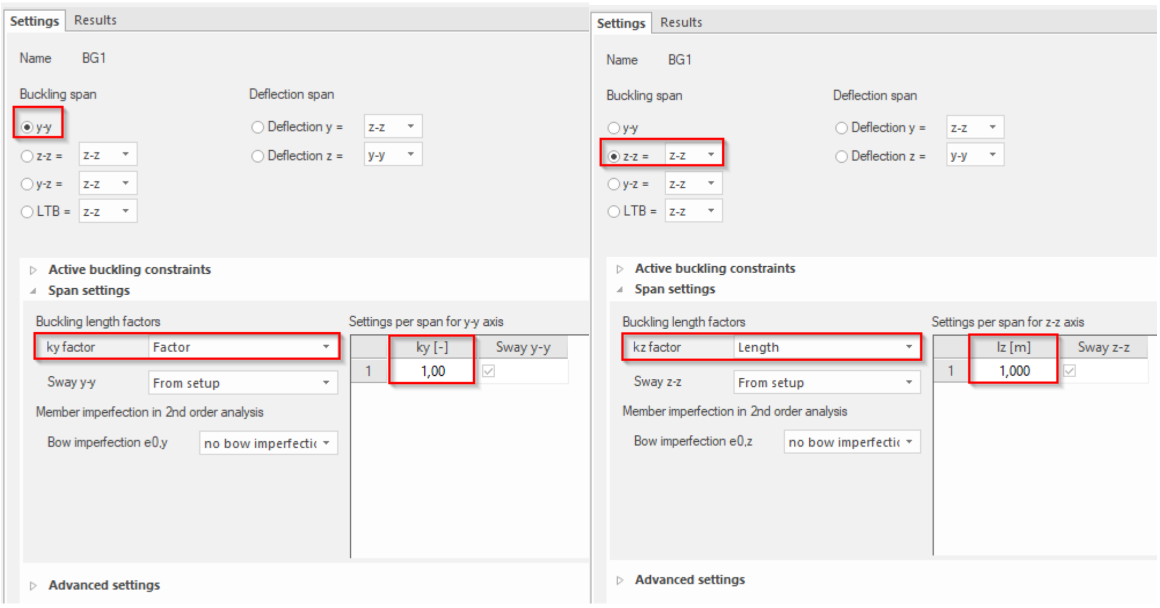


Buckling factors/lengths: manual input

The principles of the buckling factor and buckling length are explained in the previous paragraphs. Those default settings can be changed in the menu "System lengths and buckling settings", in the menu "System lengths and buckling groups" or in the Steel setup.

In the menu "Steel -> Beams -> Steel setup" the user can change the "Buckling ky, kz coefficients or buckling lengths" on "user input" by choosing the option 'Factor' or 'Length'. By changing the settings in Steel setup, the user will define these options for the whole structure.

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 for each member in the menu '**System lengths and buckling settings**':



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:

Effective properties						
Effective area	A_{eff}	1.7222e+04	mm^2			
Effective second moment of area	$I_{eff,y}$	1.6608e+09	mm^4	$I_{eff,z}$	5.2895e+07	mm^4
Effective section modulus	$W_{eff,y}$	4.4111e+06	mm^3	$W_{eff,z}$	3.9920e+05	mm^3
Shift of the centroid	$e_{N,y}$	0.00	mm	$e_{N,z}$	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 (6.4

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	6.900	6.900	m
Buckling factor k	3.35	0.99	
Buckling length L_{cr}	23.123	6.811	m
Critical Euler load N_{cr}	6438.53	2362.99	kN
Slenderness λ	77.79	128.41	
Relative slenderness λ_{rel}	0.79	1.31	
Limit slenderness $\lambda_{rel,0}$	0.20	0.20	
Buckling curve	a	b	
Imperfection α	0.21	0.34	
Reduction factor χ	0.80	0.42	
Buckling resistance $N_{b,Rd}$	3236.78	1710.75	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 = 23.115 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 = 3236.78 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 = 1710.75 kN.

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

Flexural Buckling verification		
Cross-section effective area A_{eff}	1.7222e+04	mm²
Buckling resistance $N_{b,Rd}$	1710.75	kN
Unity check	0.09	-

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:

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

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

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

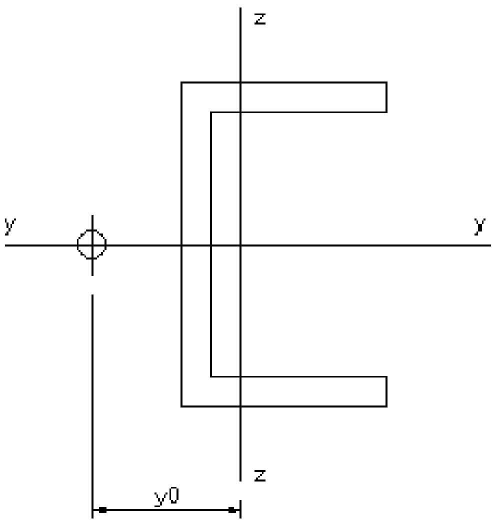
$$i_0^2 = i_y^2 + i_z^2 + y_0^2$$

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

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

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

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 L_{cr}	3.600	m
Elastic critical load $N_{cr,T}$	2156.75	kN
Elastic critical load $N_{cr,TF}$	1302.24	kN
Relative slenderness $\lambda_{rel,T}$	0.98	
Limit slenderness $\lambda_{rel,0}$	0.20	
Buckling curve	c	
Imperfection α	0.49	
Reduction factor χ	0.55	
Cross-section area A	5.3300e+03	mm²
Buckling resistance $N_{b,Rd}$	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:

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

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

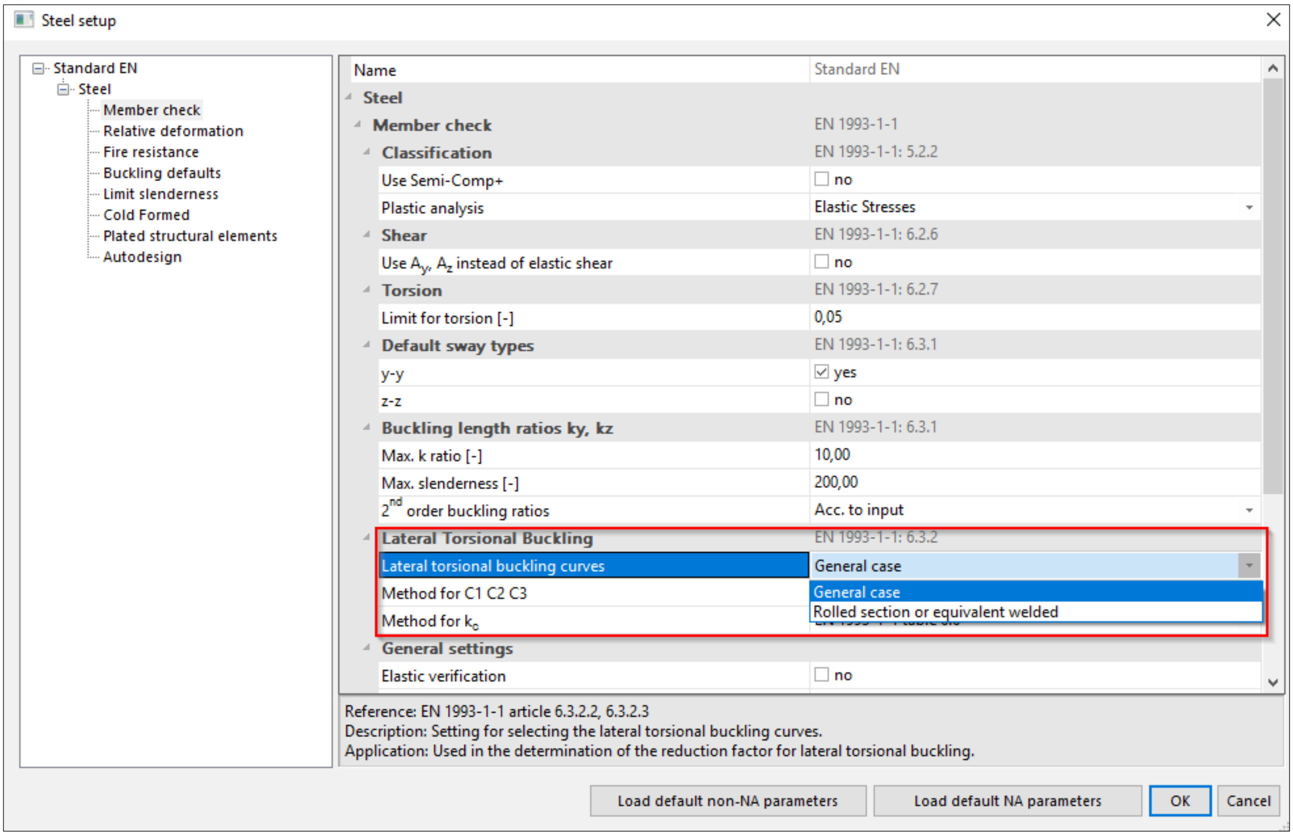
With the following recommended buckling curves for lateral torsional buckling:

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

For slenderness's $\bar{\lambda}_{LT} \leq 0{,}2$ 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” and 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

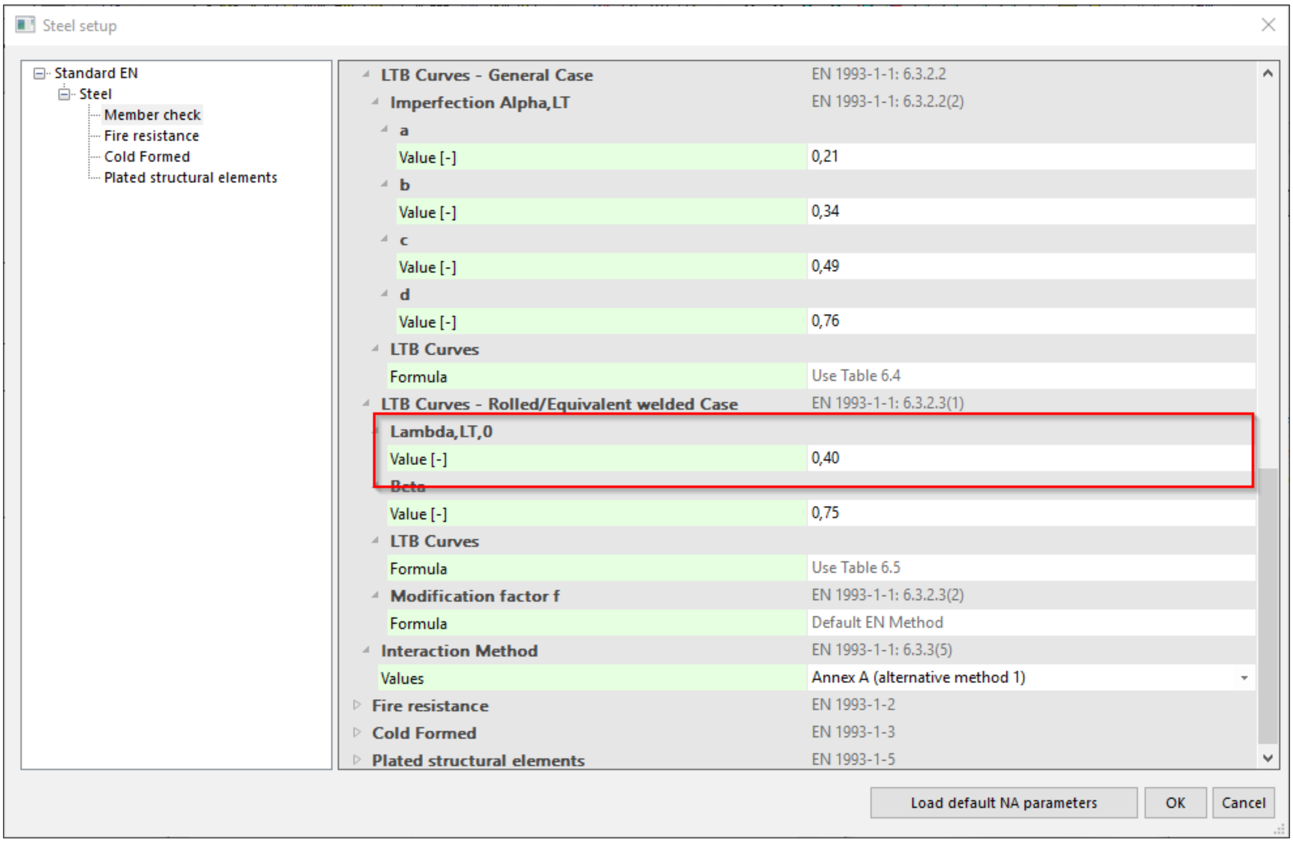
$\bar{\lambda}_{LT,0}$ = 0,4 (maximum value)

β= 0,75 (minimum value)

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

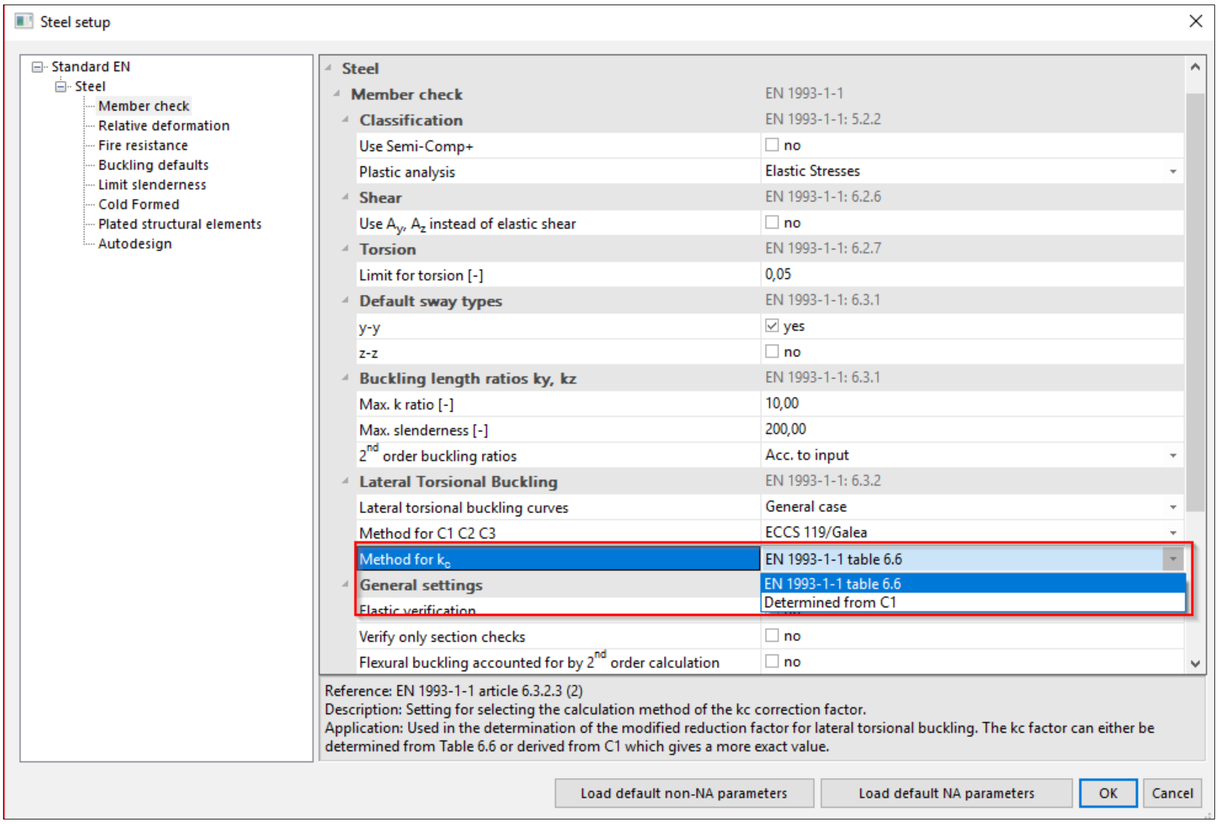
Table 6.6: Correction factors k_c

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

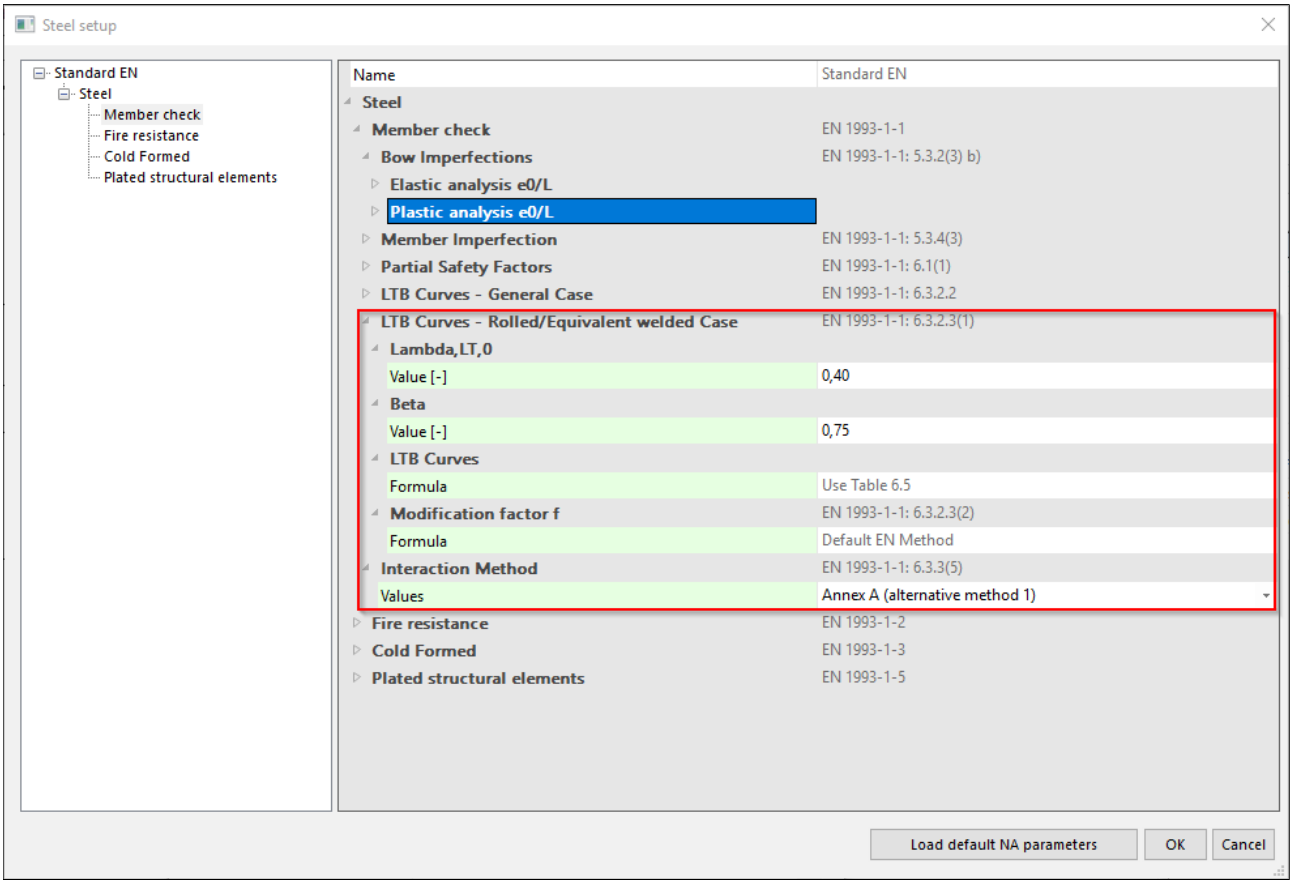
But alternatively k_c can also be calculated from the factor C_1:

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

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	
Plastic section modulus $W_{pl,y}$	3.6380e+05	mm^3
Elastic critical moment M_{cr}	56.14	kNm
Relative slenderness $\lambda_{rel,LT}$	1.23	
Relative slenderness $\lambda_{rel,T}$	0.07	
Relative slenderness $\lambda_{rel,EXTRA}$	1.31	
Limit slenderness $\lambda_{rel,LT,0}$	0.20	
LTB curve	a	
Imperfection α_{LT}	0.21	
Reduction factor χ_{LT}	0.47	
Design buckling resistance $M_{b,Rd}$	39.92	kNm
Unity check	0.46	-

Note: $\lambda_{rel,EXTRA}$ is determined according to "Design rule for lateral torsional buckling of channel sections, 2007".

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	

Mcr parameters		
Correction factor k_w	1.00	
LTB moment factor C_1	1.13	
LTB moment factor C_2	0.45	
LTB moment factor C_3	0.53	
Shear center distance d_z	0.00	mm
Distance of load application z_g	0.00	mm
Mono-symmetry constant β_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 Young 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 there 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

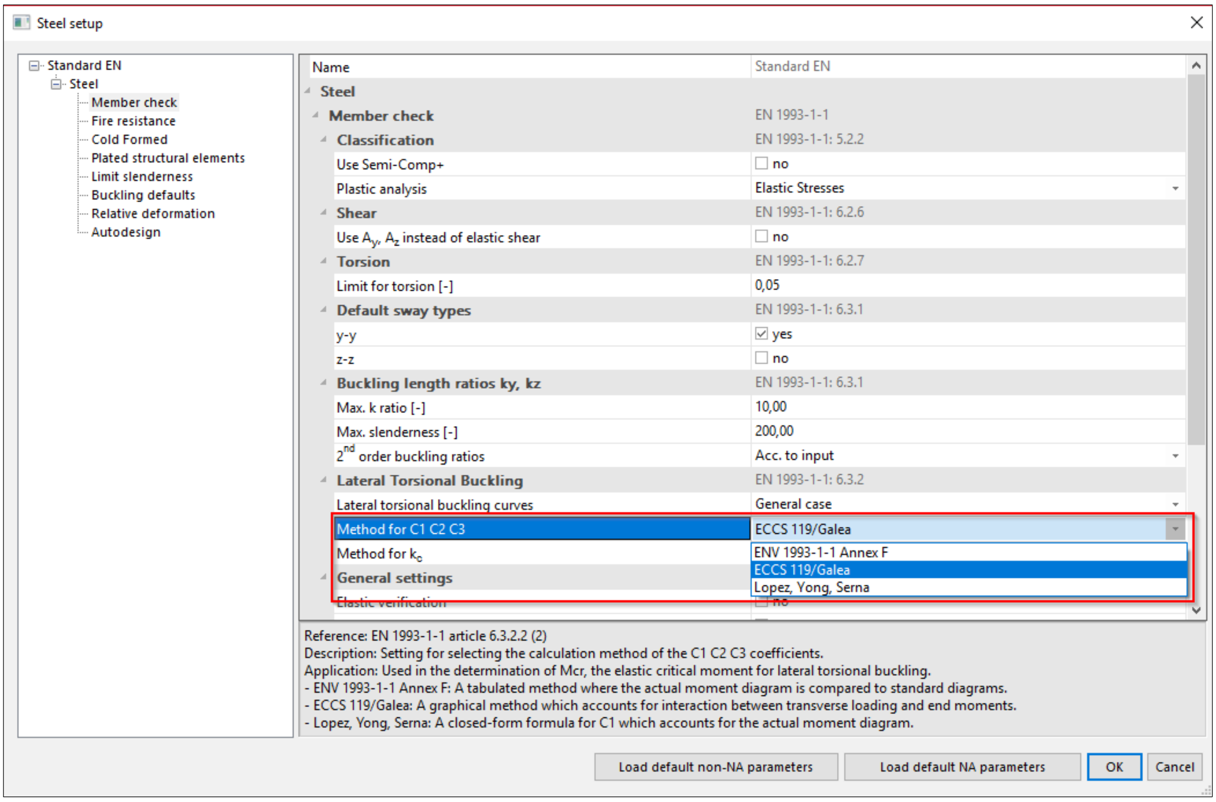
C1 : takes into account the shape of the moment diagram
C2 : takes into account the position of the loading
C3 : takes into account the asymmetry of the cross section

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:

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

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

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

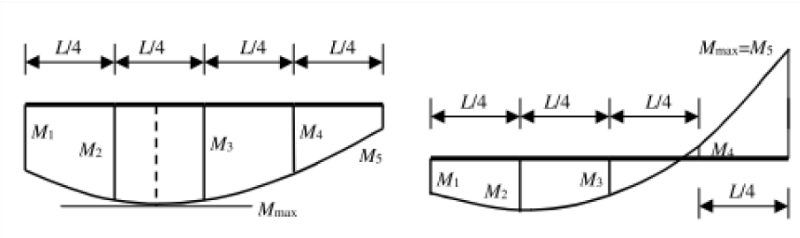
$$\alpha_1 = 1 - k_2$$

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

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

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

$$\alpha_1 = 1 - k_1$$



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 k_w	1.00
LTB moment factor C_1	1.89
LTB moment factor C_2	0.33
LTB moment factor C_3	2.64
Shear center distance d_z	0.00
Distance of load application z_g	-245.00
Mono-symmetry constant β_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 k_w	1.00
LTB moment factor C_1	1.26
LTB moment factor C_2	0.45
LTB moment factor C_3	0.41
Shear center distance d_z	0.00
Distance of load application z_g	-245.00
Mono-symmetry constant β_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 k_w	1.00
LTB moment factor C_1	1.20
LTB moment factor C_2	0.00
LTB moment factor C_3	0.00
Shear center distance d_z	0.00
Distance of load application z_g	-245.00
Mono-symmetry constant β_y	0.00
Mono-symmetry constant z_j	0.00

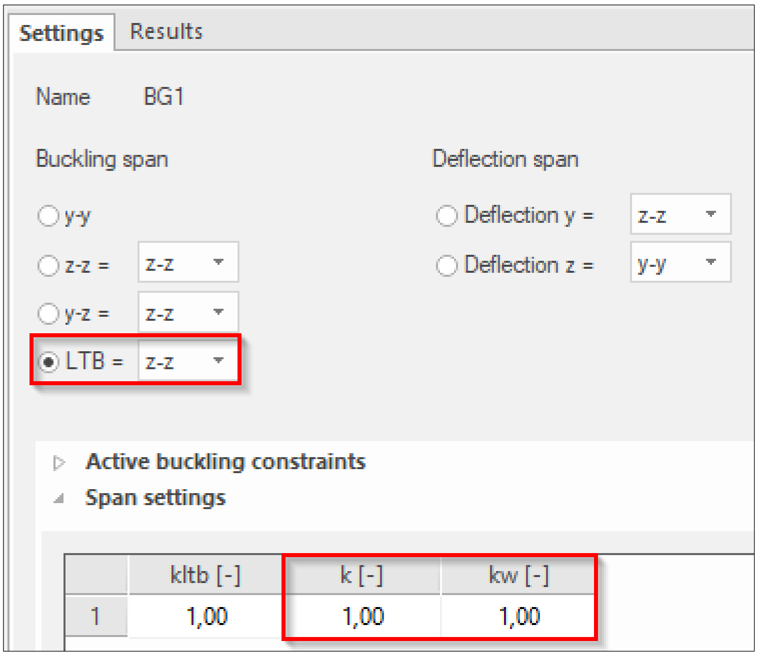
Mcr = 842.64 kNm

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 within the ‘System lengths and buckling settings’.

Select the member then open the ‘System lengths and buckling settings’ menu from the properties window:



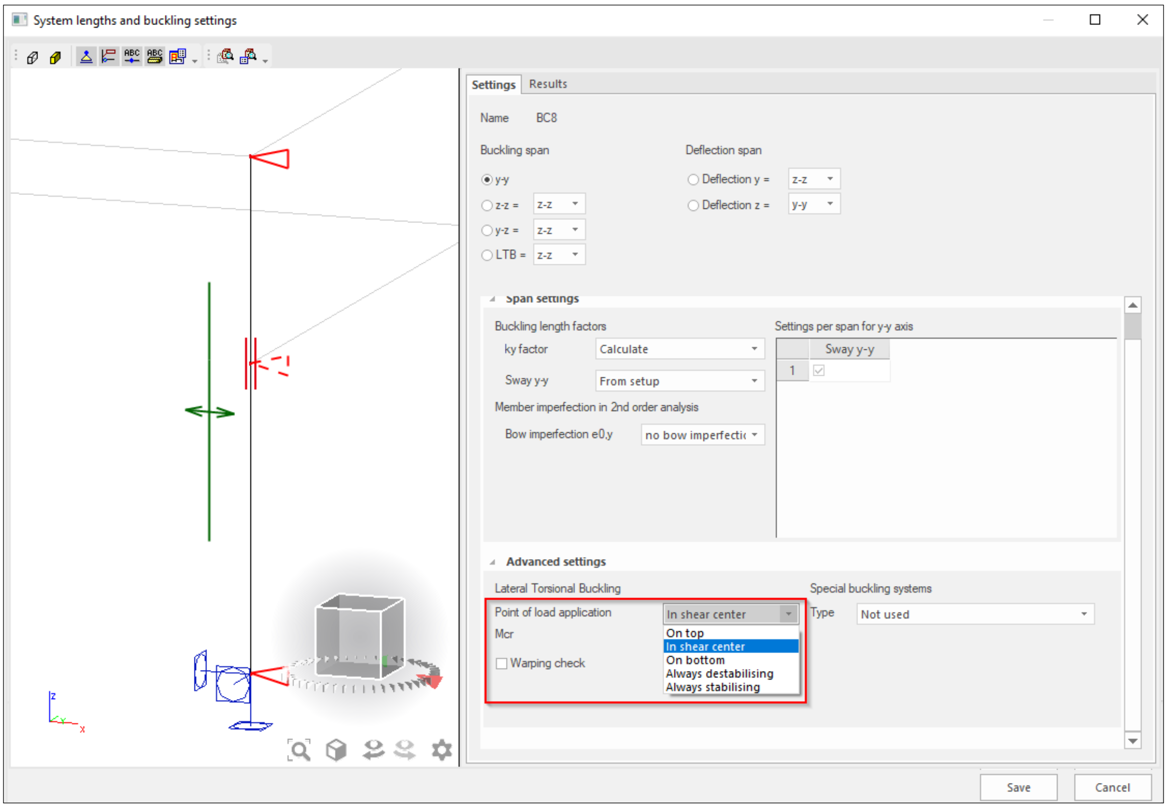
Click on LTB and in Span settings you can modify the values for k and kw.

