SCIA
BY ALLPLAN



\ SCIA ENGINEER

ADVANCED CONCEPT TRAINING

STEEL CODE CHECK

All information in this document is subject to modification without prior notice. No part of this manual may be reproduced, stored in a database or retrieval system or published, in any form or in any way, electronically, mechanically, by print, photo print, microfilm or any other means without prior written permission from the publisher. SCIA is not responsible for any direct or indirect damage because of imperfections in the documentation and/or the software.

© Copyright 2026 SCIA nv. All rights reserved.

Table of Contents

Table of Contents 3
Introduction 6
Chapter 1: Materials 7
Chapter 2: Cross-sections and classification 10
2.1. Cross-sections 10
2.2. Section classification 12
2.2.1. Elastic stresses 18
2.2.2. Yield surface intersection 19
2.2.3. Iterative approach 20
2.2.4. Semi-Comp+ 21
2.2.5. Effective section 23
Chapter 3: Shear center eccentricity 27
Chapter 4: ULS section check 29
4.1. Partial safety factors 29
4.2. Tension 30
4.3. Compression 31
4.4. Bending moment 32
4.5. Shear 33
4.6. Torsion 34
4.7. Combined check: bending, shear and axial force 39
Chapter 5: ULS stability check 41
5.1. Classification 41
5.1.1. Maximum class along the member 41
5.1.2. Utilisation factor η 42
5.2. Flexural buckling 44
5.2.1. Buckling factors 44
5.2.2. Buckling length 54
5.2.3. Flexural buckling check 58
5.3. Torsional buckling 62
5.4. Lateral torsional buckling 64
5.4.1. Calculation of Mcr 64
5.4.2. Lateral torsional buckling check 71
5.4.3. LTB restraints 76
5.4.4. Sheeting 80
5.5. Compression and bending check 84

5.6. Shear buckling check – EN 1993-1-5 86
5.6.1. General 86
5.6.2. Stiffeners 88
Chapter 6: ULS check for battened compression members 90
Chapter 7: Optimisation 94
7.1. Cross-section optimisation 94
7.2. Overall optimisation 97
7.3. Arbitrary members 100
Chapter 8: 2nd order calculation and imperfections 101
8.1. Overview 101
8.2. Alpha critical 103
8.3. Global frame imperfection ϕ 103
8.4. Bow imperfection 107
8.4.1. N_{Ed} > 25% N_{cr} 107
8.4.2. Bow imperfection e_0 107
8.5. Buckling shape as imperfection - η_{cr} 110
8.6. Superposed eigenshapes or inclination 114
Chapter 9: Physical non-linearity 116
9.1. Plastic hinges 116
9.2. General plastic analysis 120
9.2.1. Von Mises yield criterion 120
9.2.2. Finite element model 121
9.2.3. Material properties 121
9.2.4. General plasticity in SCIA Engineer 122
Chapter 10: SLS check 127
10.1. Nodal displacement 127
10.2. Relative deformation – SLS check 128
Chapter 11: Fire resistance check 133
11.1. General 133
11.1.1. Temperature-time curves 133
11.1.2. Steel temperature 134
11.1.3. Steel properties 136
11.1.4. Fire resistance properties in SCIA Engineer 138
11.2. Calculation methods 139
11.2.1. Resistance domain 140
11.2.2. Time domain 146
11.2.3. Temperature domain (iterative) 148

Annex A: Shear areas 150
Annex B: Examples on the Eurocode paths 152
References and literature 168

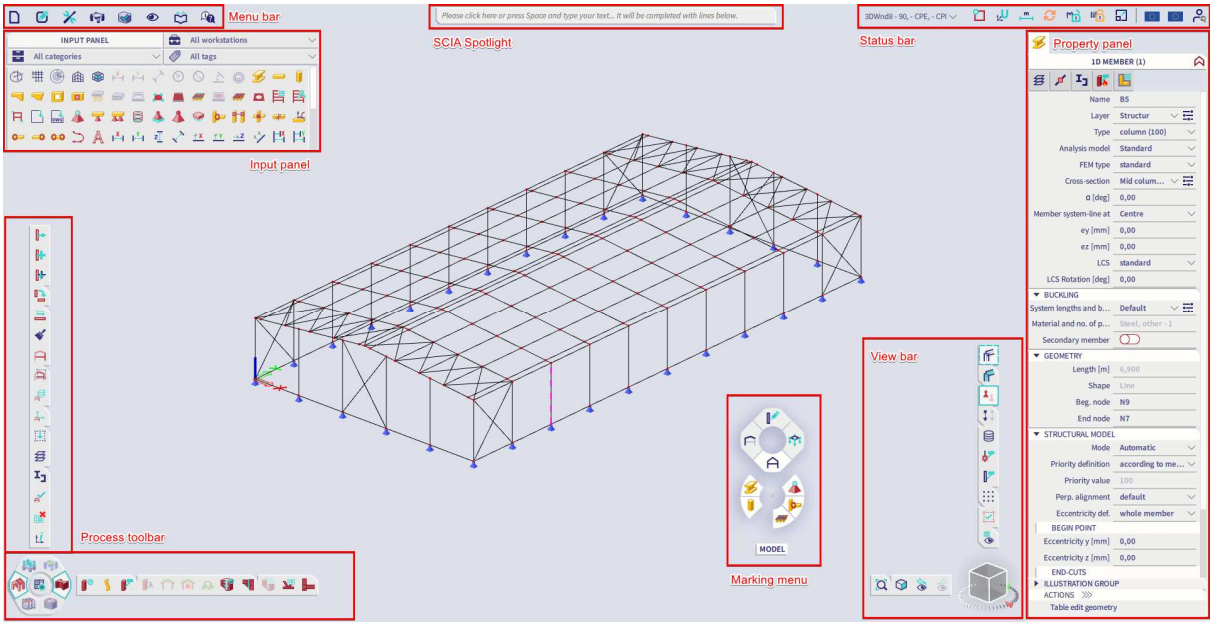
Introduction

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** (**sen.ed.co.en**).

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

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



Chapter 1: 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 ≤ 40 mm		40 mm < t ≤ 80 mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

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

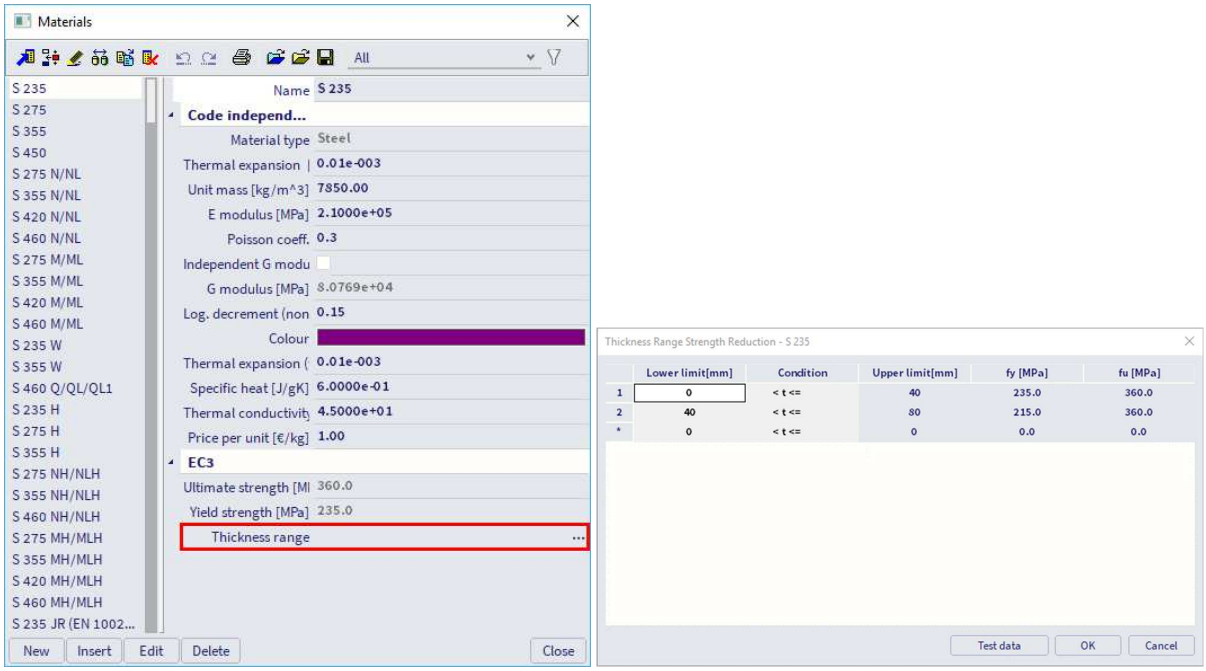
Standard and steel grade	Nominal thickness of the element t [mm]			
	t ≤ 40 mm		40 mm < t ≤ 80 mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10210-1				
S 235 H	235	360	215	340
S 275 H	275	430	255	410
S 355 H	355	510	335	490
S 275 NH/NLH	275	390	255	370
S 355 NH/NLH	355	490	335	470
S 420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

Those materials are included in SCIA Engineer. You can find them via **Menu bar > Libraries** > **Materials**

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

Also materials from the product codes are implemented in the software (for example materials from Belgian National Annex NBN NA,…).

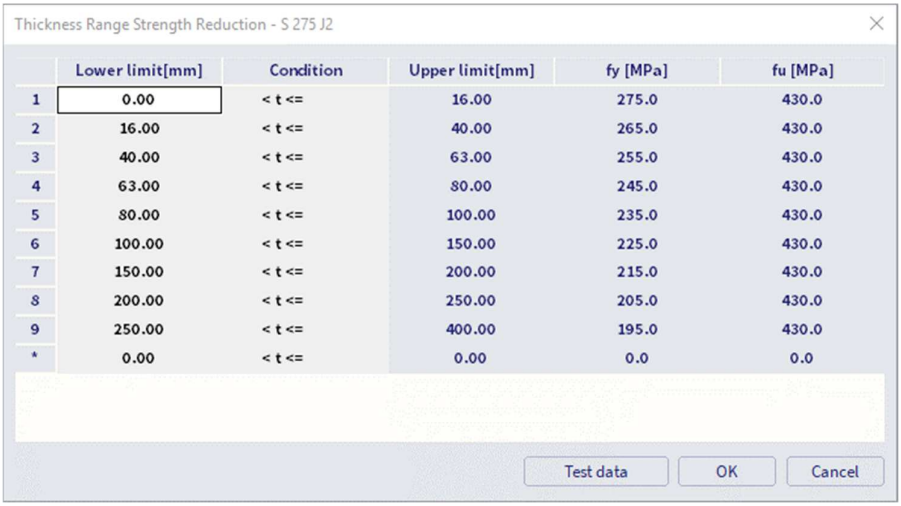
With the option '**Thickness range**' the influence of the thickness on the yield strength f_y and tensile strength f_u are defined. When inputting a new user defined material in SCIA Engineer, also the thickness range for that material should be inputted:



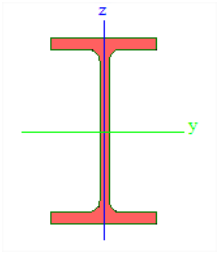
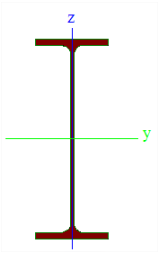
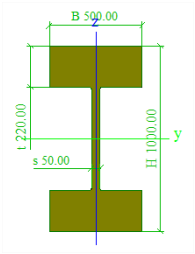
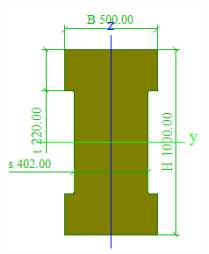
Example: **NA_Material_Strength_Application.esa**

In this example a material of the National Annex of Belgium has been inputted manually: S 275 J2.