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 ‘System lengths and buckling settings, you can see an option called ‘Point of load application’. The five possibilities are:

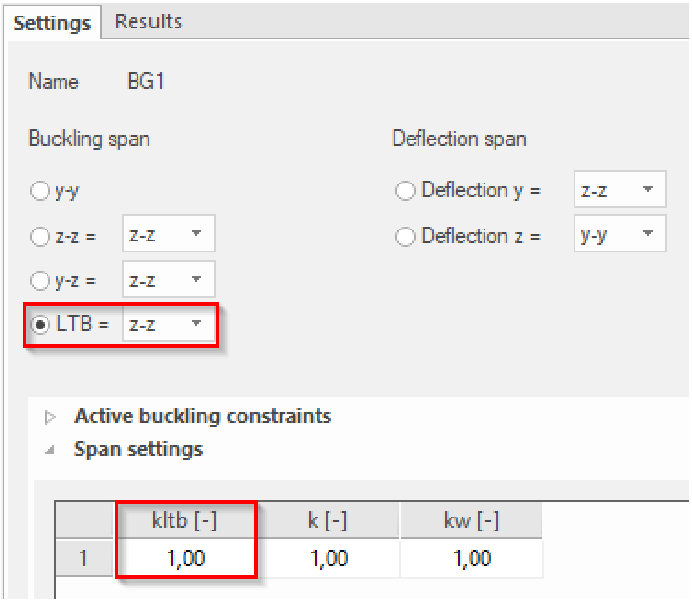
- On top: the load is applied on the top flange
- In shear center: the load is applied in the shear center
- On bottom: the load is applied at the bottom flange
- Always destabilising: the load is applied on the destabilising flange
- Always stabilising: the load is applied on the stabilising flange



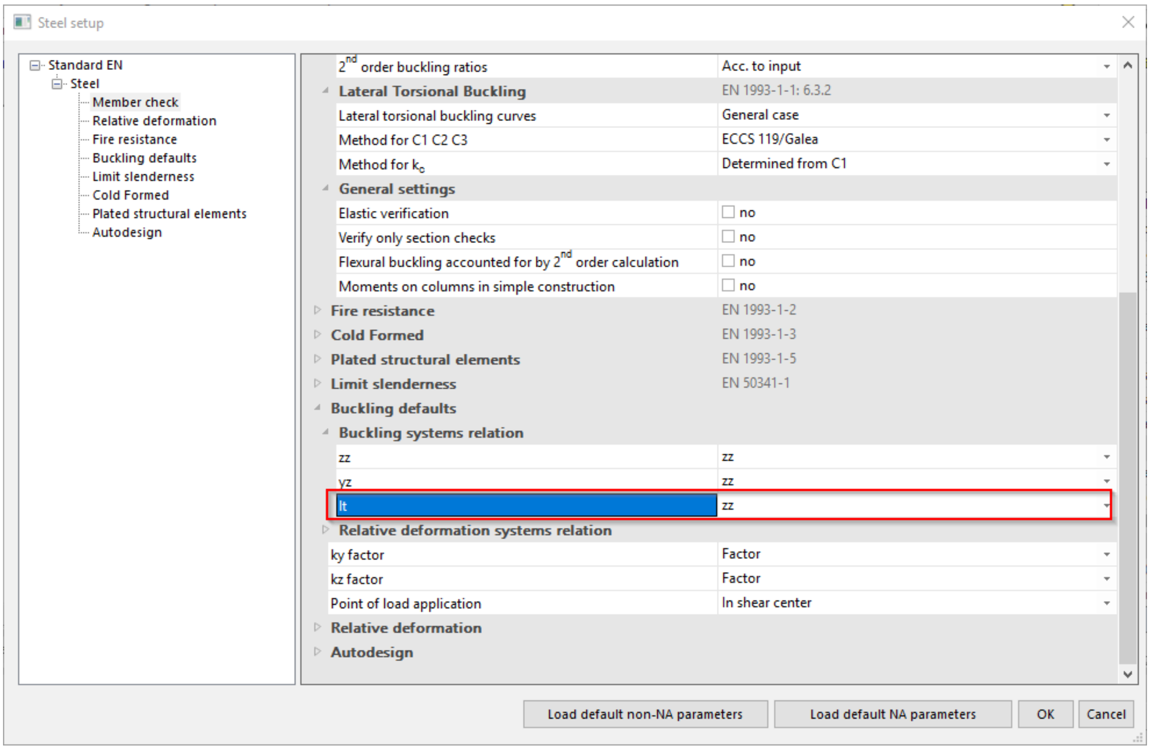
This value is set to ‘In shear center’ by default and can be adapted to influence the value of Mcr.

LTB Length

LTB length is calculated as lLTB = k_{ltb} * LLTB
k_{ltb} is by default taken equal to 1. A smaller value can be used to reduce the LTB length. You can adapt k_{ltb} within the ‘System lengths and buckling settings’:

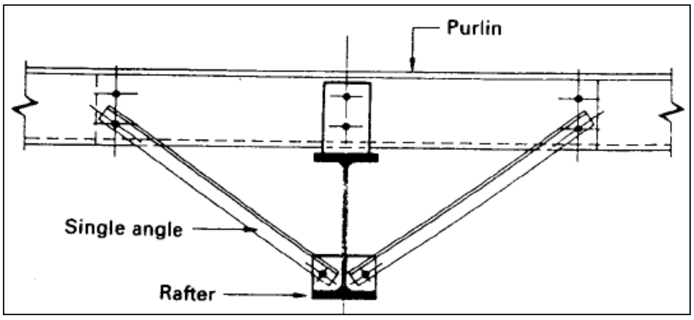


LLTB is the reference length. It is by default equal to the reference length around the weak axis (Lz) for both bottom and top flange. This can be seen in both previous window but as well in 'Steel setup' menu:



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}:

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

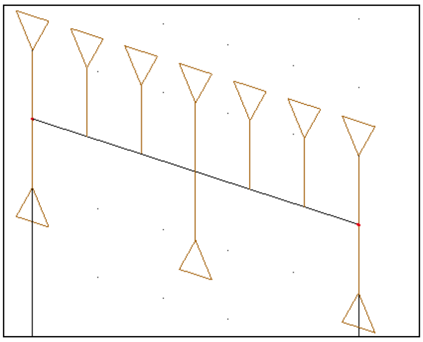
Where

$k_{LT} \cdot L$ — 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.

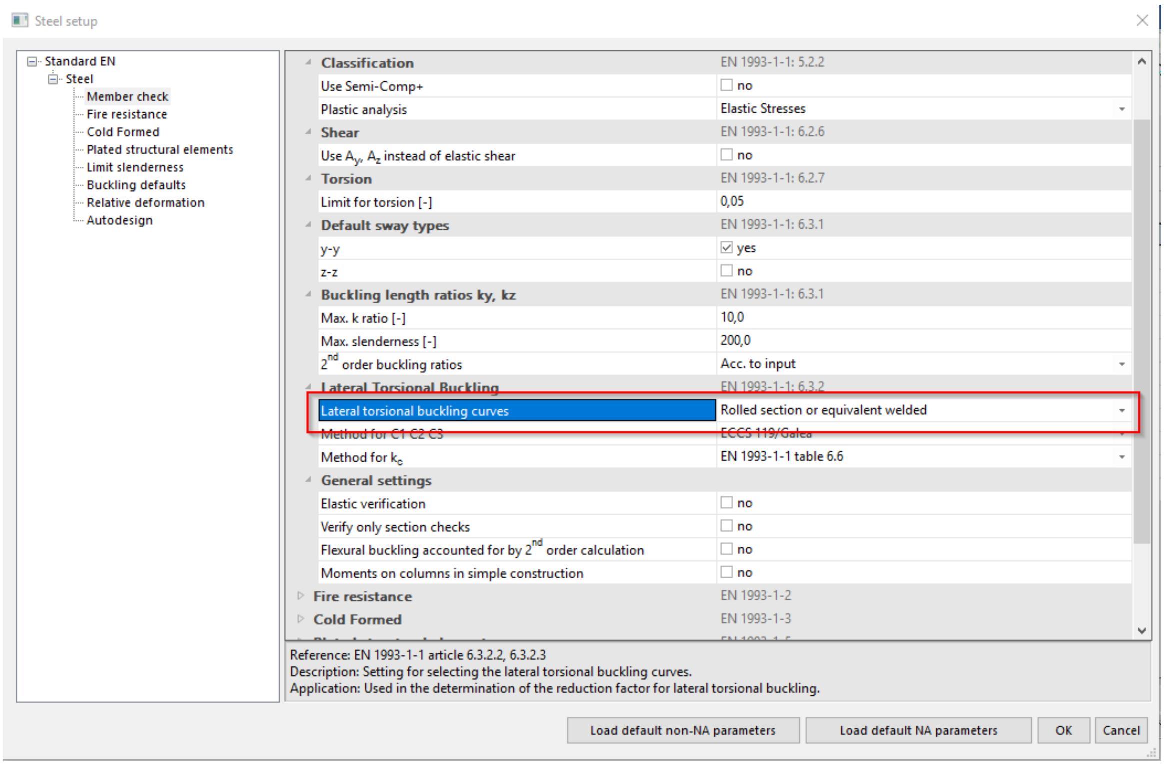
The restraints can be inputted via “Steel -> Beams -> Member Check data -> LTB Restraints”.



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

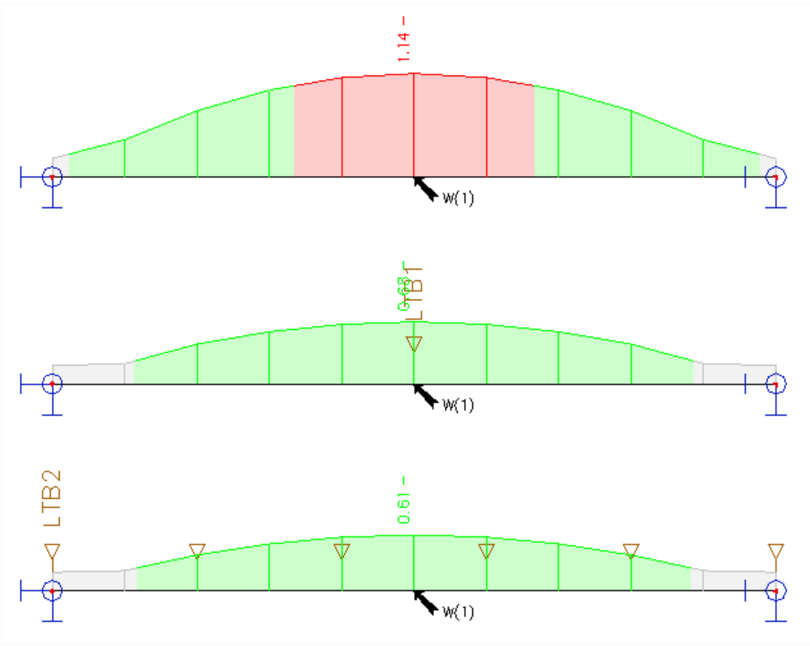
Example: LTB_Restraints.esa

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	B2

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	
Plastic section modulus $W_{pl,y}$	1.7020e-03	m^3
Elastic critical moment M_{cr}	280.68	kNm
Relative slenderness $\lambda_{rel,LT}$	1.19	
Limit slenderness $\lambda_{rel,LT,0}$	0.40	
LTB curve	c	
Imperfection α_{LT}	0.49	
LTB factor β	0.75	
Reduction factor χ_{LT}	0.53	
Correction factor k_c	0.94	
Correction factor f	0.98	
Modified reduction factor $\chi_{LT,mod}$	0.54	
Design buckling resistance $M_{b,Rd}$	215.74	kNm
Unity check	**1.14**	-

Mcr parameters		
LTB length L	7.000	m
Influence of load position	no influence	
Correction factor k	1.00	
Correction factor k_w	1.00	
LTB moment factor C_1	1.13	
LTB moment factor C_2	0.45	
LTB moment factor C_3	0.53	
Shear center distance d_z	0	mm
Distance of load application z_g	0	mm
Mono-symmetry constant β_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	
Plastic section modulus $W_{pl,y}$	1.7020e-03	m^3
Elastic critical moment M_{cr}	972.64	kNm
Relative slenderness $\lambda_{rel,LT}$	0.64	
Limit slenderness $\lambda_{rel,LT,0}$	0.40	
LTB curve	c	
Imperfection α_{LT}	0.49	
LTB factor β	0.75	
Reduction factor χ_{LT}	0.86	
Correction factor k_c	0.91	
Correction factor f	0.96	
Modified reduction factor $\chi_{LT,mod}$	0.90	
Design buckling resistance $M_{b,Rd}$	359.89	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 k_w	1.00	
LTB moment factor C_1	1.33	
LTB moment factor C_2	0.12	
LTB moment factor C_3	1.00	
Shear center distance d_z	0	mm
Distance of load application z_g	0	mm
Mono-symmetry constant β_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 $\bar{\lambda}_{LT} \leq \bar{\lambda}_{LT,0}$ or for $\frac{M_{Ed}}{M_{cr}} \leq \bar{\lambda}_{LT,0}{}^2$ lateral torsional buckling effects may be ignored. With: $\bar{\lambda}_{LT,0} = 0{,}4$

So for beam B3 the following LTB check is displayed:

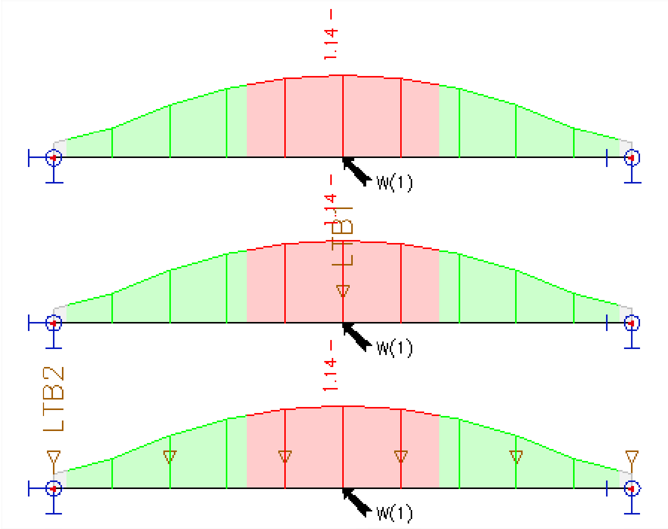
Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters		
Method for LTB curve	Alternative case	
Plastic section modulus $W_{pl,y}$	1.7020e-03	m^3
Elastic critical moment M_{cr}	4017.35	kNm
Relative slenderness $\lambda_{rel,LT}$	0.32	
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 k_w	1.00	
LTB moment factor C_1	1.01	
LTB moment factor C_2	0.02	
LTB moment factor C_3	1.00	
Shear center distance d_z	0	mm
Distance of load application z_g	0	mm
Mono-symmetry constant β_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:



Calculation 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

Calculation 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: $\chi_{LT} = \dfrac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \lambda_{EXTRA}^2}}$

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

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

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

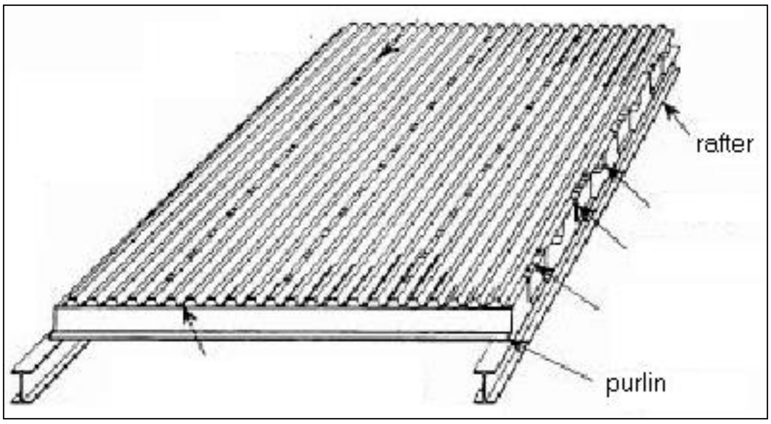
$$\boxed{\begin{array}{lll} \lambda_T = 1.0 - \lambda_{LT} & if & 0.5 \le \lambda_{LT} < 0.80 \\ \lambda_T = 0.43 - 0.29\lambda_{LT} & if & 0.80 \le \lambda_{LT} < 1.5 \\ \lambda_T = 0 & if & \lambda_{LT} > 1.5 \end{array}}$$

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

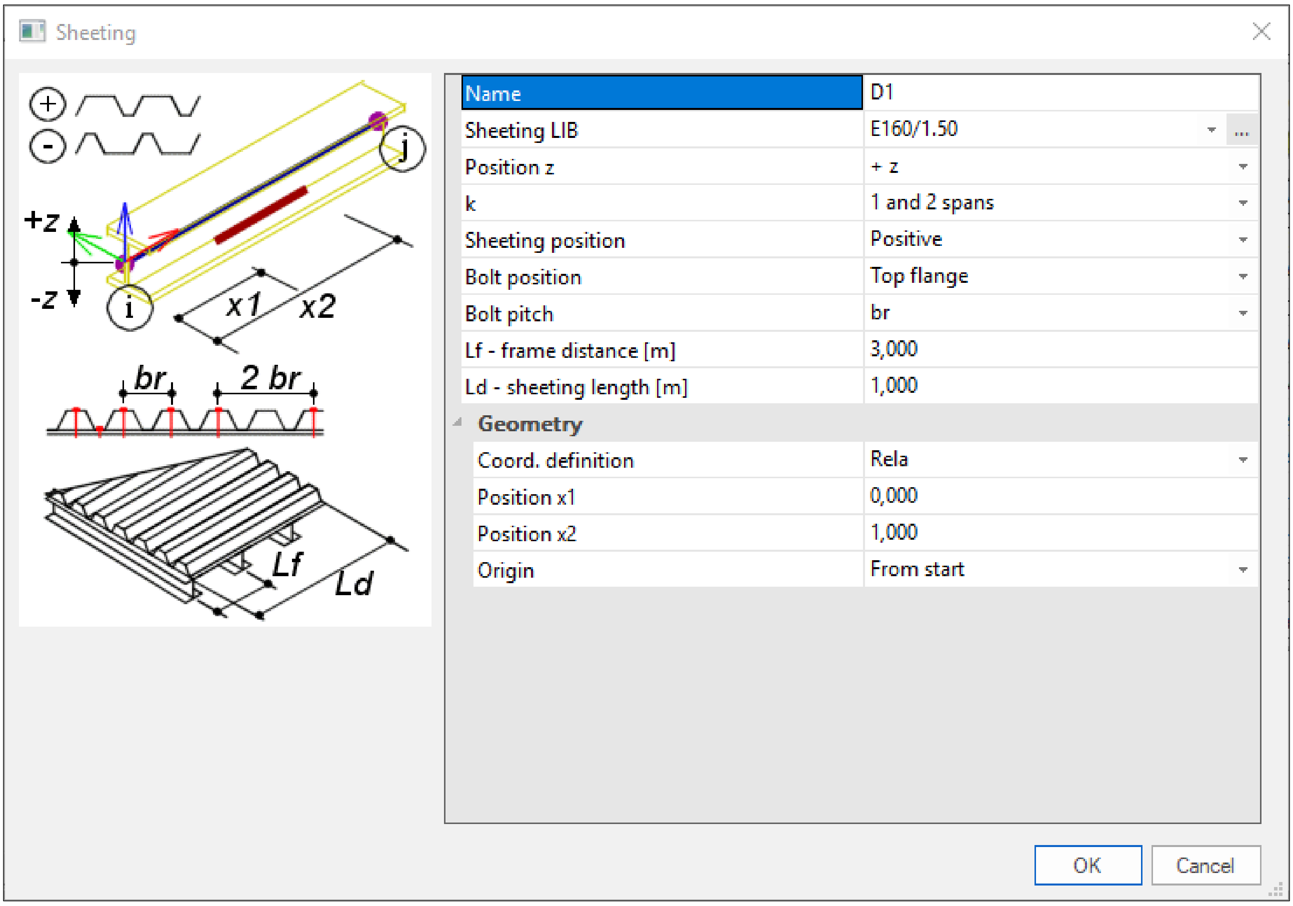
- The section concerns a Channel section (**Form Code 5**)
- The *General Case* is used for LTB (Not the ***Rolled and Equivalent Welded Case***)
- **15 <= Lltb/h <= 40** (with **Lltb** the LTB length and **h** the cross-section height)

Sheeting

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 sheeting can be inputted in SCIA Engineer via “Steel -> Beams -> Member Check data -> Sheeting”:



The settings for the diaphragm are:

Sheeting LIB	With this option the user can choose between sheetings of the library of SCIA Engineer or input his own sheeting.
Position z	The position of the sheeting according to the LCS of the beam.
k	The value of coefficient k depends on the number of spans of the sheeting: k = 2 for 1 or 2 spans, k = 4 for 3 or more spans.
Sheeting position	The position of the sheeting may be either positive or negative. Positive means that the sheeting is assembled in a way so that the width is greater at the top side. Negative means that the sheeting 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 sheeting.
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 sheeting (shear field.)

The sheeting 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 sheeting's.

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 sheeting:

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

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 the sheeting
	$C_{\theta M,k}$	the rotational stiffness of the sheeting
	$C_{\theta A,k}$	the rotational stiffness of the connection between the sheeting 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 sheeting = 4 for 3 or more spans of the sheeting
	EI_{eff}	bending stiffness of per unit width of the sheeting
	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 sheeting are shown in the preview of the steel check just before the Section check:

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

Parameters		
Sheeting name	D1	
Moment of inertia per length I	0.00	m^4/m
Position z	+ z	
Sheeting position	negative	
Bolt position	top flange	
Bolt pitch	br	
Frame distance L_f	3.000	m
Sheeting length L_d	3.000	m
Sheeting factor K1-	0.167	m/kN
Sheeting factor K2-	15.700	m^2/kN
Numerical coefficient k	4.00	

Stiffness		
Actual stiffness S	5555.21	kN
Required stiffness S_{erf}	42650.44	kN
$S < S_{erf}$	inadequately braced	
Sheeting on side	compression side	
Rotational stiffness (Sheeting) $C_{\theta M,k}$	2637.60	kNm/m
Rotational stiffness (Beam distortion) $C_{\theta P,k}$	100.15	kNm/m
Rotational stiffness (Connection) $C_{\theta A,k}$	23.75	kNm/m
Rotational coefficient C_{100}	10.00	kNm/m
Rotational stiffness vorhC_θ	19.06	kNm/m
LTB length L	7.000	m
Cross-section torsional constant I_t	6.6900e-07	m^4
Additional torsional constant $I_{t,add}$	1.1715e-06	m^4
Adapted torsional constant $I_{t,id}$	1.8405e-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	
Plastic section modulus $W_{pl,y}$	1.7020e-03	m^3
Elastic critical moment M_{cr}	404.94	kNm
Relative slenderness $\lambda_{rel,LT}$	0.99	
Limit slenderness $\lambda_{rel,LT,0}$	0.20	
LTB curve	b	
Imperfection α_{LT}	0.34	
Reduction factor χ_{LT}	0.60	
Design buckling resistance $M_{b,Rd}$	240.36	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 k_w	1.00	
LTB moment factor C_1	1.13	
LTB moment factor C_2	0.45	
LTB moment factor C_3	0.53	
Shear center distance d_z	0	mm
Distance of load application z_g	0	mm
Mono-symmetry constant β_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 only included in the ultimate 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:

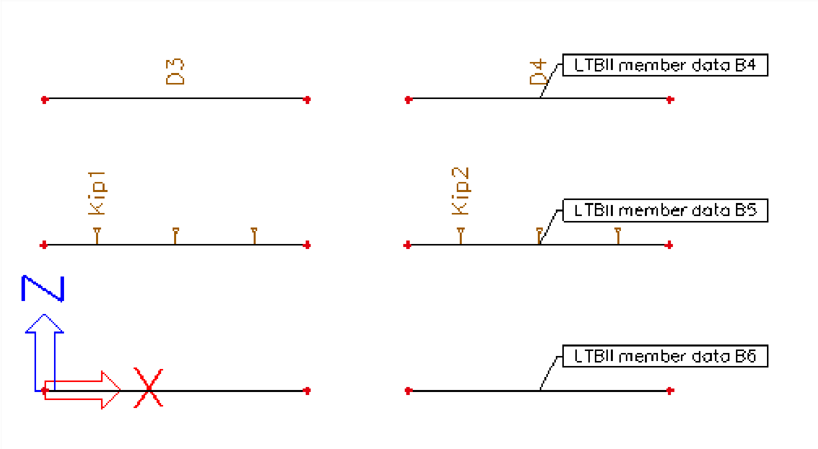
- Appropriate end conditions for torsion and warping
- End and begin forces
- Loadings
- Intermediate restraints (diaphragms, LTB restraints)

To use this option in SCIA Engineer the functionality “**7DoF 2nd order analysis for LTB**” should be activated. In previous versions (before 18.0) this option’s old setting name is ‘LTB 2nd order’.

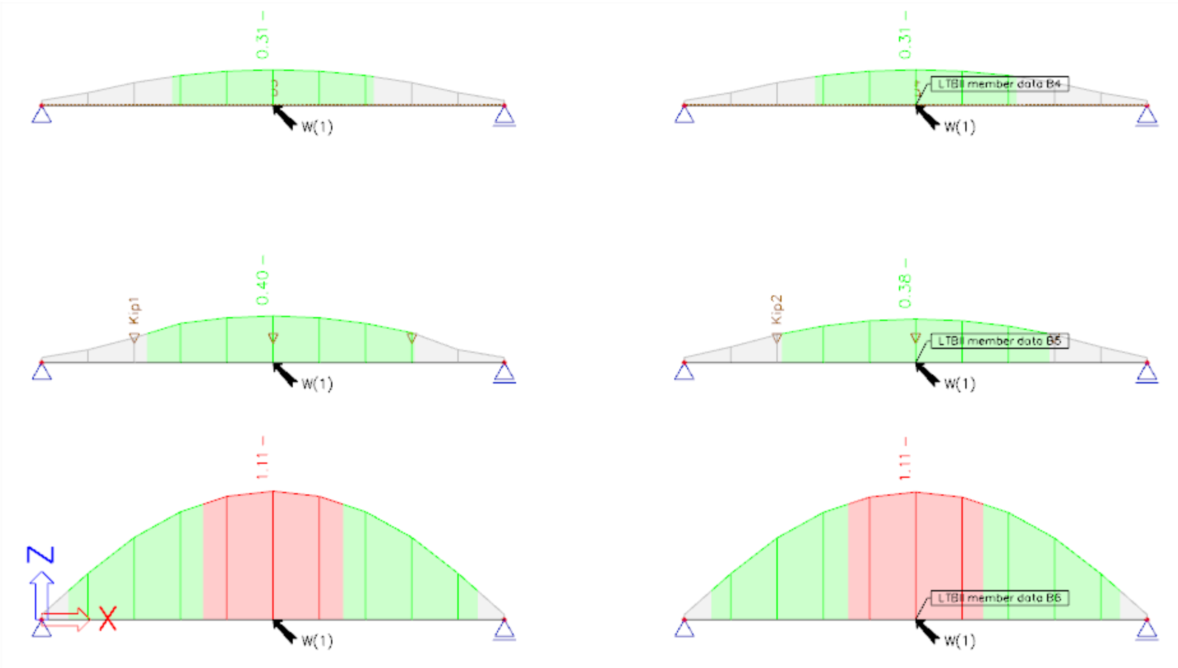
And afterward with the option “Steel -> Beams -> Member check data -> LTBII member data” the user can input LTB-data on a beam.

Example: LTBII.esa

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

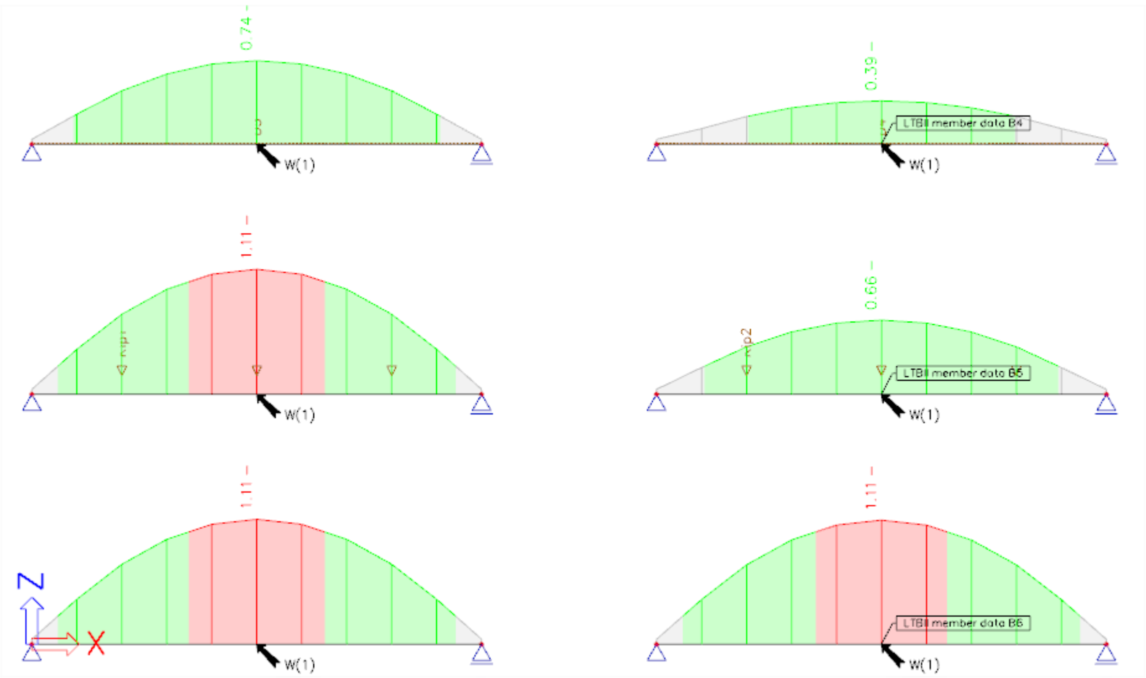


When looking at the check for LC1 there is not a big difference between the beams with or without the LTBII data:



In Frilo the rigidity of the sheeting against rotation and translation is taken into account, which results in a higher stiffness of the sheeting 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 sheeting 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 & 6.3.2.2 and formula (6.54)

LTB parameters		
Method for LTB curve	General case	
Plastic section modulus $W_{pl,y}$	1.6600e-04	m^3
Elastic critical moment M_{cr}	74.26	kNm
Relative slenderness $\lambda_{rel,LT}$	0.78	
Limit slenderness $\lambda_{rel,LT,0}$	0.20	
LTB curve	a	
Imperfection α_{LT}	0.21	
Reduction factor χ_{LT}	0.80	
Design buckling resistance $M_{b,Rd}$	36.73	kNm
Unity check	0.39	-

Note: The elastic critical moment M_{cr} 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. 6.3.3**.

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

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

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

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

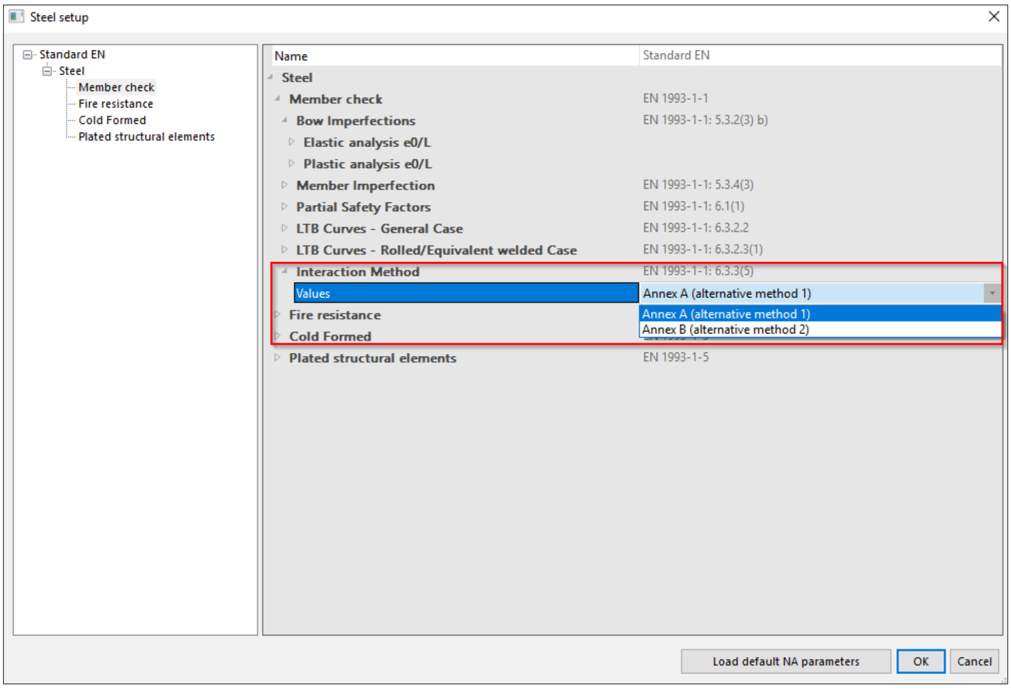
$\boldsymbol{\chi_y}$ and $\boldsymbol{\chi_z}$ are the reduction factors due to flexural buckling

$\boldsymbol{\chi_{LT}}$ 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 A_{eff}	1.7222e+04	mm^2
Effective section modulus $W_{eff,y}$	4.4111e+06	mm^3
Effective section modulus $W_{eff,z}$	3.9920e+05	mm^3
Design compression force N_{Ed}	160.01	kN
Design bending moment (maximum) $M_{y,Ed}$	-701.56	kNm
Design bending moment (maximum) $M_{z,Ed}$	-0.23	kNm
Additional moment $\Delta M_{y,Ed}$	0.00	kNm
Additional moment $\Delta M_{z,Ed}$	0.00	kNm
Characteristic compression resistance N_{Rk}	4047.10	kN
Characteristic moment resistance $M_{y,Rk}$	1036.61	kNm
Characteristic moment resistance $M_{z,Rk}$	93.81	kNm
Reduction factor χ_y	0.80	
Reduction factor χ_z	0.42	
Reduction factor χ_{LT}	0.75	
Interaction factor k_{yy}	0.98	
Interaction factor k_{yz}	0.83	
Interaction factor k_{zy}	0.94	
Interaction factor k_{zz}	0.80	

Maximum moment $M_{y,Ed}$ is derived from beam B28 position 6.900 m.
Maximum moment $M_{z,Ed}$ is derived from beam B28 position 6.900 m.