This material has a lot of thickness range strength reductions, as we can see in the thickness range table:



And for each beam the correct yield strength has been taken into account according to the inputted table above. You can check this by performing a linear calculation and asking for the detailed output for **Menu bar > Design > Steel members > ULS Check** (or **Process toolbar > Steel > Steel ULS check)** :

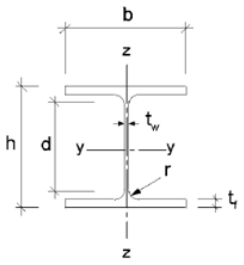
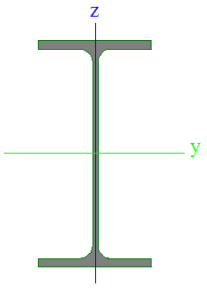
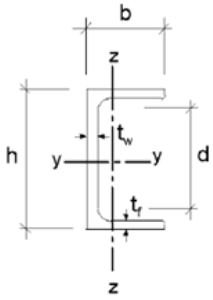
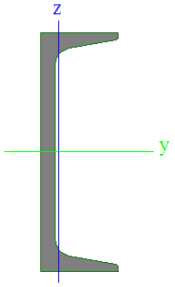
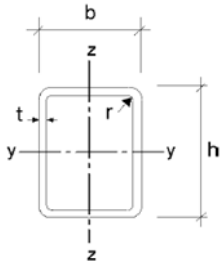
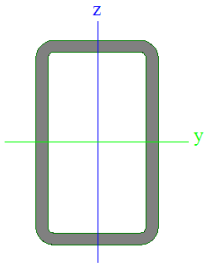
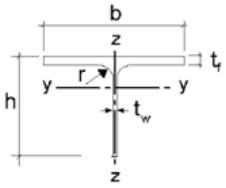
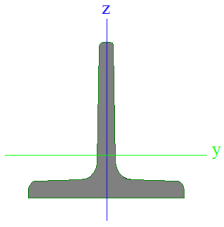
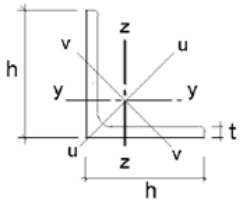
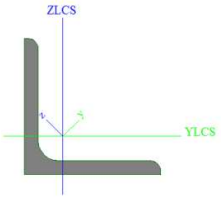
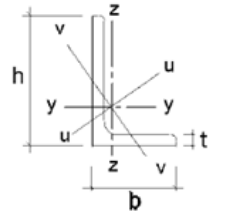
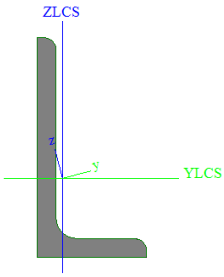
CS1	CS2	CS3	CS4
			
Formcode 1 - I section h [mm] 80.00 b [mm] 46.00 t [mm] 5.20 s [mm] 3.80 r [mm] 5.00 r1 [mm] 0.00 a [%] 0 W [mm] 0.00 wm [mm^2] 0.00	Formcode 1 - I section h [mm] 600.00 b [mm] 220.00 t [mm] 19.00 s [mm] 12.00 r [mm] 24.00 r1 [mm] 0.00 a [%] 0 W [mm] 116.00 wm [mm^2] 0.00	Formcode 1 - I section 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	Formcode 1 - I section 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 f_y 275.0 MPa Ultimate strength f_u 430.0 MPa Fabrication Rolled	Material Yield strength f_y 265.0 MPa Ultimate strength f_u 430.0 MPa Fabrication Rolled	Material Yield strength f_y 205.0 MPa Ultimate strength f_u 430.0 MPa Fabrication Rolled	Material Yield strength f_y 195.0 MPa Ultimate strength f_u 430.0 MPa Fabrication Rolled

Chapter 2: Cross-sections and classification

2.1. 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.

Be aware that the switch will also affect the buckling settings for yy and zz.

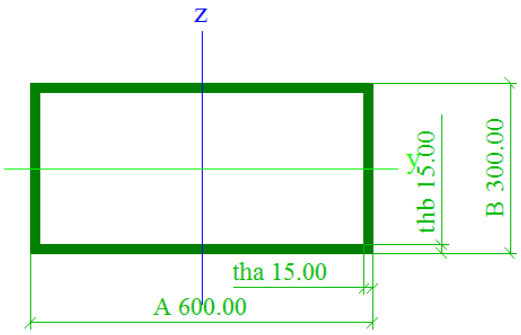
Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		non-sway	sway	
System length	L	6,000	6,000	m
Buckling factor	k	0,70	2,00	
Buckling length	l_{cr}	4,200	12,014	m
Critical Euler load	N_{cr}	144890,20	5959,77	kN
Slenderness	λ	19,32	95,27	
Relative slenderness	λ_{rel}	0,21	1,01	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	

You can avoid this behaviour by creating the cross-section differently, so the y-axis remains the strong axis. Afterwards with the properties of the member you can perform a rotation of the local coordinate system (LCS rotation) if necessary.

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 M_y and a shear force V_z 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
Normal force	N_{Ed}	0.00	kN
Shear force	$V_{y,Ed}$	300.00	kN
Shear force	$V_{z,Ed}$	0.00	kN
Torsion	T_{Ed}	0.00	kNm
Bending moment	$M_{y,Ed}$	0.00	kNm
Bending moment	$M_{z,Ed}$	-300.00	kNm

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

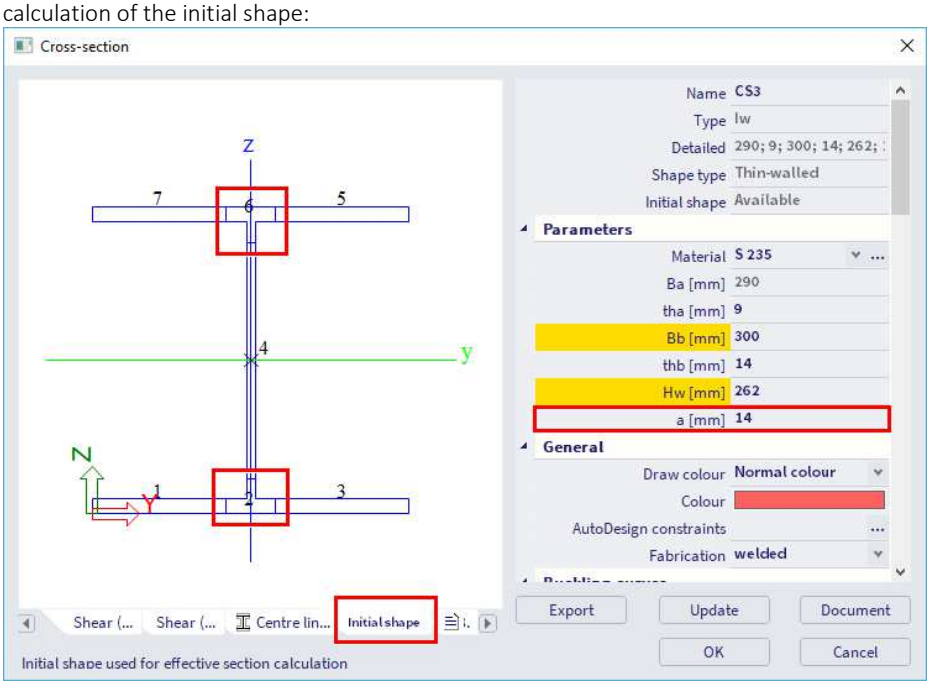
2.2. Section classification

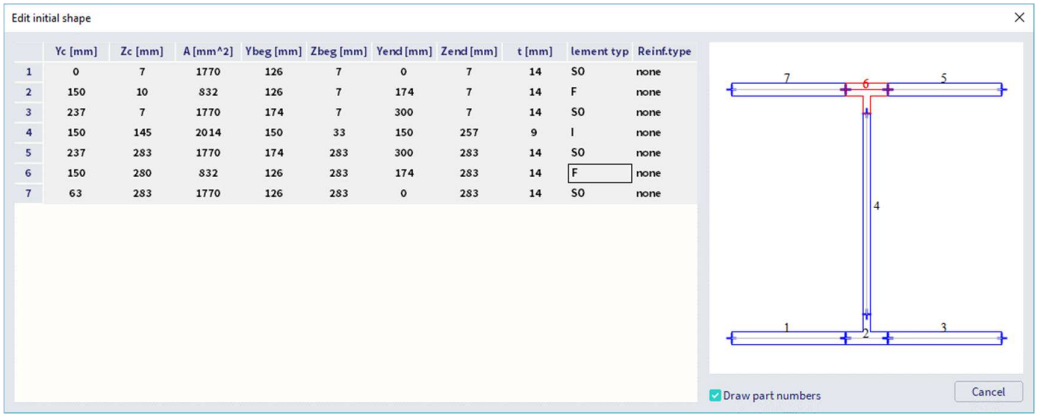
Since SCIA Engineer 17.0, all sections containing an initial shape can be classified. The sections without an initial shape cannot be classified and will be automatically checked as being class 3.

When you use a cross-section from the profile library, the initial shape will be available. When you want to use another cross-section, you can use the group '**Thin-walled geometric**', for which the initial shape and torsion properties are available.

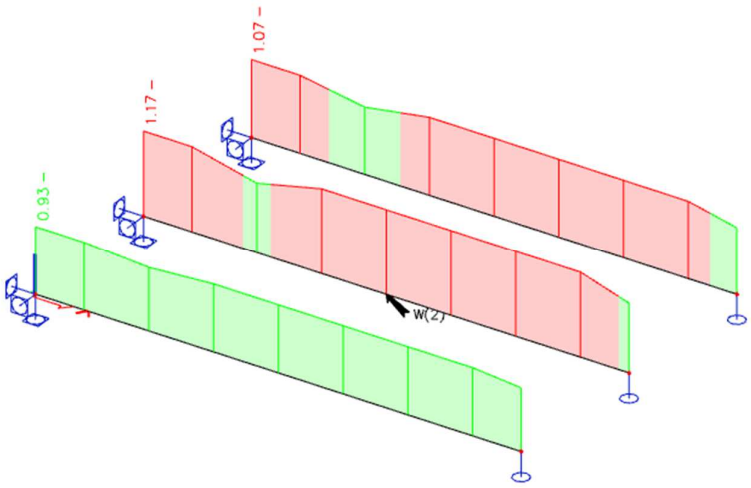
Example: **Cross-section.esa**

- 3 cross-sections:
  - HE300A from profile library
  - HE300A inputted as a general cross-section (imported from .dwg)
  - HE300A inputted as a sheet welded Iw section. By creating this section, it is important that the correct value for parameter a is defined in mm. This parameter will be recognized as the fixed part in the calculation of the initial shape:



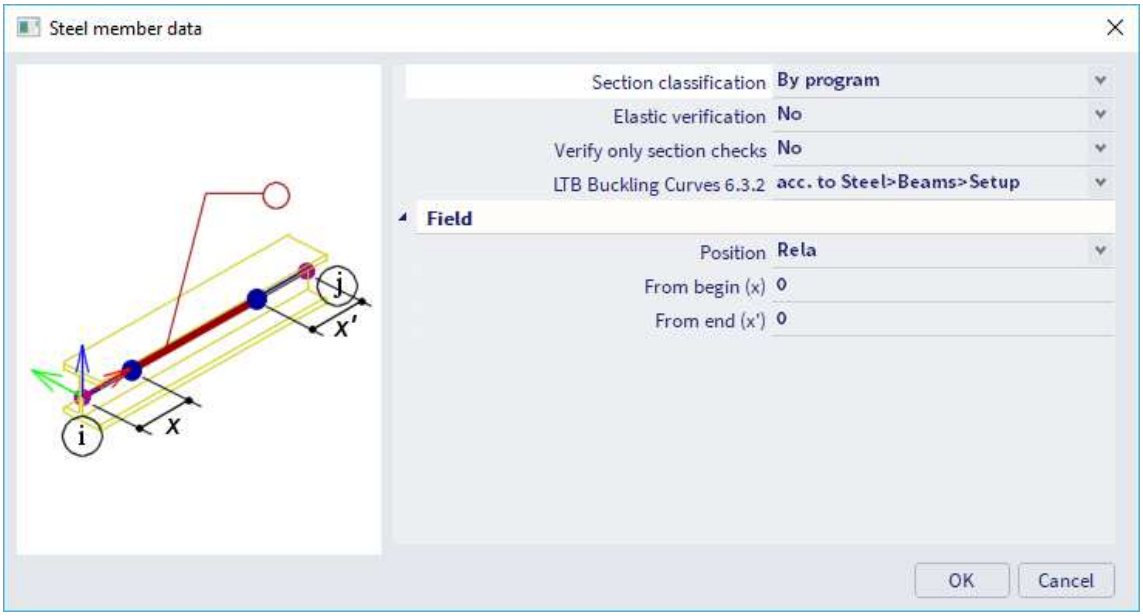


- Only the first and the third profile are recognized as a symmetric I-shape containing an initial shape. For this reason, the classification calculation can be performed.
- The second profile is not recognized as a symmetric I-shape and there is no initial shape available.
- The first and the third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



Note: the general cross-section cannot be imported in SCIA Engineer with the concept edition. For this functionality, the Professional or Ultimate edition is necessary (or module sen.05).

SCIA Engineer will calculate the classification automatically, but the calculated classification in SCIA Engineer can be overruled by 2 settings in the steel member data (**Input panel > Steel > Steel member data** or **Process toolbar > Steel member data**):

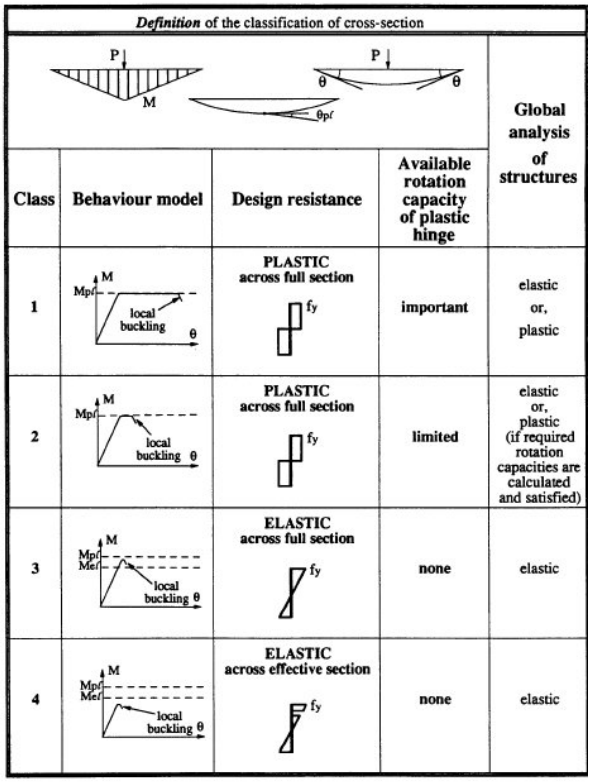


- **Section classification**: you can choose between a classification calculation '**By program'**, or can overwrite this and choose for **class 1, 2 or 3**. Since classification 4 is not described for all cross-sections in the Eurocode, this option cannot be chosen.
- **Elastic Check Only**: you can choose to perform only an elastic check. This corresponds with a class 3 check.