Interaction method 1 parameters		
Critical Euler load $N_{cr,y}$	6438.53	kN
Critical Euler load $N_{cr,z}$	2362.99	kN
Elastic critical load $N_{cr,T}$	4845.28	kN
Effective section modulus $W_{eff,y}$	4.4111e+06	mm^3
Second moment of area I_y	1.6610e+09	mm^4
Second moment of area I_z	5.2890e+07	mm^4
Torsional constant I_t	1.6200e+06	mm^4
Method for equivalent moment factor $C_{my,0}$	Table A.2 Line 1 (Linear)	
Ratio of end moments ψ_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 ψ_z	0.00	
Equivalent moment factor $C_{mz,0}$	0.78	
Factor μ_y	0.99	
Factor μ_z	0.96	
Factor ε_y	17.12	
Factor a_{LT}	1.00	
Critical moment for uniform bending $M_{cr,0}$	1008.48	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 + 0.89 + 0.00 = 0.94 -
Unity check (6.62) = 0.09 + 0.85 + 0.00 = 0.95 -

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.

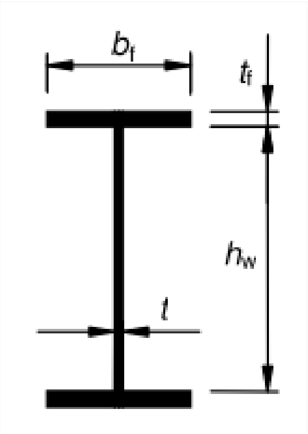
First it will be checked if the slenderness is lesser than a certain value, because for webs with a small slenderness, this check should not be executed.

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 η = 1.20 is recommended for steel grades up to and including S460. For higher steel grades η = 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: Shear buckling.esa

Consider Beam B1

This is an IPE 160 profile: h_w = 160 – 2 x 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 x 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 h_w	145	mm
Web thickness t	5	mm
Material coefficient ε	1.00	
Shear correction factor η	1.20	

Shear Buckling verification	
Web slenderness h_w/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 B2:

This is an I profile: h_w = 600 – 2 x 9 = 582 mm. And t = 6 mm

⇨ h_w / t = 97

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	6.000	m
Web	unstiffened	
End post	non-rigid	
Web height h_w	582	mm
Web thickness t	6	mm
Yield strength f_{yw}	235.0	MPa
Flange width b_f	276	mm
Flange thickness t_f	9	mm
Yield strength f_{yf}	235.0	MPa
Material coefficient ε	1.00	
Shear correction factor η	1.20	

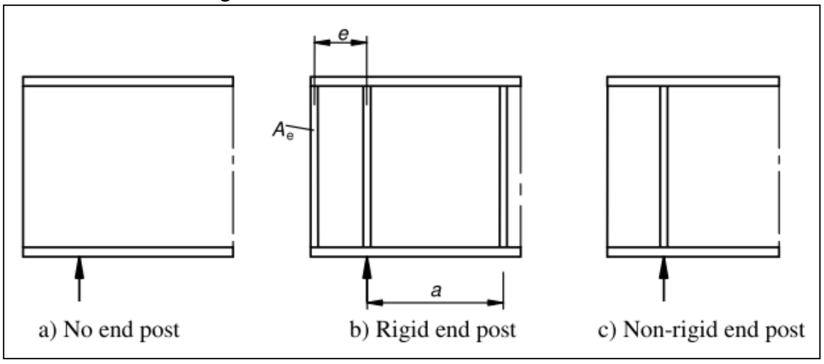
Shear Buckling verification		
Web slenderness h_w/t	97.00	
Web slenderness limit	60.00	
Plate slenderness λ_w	1.12	
Reduction factor χ_w	0.74	
Contribution of the web $V_{bw,Rd}$	350.27	kN
Capacity of the flange $M_{f,Rd}$	344.99	kNm
Flange factor c	1.606	m
Contribution of the flange $V_{bf,Rd}$	3.27	kN
Maximum resistance $V_{b,Rd,limit}$	568.54	kN
Resistance $V_{b,Rd}$	353.53	kN
Plastic resistance $M_{pl,Rd}$	502.92	kNm
Shear ratio $\eta_{3,bar}$	0.04	

Unity check (5.10) = 0.04 -

Note: The interaction between Bending and Shear Buckling does not need to be verified because the shear ratio does not exceed 0.5.

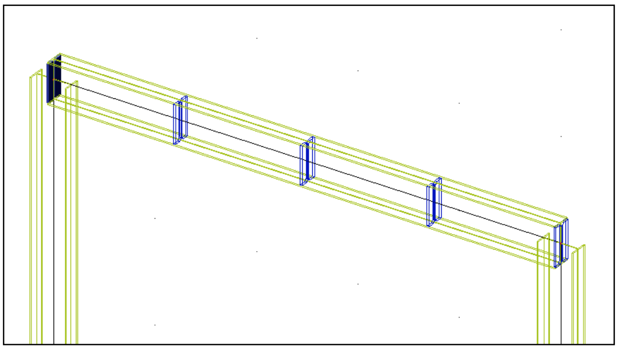
Stiffners

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”.

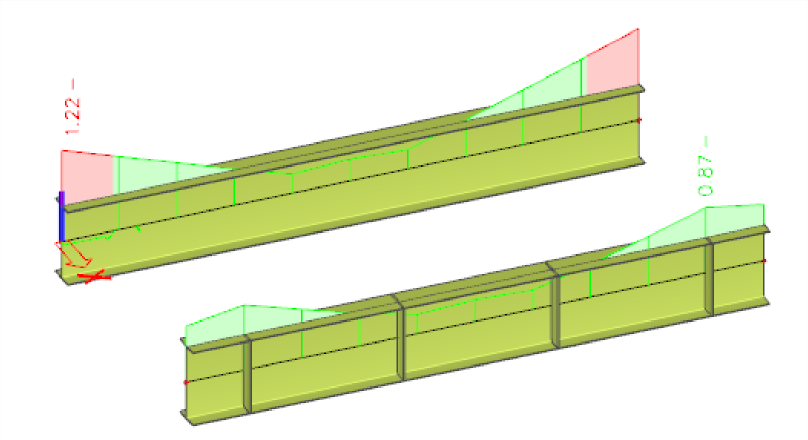


The stiffeners define the field dimensions (a,d) which are only relevant for the shear buckling check. When no stiffeners are defined, the value for ‘a’ is taken equal to the member length.

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 h_w	782.00	mm
Web thickness t	5.00	mm
Yield strength f_{yw}	235.0	MPa
Flange width b_f	275.00	mm
Flange thickness t_f	9.00	mm
Yield strength f_{yf}	235.0	MPa
Material coefficient ε	1.00	
Shear correction factor η	1.20	

Shear Buckling verification		
Web slenderness h_w/t	156.40	
Web slenderness limit	60.00	
Plate slenderness λ_w	1.81	
Reduction factor χ_w	0.46	
Contribution of the web $V_{bw,Rd}$	243.24	kN
Capacity of the flange $M_{f,Rd}$	460.07	kNm
Flange factor c	1.570	m
Contribution of the flange $V_{bf,Rd}$	1.92	kN
Maximum resistance $V_{b,Rd,limit}$	636.60	kN
Resistance $V_{b,Rd}$	245.16	kN
Plastic resistance $M_{pl,Rd}$	693.01	kNm
Shear ratio $\eta_{3,bar}$	1.23	
Moment resistance $M_{R,eff}$	580.90	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

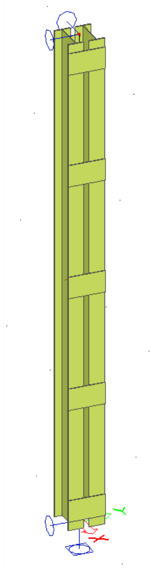
Shear buckling check:

Shear Buckling parameters		
Buckling field length a	1.600	m
Web	stiffened	
End post	non-rigid	
Web height h_w	782.00	mm
Web thickness t	5.00	mm
Yield strength f_{yw}	235.0	MPa
Flange width b_f	275.00	mm
Flange thickness t_f	9.00	mm
Yield strength f_{yf}	235.0	MPa
Material coefficient ε	1.00	
Shear correction factor η	1.20	

Shear Buckling verification		
Shear buckling coefficient k_T	6.30	
Web slenderness h_w/t	156.40	
Web slenderness limit	64.82	
Plate slenderness λ_w	1.67	
Reduction factor χ_w	0.50	
Contribution of the web $V_{bw,Rd}$	264.19	kN
Capacity of the flange $M_{f,Rd}$	460.07	kNm
Flange factor c	0.419	m
Contribution of the flange $V_{bf,Rd}$	11.38	kN
Maximum resistance $V_{b,Rd,limit}$	636.60	kN
Resistance $V_{b,Rd}$	275.57	kN
Plastic resistance $M_{pl,Rd}$	693.01	kNm
Shear ratio $\eta_{3,bar}$	0.91	
Moment resistance $M_{R,eff}$	580.90	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



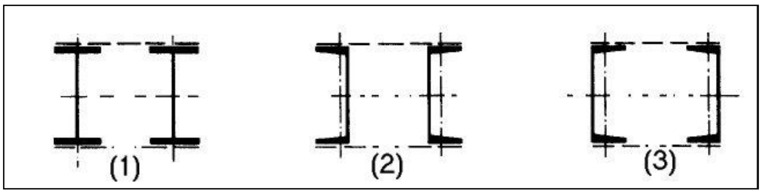
This check will be executed following the EC-EN 11993-1-1 art.6.4.1 and 6.4.3.

The following section pairs are supported as battened compression member:

(1) 2I

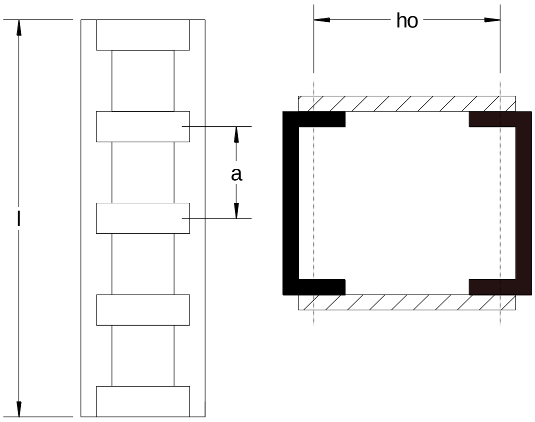
(2) 2Uo

(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

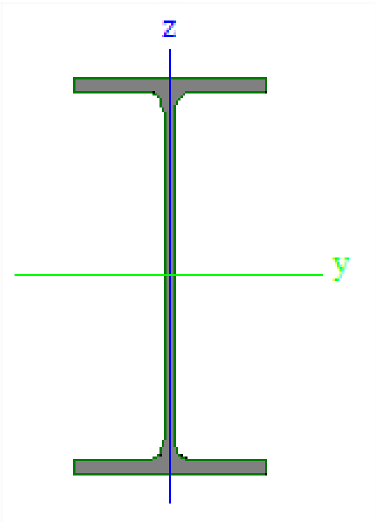
Consider member B1

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

Length L	4600.00	mm
Length a	1150.00	mm
Distance between centroids of chords h_0	210.00	mm
Second moment of area of chord I_{ch}	7.8800e+06	mm^4
Slenderness λ	41.50	
Efficiency factor μ	1.00	
Area of chord A_{ch}	6.2600e+03	mm^2
Effective second moment of area I_{eff}	1.5379e+08	mm^4
Shear stiffness S_v	24381.93	kN
Bow imperfection e_0	9.20	mm
Moment M_{Ed}^{I}	0.00	kNm
Moment M_{Ed}	4.86	kNm
Shear force V_{Ed}	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

h_0 distance between centroids of chords = 210mm

I_{ch} I_z of the IPE330 profile

A_{ch} area if one I-profile, so A of IPE 330

Lambda = l / i_0 = 4600mm / 110,833 mm = 41.5

with

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

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

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

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 => $\mu = 1.00$

I_{eff} $0.5 \cdot h0^2 \cdot Ach + 2 \cdot \mu \cdot Ich$

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

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

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

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

And

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

⇨ $Sv = 2.438\ x\ 10^4 kN$

e_0 = l/500 = 4600mm/500 = 9.2 mm

$M_{Ed,I}$ M_z in the cross section under consideration = 0 kNm

M_{Ed} 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:

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

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

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

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

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

Axial force N_G	270.78	kN
Shear force V_G	1.66	kN
Moment M_G	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:

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

V_G = V_{Ed} /2 = 3,32kN / 2 = 1,66kN

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

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

Buckling check of chord

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

Chord force $N_{ch,Ed}$	270.78	kN
Buckling length L_{cr}	1150.00	mm
Slenderness λ	32.41	
Relative slenderness λ_{rel}	0.37	
Buckling curve	b	
Imperfection α	0.34	
Reduction factor χ	1.00	
Unity check	0.16	-

$N_{ch,Ed}$ = N_{Chord} = 170,78 kN (see check of Check of chord as beam in field between battens)

Buckling length = L_{cr}

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 = $\frac{NchEd}{NbRd} = \frac{270{,}78kN}{1618{,}2\ kN} = 0.16$

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 EN 1993-1-1 article 6.4.3.1, 6.2.9.2 & 6.2.6 and formula (6.42), (6.19)

Thickness t	7.00	mm
Width of link b	220.00	mm
Moment T	9.09	kN
Moment M_G	0.95	kNm
Stress σ	16.9	MPa
Unity check	0.06	-
Stress τ	5.9	MPa
Unity check	0.04	-

t thickness of batten

b width of batten

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

M_G = 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 I_b see calculation of S_v 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: $= \frac{Tau}{\frac{fy/\sqrt{3}}{\gamma_{M0}}} = \frac{5{,}9MPa}{\frac{275N/mm^2/\sqrt{3}}{1.00}} = 0.04$

Optimisation

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

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.

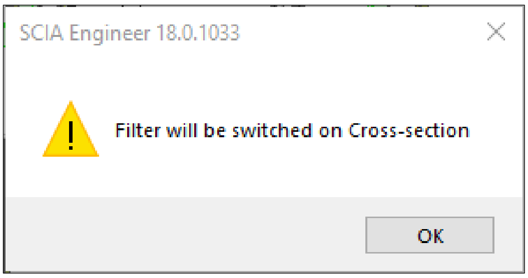
Cross section optimisation

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

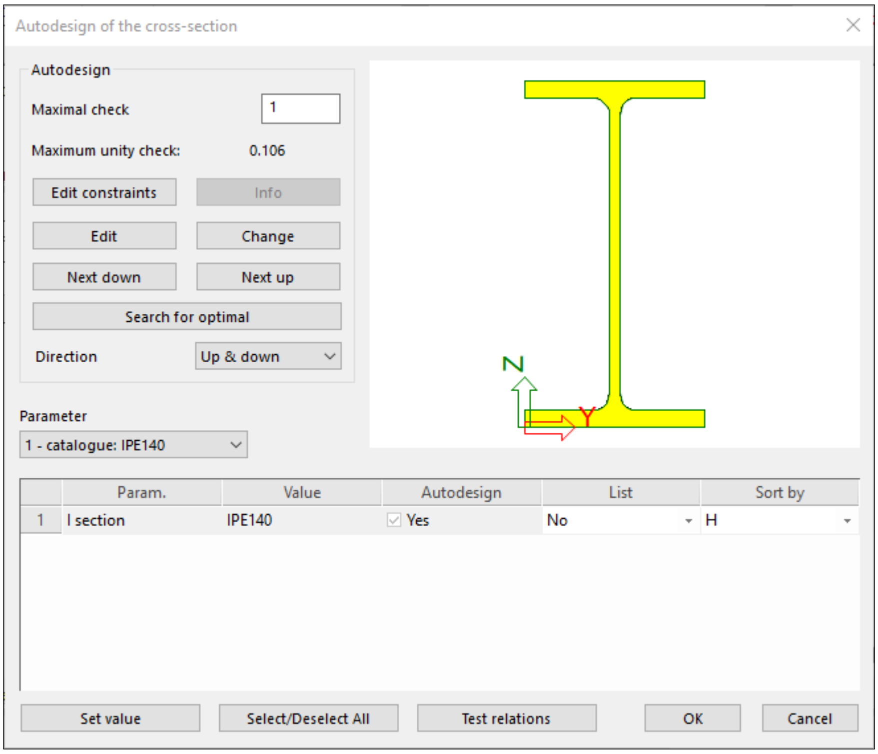
The optimisation can be found via “Steel -> Beams -> ULS Checks – EC-EN 1993 Steel Check ULS”. First the steel code check has to be calculated, so first click on “Refresh”.

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.

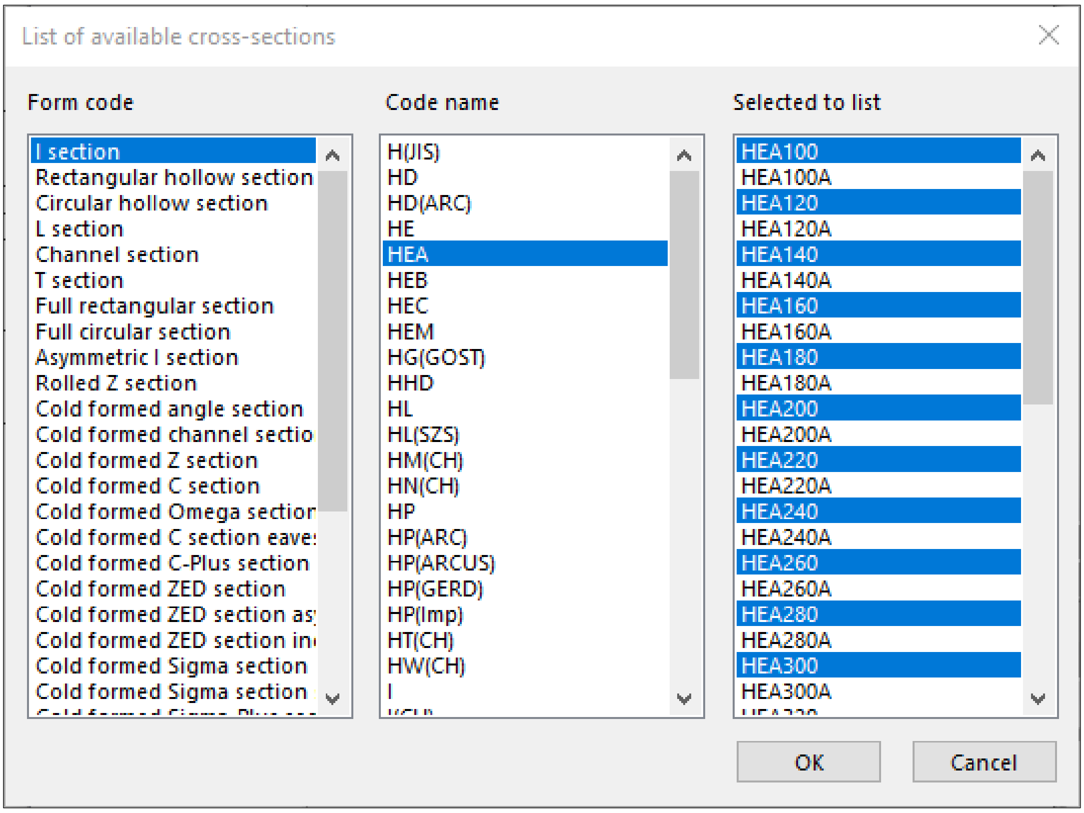
In this example an IPE 80AA will be found. When clicking on "OK" now, SCIA Engineer will replace this IPE140 profile by an IPE80AA profile automatically.

Now the steel code check of this IPE80AA 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.

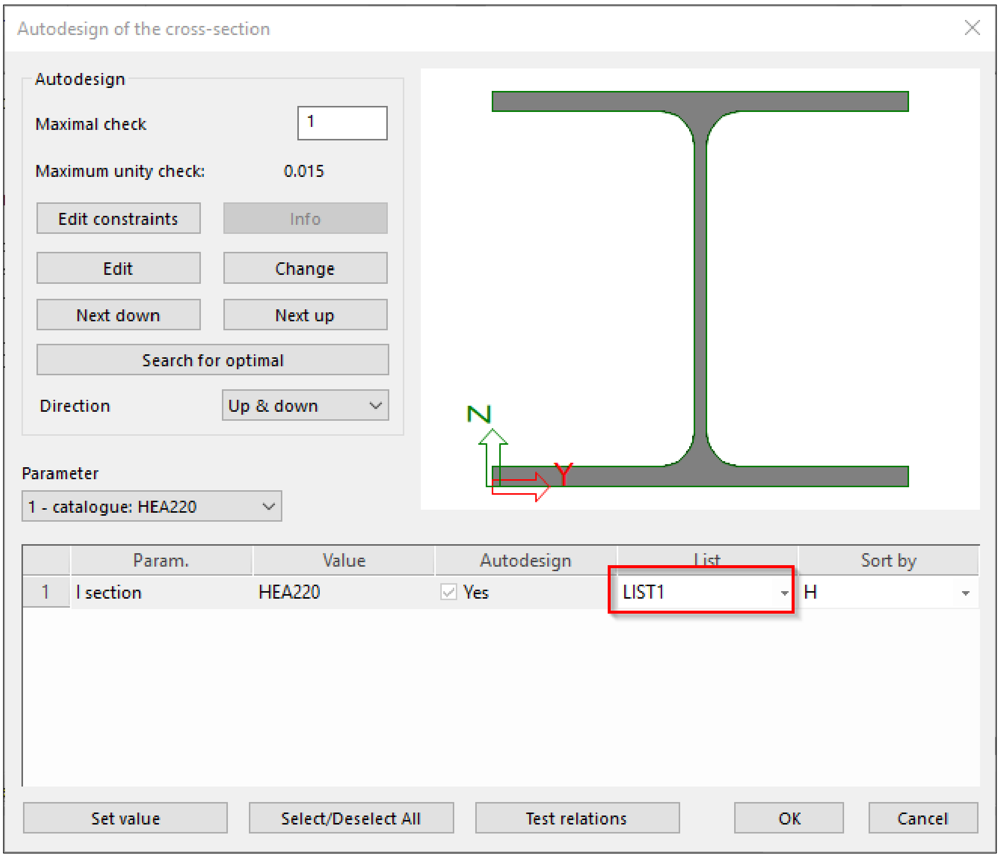
It is also possible to use a **list of sections**.
With this list it is possible to indicate which sections can be used or not. For example the section IPE140A can be filtered out the possibilities for the autodesign.

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 0.015, so SCIA Engineer will search for the smallest profile from this "LIST1" which will pass the unity check

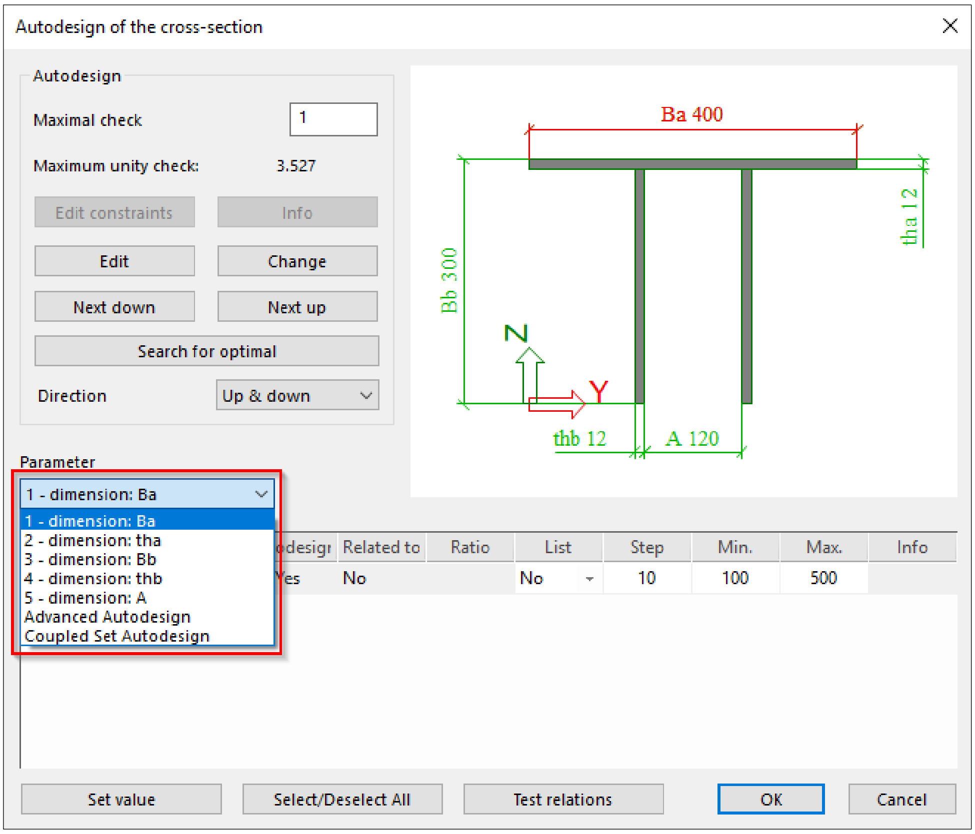
Close this example without saving! It will be used in the next paragraph again!

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

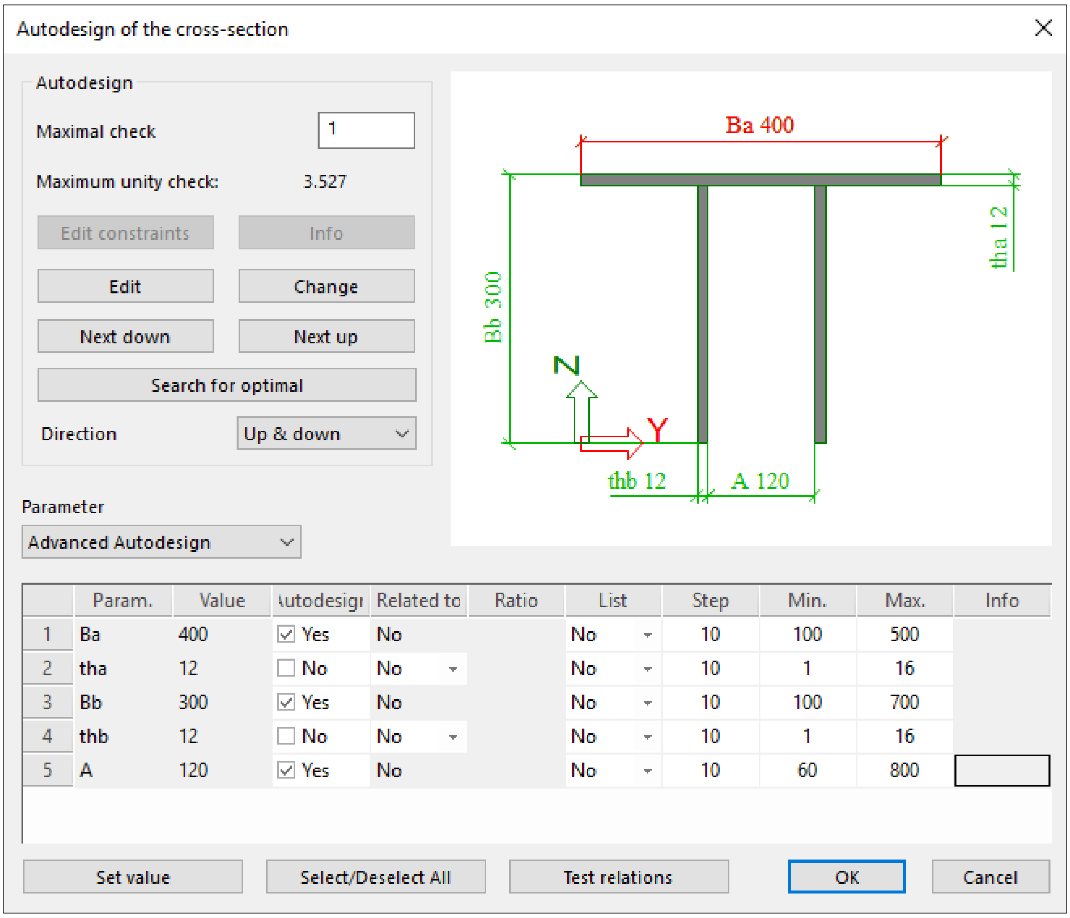
Example: Optimisation2.esa

Go here to "Steel -> Beams -> ULS Checks – EC-EN 1993 Steel Check ULS" and go to the Autodesign menu for this beam.

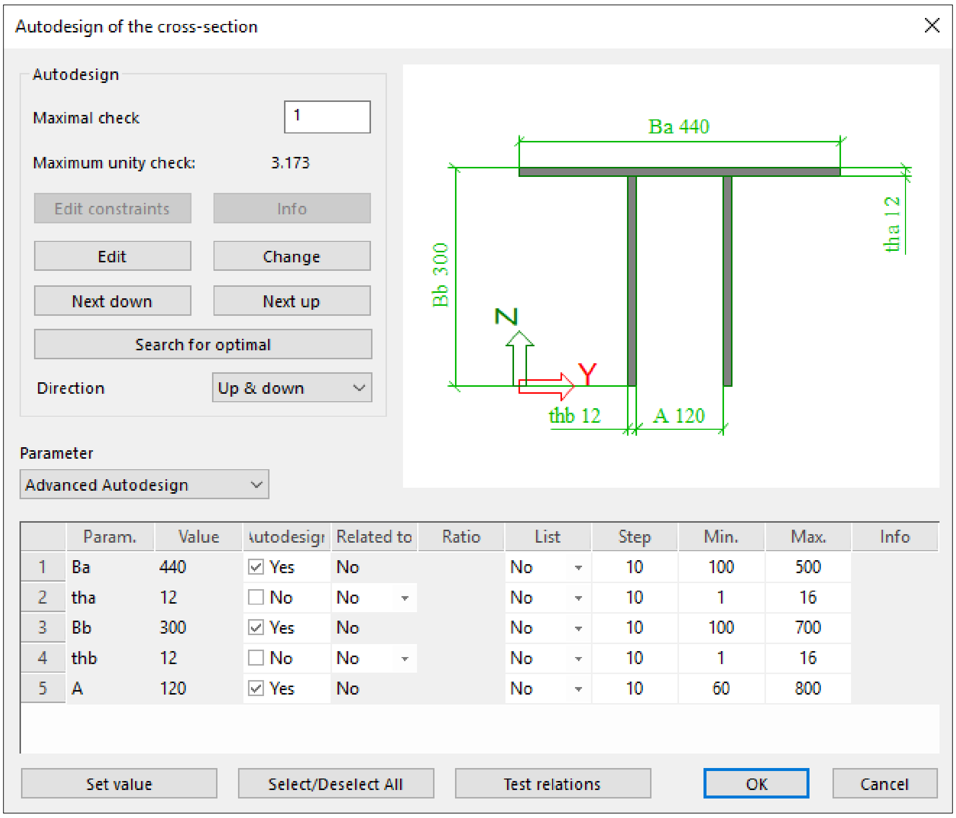
This profile has a lot of parameters, so the user can choose which parameters should be updated:



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 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.

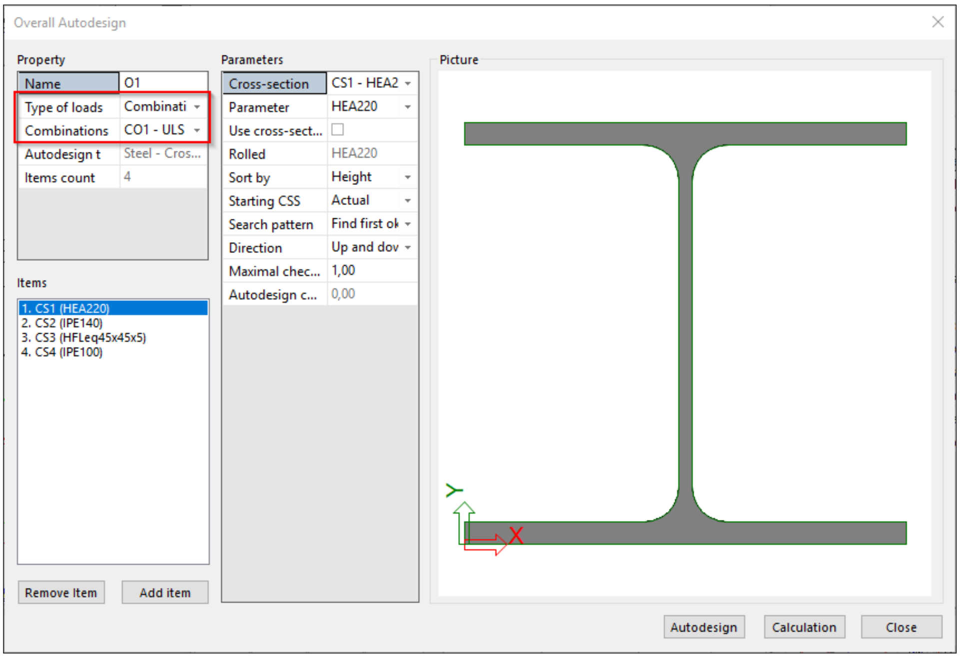
The principle of the overall optimisation process is explained by the following example.