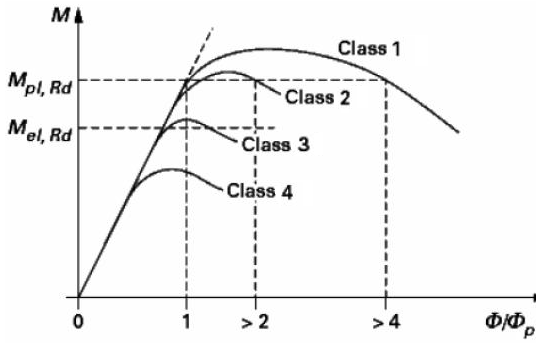
The classification of cross-sections is executed according to EN 1993-1-1, art. 5.5.

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



This classification depends on the proportions of each of its compression elements.

For each intermediary section, the classification is determined and the related section check is performed. The classification can change for each intermediary point.

For each load case/combination, the critical section classification over the member is used to perform the stability check. So, the stability section classification can change for each load case/combination.

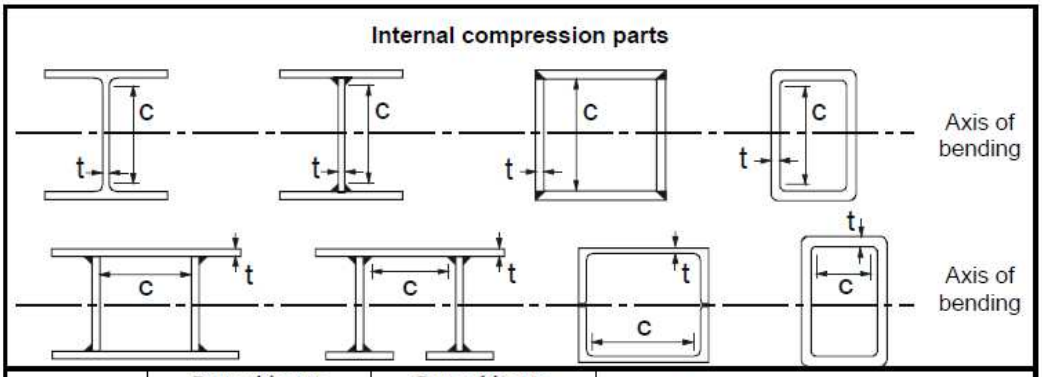
However, for non-prismatic sections, the stability section classification is determined for each intermediary section.

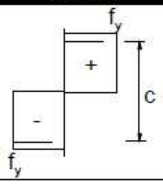
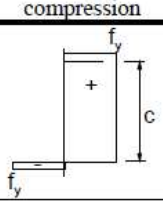
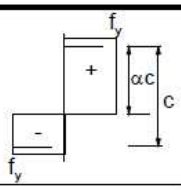
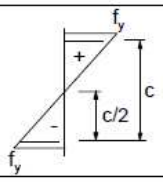
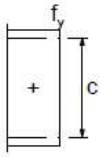
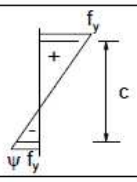
The classification check in SCIA Engineer will be executed following tables 5.2 of the EN 1993-1-1.
For standard sections, the classification is done according to the parts of the Initial Shape.

- 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 (Circular Hollow 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.

Note: 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



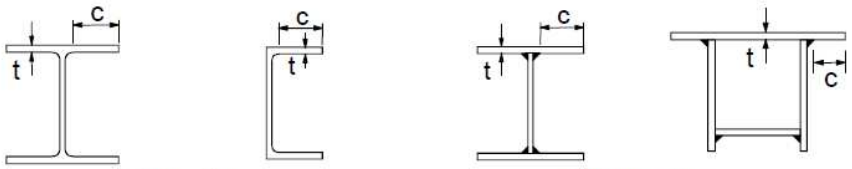
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t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

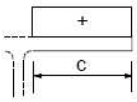
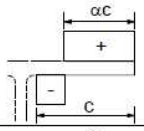
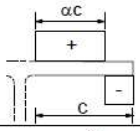
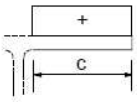
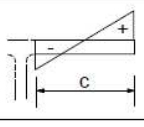
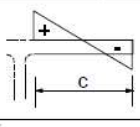
*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

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

Outstand flanges



Rolled sections | Welded sections

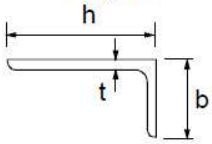
Class	Part subject to compression	Part subject to bending and compression	
		Tip in compression	Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \frac{9\varepsilon}{\alpha}$	$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$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

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

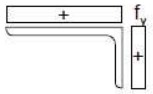
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

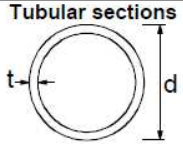
Refer also to "Outstand flanges" (see sheet 2 of 3)



Does not apply to angles in continuous contact with other components

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

Tubular sections



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

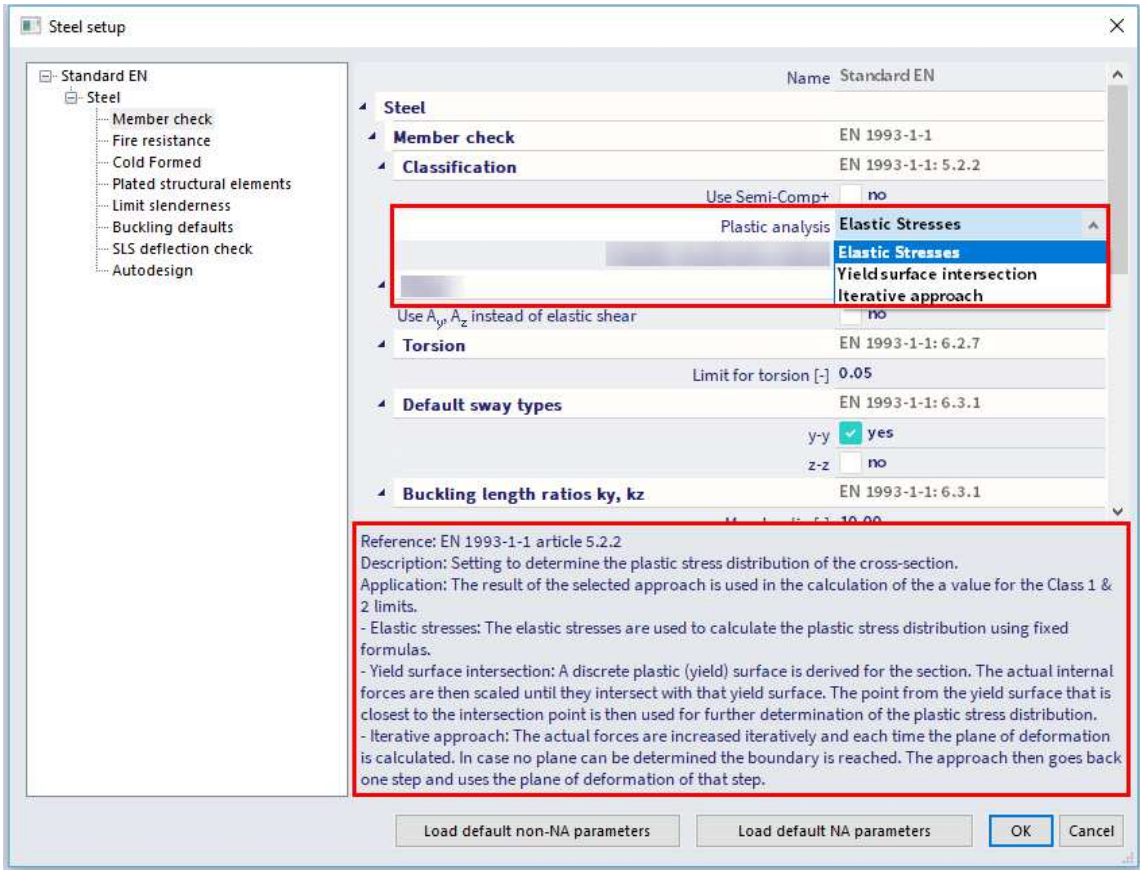
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51

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

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

- Elastic stresses;
- Yield surface intersection;
- Iterative approach.

In the cross-section properties, you 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 (**Menu bar > Design > Steel members > Settings** or **Process toolbar > Steel > Steel settings**):



2.2.1. 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 f_1 and f_2 at the ends of the parts.

Standard calculation of α

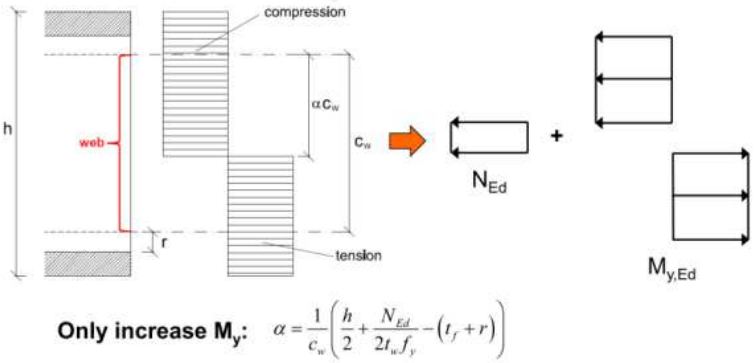
In case one stress is positive (compression) and the other stress is negative (tension), the following calculation is used:

$$\alpha = \frac{|\sigma_{compression}|}{|\sigma_{compression}| + |\sigma_{tension}|}$$

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

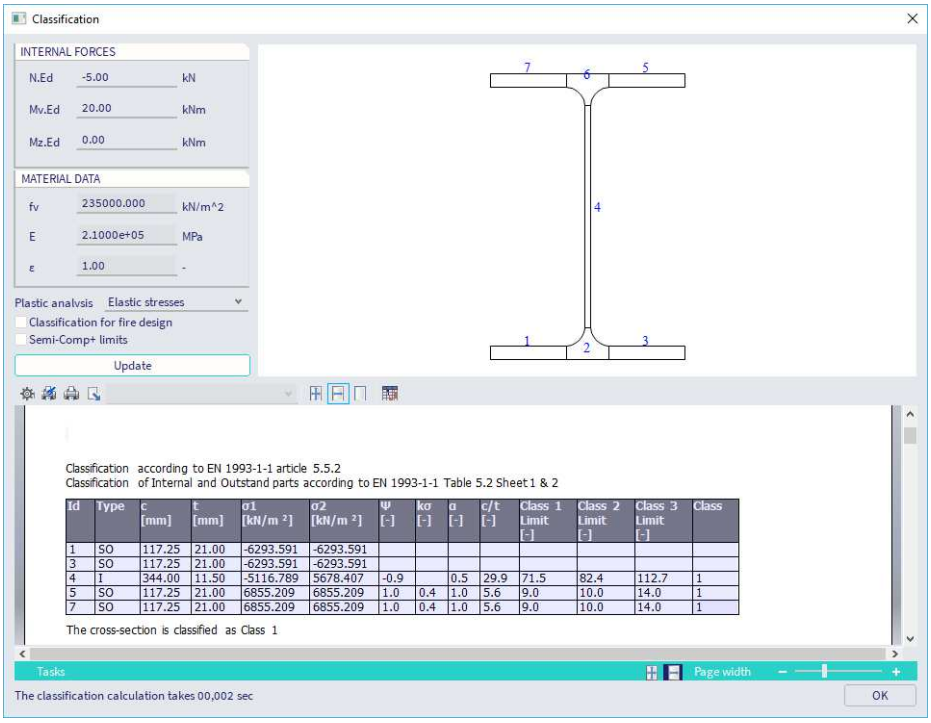
Double-symmetric I-section

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



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

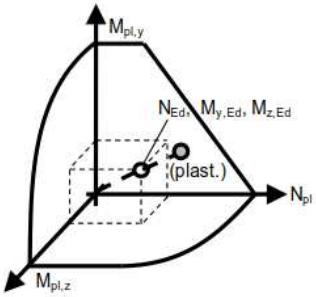
Below you can find the output for cross-section HE450A:



Id	Type	c [mm]	t [mm]	σ1 [kN/m²]	σ2 [kN/m²]	Ψ [-]	kσ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	117.25	21.00	-6293.591	-6293.591								
3	SO	117.25	21.00	-6293.591	-6293.591								
4	I	344.00	11.50	-5116.789	5678.407	-0.9		0.5	29.9	71.5	82.4	112.7	1
5	SO	117.25	21.00	6855.209	6855.209	1.0	0.4	1.0	5.6	9.0	10.0	14.0	1
7	SO	117.25	21.00	6855.209	6855.209	1.0	0.4	1.0	5.6	9.0	10.0	14.0	1