Example: Optimisation.esa

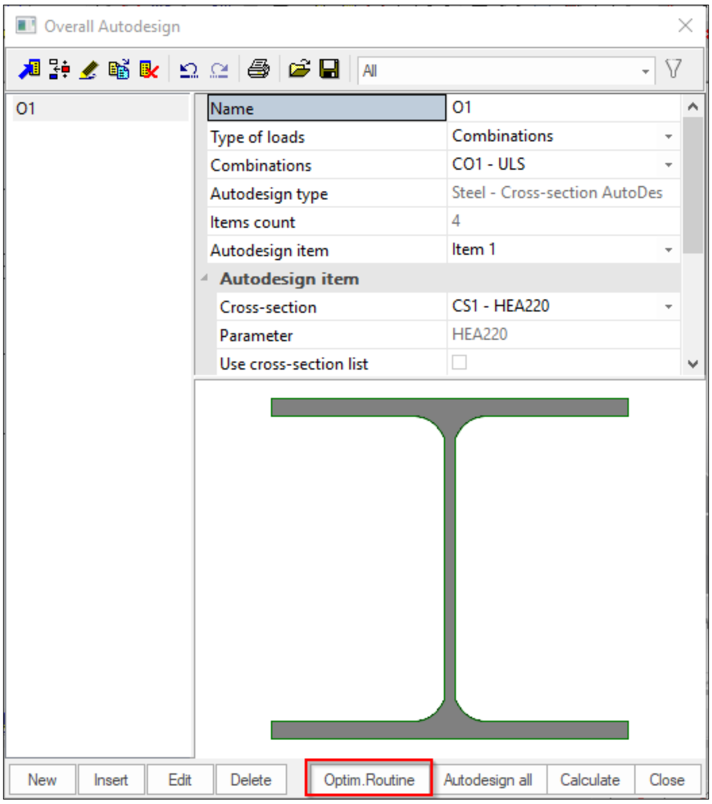
If this example is still open from the previous chapter, please close it and reopen it without saving.

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 3 iteration steps, because all profiles remain the same after the third 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 - HEA160A	HEA160A	Height	CS1 - HEA220	CS1 - HEA200	0,82
CS2 - IPE200O	IPE200O	Height	CS2 - IPE140	CS2 - IPE200O	0,89
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq45x45x5	CS3 - HFLeq80x80x8	0,94
CS4 - IPE140A	IPE140A	Height	CS4 - IPE100	CS4 - IPE140AA	0,91

Changed profiles:

- CS1: HEA220 - > HEA200
- CS2: IPE140 -> IPE200O
- CS3: HFLeq45x45x5 -> HFLeq80x80x8
- 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 - HEA160A	HEA160A	Height	CS1 - HEA200	CS1 - HEA160	0,92
CS2 - IPE200O	IPE200O	Height	CS2 - IPE200O	CS2 - IPE200	0,99
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq80x80x8	CS3 - HFLeq90x90x9	0,82
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140AA	CS4 - IPE140A	0,97

Changed profiles:

- CS1: HEA200 - > HEA160
- CS2: IPE200O -> IPE200
- CS3: HFLeq80x80x8 -> HFLeq90x90x9
- 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 - HEA160A	HEA160A	Height	CS1 - HEA160	CS1 - HEA160	0,76
CS2 - IPE200O	IPE200O	Height	CS2 - IPE200	CS2 - IPE200O	0,87
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq90x90x9	CS3 - HFLeq90x90x9	0,99
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140A	CS4 - IPE140A	0,98

Changed profiles:

- CS2: IPE200 -> IPE200O

ITERATION STEP 4:

4. Routine step: 4

Cross-section	Parameter	Sort by	Original cross-section	Autodesign of cross-section	Autodesign check [-]
CS1 - HEA160A	HEA160A	Height	CS1 - HEA160	CS1 - HEA160A	0,98
CS2 - IPE200O	IPE200O	Height	CS2 - IPE200O	CS2 - IPE200O	0,89
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq90x90x9	CS3 - HFLeq90x90x9	0,98
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140A	CS4 - IPE140A	0,98

Changed profiles:

- CS1: HEA160 - > HEA160A

ITERATION STEP 5:

5. Routine step: 5

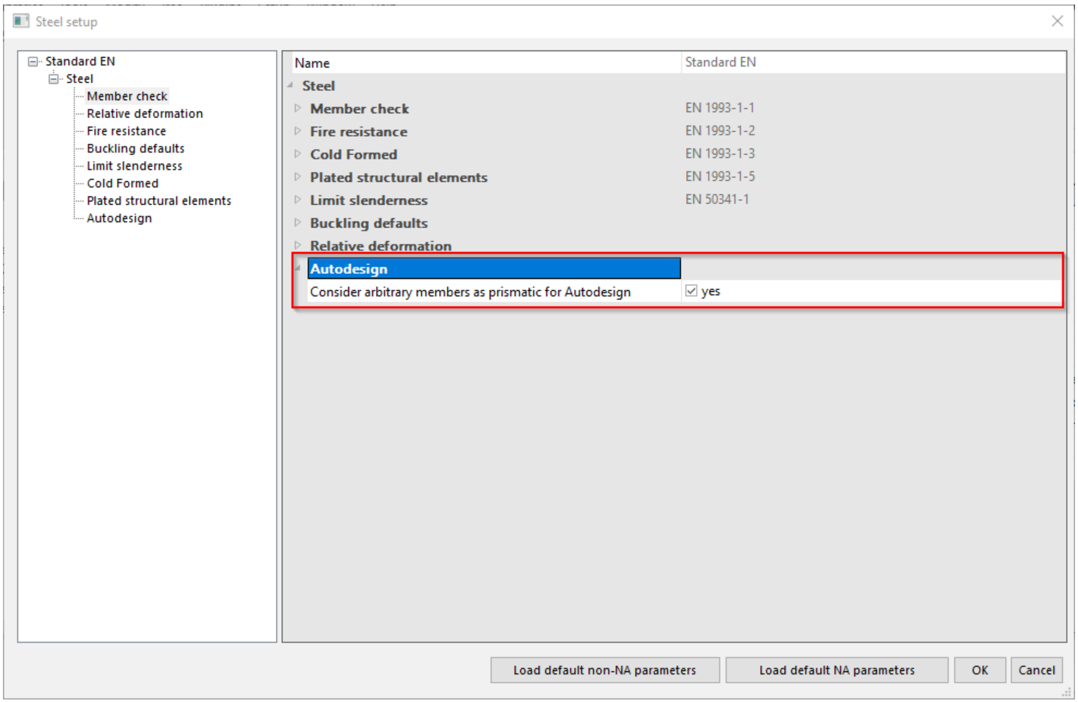
Cross-section	Parameter	Sort by	Original cross-section	Autodesign of cross-section	Autodesign check [-]
CS1 - HEA160A	HEA160A	Height	CS1 - HEA160A	CS1 - HEA160A	0,90
CS2 - IPE200O	IPE200O	Height	CS2 - IPE200O	CS2 - IPE200O	0,96
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq90x90x9	CS3 - HFLeq90x90x9	0,99
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140A	CS4 - IPE140A	0,98

As a limit number we put 5 iteration steps so the iteration process is completed.

If earlier in the iteration process no profile had been adapted, the iteration process would stop then.

Arbitrary members

Since SCIA Engineer 17.0, there is a new setting available for performing the autodesign of arbitrary members. This setting can be found in “Steel > Beams > Steel Setup”:



When this setting is activated (default setting), the member is considered as being prismatic (e.g.one cross-section) during the autodesign calculation. This speeds up the calculation.

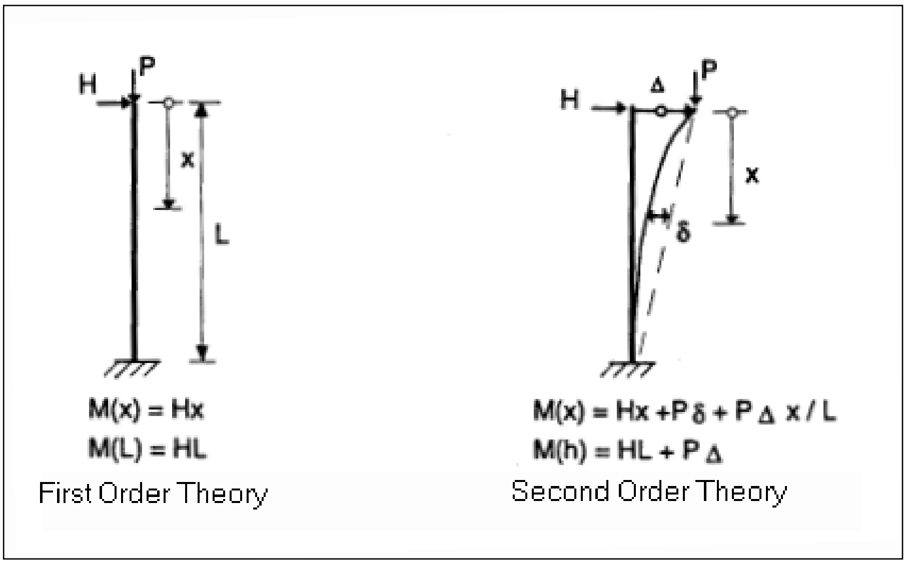
2^{nd} order calculation and imperfections

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.

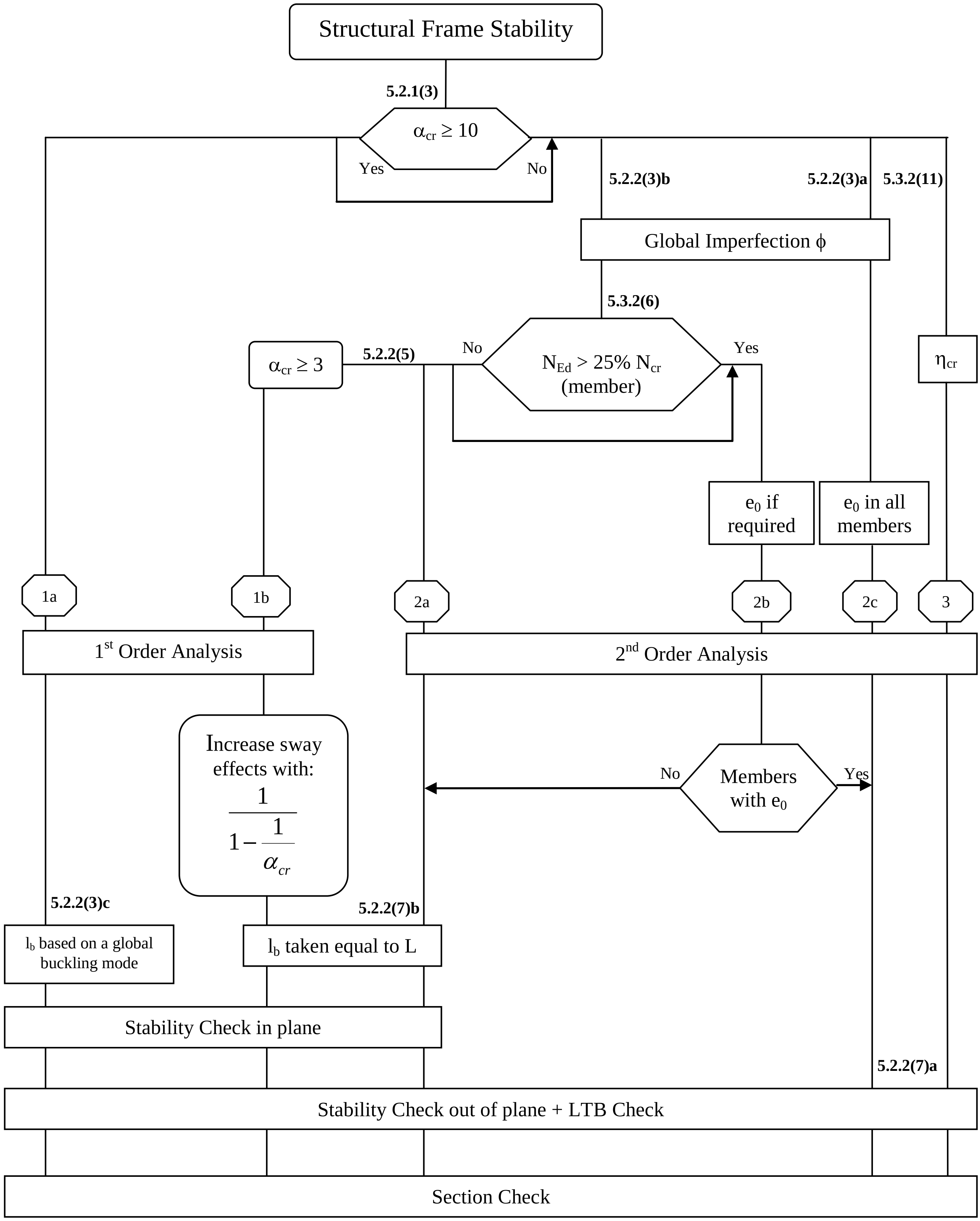
The second-order theory can be adopted in all cases, while first-order theory may be used only when the displacement effects on the structural behavior are negligible.

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:

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

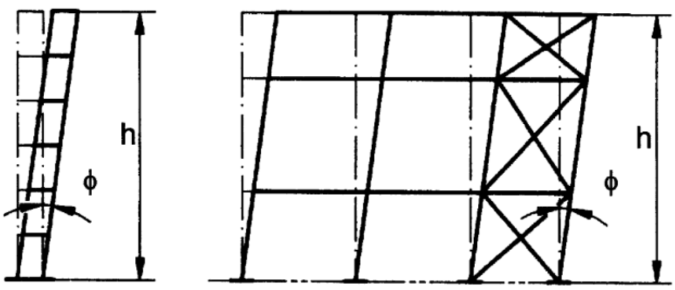
With: α_{cr}: the factor by which the design loading has to be increased to cause elastic instability in a global mode.
F_{Ed}: the design loading on the structure.
F_{cr}: the elastic critical buckling load for global instability, based on initial elastic stiffnesses.

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):



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

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

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

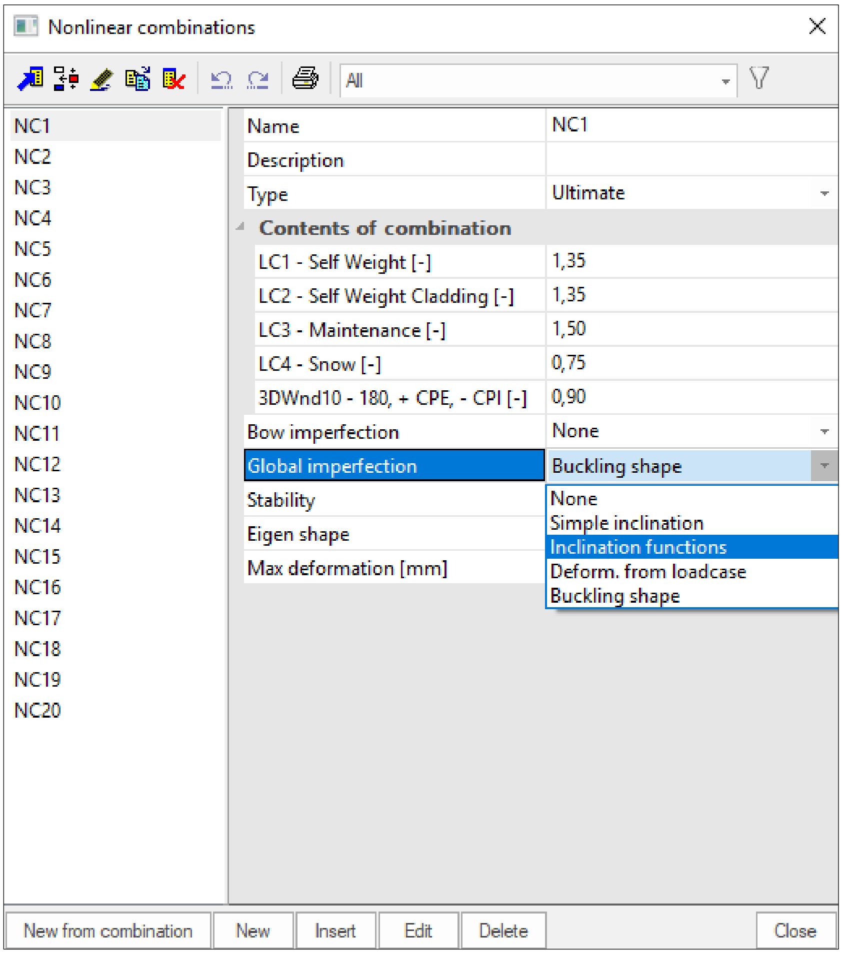
With: h The height of the structure in meters
m The number of columns in a row including only those columns which carry a vertical load N_{Ed} not less than 50% of the average value of the vertical load per column in the plane considered.

The global imperfection can be inputted in SCIA Engineer in the nonlinear 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.

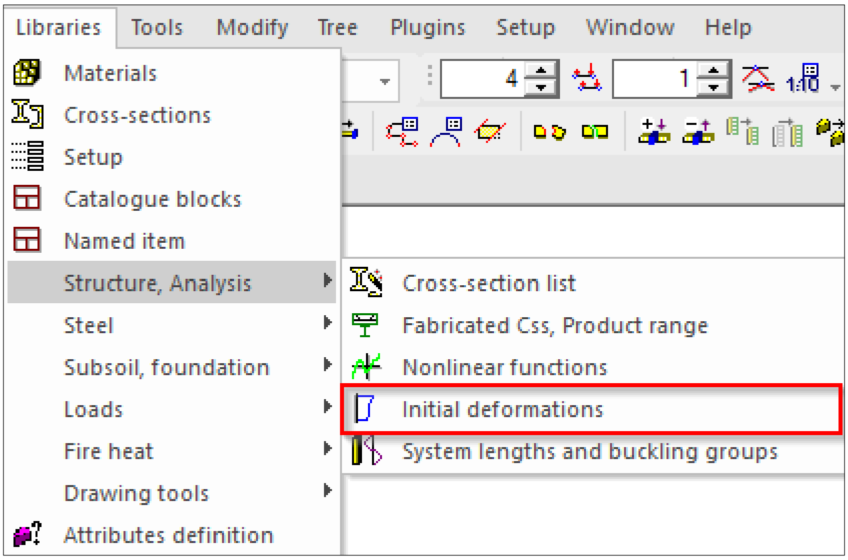
In SCIA Engineer the user can choose between five options for the **global imperfection** in the nonlinear combinations:



- **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.
- **Buckling shape**: the imperfection is derived from the buckling data.

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

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



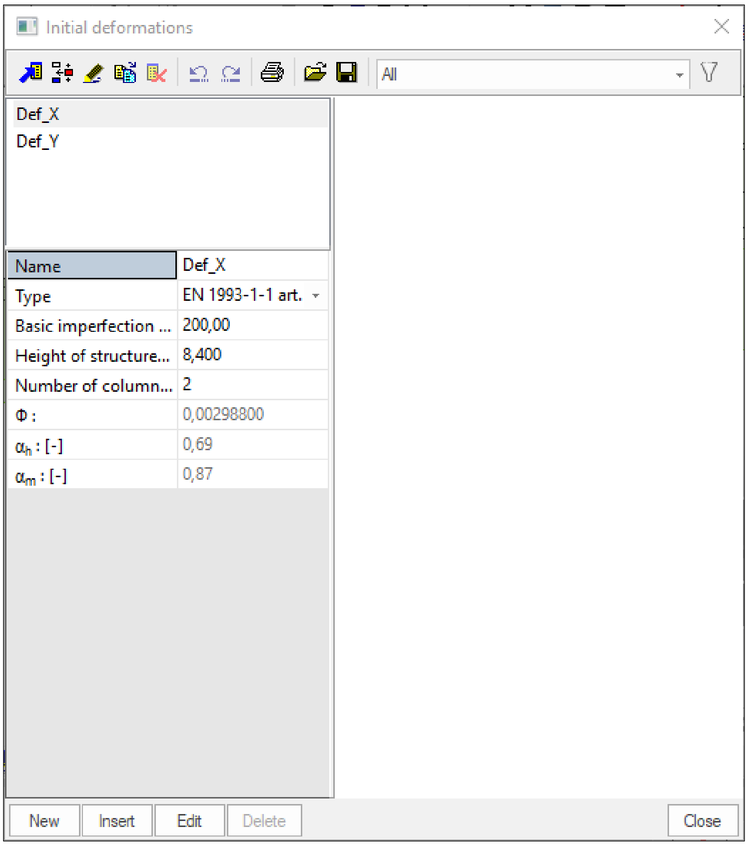
The Type is chosen as “EN 1993-1-1 art. 5.3.2(3)”, with a standard imperfection of 1/200.

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:



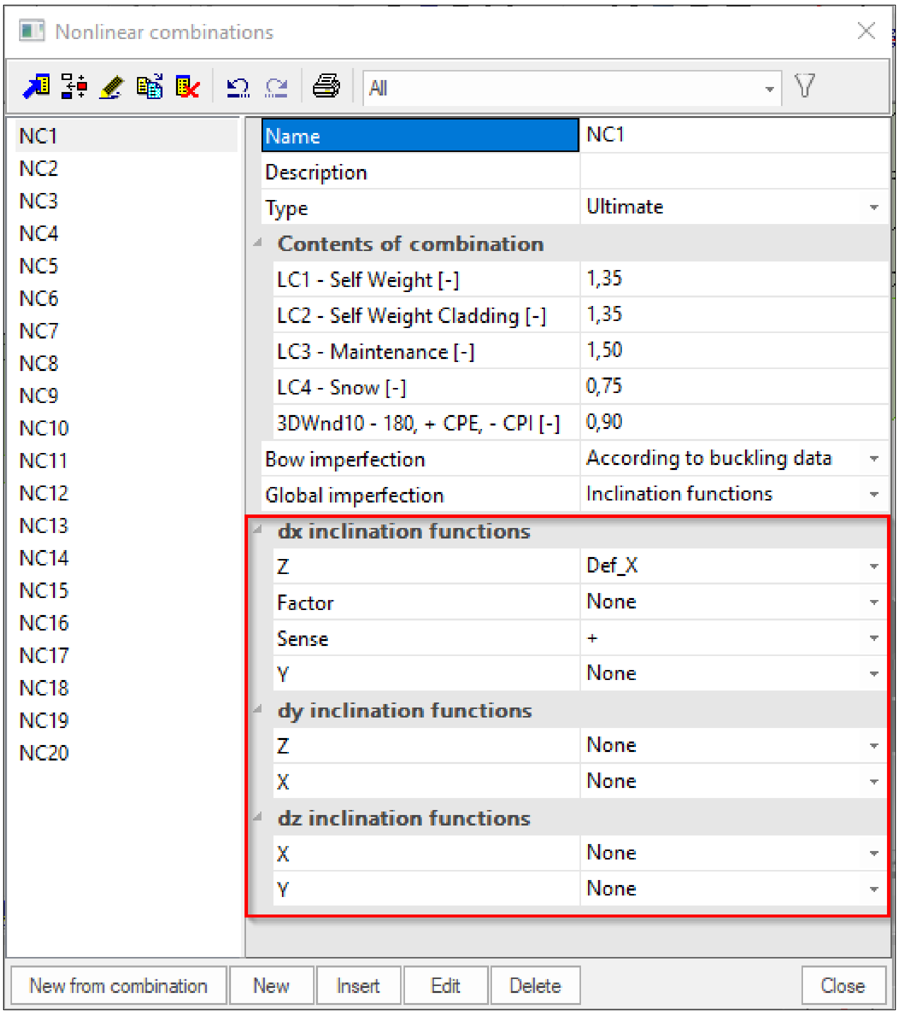
In this example 5 nonlinear combinations are inputted:

- 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



Tip: The nonlinear combinations can be copied from the linear combinations using the button “New from linear combinations”. If code or envelope combinations were used to perform the linear calculation, these combinations first need to be exploded to linear combinations before it is possible to copy them to nonlinear combinations.

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

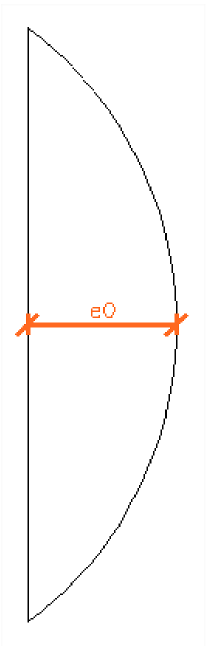
Thus: $\sqrt{\frac{A \cdot f_y}{N_{cr}}} > 0{,}5\sqrt{\frac{A \cdot f_y}{N_{Ed}}}$

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

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

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

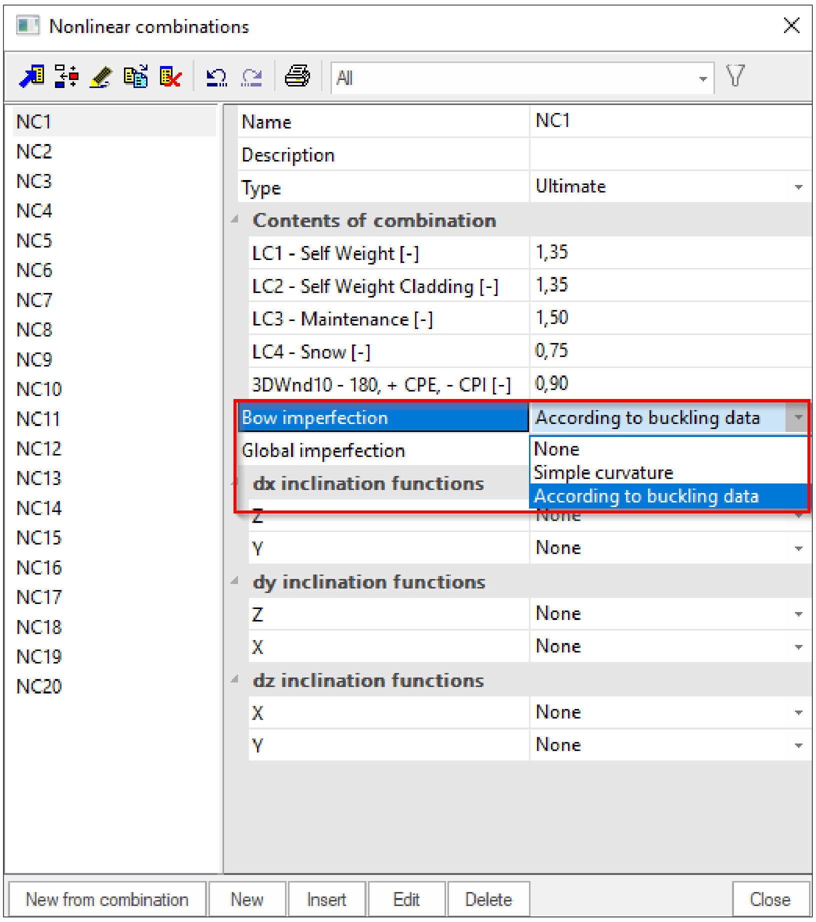
Where L is the member length.

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

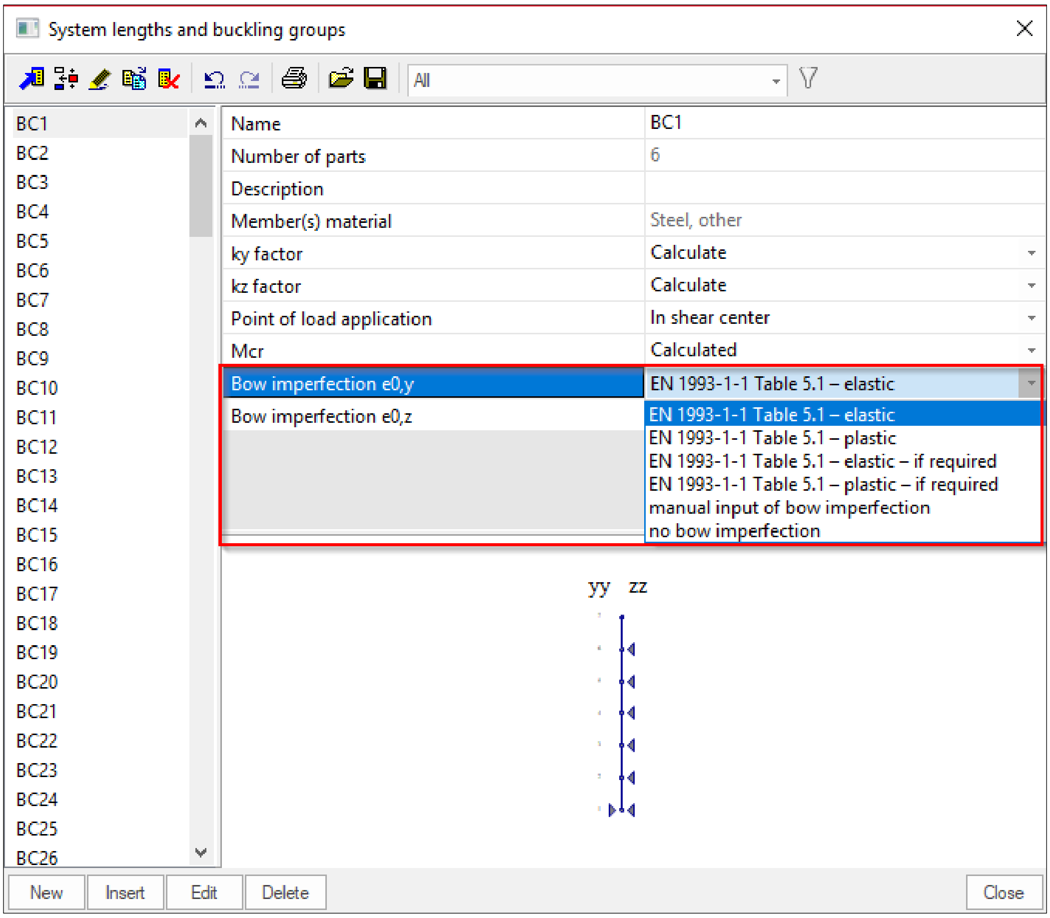
In SCIA Engineer the user can choose between three options for the **bow imperfection** in the nonlinear combinations:



- **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 -> System lengths and buckling groups



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.

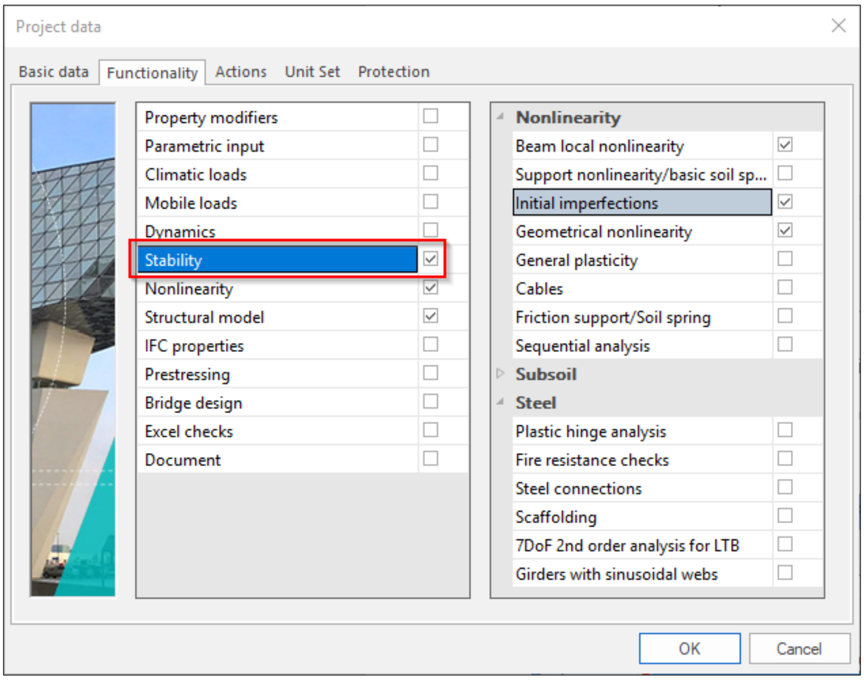
In this example on all beams the bow imperfection “According to code – elastic” has been inputted. Except for the diagonals no bow imperfection has been inputted in this example.

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

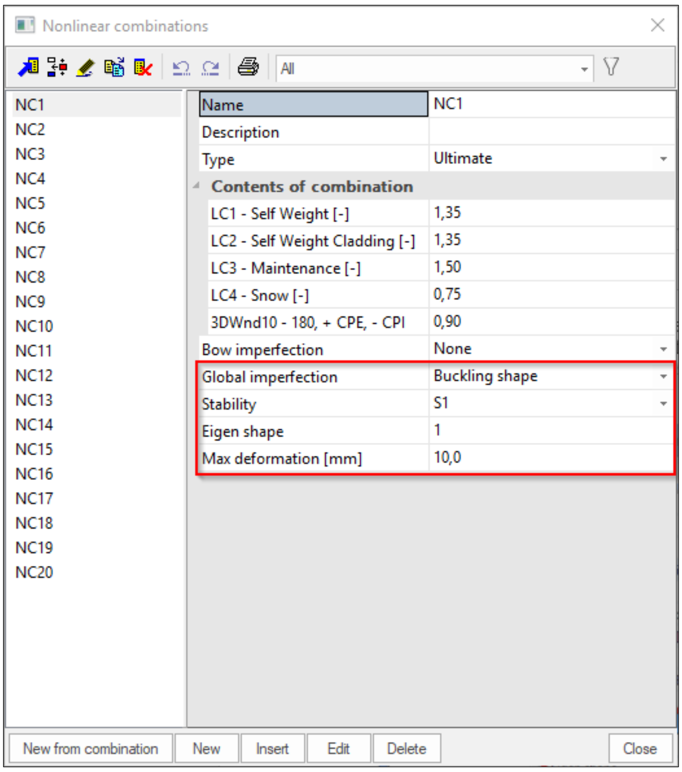
The calculation of alpha critical is done by a stability calculation in SCIA Engineer. For this calculation a Professional or an Expert edition is necessary. The stability calculation has been inputted in module esas.13.

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 imperfections** and **Stability** must be activated.



So first the stability calculation (linear or nonlinear) is calculated. Afterwards in the nonlinear combinations, the user can choose the buckling shape he/she wants to take into account. So first he/she can choose for the stability combination and just below for the calculated mode:



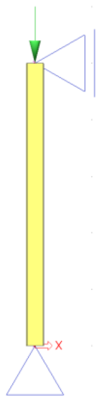
The last option that has to be inputted is a value for the maximum deformation. This is the deformation of the node which has the biggest deformation of the construction. SCIA Engineer will recalculate all the displacement of the other nodes using this maximal deformation.

Since the buckling shape is dimensionless, EN 1993-1-1 gives the formula to calculate the amplitude η_{init} of the imperfection. In example below 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:

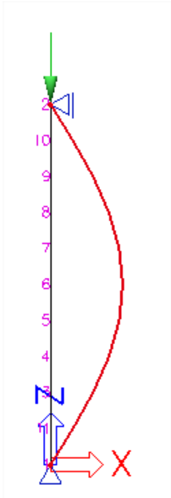
Critical load coefficients

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:

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

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

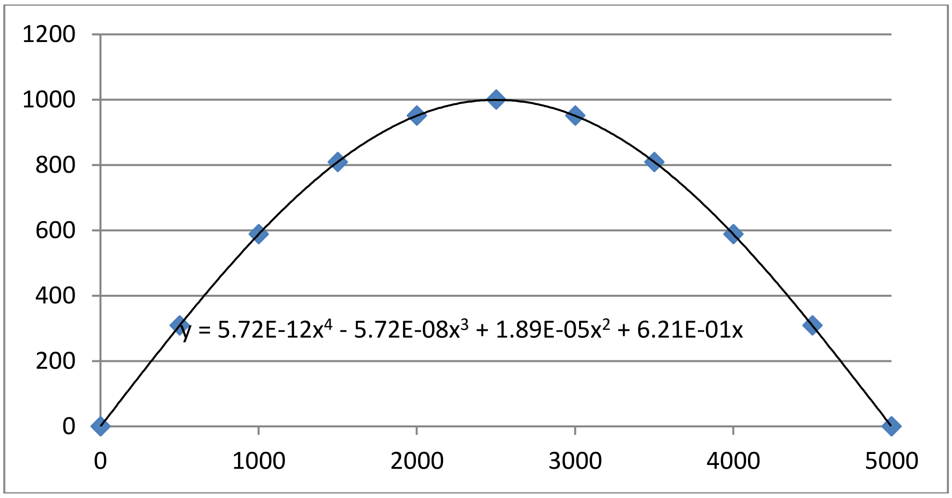


Displacement of nodes

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.

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

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

Calculation of e_0

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

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

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

$\alpha = 0{,}21$ 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	

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

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

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

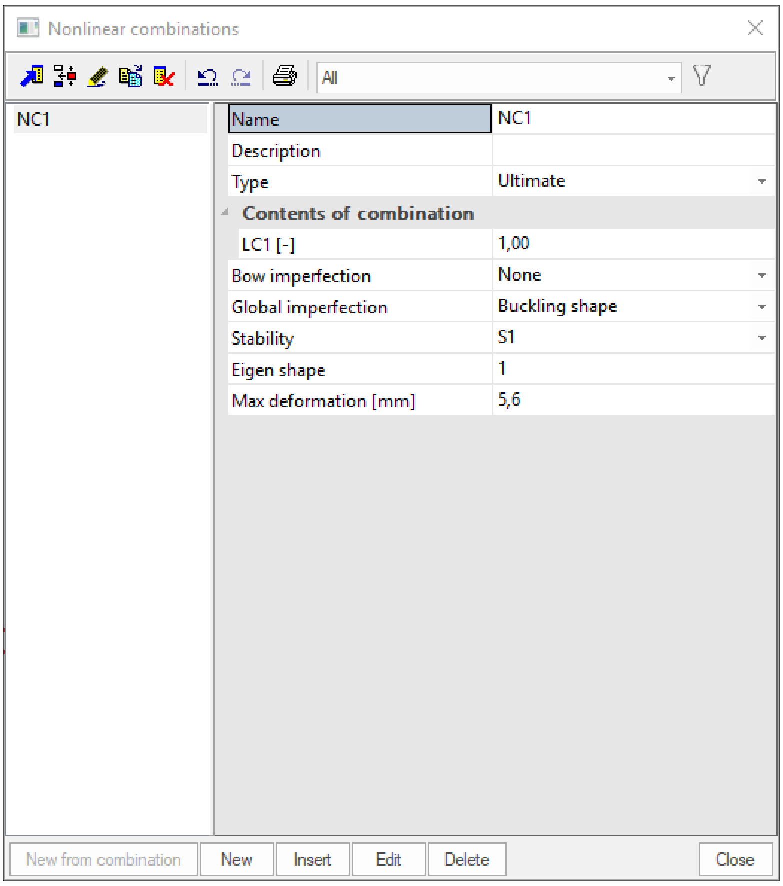
$\eta''_{cr,max}$ at mid section = -3,912 $\cdot 10^{-4} \cdot 1/mm^2$

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

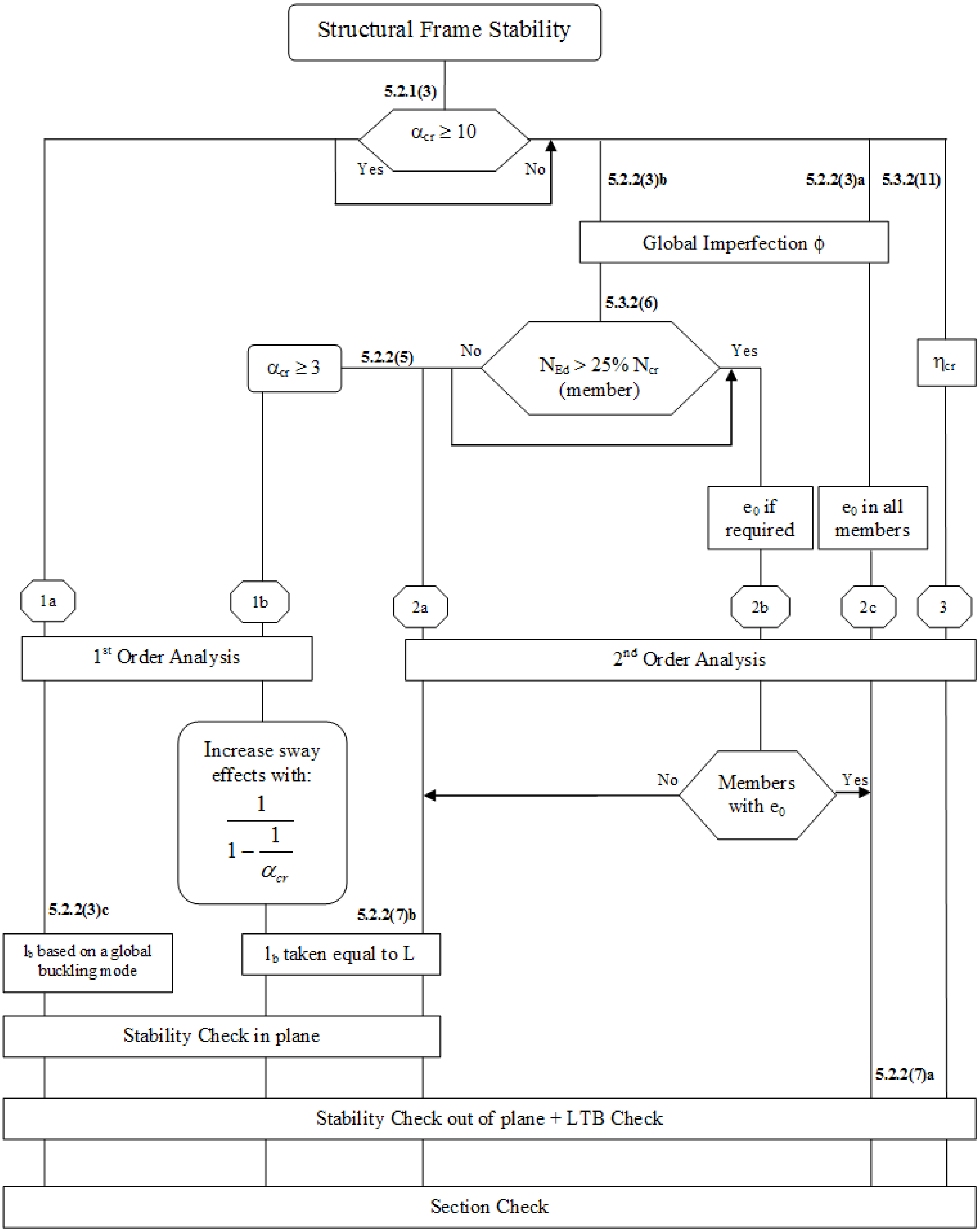
⇨ $\eta_{init} = 5{,}642\ mm \frac{6885280\ N}{210000 N/mm^2\cdot 83560000\ mm^4\cdot 3{,}912\cdot 10^{-4}\cdot 1/mm^2} \cdot 1000$

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

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


Nonlinear combinations
All
NC1
Name
NC1
Description
Type
Ultimate
Contents of combination
LC1 [-]
1,00
Bow imperfection
None
Global imperfection
Buckling shape
Stability
S1
Eigen shape
1
Max deformation [mm]
5,6
New from combination
New
Insert
Edit
Delete
Close

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

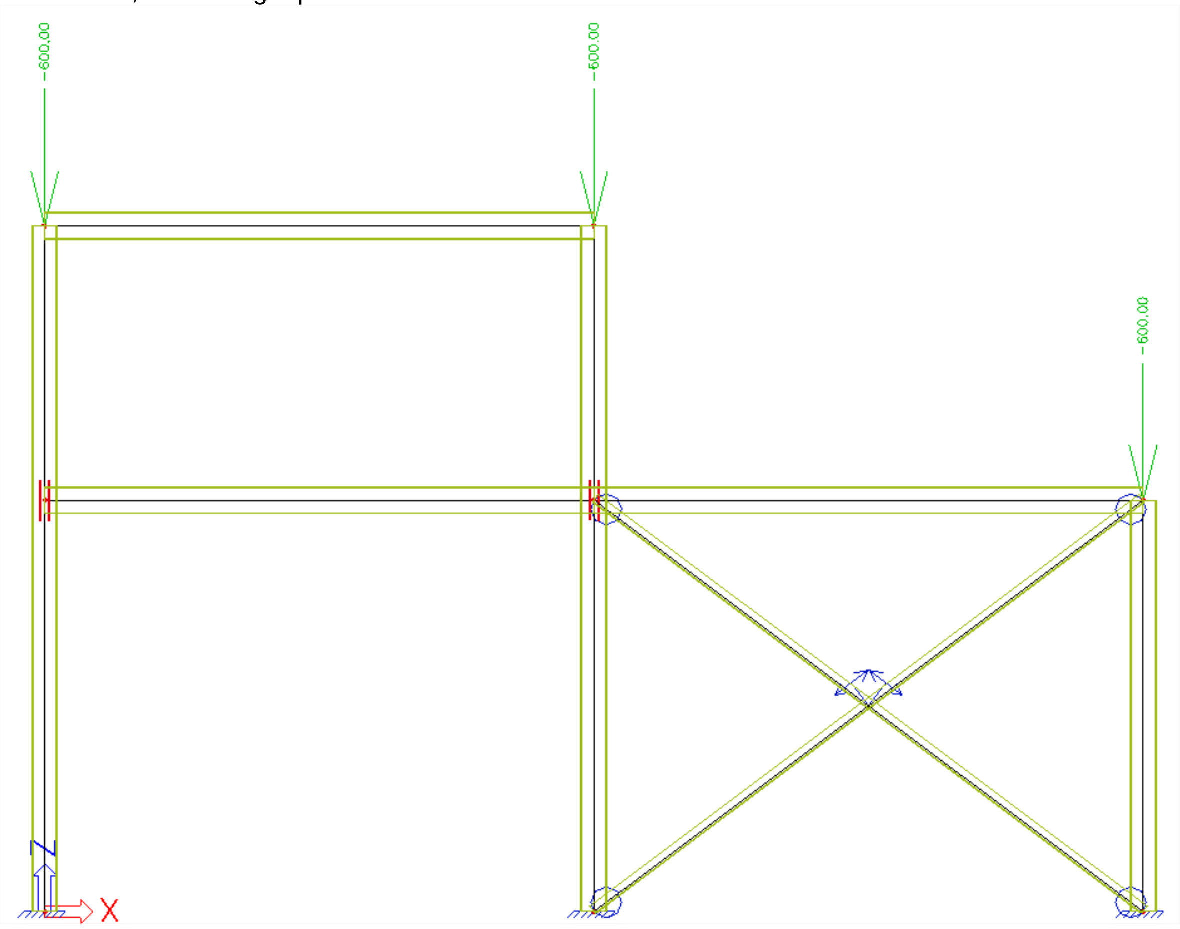
Example Overview Paths_path 2b.esa

Example Overview Paths_path 2c.esa

Example Overview Paths_path 3.esa

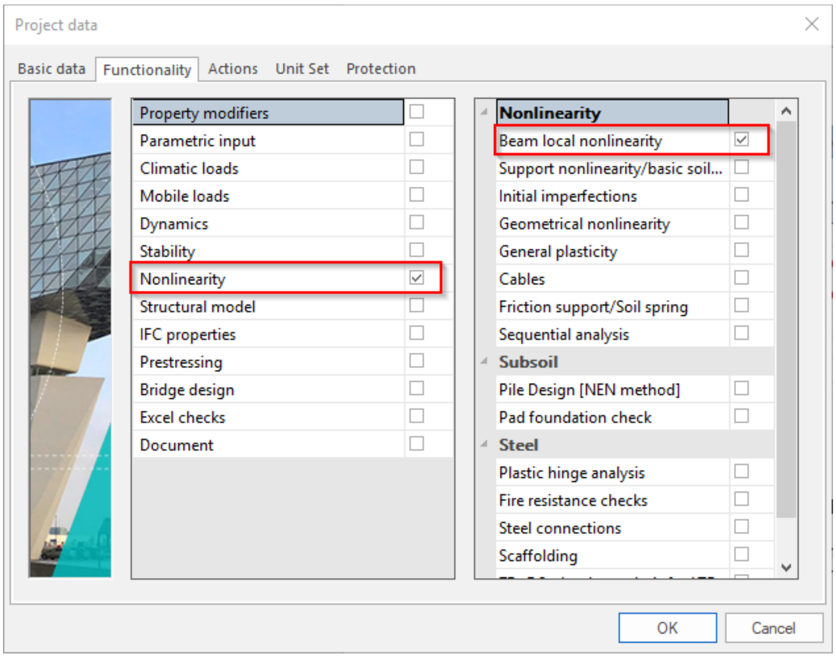
Example Overview

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



Path 1a 1st order analysis

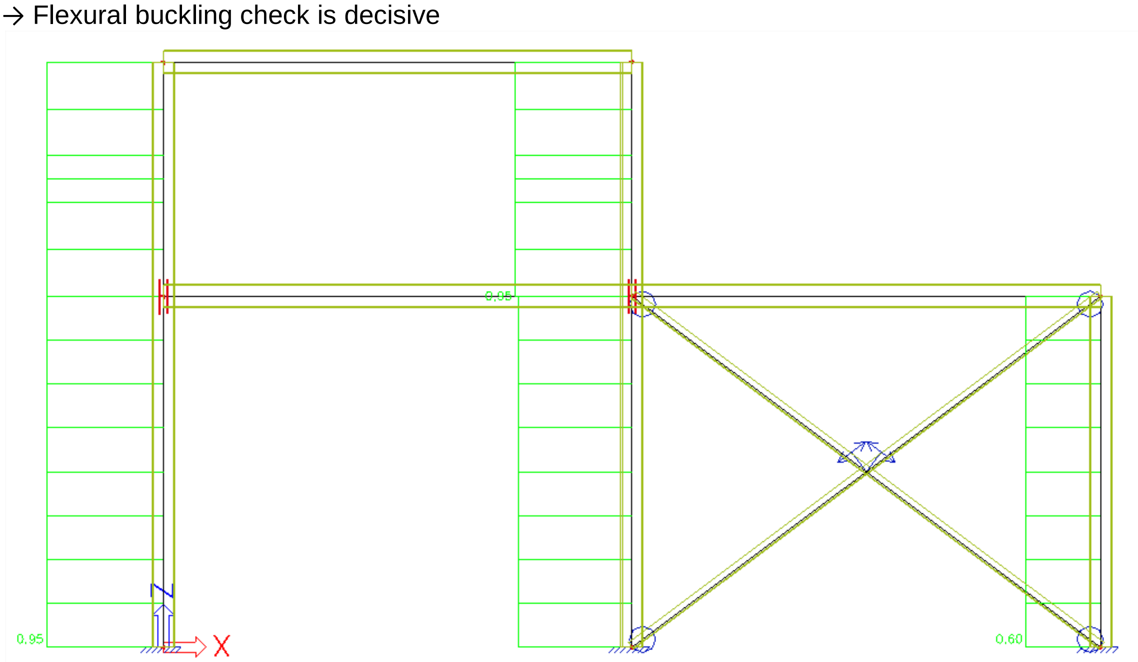
1) Functionalities



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

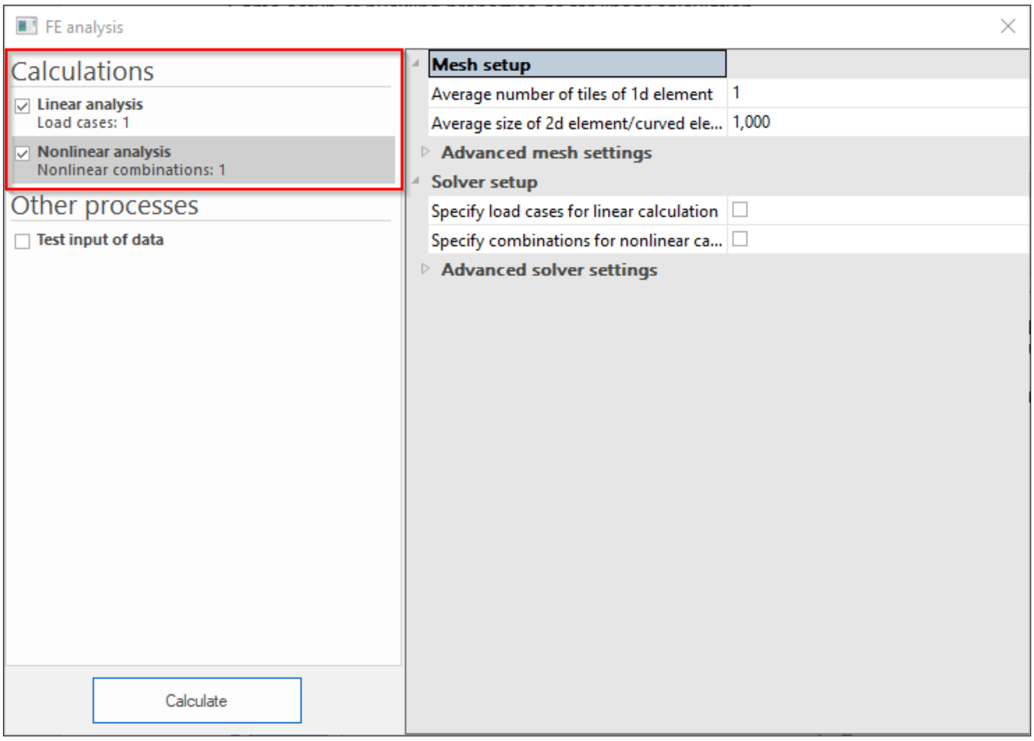
Unity check for ULS
→ Flexural buckling check is decisive



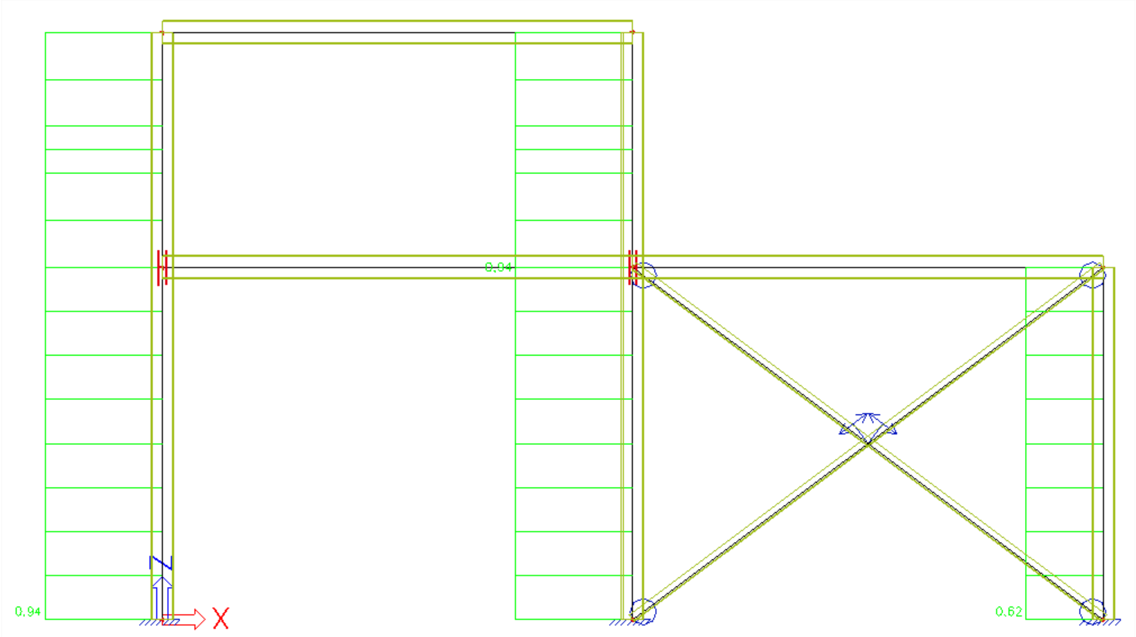
3) Nonlinear calculation

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 both linear and nonlinear calculation/analysis, because the buckling properties are calculated only during the linear calculation! In calculation dialog for version 16 and earlier the user would choose Batch analysis (linear and nonlinear analysis).

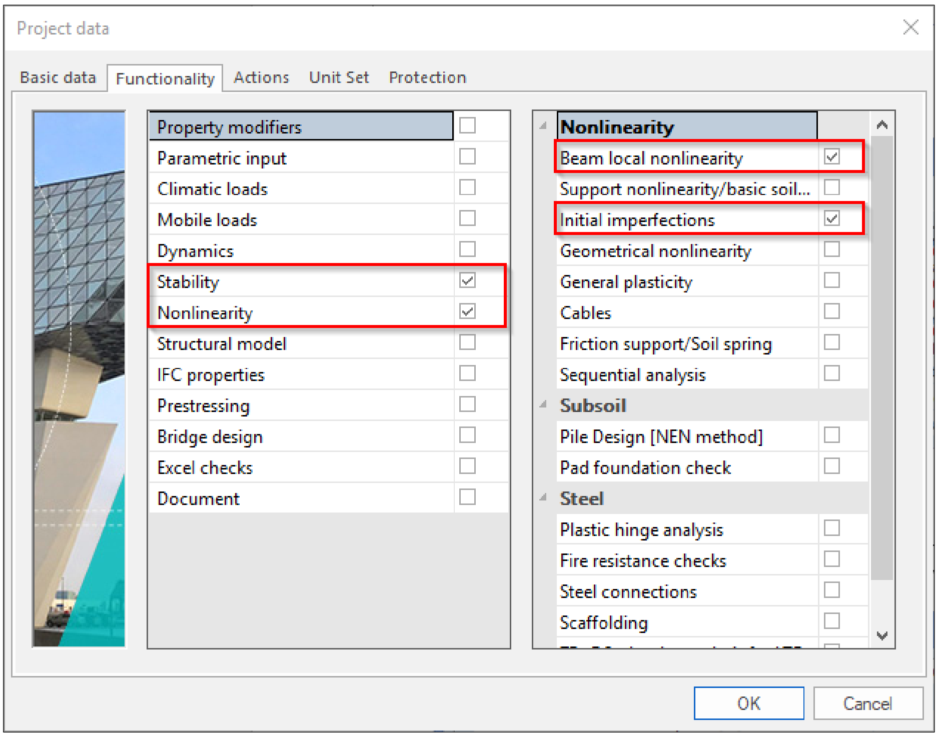


Unity check for ULS
→ Flexural buckling check is decisive



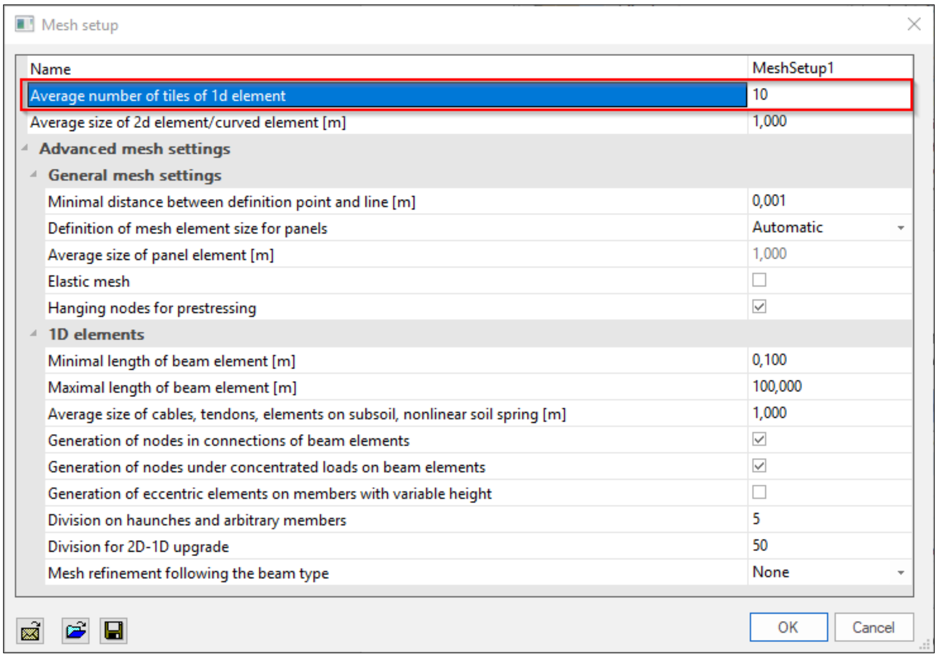
Path 1b 1st order analysis

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**.



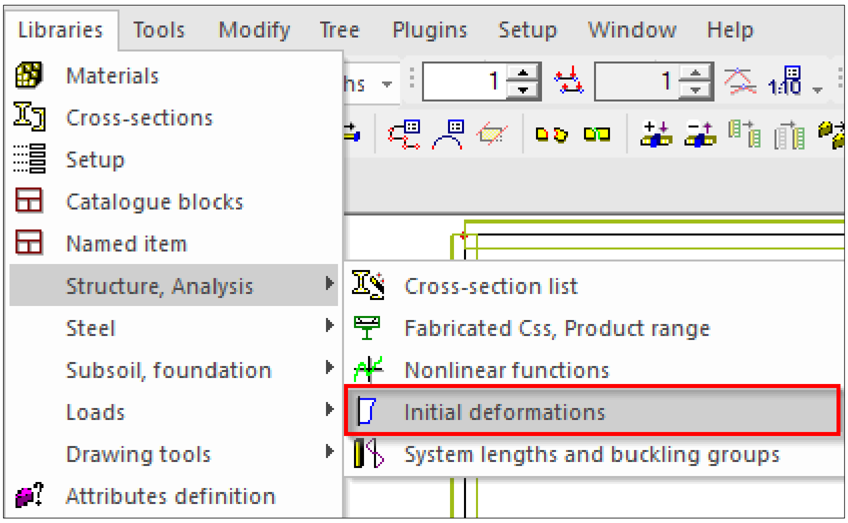
The critical alpha value has to be higher than 3 to be able to use path b of the structural frame stability scheme. In this example, the critical load coefficient is equal to 7,03 (which is in between 3 and 10).