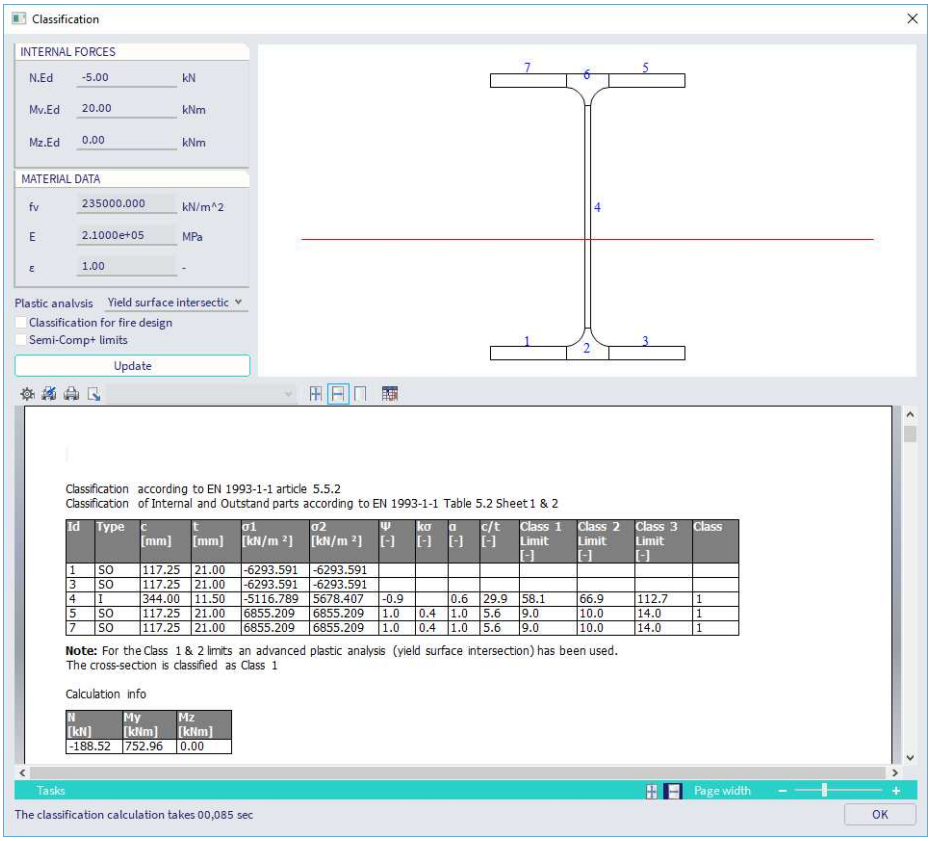
2.2.2. Yield surface intersection

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 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.

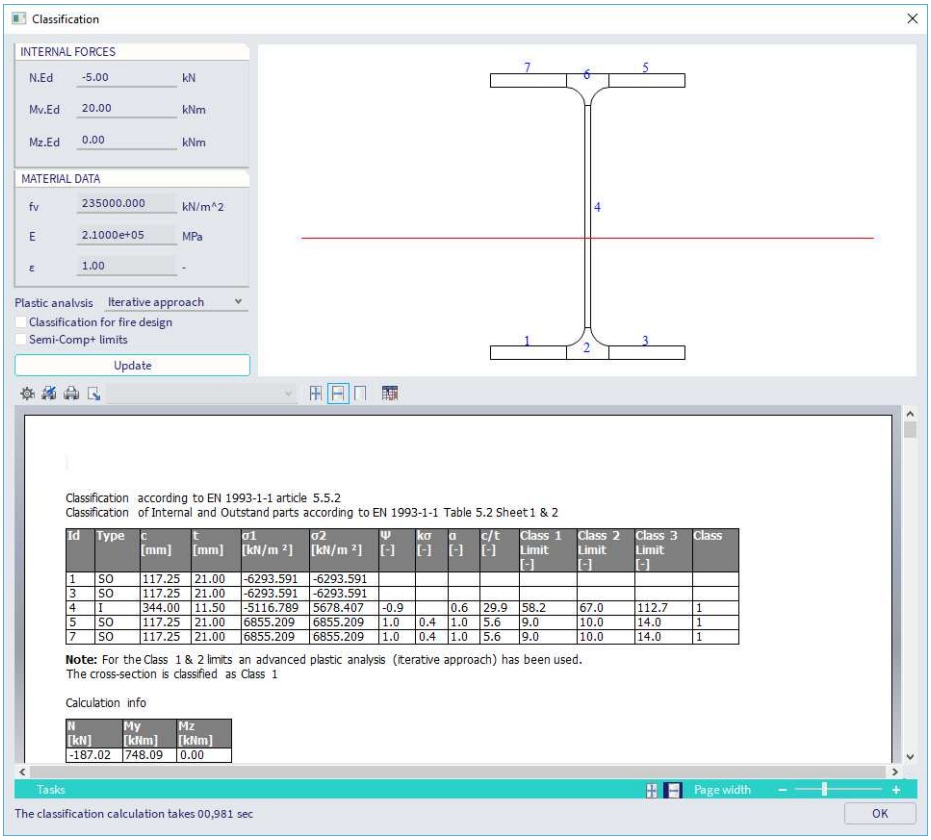


Note: it can happen that the plastic neutral axis is rotated, but this has a negligible effect on α. This happens when the closest point on the yield surface has a minor M_z moment.

2.2.3. 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 yielding plateau.

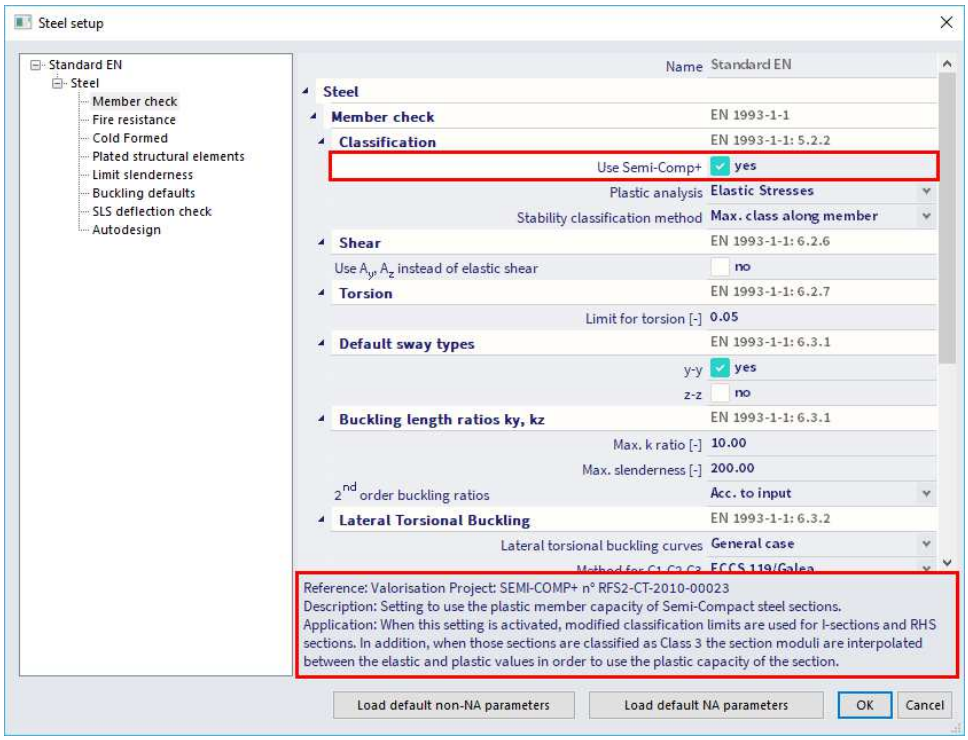
The actual plane of deformation for the given internal forces is determined iteratively, which provides an exact solution.



2.2.4. 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.

The Semi-Comp+ 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

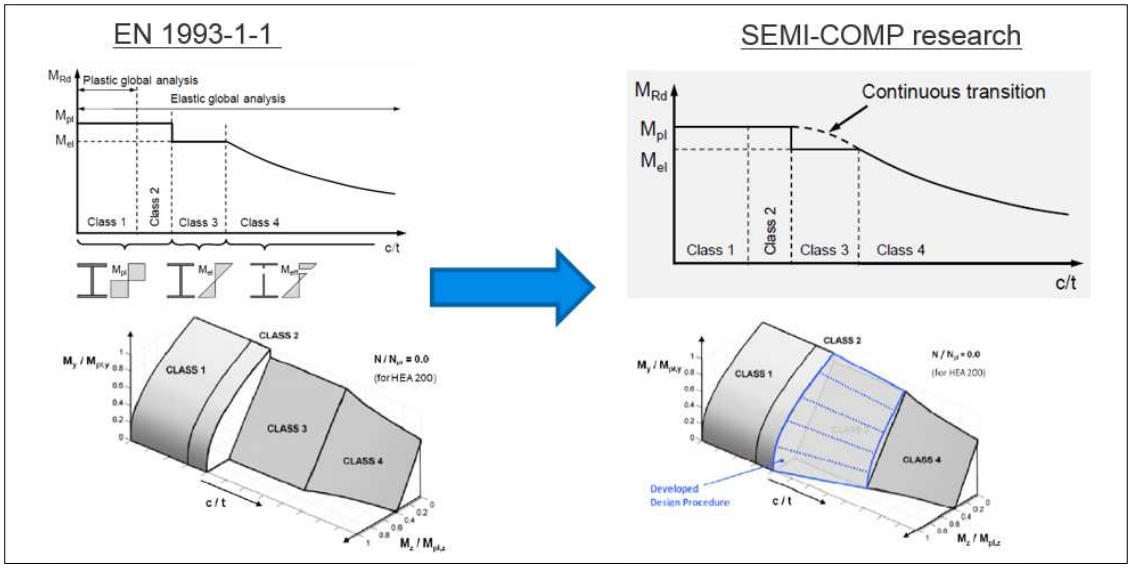
Currently, there is a discrepancy in the used classification limits between EN1993-1-1 and EN1993-1-5. The Semi-Comp+ publication identified this discrepancy and proposed new classification limits (Ref. [36]):

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 (*ε)	
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

Note: these 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 considered for class 3 sections.



2.2.5. Effective section

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:

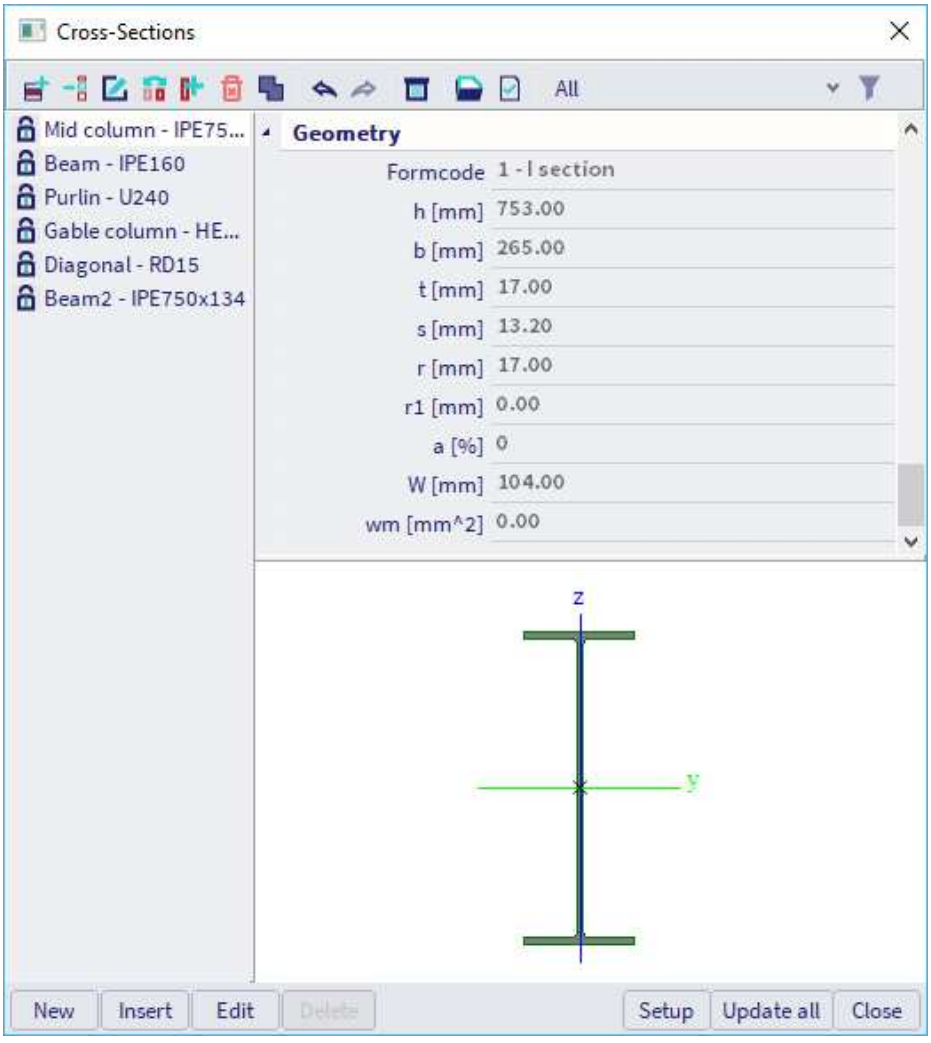
- A_{eff} is the effective area of the cross-section when subject to uniform compression;
- W_{eff} is the effective section modulus of the cross-section when subject only to moment about the relevant axis;
- e_N is the shift of the relevant centroidal axis when the cross-section is subject to uniform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

With these critical properties, the steel code check will be executed.

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.

We consider column B28:

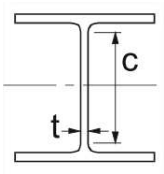


The classification has been executed at the bottom of the column (position = 0,00 m)

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

With table 5.2 of EN 1993-1-1 we can determine the class for each component of the cross-section.