Critical load coefficients

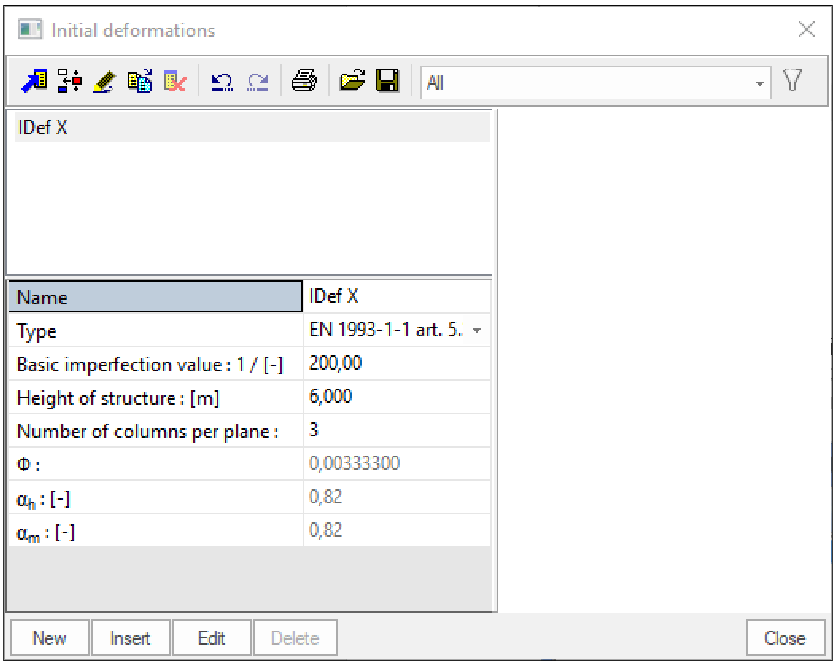
N -	f []
Stability combination : S1	
1	7,03
2	22,78
3	27,07
4	35,01

3) Global imperfection

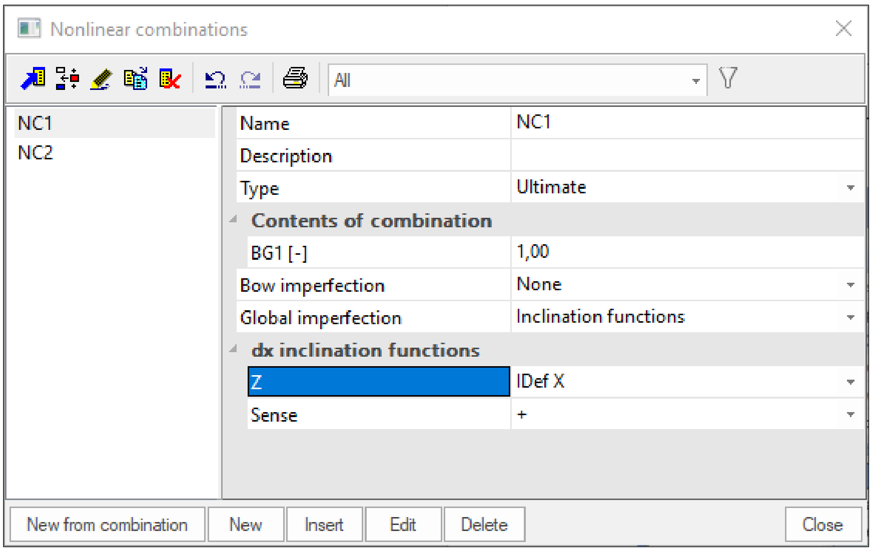
The next step is to insert the global imperfection. This will be done using “Libraries > Structure, Analysis > Initial deformations”.



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).

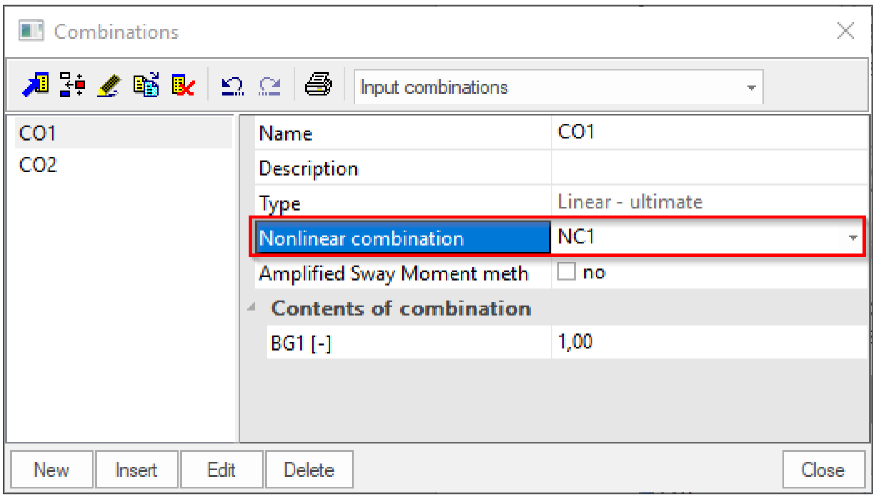


This global imperfection can now be inserted into the nonlinear combination. Once in the positive x-direction and once in the negative x-direction.



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.



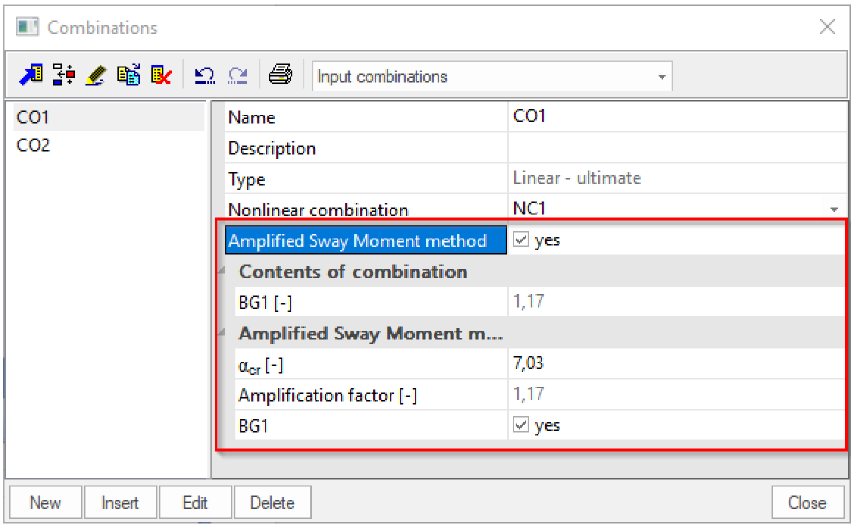
The **Amplified Sway Moment method** may only be used, if the actual normal force N_{Ed} is smaller than 25% of the critical Euler force N_{cr}. Both values are printed in the detailed output of the steel code check. For column S1:

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

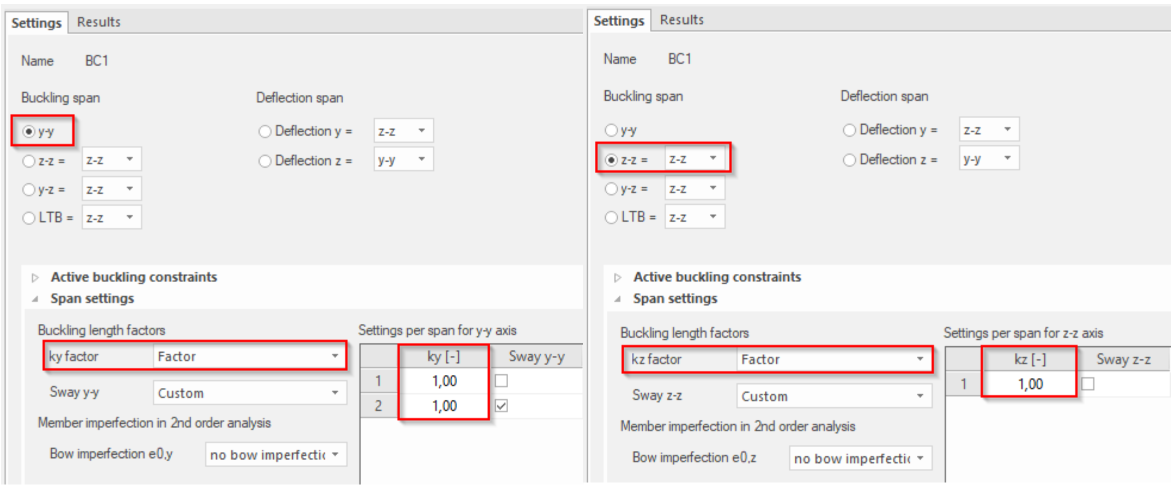
Buckling parameters	yy
Sway type	non-sway
System length L	3,000
Buckling factor k	0,60
Buckling length Lcr	1,802
Critical Euler load Ncr	23558,43
Slenderness Lambda	21,76
Relative slenderness Lambda,rel	0,23
Limit slenderness Lambda,rel,0	0,20
Buckling curve	b
Imperfection Alpha	0,34
Reduction factor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

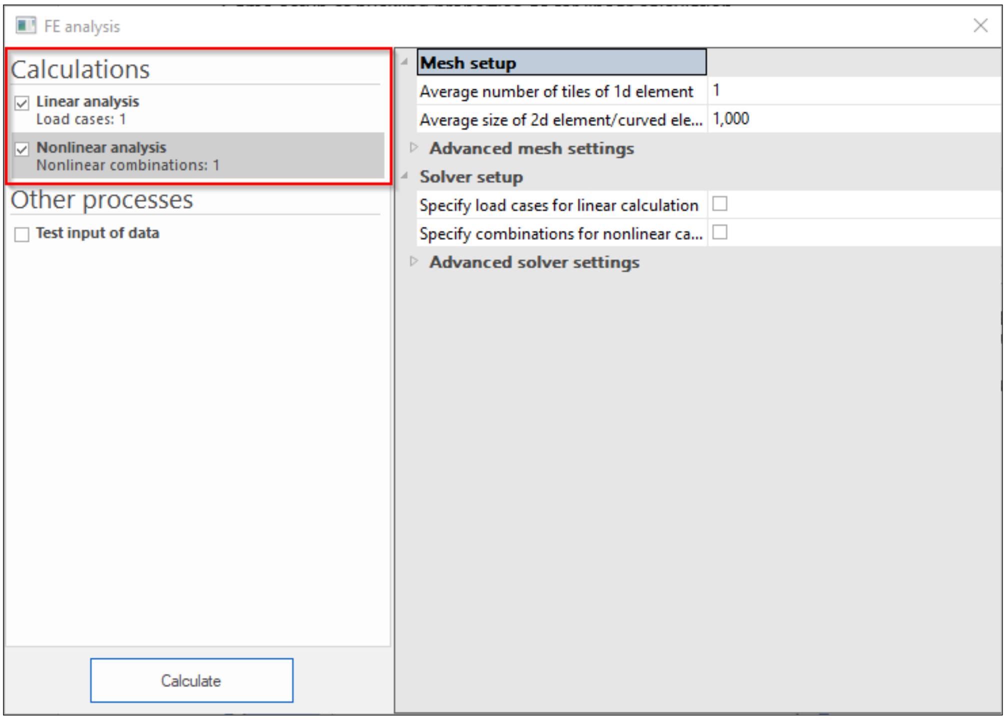


5) Linear calculation

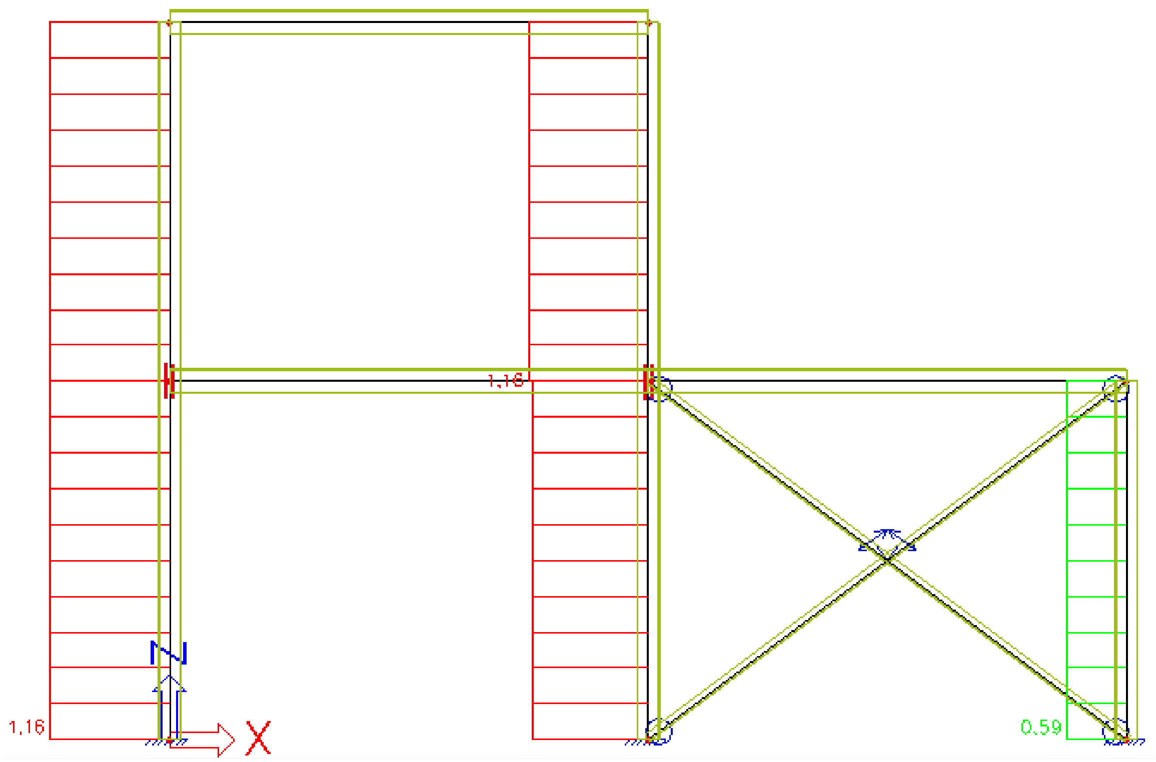
The buckling lengths are taken equal to the system lengths. This can be done by inserting buckling ratios equal to 1 manually in ‘System lengths and buckling settings’:



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.

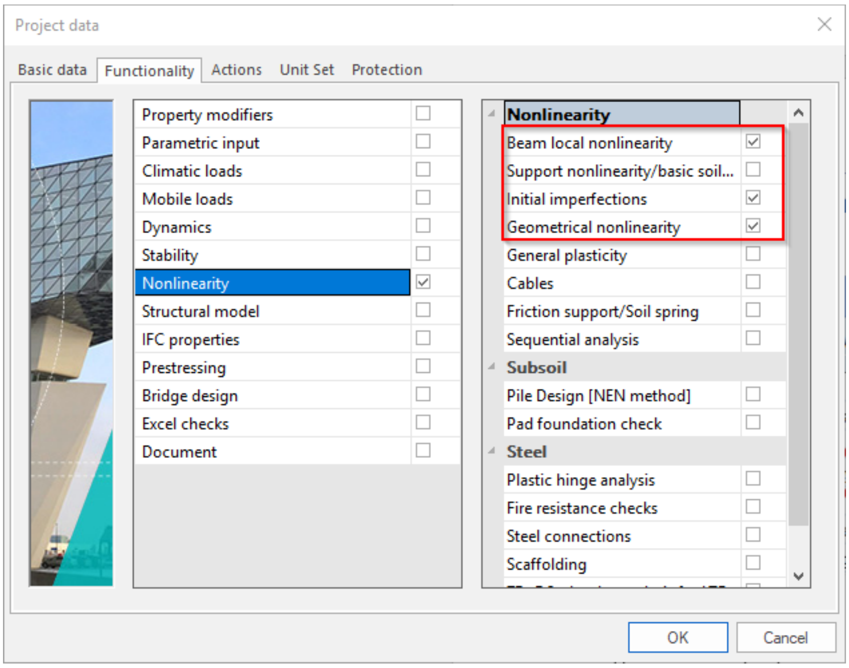


Unity check for ULS
→ Flexural buckling check is decisive



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

1) Functionalities

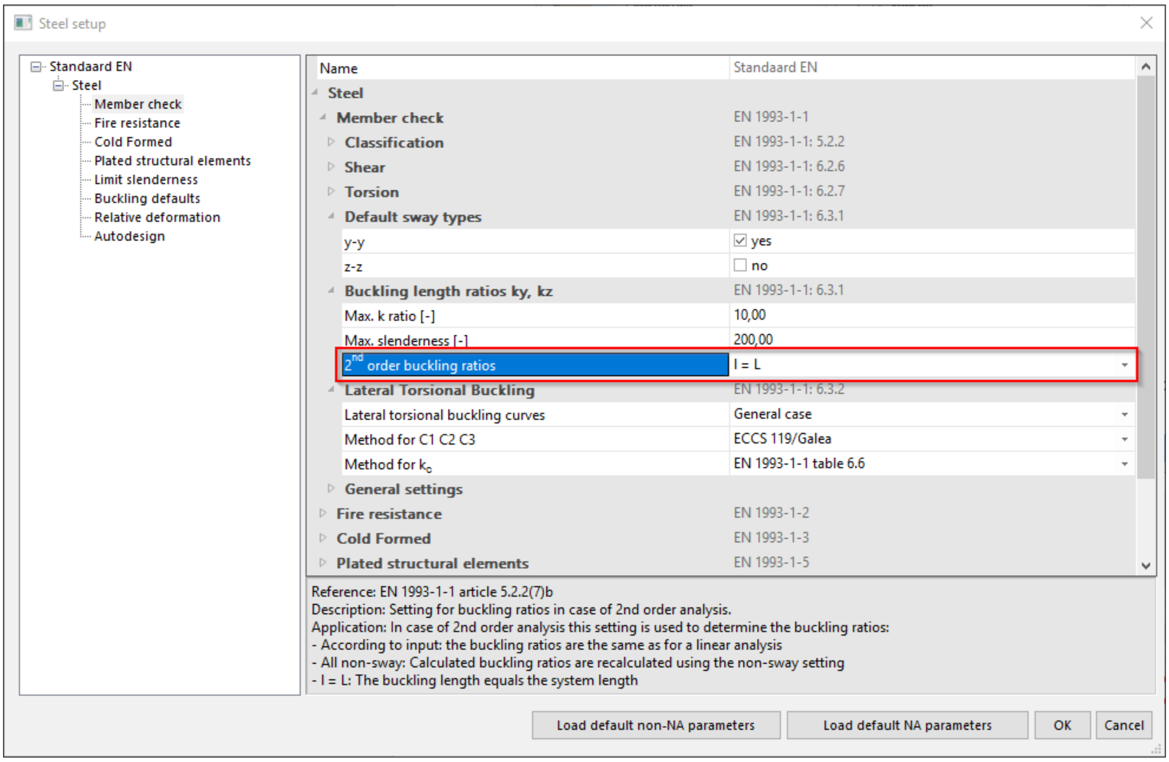


2) Global imperfection

See previous chapter.

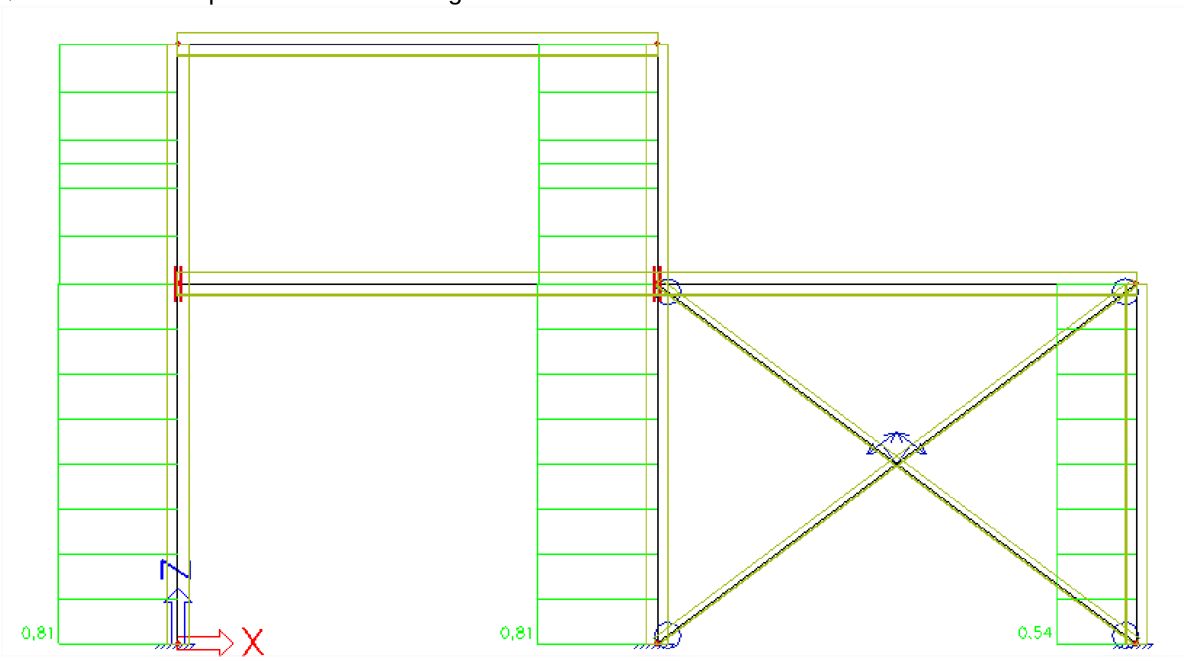
3) Steel setup

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.



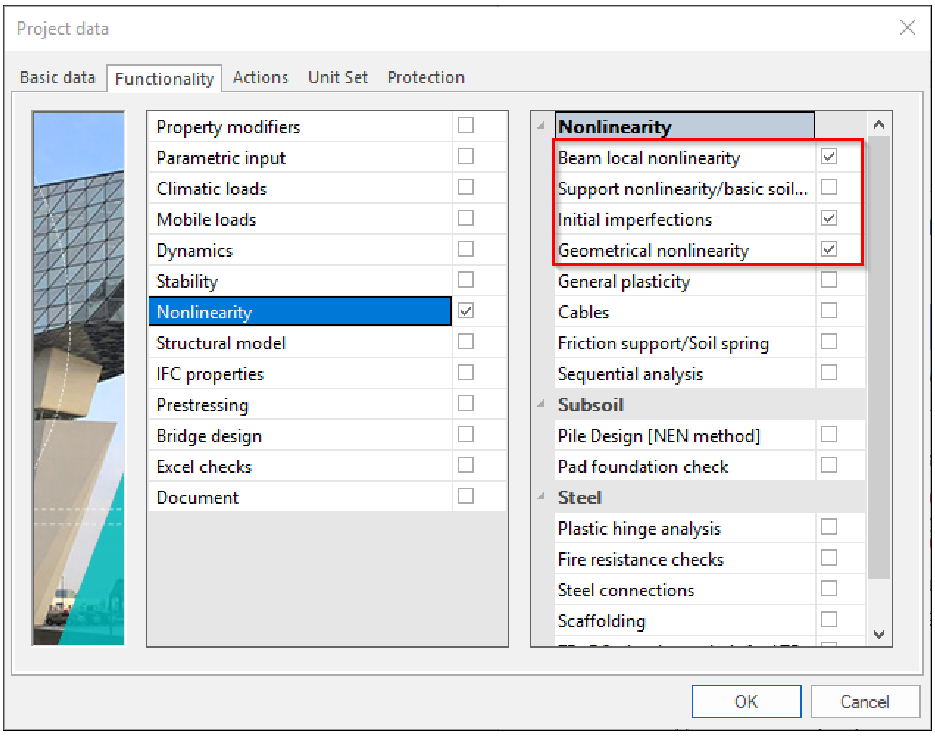
Unity check for ULS

→ Combined compression and bending check is decisive



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

1) Functionalities

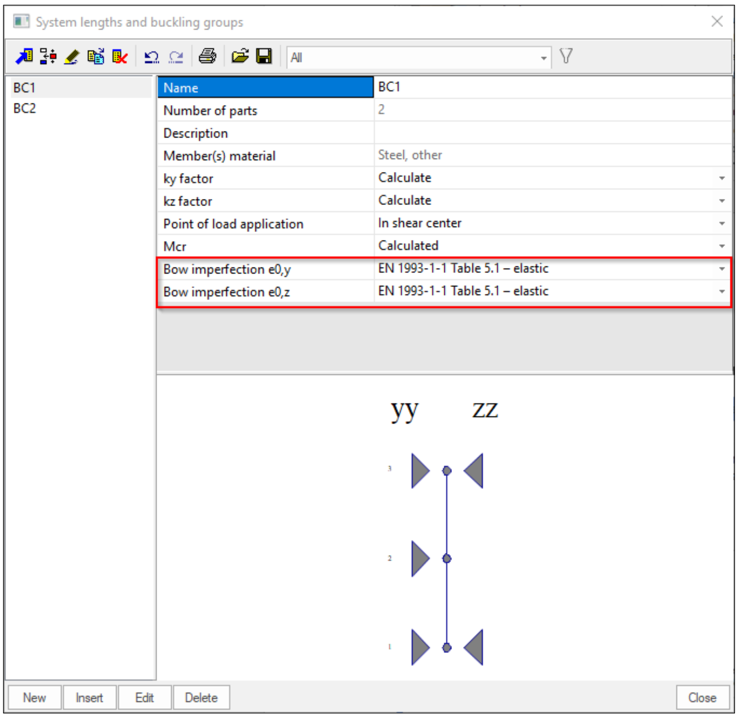


2) Global imperfection

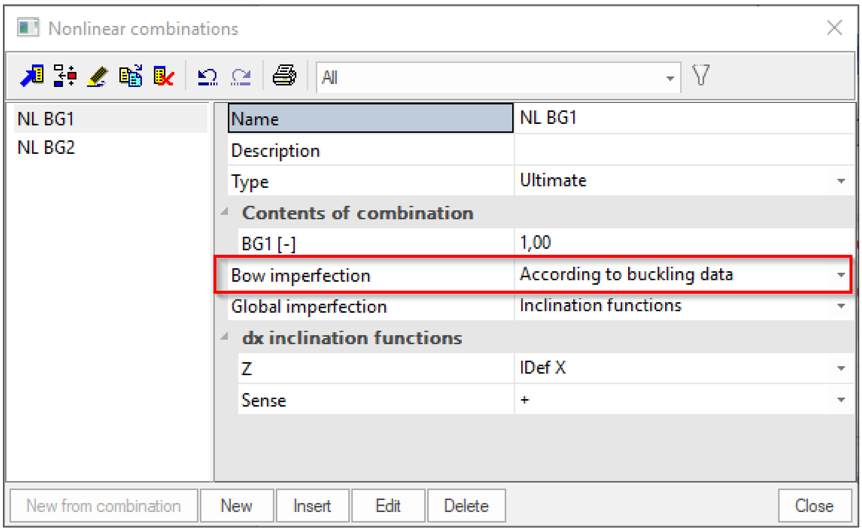
See previous chapters.

3) Bow imperfection

The bow imperfection e0 is inserted in the ‘System lengths and buckling groups’.

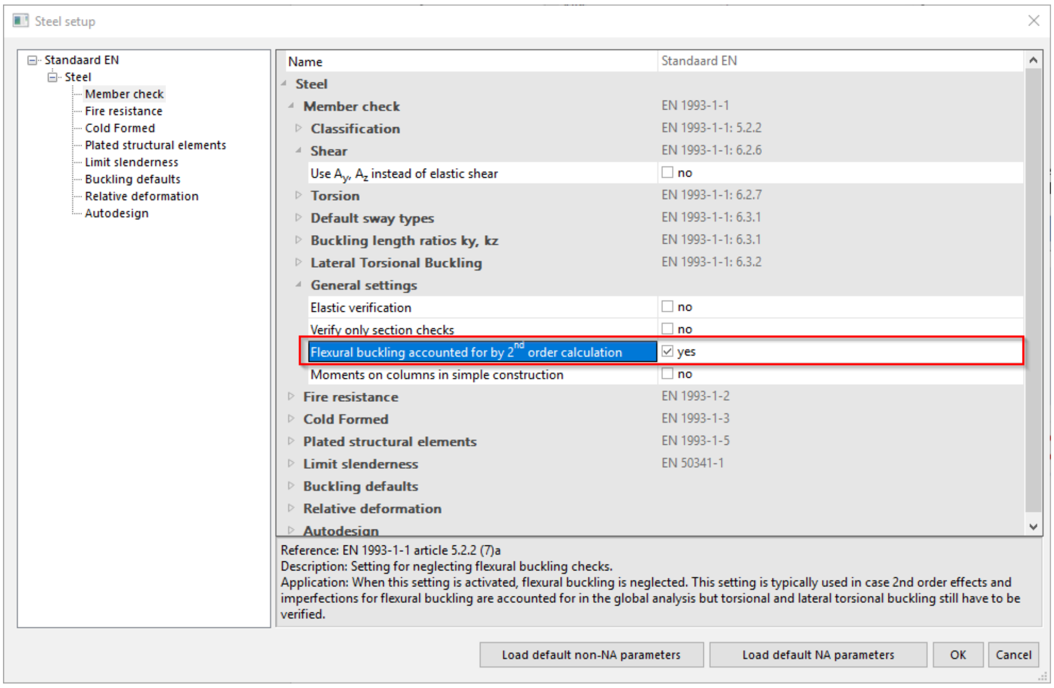


And in the nonlinear combination, a reference is made to this inserted buckling data:



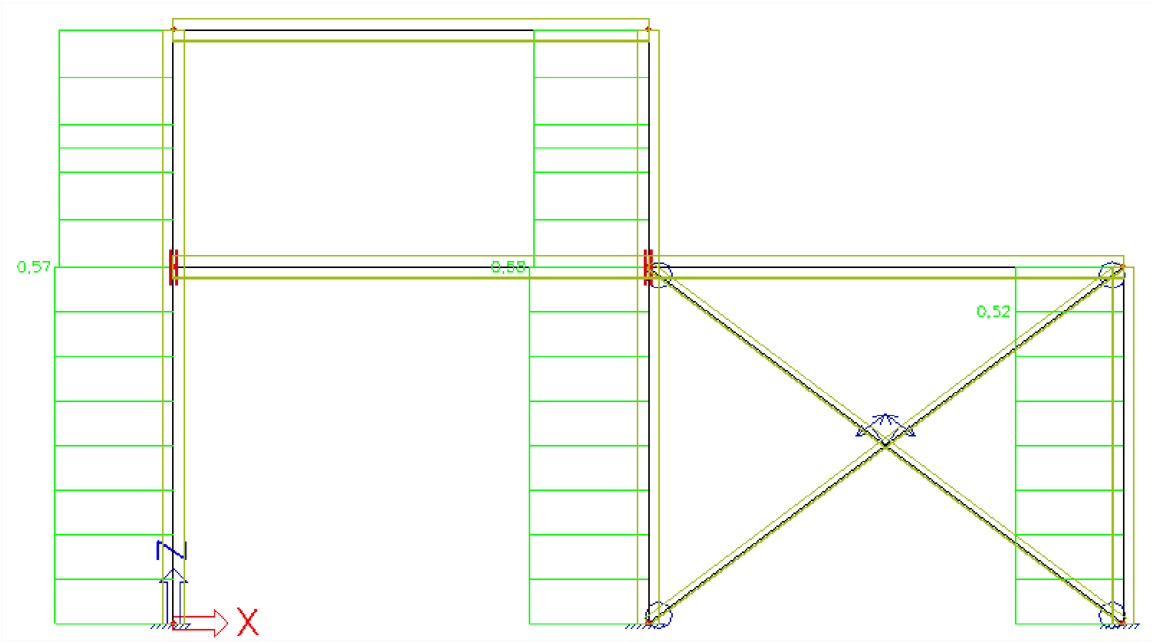
4) Steel setup

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.