Web according to table 5.2 (sheet 1):

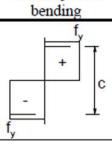
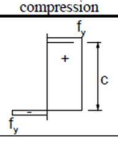
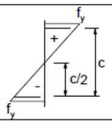
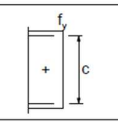


c = H – 2 * thickness flange – 2 * radius

c = 753 mm – 2*17 mm – 2*17 mm = 685 mm

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

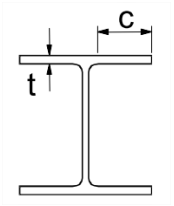
Maximum ratio for class 1: 33

Maximum ratio for class 2: 38

Maximum ratio for class 3: 42

→ c/t = 51,89 > 42 → Class 4

Flange according to table 5.2 (sheet 2):

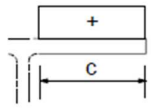
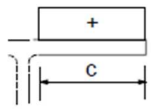


c = B/2 – (thickness web)/2 – radius

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

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 for class 1: 9

Maximum ratio for class 2: 10

Maximum ratio for class 3: 14

→ c/t = 6,40 < 9 → Class 1

This corresponds with the classification of this cross-section for the stability check 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	8656.406	8656.406	1.0	0.4	1.0	6.4	9.0	10.0	14.0	1
3	SO	108.90	17.00	8656.406	8656.406	1.0	0.4	1.0	6.4	9.0	10.0	14.0	1
4	I	685.00	13.20	8656.406	8656.406	1.0		1.0	51.9	33.0	38.0	42.0	4
5	SO	108.90	17.00	8656.406	8656.406	1.0	0.4	1.0	6.4	9.0	10.0	14.0	1
7	SO	108.90	17.00	8656.406	8656.406	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 must be calculated.

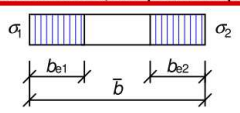
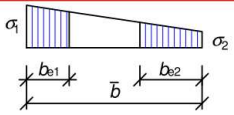
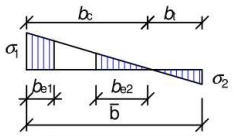
Those properties are also given in SCIA Engineer in the preview of the steel code check, just below the classification calculation.

Effective properties						
Effective area	A_{eff}	1.7222e+04	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

The calculation of the sectional area A_{eff} is given 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}
	$\psi = 1$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = 0,5\ b_{eff}$ $b_{e2} = 0,5\ b_{eff}$
	$1 > \psi \geq 0$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = \frac{2}{5-\psi} b_{eff}$ $b_{e2} = b_{eff} - b_{e1}$
	$\psi < 0$: $b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$ $b_{e1} = 0,4\ b_{eff}$ $b_{e2} = 0,6\ b_{eff}$

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

For ψ = 1 and internal compression elements:

$$\overline{\lambda_p} \leq 0,5 + \sqrt{0,085 - 0,055\psi} = 0,673 \rightarrow \rho = 1$$

$$\overline{\lambda_p} > 0,5 + \sqrt{0,085 - 0,055\psi} = 0,673 \rightarrow \rho = (\overline{\lambda_p} - 0,22)/\overline{\lambda_p}^2$$

For our cross-section:

$$\overline{\lambda_p} = [f_y/\sigma_{cr}]^{0.5} = \frac{\bar{b}/t}{28,4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} = \frac{51,89}{28,4 \cdot 1,00 \cdot \sqrt{4,0}} = 0,91$$

With:

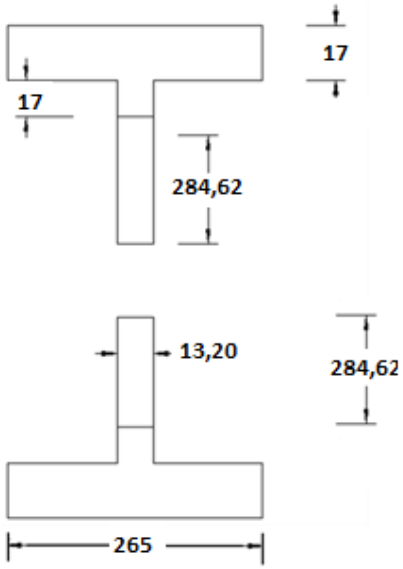
- ε = 1,00 (S235)
- k_σ = 4,0 (table 4.1 of EN 1993-1-5:2006)

So:

$$\rho = \frac{0,91 - 0,22}{(0,91)^2} = 0,83$$

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

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



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:

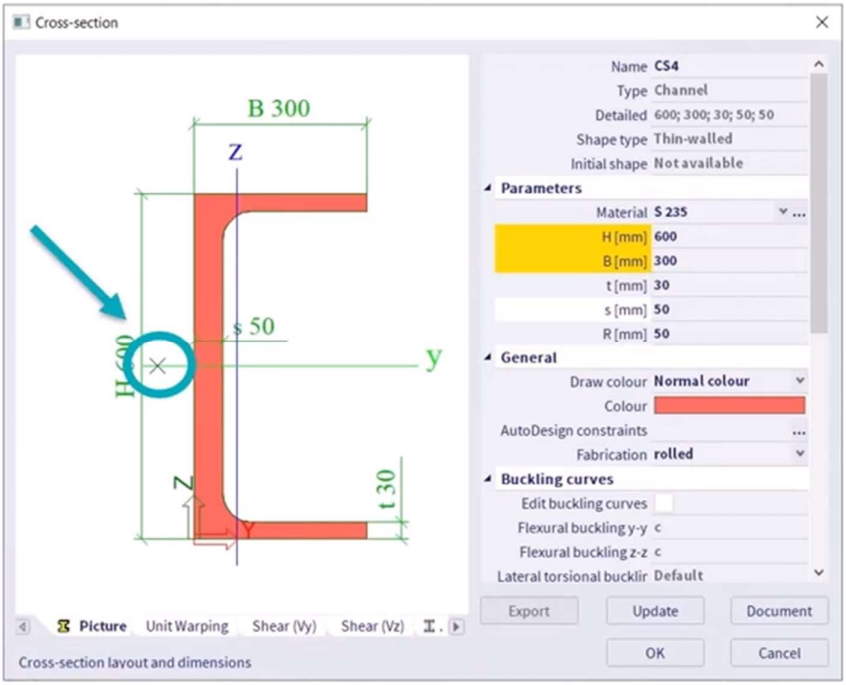
Effective properties						
Effective area	A_{eff}	1.7222e+04	mm^2			
Effective second moment of area	$I_{eff,y}$	1.6608e+09	mm^4	$I_{eff,z}$	5.2895e+07	mm^4
Effective section modulus	$W_{eff,y}$	4.4111e+06	mm^3	$W_{eff,z}$	3.9920e+05	mm^3
Shift of the centroid	$e_{N,y}$	0.00	mm	$e_{N,z}$	0.00	mm

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

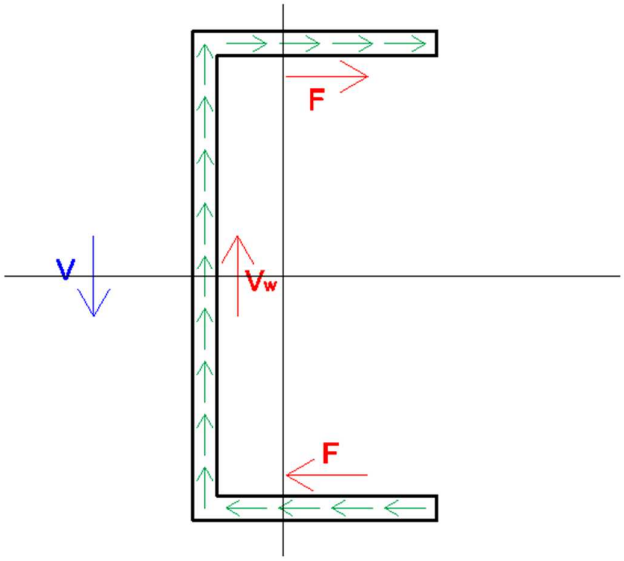
Note: for cold-formed cross-sections the rules are adapted (according to EN 1993-1-3).

Chapter 3: Shear center eccentricity

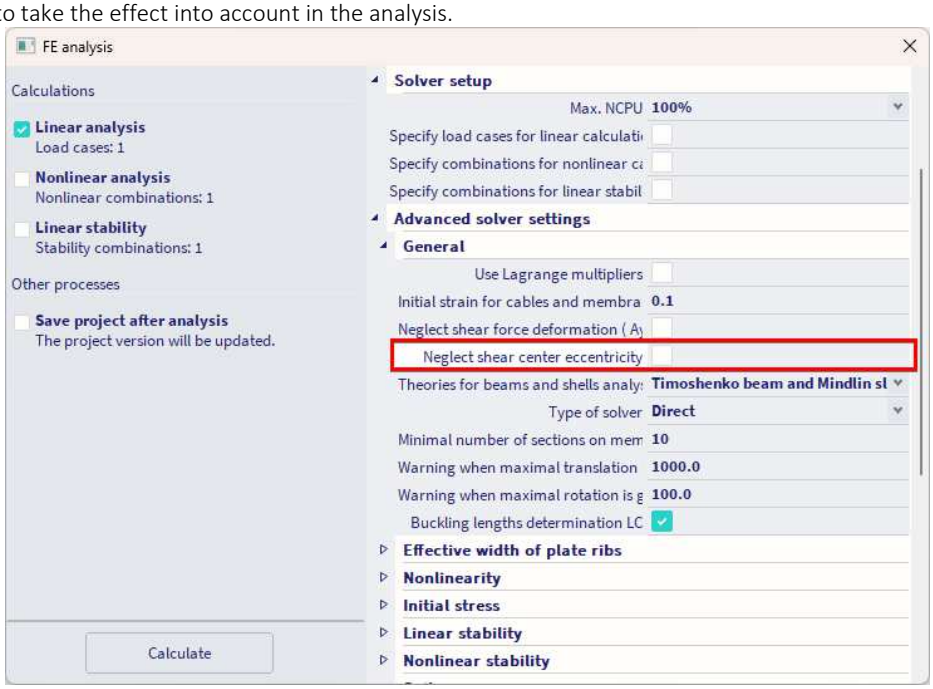
Definition shear centre: the point at which a load applied parallel to the plane of the section will produce no torsion. For doubly symmetric profiles this is always the center of gravity, but for non-symmetrical profiles this is not the case.



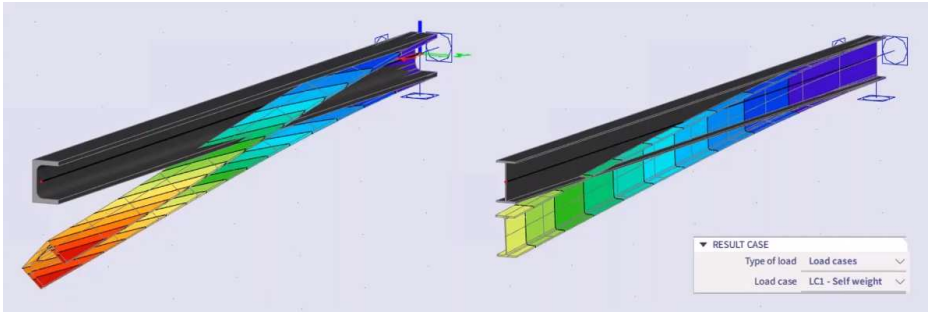
When a shear is applied to a non-symmetrical section, there will be opposite stresses in the section to counter this force and these will continue in the flanges. When it's symmetrical these flanges will lift each other up but here both of them will create forces which will create an additional torsional moment. This torsional moment can be countered by applying the load with an eccentricity, which is called the shear center eccentricity.



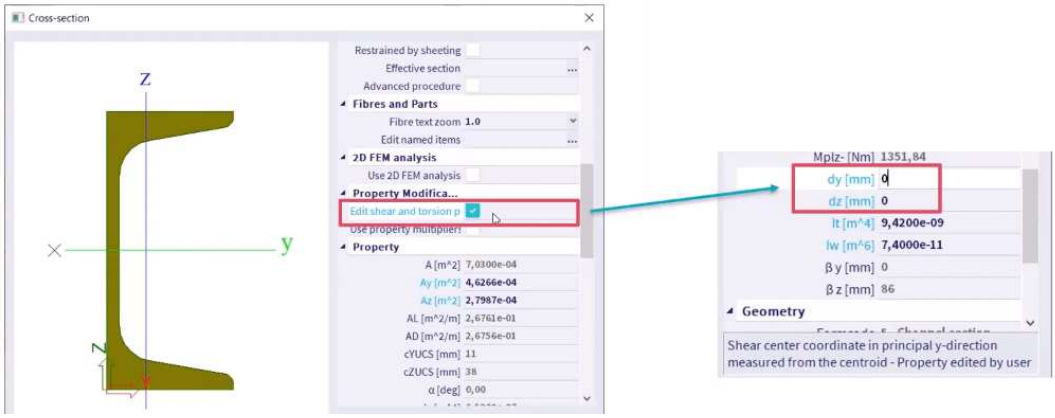
Since SCIA Engineer 22.0 the shear center eccentricity is considered in the analysis. In older versions the additional torsion isn't considered in the analysis since the force was placed in the center of gravity. When opening a project created in an older version, the advanced solver option 'Neglect shear center eccentricity' is activated and you can deactivate it to take the effect into account in the analysis.