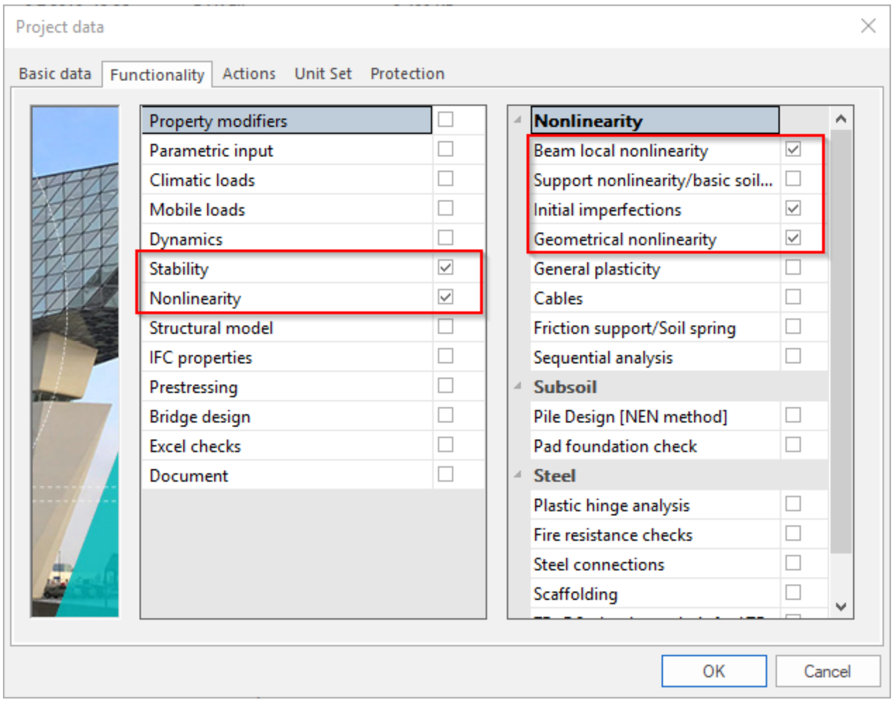
Unity check for ULS

→ 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

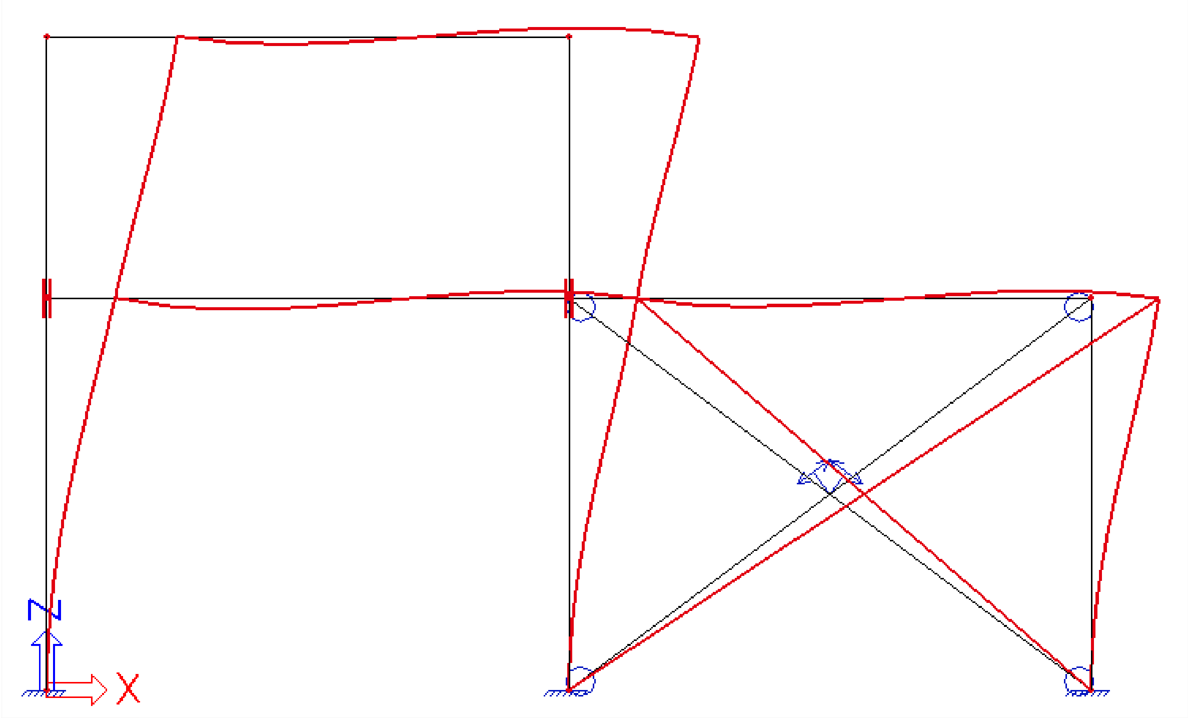
1) Functionalities



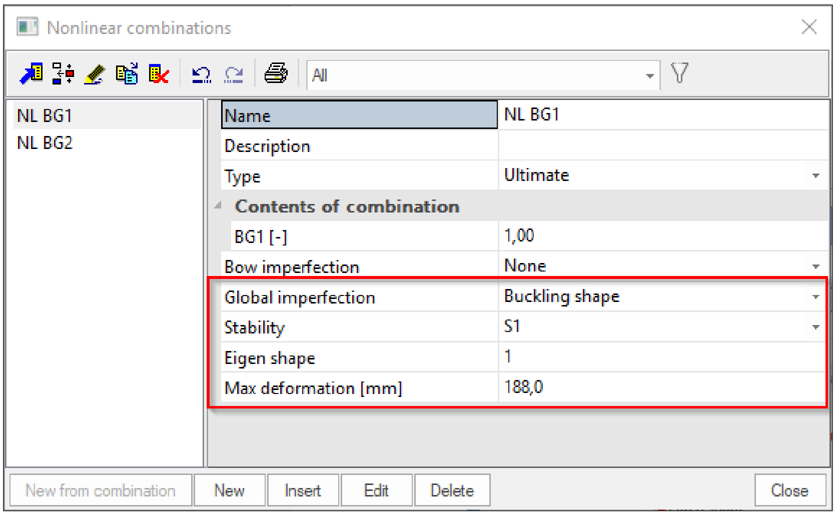
2) Stability calculation

See previous chapters.

The first buckling mode looks like this:

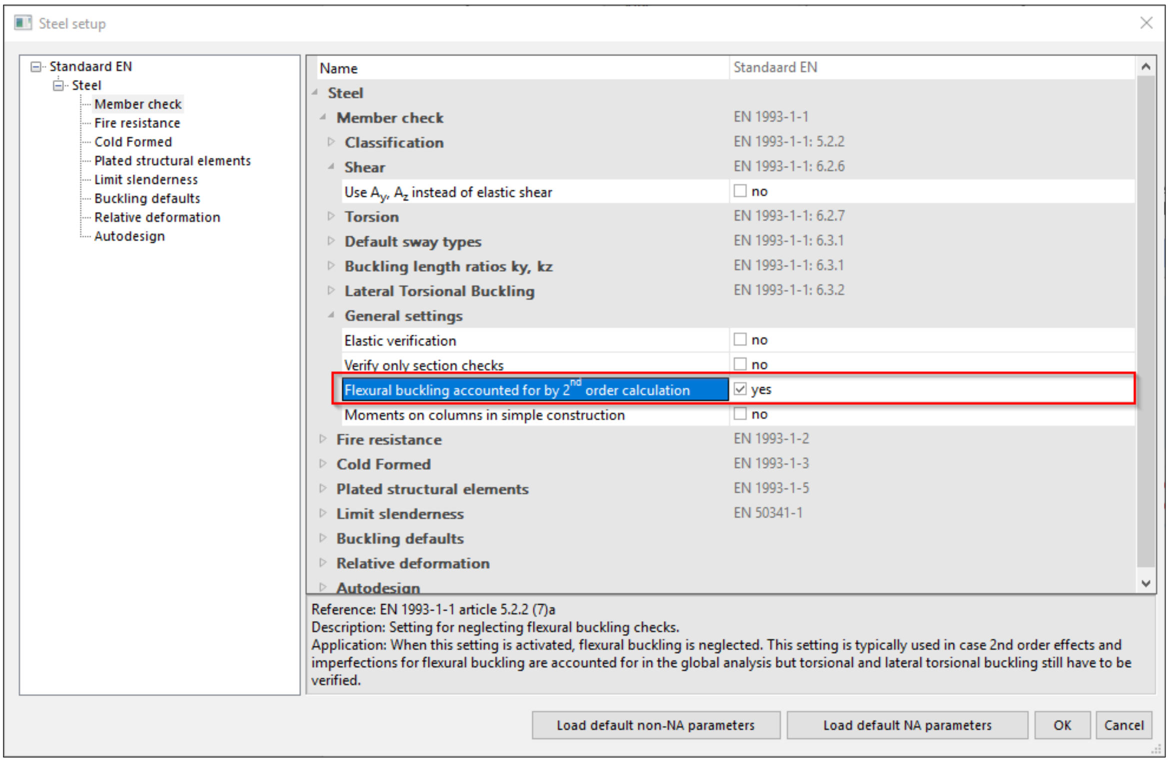


Calculation of η_{init} → To be filled in as ‘max. deformation’ (see ‘Nonlinear combinations’ window)



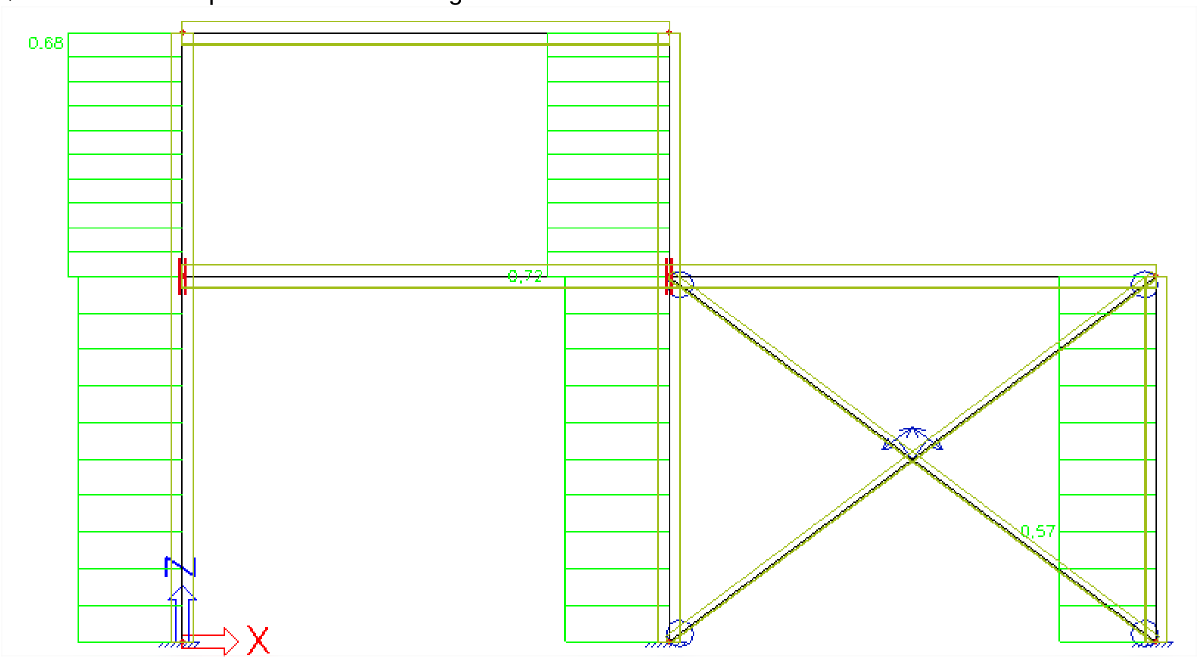
3) Steel setup

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.

Horizontal deformation
The maximum displacement in the X direction is 26,1mm on a height of 6.9m.
And in the Y-direction 28,0mm on a height of 8.1m.

Limit for Limit for horizontal deflection δ is h/150

⇨ 6900/150 = 46 mm → 26,1mm < 46 mm → OK

⇨ 8100/150 = 54 mm → 28,0mm < 54mm → OK

Vertical deformation
The maximum displacement in the Z direction is 59,0 mm

Limit for Limit for vertical deflection δ is L/200

⇨ 30000/200 = 150 mm → 59,0 mm < 150 mm → OK

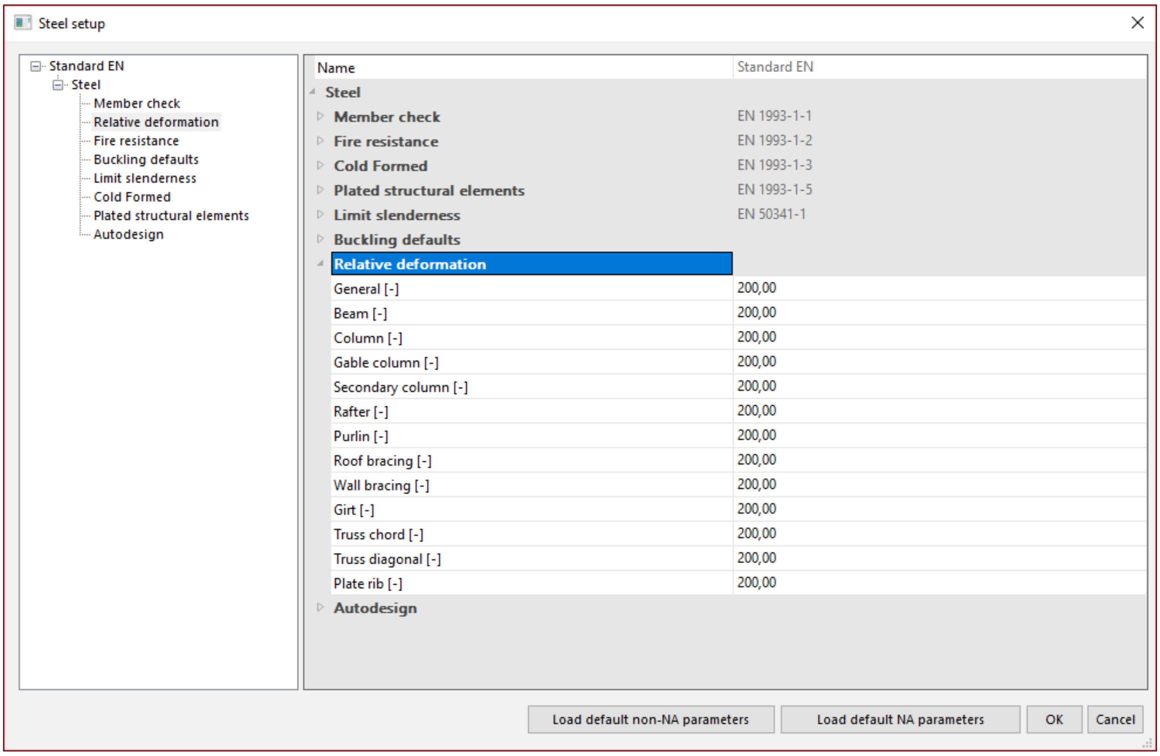
Displacement of nodes

Linear calculation
Combination: CO2
Extreme: Global
Selection: All

Name	Case	U_x [mm]	U_y [mm]	U_z [mm]	Φ_x [mrad]	Φ_y [mrad]	Φ_z [mrad]	U_{total} [mm]
N113	CO2/1	**-26.1**	-0.5	-0.1	-1.4	0.3	0.0	26.1
N114	CO2/2	**26.1**	-0.5	-0.1	-1.4	-0.3	0.0	26.1
N70	CO2/3	0.0	**-28.0**	0.0	1.5	0.0	1.3	28.0
N60	CO2/4	0.0	**28.0**	0.0	-1.5	0.0	1.3	28.0
N109	CO2/5	13.1	0.1	**-59.0**	-0.1	-0.3	0.0	**60.4**
N82	CO2/6	9.9	-9.3	**14.5**	-26.8	-0.2	1.9	19.8
N80	CO2/6	9.6	-10.3	0.0	**-29.7**	0.0	1.3	14.1
N179	CO2/7	-9.6	10.3	0.0	**29.7**	0.0	1.3	14.1
N116	CO2/8	-9.6	-0.1	-21.5	-0.4	**-5.9**	0.0	23.6
N112	CO2/5	9.6	-0.1	-21.5	-0.4	**5.9**	0.0	23.6
N95	CO2/7	-9.6	-9.5	0.0	-27.7	0.0	**-4.3**	13.6
N87	CO2/6	9.6	-9.5	0.0	-27.7	0.0	**4.3**	13.6

Relative deformation

For each beam type, limiting values for the relative deflections are set, using the menu “Steel -> Beams -> Steel Setup -> Relative deformation”:



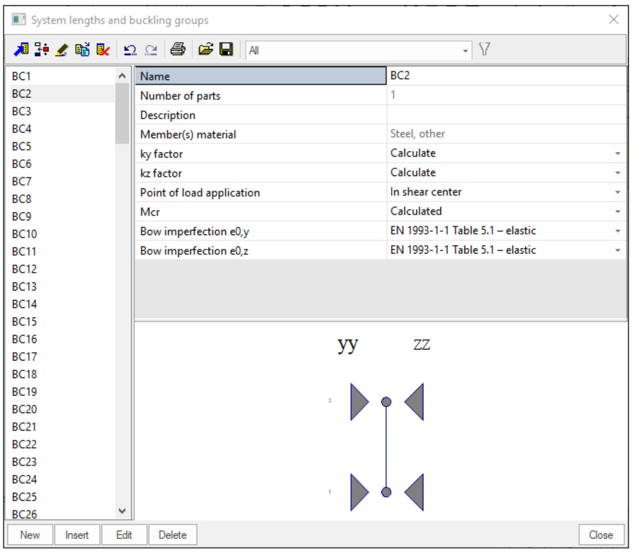
With the option “Steel -> Beams -> SLS Checks - Relative deformation”, the relative deformations can be checked. The relative deformations are given as absolute value, relative value related to the span, or as unity check related to the limit for the relative value to the span.

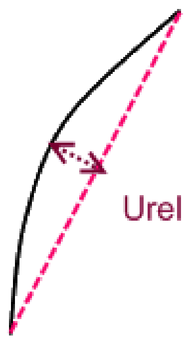
The span is defined in the menu “System lengths and buckling groups”.

Two options for this span are possible and explained below:

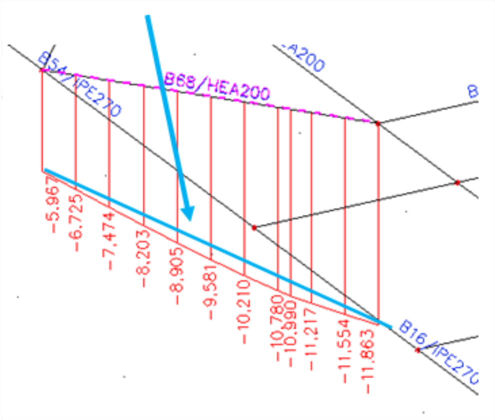
2 nodes supported

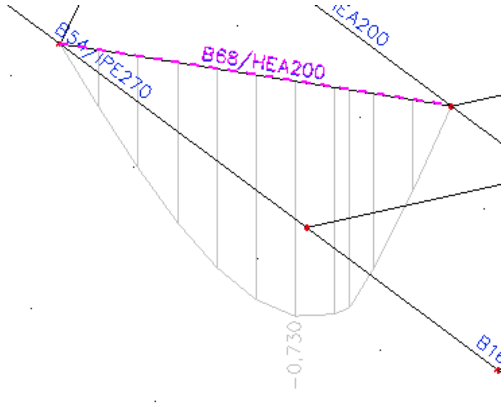
When two nodes are supported in this menu as shown below, the deformation is taken as the maximum deflection of the beam in comparison with a line connecting the two end nodes:





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):

$$uz_{blue\ line;0,979m} = 5,967mm + \frac{0,979m}{1,632m} * (11,863mm - 5,967mm) = 9,504mm$$

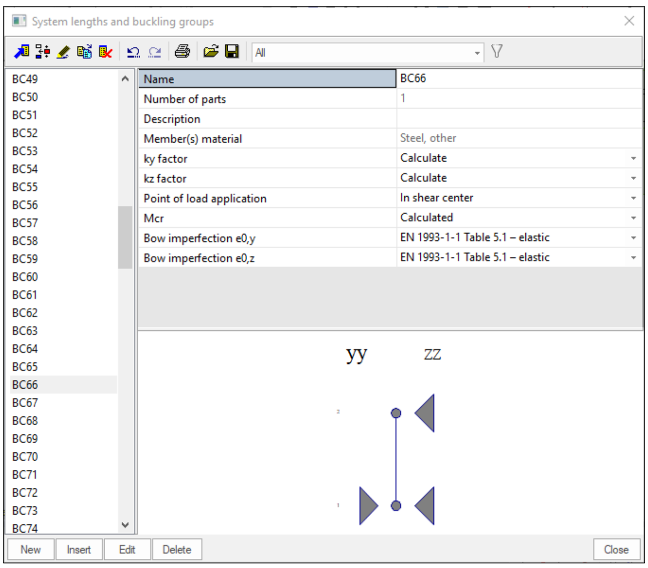
And $u_{z,\ relative}$ = 10,2mm – 9,5mm = 0,7mm

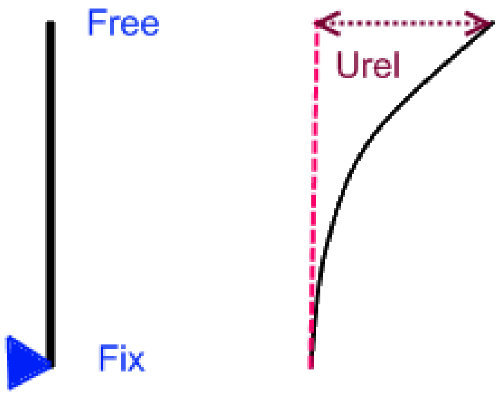
And suppose the length of this beam = 1632mm:

$$\Rightarrow \quad rel\ uz = \frac{0,73mm}{1632mm} = 1/2236$$

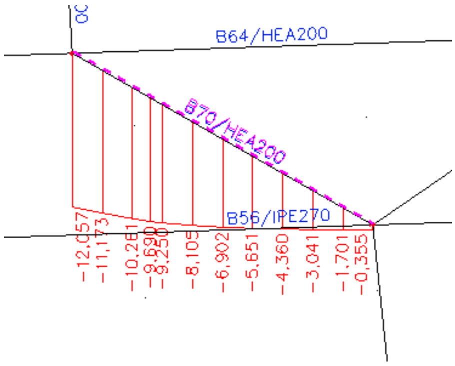
<u>1 node supported and the other free</u>

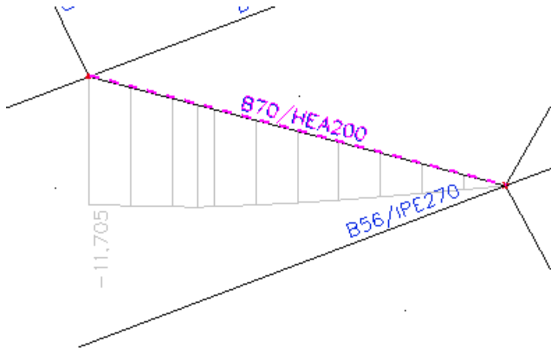
This is the case if one end node is free:





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 $u_{z,\,\text{relative}}$ = 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

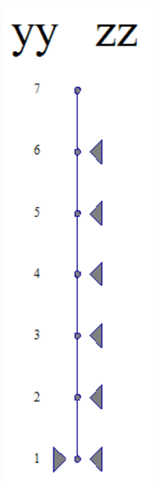
Consider beam B10.

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	CO2/1	**-0.2**	**1/3095**	-28.4	1/531	**0.06**	0.38
B10	15.075	CO2/2	**0.2**	**1/3161**	10.0	1/1505	**0.06**	0.13
B10	13.751	CO2/3	0.0	1/10000	**-59.0**	**1/255**	0.00	**0.78**
B10	12.029	CO2/4	0.0	1/10000	**13.8**	**1/1092**	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 = 59,0 mm (and 0 mm at the beginning of the beam)

uz_{relative} = 59,0 mm – 0.0 mm = 59,0 mm

uz_{relative} / Length = 59,0 mm / 15075 mm = 1/255

Check = $\frac{1/255}{1/200} = 0{,}78$

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.

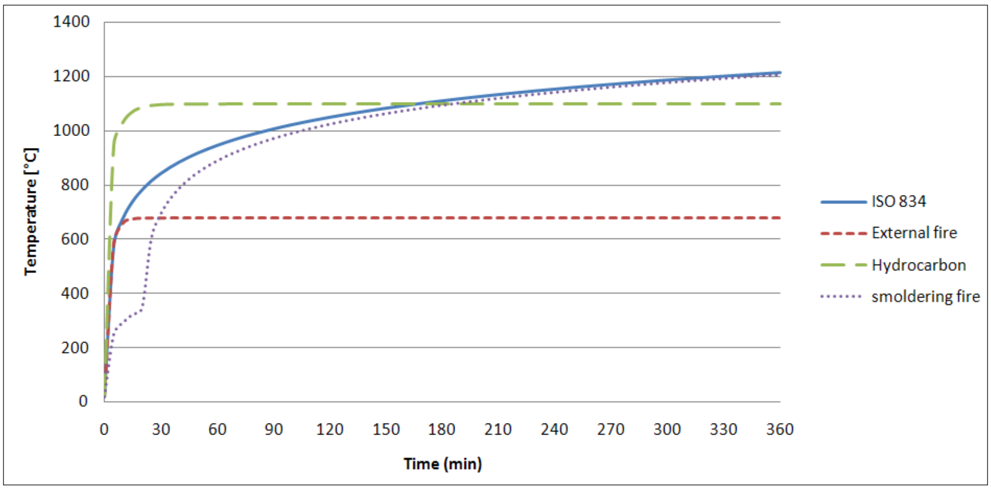
For a selected temperature curve, the temperature in the material after a required period is calculated. And with this material temperature, the material characteristics are adapted.

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]

$\alpha_c = 25\ \mathrm{W/m^2K}$

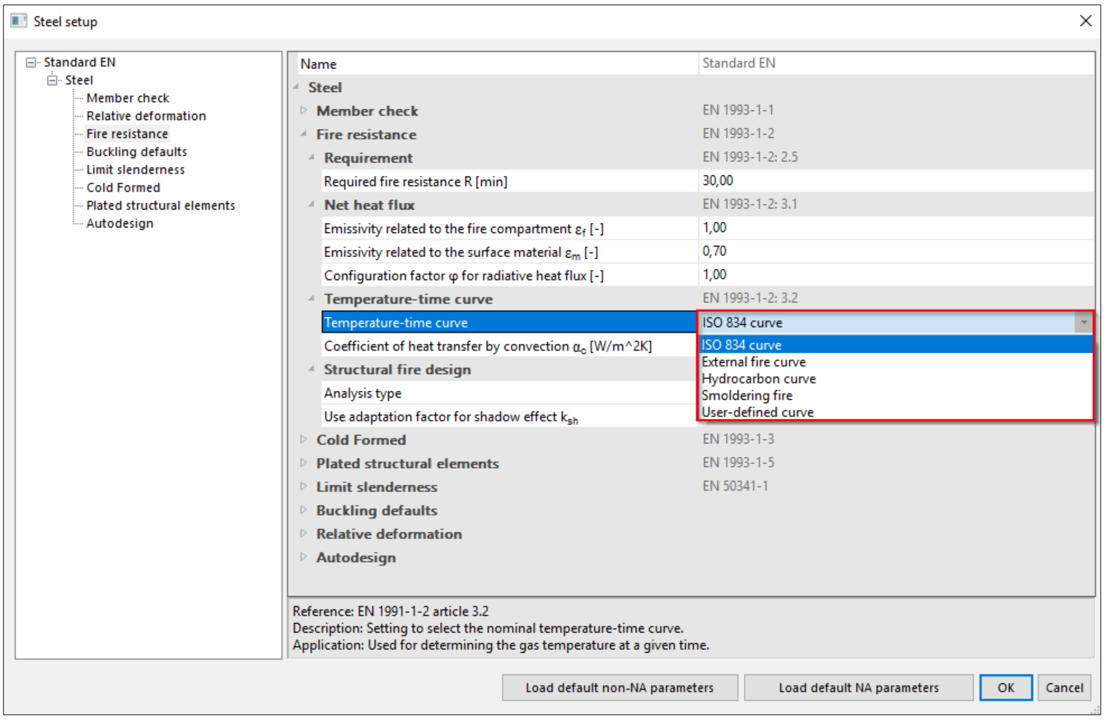
Hydrocarbon curve : $\Theta_g = 1080\,(1 - 0{,}325\, e^{-0{,}167\,t} - 0{,}675\, e^{-2{,}5\,t}) + 20$ [°C]

$\alpha_c = 50\ \mathrm{W/m^2K}$

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:

$$\dot{h}_{net} = \dot{h}_{net,r} + \dot{h}_{net,c}$$

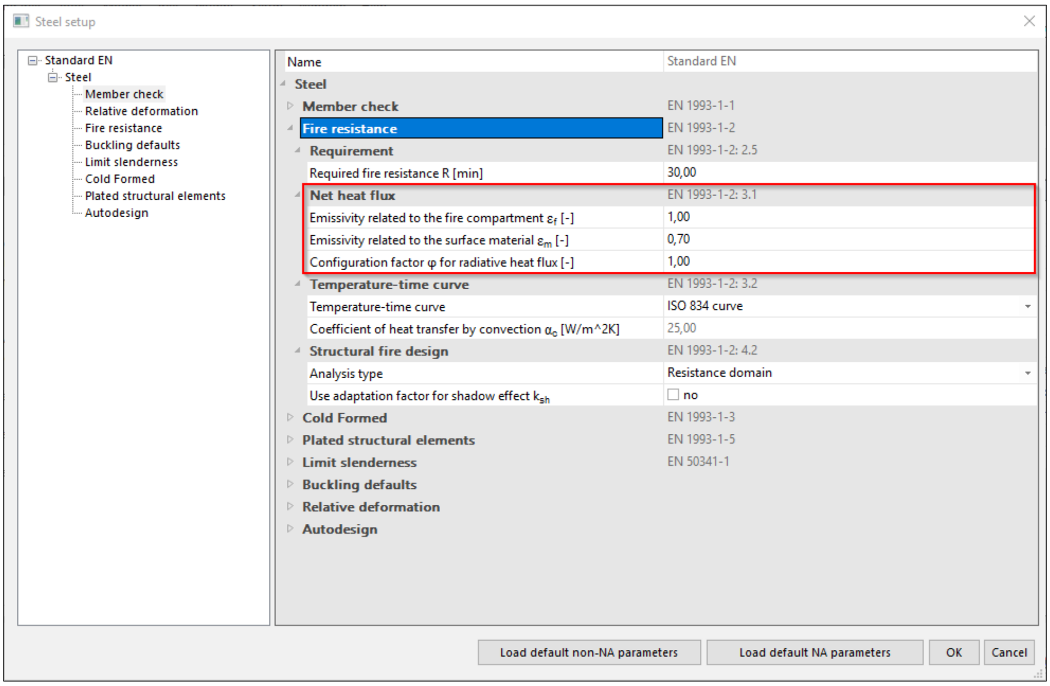
$\dot{h}_{net,c}$ = heat transfer by convection
$= \alpha_c(\Theta_g - \Theta_m)$ [W/m²]

$\dot{h}_{net,r}$ = 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

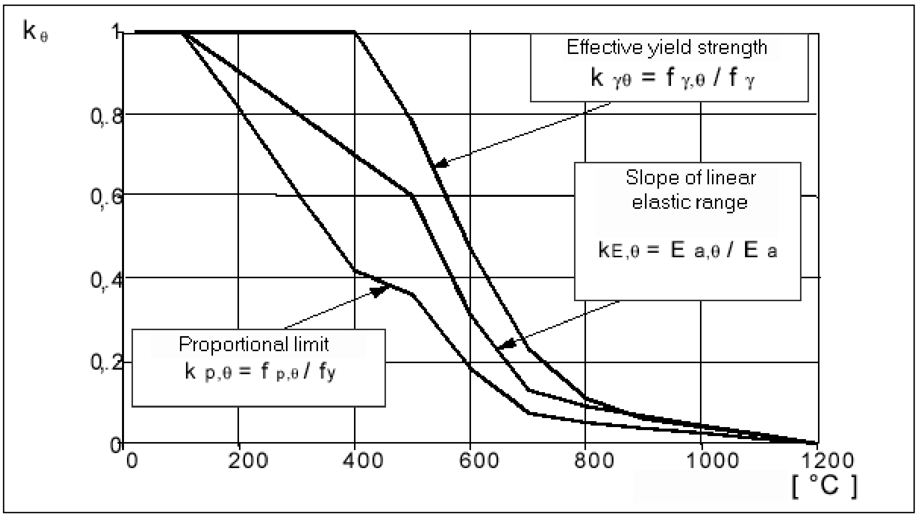
Most of the steel properties will change by a different temperature, so once the steel temperature is known, the steel properties can be calculated. In below the properties for carbon steel from EN 1993-1-2, art.3 are used. The properties for stainless steel can be found in EN 1993-1-2: 2005: Annex C.