Below you can find a small example of the effect, left with the shear center eccentricity taken into account and right without the effect:



If wanted, you can neglect the effect for a specific cross-section by editing the torsion properties for the cross-section:



Note that in SCIA Engineer only 6 degrees of freedom are available, so only St. Venant torsion can be considered in the analysis. It is possible to consider warping in the steel code check. Further explanation on this is given in the torsion check in chapter 4.

Chapter 4: 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 **Menu bar > Design > Steel members** > **ULS Check** or **Process toolbar > Steel > Steel ULS check**. In the properties of the check, you 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 on one page;
- 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. Since SCIA18.0 also **all formulas** can be shown in the output.

Further in this chapter, the section checks will be explained and the detailed results are shown.

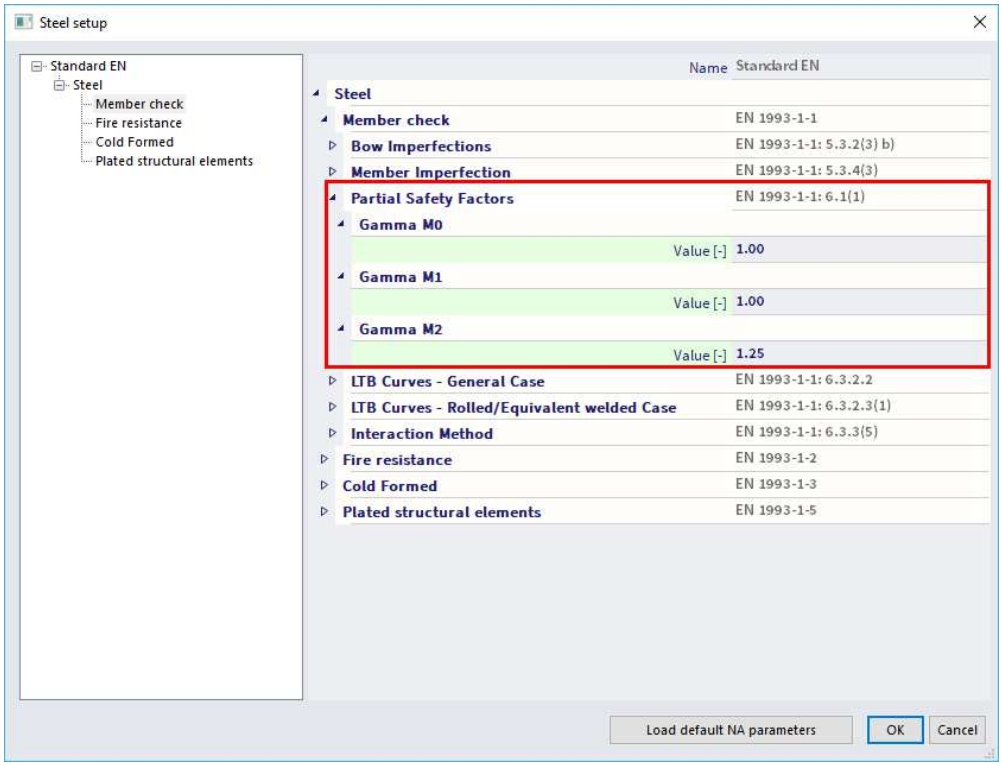
4.1. Partial safety factors

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

Following safety factors are considered:

- γ_{M0} = 1,00: resistance of cross-sections;
- γ_{M1} = 1,00: resistance of members to instability accessed by member checks;
- γ_{M2} = 1,25: resistance of cross-sections in tension to fracture.

Those factors can also be found in the **National Annex** of EN 1993-1-1 in SCIA Engineer (via **Status bar > National annex > Manage annexes**):



4.2. Tension

The tension check will be executed following EN 1993-1-1 art. 6.2.3 formula (6.5):

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

With:

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

---

Example: **Industrial Hall.esa**

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

Internal forces		Calculated	Unit
Normal force	N_{Ed}	30,77	kN
Shear force	$V_{y,Ed}$	0,00	kN
Shear force	$V_{z,Ed}$	0,00	kN
Torsion	T_{Ed}	0,00	kNm
Bending moment	$M_{y,Ed}$	0,00	kNm
Bending moment	$M_{z,Ed}$	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)

Cross-section area	A	1,7663e+02	mm^2
Plastic tension resistance	$N_{pl,Rd}$	41,51	kN
Ultimate tension resistance	$N_{u,Rd}$	45,78	kN
Tension resistance	$N_{t,Rd}$	41,51	kN
Unity check		0,74	-

$$N_{pl,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{1,7663 \cdot 10^2[mm^2] \times 235,0[MPa]}{1,00} = 41,51[kN] \quad \text{(EC3-1-1: 6.6)}$$

$$N_{u,Rd} = \frac{0,9 \times A \times f_u}{\gamma_{M2}} = \frac{0,9 \times 1,7663 \cdot 10^2[mm^2] \times 360,0[MPa]}{1,25} = 45,78[kN] \quad \text{(EC3-1-1: 6.7)}$$

$$N_{t,Rd} = \min(N_{pl,Rd}; N_{u,Rd}) = \min(41,51[kN]; 45,78[kN]) = 41,51[kN]$$

$$\text{Unity check} = \frac{N_{Ed}}{N_{t,Rd}} = \frac{30,77[kN]}{41,51[kN]} = \mathbf{0,74 \leq 1,00} \quad \text{(EC3-1-1: 6.5)}$$

4.3. Compression

The compression check will be executed following EN 1993-1-1 art. 6.2.4 formula (6.9):

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

With:

- $N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ 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
Normal force	N_{Ed}	-158,70	kN
Shear force	$V_{y,Ed}$	-0,05	kN
Shear force	$V_{z,Ed}$	-101,46	kN
Torsion	T_{Ed}	0,00	kNm
Bending moment	$M_{y,Ed}$	-116,71	kNm
Bending moment	$M_{z,Ed}$	-0,06	kNm

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

Compression check

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

Cross-section area	A	1,8800e+04	mm^2
Compression resistance	$N_{c,Rd}$	4418,00	kN
Unity check		0,04	-

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{1,8800 \cdot 10^4[mm^2] \times 235,0[MPa]}{1,00} = 4418,00[kN]$$ (EC3-1-1: 6.10)

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{c,Rd}} = \frac{|-158,70[kN]|}{4418,00[kN]} = \mathbf{0,04 \leq 1,00}$$ (EC3-1-1: 6.9)

4.4. Bending moment

The bending moment check for M_y and M_z will be executed following EN 1993-1-1 art. 6.2.5 formula (6.12):

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

With:

- $M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}}$ 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

Note: an interpolated section modulus can be used in case of class 3 cross-sections when the Semi-Comp+ method is activated (see chapter 2)

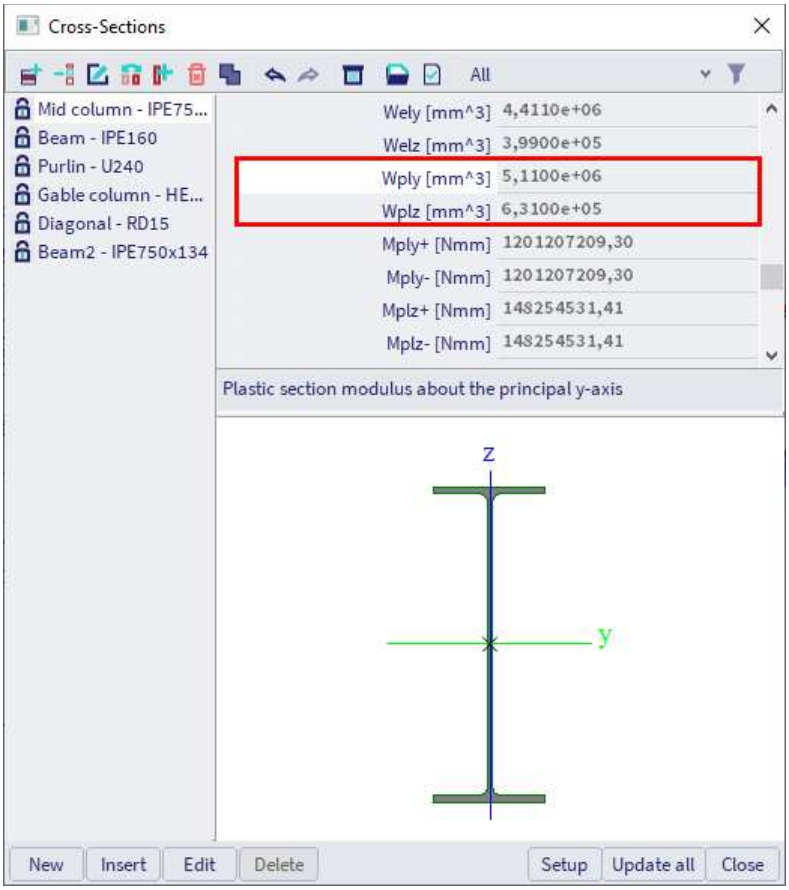
Example: **Industrial Hall.esa**

Consider column B28 (for combination CO1-ULS).

The critical check is on position 1,150 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-158,70	kN
Shear force	$V_{y,Ed}$	-0,05	kN
Shear force	$V_{z,Ed}$	-101,46	kN
Torsion	T_{Ed}	0,00	kNm
Bending moment	$M_{y,Ed}$	-116,71	kNm
Bending moment	$M_{z,Ed}$	-0,06	kNm

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



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

Plastic section modulus	$W_{pl,y}$	5,1100e+06	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	1200,85	kNm
Unity check		0,10	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{5,1100 \cdot 10^6[mm^3] \times 235,0[MPa]}{1,00} = 1200,85[kNm]$$ (EC3-1-1: 6.13)

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|-116,71[kNm]|}{1200,85[kNm]} = \mathbf{0,10 \leq 1,00}$$ (EC3-1-1: 6.12)

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

Plastic section modulus	$W_{pl,z}$	6,3100e+05	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	148,28	kNm
Unity check		0,00	-

$$M_{pl,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{6,3100 \cdot 10^5[mm^3] \times 235,0[MPa]}{1,00} = 148,28[kNm]$$ (EC3-1-1: 6.13)

$$\text{Unity check} = \frac{|M_{z,Ed}|}{M_{pl,z,Rd}} = \frac{|-0,06[kNm]|}{148,28[kNm]} = \mathbf{0,00 \leq 1,00}$$ (EC3-1-1: 6.12)

4.5. Shear

The shear check for V_y and V_z will be executed following EN 1993-1-1 art. 6.2.6 formula (6.17):

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

For **plastic design** $V_{c,Rd}$ (in the absence of torsion) is the design plastic shear resistance $V_{pl,Rd}$, according to formula (6.18):

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

With:

- A_v shear area, the formula for A_v depends on the cross-section (see EN 1993-1-1 article 6.2.6(3) and several ECCS publications, see annex A).

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 (formula (6.19)) unless the buckling verification in section 5 of EN 1993-1-5 applies:

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

With (formula (6.20)):

$$\tau_{Ed} = \frac{V_{Ed} \cdot S}{I_t}$$

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
Normal force	N_{Ed}	-158,70	kN
Shear force	$V_{y,Ed}$	-0,05	kN
Shear force	$V_{z,Ed}$	-101,46	kN
Torsion	T_{Ed}	0,00	kNm
Bending moment	$M_{y,Ed}$	-116,71	kNm
Bending moment	$M_{z,Ed}$	-0,06	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)

Shear correction factor	η	1,20	
Shear area	A_v	9,4086e+03	mm²
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	1276,54	kN
Unity check		0,00	-

$$V_{pl,y,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{9,4086 \cdot 10^3[mm^2] \times \frac{235,0[MPa]}{\sqrt{3}}}{1,00} = 1276,54[kN]$$ (EC3-1-1: 6.18)

$$\text{Unity check} = \frac{|V_{y,Ed}|}{V_{c,y,Rd}} = \frac{|-0,05[kN]|}{1276,54[kN]} = \mathbf{0,00 \leq 1,00}$$ (EC3-1-1: 6.17)

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

Shear correction factor	η	1,20	
Shear area	A_v	1,1389e+04	mm²
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	1545,22	kN
Unity check		0,07	-

$$V_{pl,z,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{1,1389 \cdot 10^4[mm^2] \times \frac{235,0[MPa]}{\sqrt{3}}}{1,00} = 1545,22[kN]$$ (EC3-1-1: 6.18)

$$\text{Unity check} = \frac{|V_{z,Ed}|}{V_{c,z,Rd}} = \frac{|-101,46[kN]|}{1545,22[kN]} = \mathbf{0,07 \leq 1,00}$$ (EC3-1-1: 6.17)

4.6. Torsion

The torsion check will be executed following EN 1993-1-1 article 6.2.7 formula (6.23):

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

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

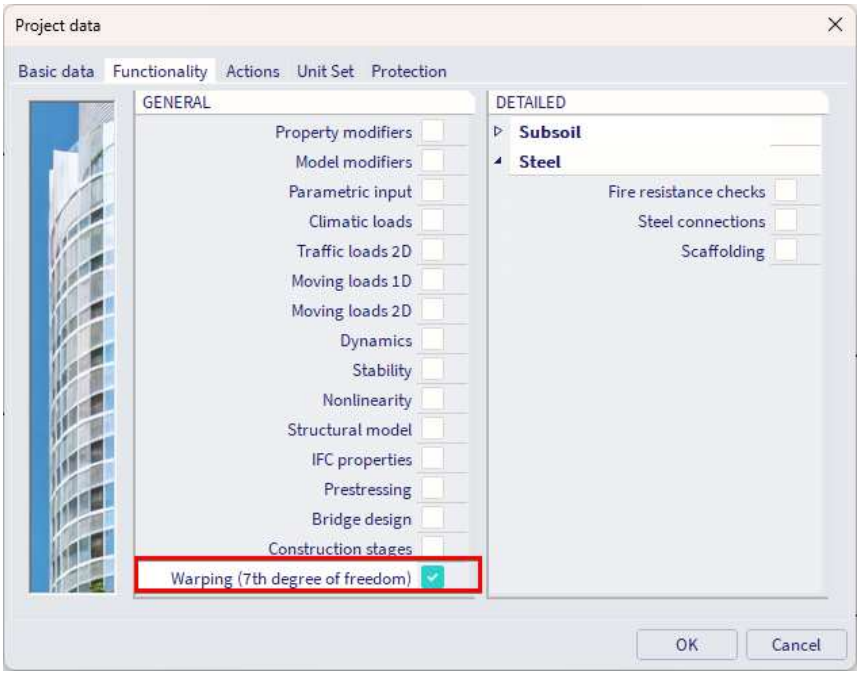
The total torsional moment T_{Ed} should be considered as the sum of two internal effects (formula (6.24)):

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

With:

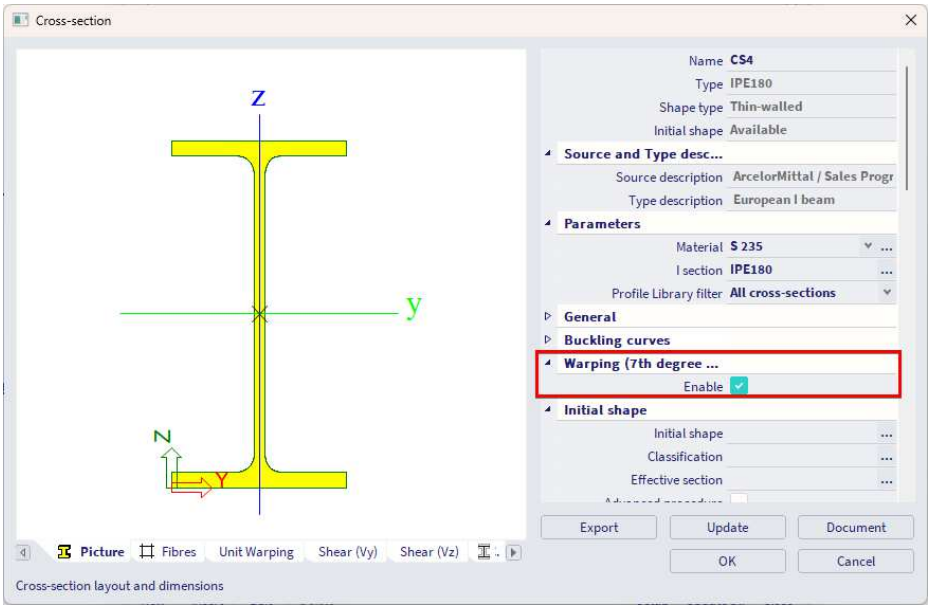
- $T_{t,Ed}$ the internal (primary) St. Venant torsion
- $T_{w,Ed}$ the internal (secondary) warping torsion

SCIA Engineer will take into account the St. Venant torsion automatically. If you would like to calculate also with **warping torsion**, you should activate the "**Warping (7th degree of freedom)**" functionality in the Project settings:



Note: For this functionality, available since SCIA Engineer 26, the Professional or Ultimate edition is needed (or module sens.07). The functionality only supports the 3D project environments "General XYZ" and "Frame XYZ".

For the cross-sections you want to take warping into account, you need to enable the **Warping** option in the Cross-section dialog:



Additional constraint properties related to warping are available for hinges on 1D members and nodal supports.
For hinges this restraint can be defined as "Free" (no transition of the warping), "Flexible" (stiffness needs to be defined) or "Rigid" (the transition condition between the warping deformation ω of two nodes of the corresponding hinge is defined through parameter **"transfer coefficient k"**).
For nodal supports the warping restraint can be defined as "Free" (no transition of the warping), "Flexible" (stiffness needs to be defined) or "Rigid".

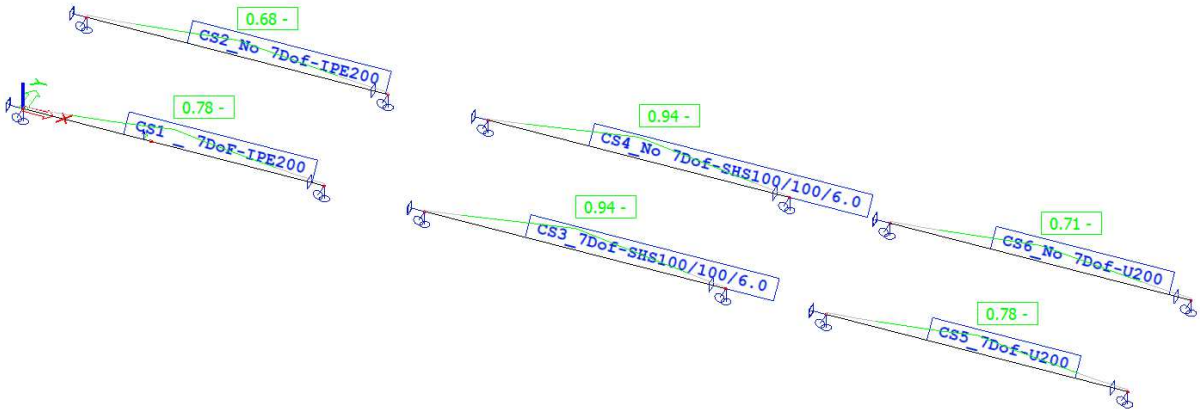
Example: **Warping.esa**

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 6 beams are inputted:

- IPE200 – warping (7 degrees of freedom) has been activated in the project settings, cross-section and supports/hinges
- IPE200 – no warping
- SHS100/100/6.0 – warping has been activated in the project settings, cross-section and supports/hinges
- SHS100/100/6.0 – no warping
- U200 – warping (7 degrees of freedom) has been activated in the project settings, cross-section and supports/hinges
- U200 – no warping

On all beams, a line force of -7,8 kN is inputted. For the I section and the SHS section an eccentricity ey of 0,100 m is added to initiate torsion. For the U section no eccentricity is added since the shear eccentricity will lead to torsion during the analysis.



For the **SHS profiles**, there is mainly no difference between warping activated or not. This was expected as warping is not important for closed hollow sections.

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

Member B2 - Torsion check IPE200 without warping activated

The critical check is on position 3.000 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-12.60	kN
Shear force	$V_{y,Ed}$	0.90	kN
Shear force	$V_{z,Ed}$	3.90	kN
Torsion	T_{Ed}	-0.09	kNm
Bending moment	$M_{y,Ed}$	11.70	kNm
Bending moment	$M_{z,Ed}$	2.70	kNm

Torsion check

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

Index of fibre	Fibre	2	
Total torsional moment	T_{Ed}	11.0	MPa
Elastic shear resistance	T_{Rd}	135.7	MPa
Unity check		0.08	-

$$\tau_{Ed} = \left| \frac{T_{Ed}}{T_{Ed,unit}} \times \tau_{Ed,unit} \right| = \left| \frac{-0.09[kNm]}{1.00[kNm]} \times 121774.408[kN/m^2] \right| = 11.0[MPa]$$

$$\tau_{Rd} = \frac{f_y}{\sqrt{3} \times \gamma_{M0}} = \frac{235.0[MPa]}{\sqrt{3} \times 1.00} = 135.7[MPa]$$

$$\text{Unity check} = \frac{\tau_{Ed}}{\tau_{Rd}} = \frac{11.0[MPa]}{135.7[MPa]} = \mathbf{0.08 \leq 1.00} \qquad \text{(EC3-1-1: 6.23)}$$

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		13	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	4.4	MPa
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	60.2	MPa
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	94.9	MPa
Total longitudinal stress	$\sigma_{tot,Ed}$	159.5	MPa
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0.0	MPa
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0.0	MPa
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	11.0	MPa
Total shear stress	$\tau_{tot,Ed}$	11.0	MPa
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	160.6	MPa
Unity check		0.68	-

$$\sigma_{N,Ed} = \frac{-N_{Ed}}{A} = \frac{-(-12.60[kN])}{2.8500 \cdot 10^{-3}[m^2]} = 4.4[MPa]$$

$$\sigma_{My,Ed} = \frac{M_{y,Ed} \times z}{I_y} = \frac{11.70[kNm] \times 100[mm]}{1.9430 \cdot 10^{-5}[m^4]} = 60.2[MPa]$$

$$\sigma_{Mz,Ed} = \frac{M_{z,Ed} \times y}{I_z} = \frac{2.70[kNm] \times 50[mm]}{1.4230 \cdot 10^{-6}[m^4]} = 94.9[MPa]$$

$$\sigma_{tot,Ed} = \sigma_{N,Ed} + \sigma_{My,Ed} + \sigma_{Mz,Ed} + \sigma_{w,Ed} = 4.4[MPa] + 60.2[MPa] + 94.9[MPa] + 0.0[MPa] = 159.5[MPa]$$

$$\tau_{Vy,Ed} = \left| \frac{V_{y,Ed}}{V_{y,Ed,unit}} \times \tau_{Vy,Ed,unit} \right| = \left| \frac{0.90[kN]}{1.00[kN]} \times 0.000[kN/m^2] \right| = 0.0[MPa]$$

$$\tau_{Vz,Ed} = \left| \frac{V_{z,Ed}}{V_{z,Ed,unit}} \times \tau_{Vz,Ed,unit} \right| = \left| \frac{3.90[kN]}{1.00[kN]} \times 0.000[kN/m^2] \right| = 0.0[MPa]$$

$$\tau_{t,Ed} = \left| \frac{T_{t,Ed}}{M_{xp,Ed,unit}} \times \tau_{t,Ed,unit} \right| = \left| \frac{-0.09[kNm]}{1.00[kNm]} \times 121774.408[kN/m^2] \right| = 11.0[MPa]$$

$$\tau_{tot,Ed} = \tau_{Vz,Ed} + \tau_{Vy,Ed} + \tau_{t,Ed} + \tau_{w,Ed} = 0.0[MPa] + 0.0[MPa] + 11.0[MPa] + 0.0[MPa] = 11.0[MPa]$$

$$\sigma_{von\ Mises,Ed} = \sqrt{\sigma_{tot,Ed}^2 + 3 \times \tau_{tot,Ed}^2} = \sqrt{159.5[MPa]^2 + 3 \times 11.0[MPa]^2} = 160.6[MPa]$$

$$\text{Unity check} = \frac{\sigma_{von\ Mises,Ed}}{\frac{f_y}{\gamma_{M0}}} = \frac{160.6[MPa]}{\frac{235.0[MPa]}{1.00}} = \mathbf{0.68 \leq 1.00} \qquad \text{(EC3-1-1: 6.1)}$$

Member B1 - Torsion check IPE200 with warping activated

The critical check is on position 3.000 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-12.60	kN
Shear force	$V_{y,Ed}$	0.90	kN
Shear force	$V_{z,Ed}$	3.90	kN
Torsion	T_{Ed}	-0.09	kNm
St. Venant torsion	$T_{t,Ed}$	0.00	kNm
Warping torsion	$T_{w,Ed}$	-0.09	kNm
Bimoment	B_{Ed}	0.06	kNm^2
Bending moment	$M_{y,Ed}$	11.70	kNm
Bending moment	$M_{z,Ed}$	2.70	kNm

You can notice that the torsional moment T_{Ed} is split into a primary torsional moment (St. Venant torsion) $T_{t,Ed}$ and a secondary torsional moment (warping torsion) $T_{w,Ed}$.