Reduction factors

Effective yield strength, relative to yield strength at 20°C: $k_{y,\theta} = f_{y,\theta}/f_y$

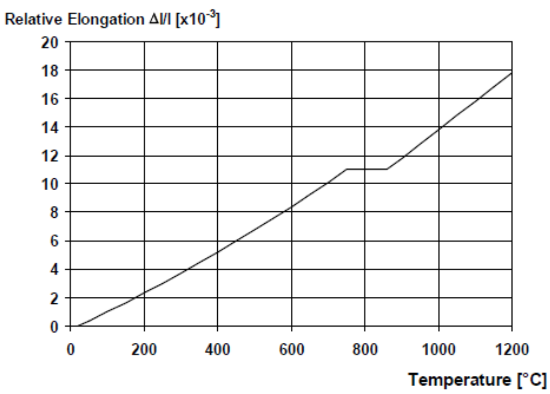
Proportional limit, relative to yield strength at 20°C: $k_{P,\theta} = f_{P,\theta}/f_y$

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 $20°C \leq \theta_a < 750°C$:

$$\frac{\Delta l}{l} = 1{,}2\ x\ 10^{-5}\theta_a + 0{,}4\ x\ 10^{-8}\theta_a^2 - 2{,}416\ x\ 10^{-4}$$

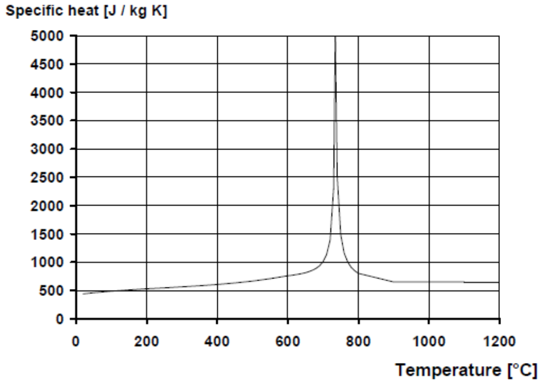
For $750°C \leq \theta_a < 860°C$:

$$\frac{\Delta l}{l} = 1{,}1\ x\ 10^{-2}$$

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



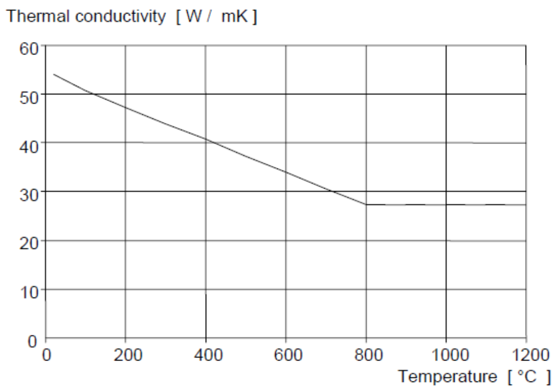
For $20°C \leq \theta_a < 600°C$: $c_a = 425 + 7{,}73\ x\ 10^{-1}\ \theta_a - 1{,}69\ x\ 10^{-3}\theta_a^2 + 2{,}22\ x\ 10^{-6}\theta_a^3\ \ J/kgK$

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 $20°C \leq \theta_a < 800°C$:

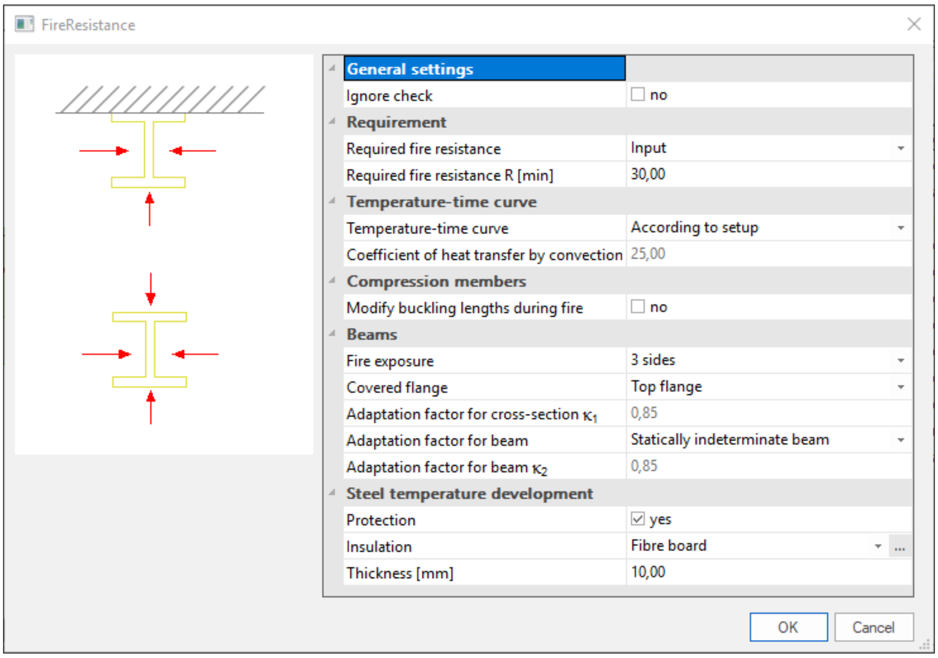
$$\lambda_a = 54 - 3{,}33\ x\ 10^{-2}\ \theta_a\ \ W/mK$$

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 α_c: 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.

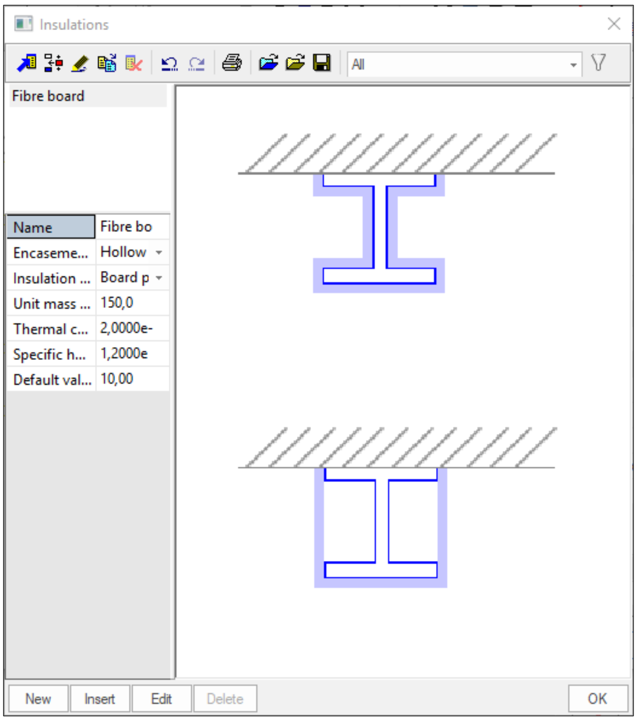
Adaption factor for cross-section κ_1: This parameters is the adaptation factor for non-uniform temperature distribution across a cross-section. This factor κ_1 is used for the check on the design moment resistance $M_{fi,\theta,Rd}$.

Adaption factor for beam: At the supports of a statically indeterminate beam or all other cases.

Adaption factor for beam κ_2: 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.



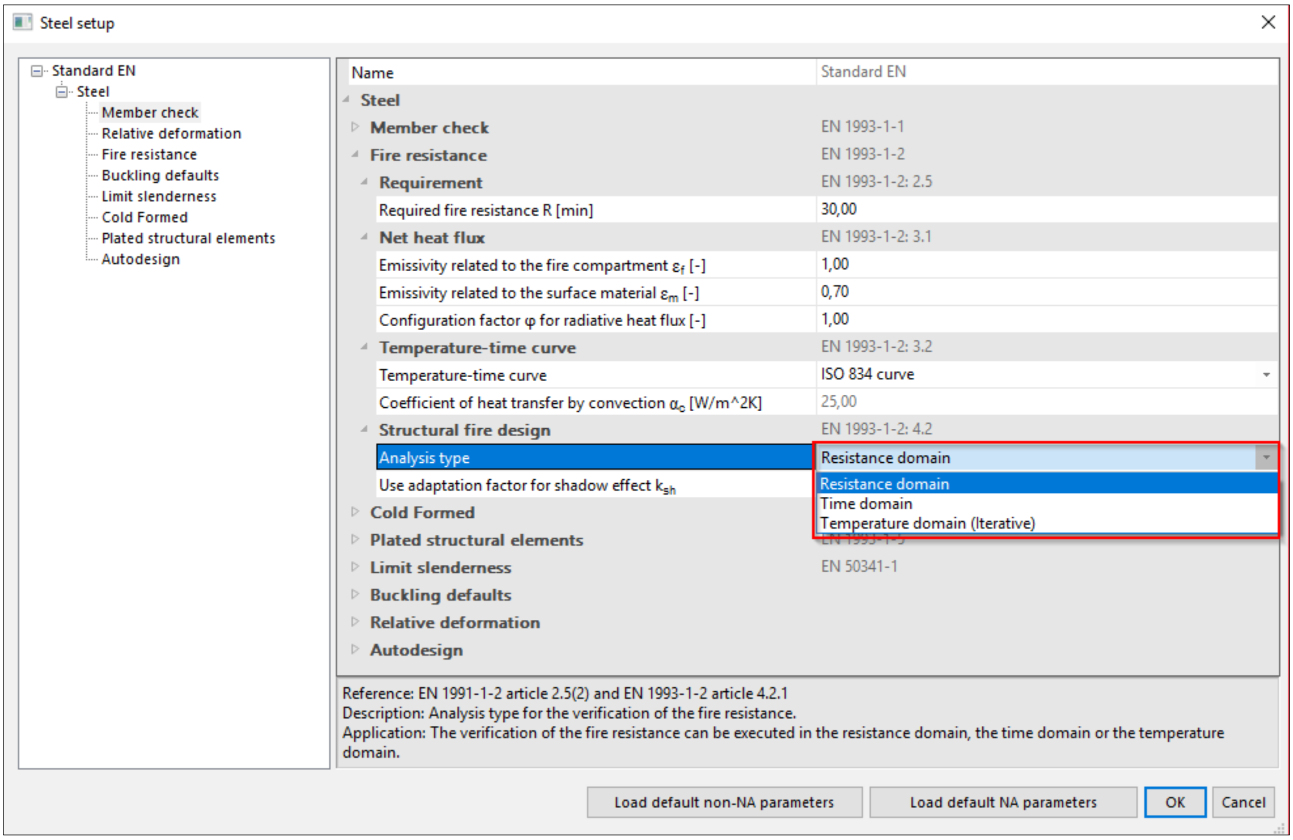
Thickness [mm]: Thickness of the insulation.

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.

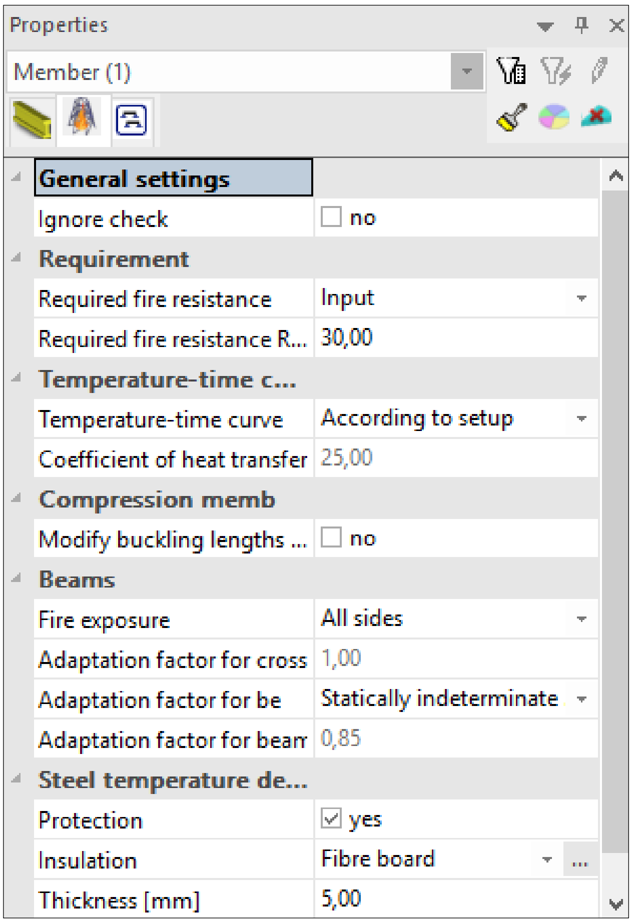
Example in SCIA Engineer

This principle is explained with an example in SCIA Engineer.

Example: Industrial hall.esa

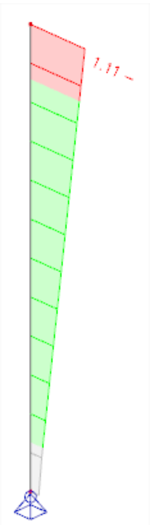
Consider member B28:

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 - EC-EN 1993 Steel Check Fire” for this column and for combination “CO3 - Fire”, resulting in a unity check of 1,11:



When looking at the detailed output, this calculation is given by SCIA Engineer:

First the partial safety factors are given:

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{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 f_y	235.0	MPa
Ultimate strength f_u	360.0	MPa
Fabrication	Rolled	

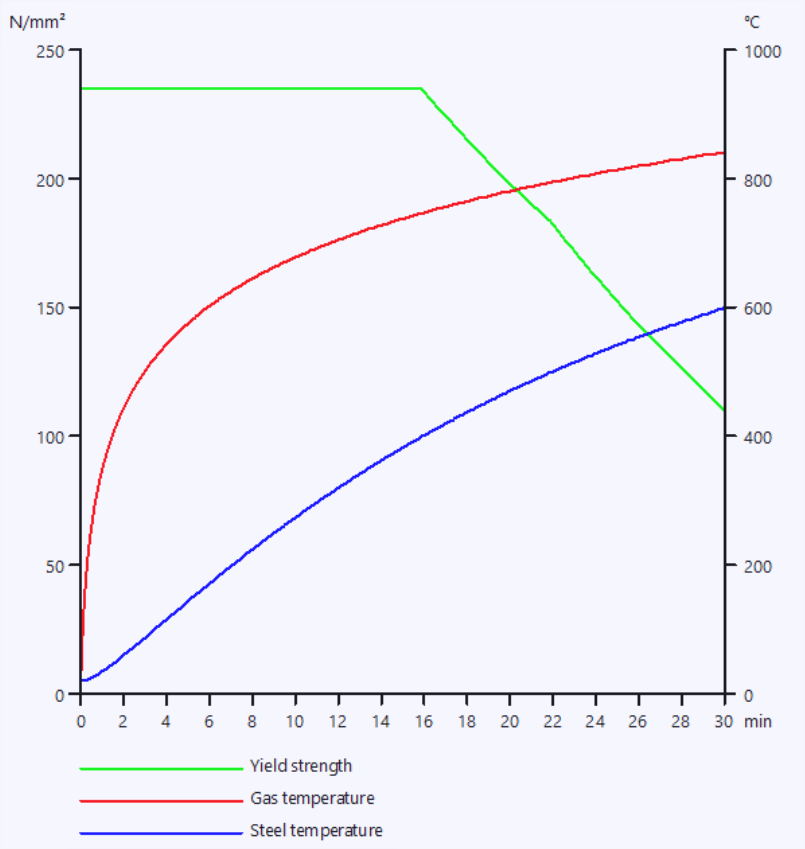
And the fire resistance properties as inputted in SCIA Engineer. Here is also indicated that the fire resistance check has been executed after 30 minutes of fire.

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 α_c	25.00	W/m²K
Emissivity related to fire compartment ε_f	1.00	
Emissivity related to surface material ε_m	0.70	
Configuration factor for radiation heat flux φ	1.00	
Required fire resistance R	30.00	min
Gas temperature θ_g	841.80	°C
Material temperature $\theta_{a,t}$	600.06	°C
Beam exposure	All sides	
Adaptation factor for cross-section κ_1	1.00	
Adaptation factor for beam κ_2	0.85	
Reduction factor for the 0.2% proof strength $k_{0.2p,\theta}$	0.47	
Reduction factor for the E modulus $k_{E,\theta}$	0.31	

Insulation properties		
Name	Fibre board	
Encasement type	Hollow encasement	
Insulation type	Board	
Thickness d_p	5.00	mm
Unit mass ρ_p	150.0	kg/m³
Thermal conductivity λ_p	2.0000e-01	W/mK
Specific heat c_p	1.2000e+00	J/gK
Section factor for insulated steel members A_p/V	1.0830e-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::...

The critical check is on position 6.900 m

Internal forces	Calculated	Unit
$N_{fi,Ed}$	-60.55	kN
$V_{y,fi,Ed}$	-0.01	kN
$V_{z,fi,Ed}$	-34.98	kN
$T_{fi,Ed}$	0.00	kNm
$M_{y,fi,Ed}$	-241.35	kNm
$M_{z,fi,Ed}$	-0.05	kNm

Classification for cross-section design

Classification according to EN 1993-1-2 article 4.2.2

Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m ²]	σ_2 [kN/m ²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	108.90	17.00	56728.614	56823.237	1.0	0.4	1.0	6.4	7.6	8.5	11.7	1
3	SO	108.90	17.00	56687.602	56592.980	1.0	0.4	1.0	6.4	7.6	8.5	11.7	1
4	I	685.00	13.20	53002.386	-46543.485	-0.9		0.5	51.9	59.2	68.2	93.9	1
5	SO	108.90	17.00	-50269.713	-50364.335								
7	SO	108.90	17.00	-50228.701	-50134.079								

The cross-section is classified as Class 1

Compression check

According to EN 1993-1-2 article 4.2.3.2 and formula (4.5)

A	1.8800e+04	mm²
$N_{fi,t,Rd}$	2075.86	kN
Unity check	0.03	-

Bending moment check for M_y
According to EN 1993-1-2 article 4.2.3.3 and formula (4.10)

$W_{pl,y}$	5.1100e+06	mm^3
$M_{pl,y,Rd}$	1200.85	kNm
$M_{y,fi,\theta,Rd}$	564.24	kNm
$M_{y,fi,t,Rd}$	663.81	kNm
Unity check	0.36	-

Bending moment check for M_z
According to EN 1993-1-2 article 4.2.3.3 and formula (4.10)

$W_{pl,z}$	6.3100e+05	mm^3
$M_{pl,z,Rd}$	148.28	kNm
$M_{z,fi,\theta,Rd}$	69.67	kNm
$M_{z,fi,t,Rd}$	81.97	kNm
Unity check	0.00	-

Shear check for V_y
According to EN 1993-1-2 article 4.2.3.3 and formula (4.16)

η	1.20	
A_v	9.4086e+03	mm^2
$V_{pl,y,Rd}$	1276.54	kN
$V_{y,fi,t,Rd}$	599.80	kN
Unity check	0.00	-

Shear check for V_z
According to EN 1993-1-2 article 4.2.3.3 and formula (4.16)

η	1.20	
A_v	1.1389e+04	mm^2
$V_{pl,z,Rd}$	1545.22	kN
$V_{z,fi,t,Rd}$	726.05	kN
Unity check	0.05	-

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)

$M_{y,fi,t,Rd}$	663.81	kNm
α	2.00	
$M_{z,fi,t,Rd}$	81.97	kNm
β	1.00	

Unity check (4.9) = 0.13 + 0.00 = 0.13 -

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 according to EN 1993-1-2 article 4.2.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ1 [kN/m 2]	σ2 [kN/m 2]	ψ [-]	kσ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	108.90	17.00	3762.225	3762.225	1.0	0.4	1.0	6.4	7.6	8.5	11.9	1
3	SO	108.90	17.00	3762.225	3762.225	1.0	0.4	1.0	6.4	7.6	8.5	11.9	1
4	I	685.00	13.20	3762.225	3762.225	1.0		1.0	51.9	28.0	32.3	35.7	4
5	SO	108.90	17.00	3762.225	3762.225	1.0	0.4	1.0	6.4	7.6	8.5	11.9	1
7	SO	108.90	17.00	3762.225	3762.225	1.0	0.4	1.0	6.4	7.6	8.5	11.9	1

The cross-section is classified as Class 4

Effective section N-
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	bp [mm]	σ1 [kN/m 2]	σ2 [kN/m 2]	ψ [-]	kσ [-]	λp [-]	ρ [-]	be [mm]	be1 [mm]	be2 [mm]
1	SO	108.90	235000.000	235000.000	1.0	0.4	0.3	1.0	108.90		
3	SO	108.90	235000.000	235000.000	1.0	0.4	0.3	1.0	108.90		
4	I	685.00	235000.000	235000.000	1.0	4.0	0.9	0.8	569.22	284.61	284.61
5	SO	108.90	235000.000	235000.000	1.0	0.4	0.3	1.0	108.90		
7	SO	108.90	235000.000	235000.000	1.0	0.4	0.3	1.0	108.90		

Effective section My-
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	bp [mm]	σ1 [kN/m 2]	σ2 [kN/m 2]	ψ [-]	kσ [-]	λp [-]	ρ [-]	be [mm]	be1 [mm]	be2 [mm]
1	SO	108.90	235000.000	235000.000	1.0	0.4	0.3	1.0	108.90		
3	SO	108.90	235000.000	235000.000	1.0	0.4	0.3	1.0	108.90		
4	I	685.00	218716.033	-218716.033	-1.0	23.9	0.4	1.0	342.50	137.00	205.50
5	SO	108.90	-235000.000	-235000.000							
7	SO	108.90	-235000.000	-235000.000							

Effective section Mz-
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	b_p [mm]	σ_1 [kN/m ²]	σ_2 [kN/m ²]	ψ [-]	k_σ [-]	λ_p [-]	ρ [-]	b_e [mm]	b_{e1} [mm]	b_{e2} [mm]
1	SO	108.90	235000.000	41856.604	0.2	0.5	0.3	1.0	108.90		
3	SO	108.90	-41856.604	-235000.000							
4	I	685.00	0.000	0.000							
5	SO	108.90	-41856.604	-235000.000							
7	SO	108.90	235000.000	41856.604	0.2	0.5	0.3	1.0	108.90		

Effective properties						
Effective area	A_{eff}	1.7222e+04	mm²			
Effective second moment of area	$I_{eff,y}$	1.6608e+09	mm⁴	$I_{eff,z}$	5.2895e+07	mm⁴
Effective section modulus	$W_{eff,y}$	4.4111e+06	mm³	$W_{eff,z}$	3.9920e+05	mm³
Shift of the centroid	$e_{N,y}$	0.00	mm	$e_{N,z}$	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	3.35	0.99	
Buckling length L_{cr}	23.123	6.811	m
Critical Euler load N_{cr}	6438.53	2362.99	kN
Slenderness λ	77.79	128.41	
Relative slenderness λ_{rel}	0.79	1.31	
Relative slenderness $\lambda_{rel,\theta}$	0.98	1.61	
Imperfection α	0.65	0.65	
Reduction factor χ_{fi}	0.47	0.25	
Buckling resistance $N_{b,fi,t,Rd}$	887.42	476.10	kN

Flexural Buckling verification		
Cross-section effective area A_{eff}	1.7222e+04	mm²
Buckling resistance $N_{b,fi,t,Rd}$	476.10	kN
Unity check	0.13	-

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		
Effective section modulus $W_{eff,y}$	4.4111e+06	mm³
Elastic critical moment M_{cr}	1785.02	kNm
Relative slenderness $\lambda_{rel,LT}$	0.76	
Relative slenderness $\lambda_{rel,LT,\theta}$	0.94	
Imperfection α_{LT}	0.65	
Reduction factor $\chi_{LT,fi}$	0.48	
Design buckling resistance $M_{b,fi,t,Rd}$	236.01	kNm
Unity check	**1.02**	-

Mcr parameters		
LTB length L	6.900	m
Influence of load position	no influence	
Correction factor k	1.00	
Correction factor k_w	1.00	
LTB moment factor C_1	1.77	
LTB moment factor C_2	0.00	
LTB moment factor C_3	1.00	
Shear center distance d_z	0.00	mm
Distance of load application z_g	0.00	mm
Mono-symmetry constant β_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 A_{eff}	1.7222e+04	mm^2
Effective section modulus $W_{eff,y}$	4.4111e+06	mm^3
Effective section modulus $W_{eff,z}$	3.9920e+05	mm^3
Design compression force $N_{fi,Ed}$	60.55	kN
Design bending moment $M_{y,fi,Ed}$	-241.35	kNm
Design bending moment $M_{z,fi,Ed}$	-0.05	kNm
Reduction factor $\chi_{min,fi}$	0.25	
Reduction factor $\chi_{z,fi}$	0.25	
Reduction factor $\chi_{LT,fi}$	0.48	
Equivalent moment factor $\beta_{M,y}$	1.80	
Factor μ_y	-0.28	
Interaction factor k_y	1.02	
Equivalent moment factor $\beta_{M,z}$	1.80	
Factor μ_z	-0.37	
Interaction factor k_z	1.05	
Equivalent moment factor $\beta_{M,LT}$	1.80	
Factor μ_{LT}	0.29	
Interaction factor k_{LT}	0.96	

Unity check (4.21c) = 0.13 + 0.51 + 0.00 = 0.63 -
Unity check (4.21d) = 0.13 + 0.99 + 0.00 = **1.11** -

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 h_w	719.00	mm
Web thickness t	13.20	mm
Yield strength f_{yw}	235.0	MPa
Flange width b_f	265.00	mm
Flange thickness t_f	17.00	mm
Yield strength f_{yf}	235.0	MPa
Material coefficient ε	0.85	
Shear correction factor η	1.20	

Shear Buckling verification		
Web slenderness h_w/t	54.47	
Web slenderness limit	51.00	
Plate slenderness $\lambda_{w,\theta}$	0.78	
Reduction factor $\chi_{w,fi}$	1.07	
Contribution of the web $V_{bw,fi,t,Rd}$	646.91	kN
Capacity of the flange $M_{f,fi,t,Rd}$	343.83	kNm
Flange factor c	1.849	m
Contribution of the flange $V_{bf,fi,t,Rd}$	2.32	kN
Maximum resistance $V_{b,fi,t,Rd,limit}$	726.05	kN
Resistance $V_{b,fi,t,Rd}$	649.23	kN
Plastic resistance $M_{pl,fi,t,Rd}$	564.24	kNm
Shear ratio $\eta_{3,bar}$	0.05	

Unity check (5.10) = 0.05 -
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 **Bending and axial compression check** will result in a unity check of **1,11**.

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

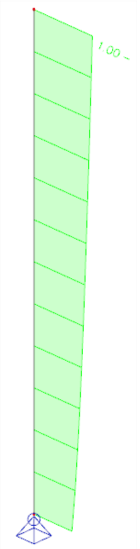
This principle is explained with an example in SCIA Engineer.

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 1,00:



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 Bending and axial compression check will result in the highest unity check = 0.45 = 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 A_{eff}	1.7222e+04	mm^2
Effective section modulus $W_{eff,y}$	4.4111e+06	mm^3
Effective section modulus $W_{eff,z}$	3.9920e+05	mm^3
Design compression force $N_{fi,Ed}$	60.55	kN
Design bending moment $M_{y,fi,Ed}$	-241.35	kNm
Design bending moment $M_{z,fi,Ed}$	-0.05	kNm
Reduction factor $\chi_{min,fi}$	0.33	
Reduction factor $\chi_{z,fi}$	0.33	
Reduction factor $\chi_{LT,fi}$	0.57	
Equivalent moment factor $\beta_{M,y}$	1.80	
Factor μ_y	-0.03	
Interaction factor k_y	1.00	
Equivalent moment factor $\beta_{M,z}$	1.80	
Factor μ_z	-0.11	
Interaction factor k_z	1.00	
Equivalent moment factor $\beta_{M,LT}$	1.80	
Factor μ_{LT}	0.20	
Interaction factor k_{LT}	0.99	

Unity check (4.21c) = 0.04 + 0.23 + 0.00 = 0.28 -
Unity check (4.21d) = 0.04 + 0.40 + 0.00 = 0.45 -

This value is used in the simple formula for the critical steel temperature:

$$\theta_{a,cr} = 39{,}19\ ln\left[\frac{1}{0{,}9674\ \mu_0^{3{,}833}} - 1\right] + 482$$
$$\theta_{a,cr} = 39{,}19\ ln\left[\frac{1}{0{,}9674\ (0.45)^{3.833}} - 1\right] + 482 = 602{,}48\ °C$$

And the steel temperature after 30 minutes is 600,06°C.

The unity check is: $\frac{600{,}06°C}{602{,}48°C} = 0{,}996 = 1{,}00$

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 α_c	25.00	W/m^2K
Emissivity related to fire compartment ε_f	1.00	
Emissivity related to surface material ε_m	0.70	
Configuration factor for radiation heat flux φ	1.00	
Required fire resistance R	30.00	min
Gas temperature θ_g	841.80	°C
Material temperature $\theta_{a,t}$	600.06	°C
Degree of utilization μ_0	0.45	
Critical material temperature $\theta_{a,cr}$	602.48	°C
Fire resistance t_{cr}	30.22	min
Beam exposure	All sides	
Adaptation factor for cross-section κ_1	1.00	
Adaptation factor for beam κ_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	1.00	-

And also the **Fire resistance time** is given in this table:
This member can resist fire for 30,22 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

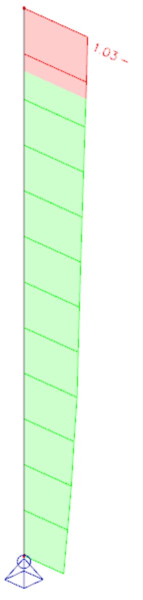
This principle is explained with an example in SCIA Engineer.

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 Checks -> EC-EN 1993 Steel Check Fire” 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 the critical material temperature $\theta_{a,cr}$ = 584.65 °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 $\theta_{a,cr}$ = 584,65°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 A_{eff}	1.7222e+04	mm^2
Effective section modulus $W_{eff,y}$	4.4111e+06	mm^3
Effective section modulus $W_{eff,z}$	3.9920e+05	mm^3
Design compression force $N_{fi,Ed}$	60.55	kN
Design bending moment $M_{y,fi,Ed}$	-241.35	kNm
Design bending moment $M_{z,fi,Ed}$	-0.05	kNm
Reduction factor $\chi_{min,fi}$	0.26	
Reduction factor $\chi_{z,fi}$	0.26	
Reduction factor $\chi_{LT,fi}$	0.49	
Equivalent moment factor $\beta_{M,y}$	1.80	
Factor μ_y	-0.26	
Interaction factor k_y	1.02	
Equivalent moment factor $\beta_{M,z}$	1.80	
Factor μ_z	-0.34	
Interaction factor k_z	1.04	
Equivalent moment factor $\beta_{M,LT}$	1.80	
Factor μ_{LT}	0.28	
Interaction factor k_{LT}	0.97	

Unity check (4.21c) = 0.11 + 0.46 + 0.00 = 0.57 -
Unity check (4.21d) = 0.11 + 0.88 + 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: $\frac{600,06°C}{584,65°C} = 1.03$

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 α_c	25.00	W/m^2K
Emissivity related to fire compartment ε_f	1.00	
Emissivity related to surface material ε_m	0.70	
Configuration factor for radiation heat flux φ	1.00	
Required fire resistance R	30.00	min
Gas temperature θ_g	841.80	°C
Material temperature $\theta_{a,t}$	600.06	°C
Critical material temperature $\theta_{a,cr}$	584.65	°C
Fire resistance t_{cr}	28.57	min
Beam exposure	All sides	
Adaptation factor for cross-section κ_1	1.00	
Adaptation factor for beam κ_2	0.85	
Reduction factor for the 0.2% proof strength $k_{0.2p,\theta}$	0.52	
Reduction factor for the E modulus $k_{E,\theta}$	0.35	
Unity check	**1.03**	-

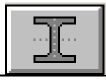
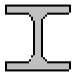
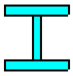
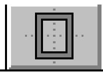
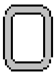
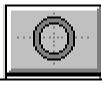
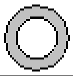
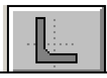
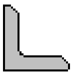
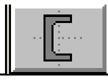
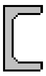
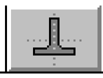
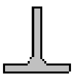
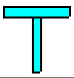
And also the **Fire resistance time** is given in this table:
This member can resist fire for 28,57 minutes.

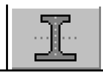
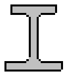
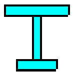
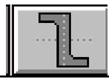
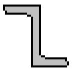
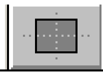
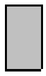
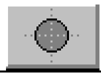
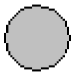
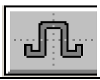
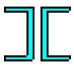
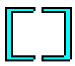
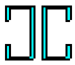
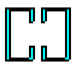
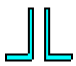
Annex A: Classification in SCIA Engineer versions older than 17.0

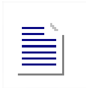
As mentioned in the chapter **Cross-sections**, the new classification tool is only valid in SCIA Engineer 17.0 versions or newer. When using SCIA Engineer 16.1 or older, the following rules are applied.

The following sections are classified during the steel code checks. If another section is used which is not mentioned in the list below, SCIA Engineer can't perform the classification calculation and the profile will be automatically classified as being class 3.

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]).

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Background Information
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