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)

Elastic verification			
Fibre		13	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	4.4	MPa
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	60.2	MPa
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	94.9	MPa
Normal stress due to the bimoment M_w	$\sigma_{w,Ed}$	23.2	MPa
Total longitudinal stress	$\sigma_{tot,Ed}$	182.7	MPa
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0.0	MPa
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0.0	MPa
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0.0	MPa
Warping influence	$\tau_{w,Ed}$	0.0	MPa
Total shear stress	$\tau_{tot,Ed}$	0.0	MPa
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	182.7	MPa
Unity check		0.78	-

$$\sigma_{N,Ed} = \frac{-N_{Ed}}{A} = \frac{-(-12.60[kN])}{2.8500 \cdot 10^{-3}[m^2]} = 4.4[MPa]$$

$$\sigma_{My,Ed} = \frac{M_{y,Ed} \times z}{I_y} = \frac{11.70[kNm] \times 100[mm]}{1.9430 \cdot 10^{-5}[m^4]} = 60.2[MPa]$$

$$\sigma_{Mz,Ed} = \frac{M_{z,Ed} \times y}{I_z} = \frac{2.70[kNm] \times 50[mm]}{1.4230 \cdot 10^{-6}[m^4]} = 94.9[MPa]$$

$$\sigma_{w,Ed} = \frac{-B_{Ed}}{M_{w,Ed,unit}} \times \sigma_{w,Ed,unit} = \frac{-0.06[kNm^2]}{1.00[kNm^2]} \times (-368606.973[kN/m^2]) = 23.2[MPa]$$

$$\sigma_{tot,Ed} = \sigma_{N,Ed} + \sigma_{My,Ed} + \sigma_{Mz,Ed} + \sigma_{w,Ed} = 4.4[MPa] + 60.2[MPa] + 94.9[MPa] + 23.2[MPa] = 182.7[MPa]$$

$$\tau_{Vy,Ed} = \left| \frac{V_{y,Ed}}{V_{y,Ed,unit}} \times \tau_{Vy,Ed,unit} \right| = \left| \frac{0.90[kN]}{1.00[kN]} \times 0.000[kN/m^2] \right| = 0.0[MPa]$$

$$\tau_{Vz,Ed} = \left| \frac{V_{z,Ed}}{V_{z,Ed,unit}} \times \tau_{Vz,Ed,unit} \right| = \left| \frac{3.90[kN]}{1.00[kN]} \times 0.000[kN/m^2] \right| = 0.0[MPa]$$

$$\tau_{t,Ed} = \left| \frac{T_{t,Ed}}{M_{xp,Ed,unit}} \times \tau_{t,Ed,unit} \right| = \left| \frac{0.00[kNm]}{1.00[kNm]} \times 121774.408[kN/m^2] \right| = 0.0[MPa]$$

$$\tau_{w,Ed} = \left| \frac{T_{w,Ed}}{M_{xs,Ed,unit}} \times \tau_{w,Ed,unit} \right| = \left| \frac{-0.09[kNm]}{1.00[kNm]} \times 0.000[kN/m^2] \right| = 0.0[MPa]$$

$$\tau_{tot,Ed} = \tau_{Vz,Ed} + \tau_{Vy,Ed} + \tau_{t,Ed} + \tau_{w,Ed} = 0.0[MPa] + 0.0[MPa] + 0.0[MPa] + 0.0[MPa] = 0.0[MPa]$$

$$\sigma_{von\ Mises,Ed} = \sqrt{\sigma_{tot,Ed}^2 + 3 \times \tau_{tot,Ed}^2} = \sqrt{182.7[MPa]^2 + 3 \times 0.0[MPa]^2} = 182.7[MPa]$$

$$\text{Unity check} = \frac{\sigma_{von\ Mises,Ed}}{\frac{f_y}{\gamma_{M0}}} = \frac{182.7[MPa]}{\frac{235.0[MPa]}{1.00}} = \mathbf{0.78 \leq 1.00} \qquad \text{(EC3-1-1: 6.1)}$$

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

4.7. 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 according to formula (6.41):

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

in which α and β are defined as follows:

- I and H sections: α = 2; β = 5n but β ≥ 1
- Circular hollow sections: α = 2; β = 2
- Rectangular hollow sections: $\alpha = \beta = \frac{1,66}{1-1,13n^2}$ but α = β ≤ 6

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

In case of one of the following, the influence of the shear force on the bending resistance cannot be accounted for using the specified article:

- No plastic shear resistance is available because an elastic shear check was done, which means the reduction factor ρ cannot be determined.
- Due to extreme shear the reduction factor ρ > 1 which would lead to a negative reduction.
- In case there is no corresponding bending moment the reduction for shear cannot be applied (for example V_z combined with M_z and thus no corresponding M_y).

In each of those cases an elastic verification using the yield criterion according to art. 6.2.1(5) (formula (6.1)) will be done instead.

The values $M_{N,y,Rd}$ and $M_{N,z,Rd}$ depend on the moment resistance, reduced with a factor depending on "*n*", the check of the normal force (formulas (6.39) and (6.40)):

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

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

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 according to formula (6.29):

$$(1-\rho)\cdot f_y$$

Where:

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

and:

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

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 a shear force, for class 3 cross-sections the maximum longitudinal stress should satisfy the criterion according to formula (6.42):

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

Class 4 cross-sections (EN 1993-1-1 article 6.2.9.3):

In absence of a shear force, for class 4 cross-sections the maximum longitudinal stress should satisfy the criterion according to formula (6.43):

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

The following criterion according to formula (6.44) should be met:

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

With:

- A_{eff} effective area of the cross-section when subjected to uniform compression
- $W_{eff,\,min}$ effective section modulus of the cross-section when subjected only to moment about the relevant axis
- e_N 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)

Cross-section area	A	1.8800e+04	mm²
Compression resistance	$N_{c,Rd}$	4418.00	kN
Unity check		0.04	-

Unity check (or $N_{Ed}/N_{c,Rd}$) = 0.04, so n = 0,04

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

Shear correction factor	η	1.20	
Shear area	A_v	9.4086e+03	mm²
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	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)

Shear correction factor	η	1.20	
Shear area	A_v	1.1389e+04	mm²
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	1545.22	kN
Unity check		0.07	-

The unity check for shear force is smaller than 0,5, thus there is 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)

Plastic bending moment	$M_{pl,y,Rd}$	1200,85	kNm
Exponent of bending ratio y	α	2,00	
Plastic bending moment	$M_{pl,z,Rd}$	148,28	kNm
Exponent of bending ratio z	β	1,00	

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

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{5,1100 \cdot 10^6[mm^3] \times 235,0[MPa]}{1,00} = 1200,85[kNm] \quad \text{(EC3-1-1: 6.13)}$$

$$\alpha = 2,00$$

$$M_{pl,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{6,3100 \cdot 10^5[mm^3] \times 235,0[MPa]}{1,00} = 148,28[kNm] \quad \text{(EC3-1-1: 6.13)}$$

$$\beta = 1,00$$

$$\text{Unity check} = \left(\frac{|M_{y,Ed}|}{M_{pl,y,Rd}}\right)^{\alpha} + \left(\frac{|M_{z,Ed}|}{M_{pl,z,Rd}}\right)^{\beta} = \left(\frac{|-116,71[kNm]|}{1200,85[kNm]}\right)^{2,00} + \left(\frac{|-0,06[kNm]|}{148,28[kNm]}\right)^{1,00} = \mathbf{0,01 \leq 1,00} \quad \text{(EC3-1-1: 6.41)}$$

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.

Chapter 5: ULS stability check

The stability check can be found in SCIA Engineer under **Menu bar > Design > Steel members** > **ULS Check** or **Process toolbar > Steel > Steel ULS check**. In the properties of the check, you can choose to look at the **Brief**, **Summary** or **Detailed** output, as explained in the previous chapter.
In below the stability check will be explained and the detailed results are shown.

5.1. Classification

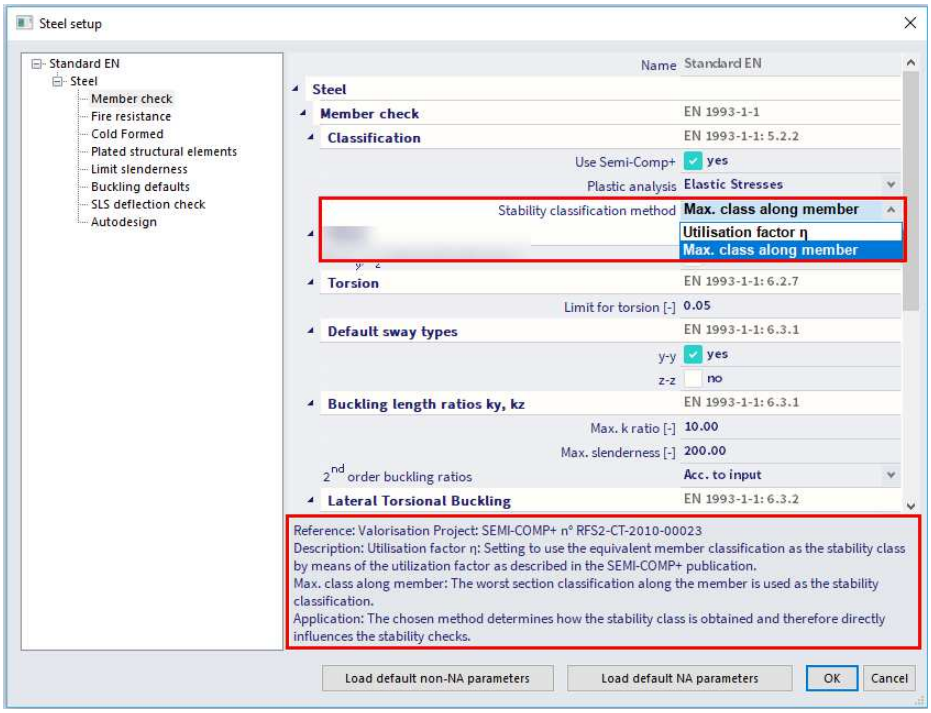
The classification can be different for the section check and for the stability check.

Classification in the section check: 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.

For the **classification in the stability check** there are two approaches possible:

- Maximum class along the member
- Utilisation factor η (eta) (since SCIA Engineer 21.0)

You can define the stability classification method in the steel settings:



5.1.1. Maximum class along the member

For each load case/combination, the classification for member buckling design is determined as the maximum class along the member. This class is used to perform the stability check since stability effects are related to the whole member and not to a single cross-section.

To determine this critical classification, all sections in the Ly and Lz system lengths of the **buckling system** are checked and the **worst classification is used** as the critical. Note that only sections on the actual member are used so in case the system length spans multiple members, only the sections of the actual member are used to determine the critical classification.

For non-prismatic sections, the stability section classification is determined for each intermediary section.

5.1.2. Utilisation factor η

The utilisation factor η is in essence a value that reflects how much the section is utilized. The stability classification is taken as the section classification at the section with the maximum utilisation factor η. This is also called as the equivalent section class for member buckling design.

Notes:

- *This approach leads to a more economic design of steel structures as the stability classification is based on the section with the highest utilisation in contrary to the previous approach which conservatively takes the worst classification along the member.*
- *This implementation has been verified with benchmarks based on all the worked examples provided by Ref.[36] on p. 141-193. With one minor remark in which worked example 7 of Ref.[40] contains wrong values for η due to mixing up steel grades (S355 & S235) between the classification and individual checks.*
- *Generating the steel code check with utilisation factor instead of max. class along member can take more time, especially in case the determination of the utilisation factor requires an iterative approach.*

Utilisation factor calculation via direct formulas:

The utilisation factor η can be derived via direct formulas in certain use cases.

- Only single section checks are used:
  Basically if the only section checks are single section checks such as single bending and/or compression, meaning no combined section checks are present. In such cases the utilisation factor η can be determined as follows depending which single section check has the highest unity check:
  - maximum section check is coming from the compression check
    $$\eta = \mathrm{UC_{N-}}$$
  - maximum section check is coming from the bending M_y check
    $$\eta = \mathrm{UC_{M_y}}$$
  - maximum section check is coming from the bending M_z check
    $$\eta = \mathrm{UC_{M_z}}$$
- Certain linear combined section checks are used:
  - linear summation (eq. 6.2 of EN1993-1-1):
    $$\eta = \left(\frac{|\mathrm{N_{Ed}}|}{\mathrm{N_{Rd}}} + \frac{|\mathrm{M_{y,Ed}}|}{\mathrm{M_{V,y,Rd}}} + \frac{|\mathrm{M_{z,Ed}}|}{\mathrm{M_{V,z,Rd}}}\right)$$
  - maximum longitudinal stress in case of class 3 (eq. 6.42 of EN1993-1-1)
    $$\eta = \frac{\gamma_{\mathrm{M0}}}{\mathrm{f_y} \cdot (1 - \rho_{\mathrm{max}})} \cdot \left(\frac{-\mathrm{N_{Ed}}}{\mathrm{A_{eff}}} + \frac{\mathrm{M_{y,Ed}} \cdot \mathrm{z}}{\mathrm{I_y}} + \frac{\mathrm{M_{z,Ed}} \cdot \mathrm{y}}{\mathrm{I_z}}\right)$$
  - maximum longitudinal stress in case of class 4 (eq. 6.43 of EN1993-1-1)
    $$\eta = \frac{\gamma_{\mathrm{M0}}}{\mathrm{f_y} \cdot (1 - \rho_{\mathrm{max}})} \cdot \left(\frac{|\mathrm{N_{Ed}}|}{\mathrm{A_{eff}}} + \frac{|\mathrm{M_{y,Ed}}|}{\mathrm{W_{eff,y}}} + \frac{|\mathrm{M_{z,Ed}}|}{\mathrm{W_{eff,z}}}\right)$$
  - linear effective summation in case of class 4 (eq. 6.44 of EN1993-1-1)
    $$\eta = \frac{\gamma_{\mathrm{M0}}}{\mathrm{f_y} \cdot (1 - \rho_{\mathrm{max}})} \cdot \left[\frac{|\mathrm{N_{Ed}}|}{\mathrm{A_{eff}}} + \frac{|\mathrm{M_{y,Ed}}| + |\mathrm{N_{Ed}}| \cdot \mathrm{e_{N_y}}}{\mathrm{W_{eff,y,min}}} + \frac{|\mathrm{M_{z,Ed}}| + |\mathrm{N_{Ed}}| \cdot \mathrm{e_{N_z}}}{\mathrm{W_{eff,z,min}}}\right]$$

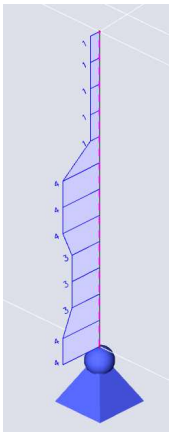
Utilisation factor calculation via an iterative approach:

An iterative approach is used for more complicated combined section checks in which a direct formula cannot be derived.

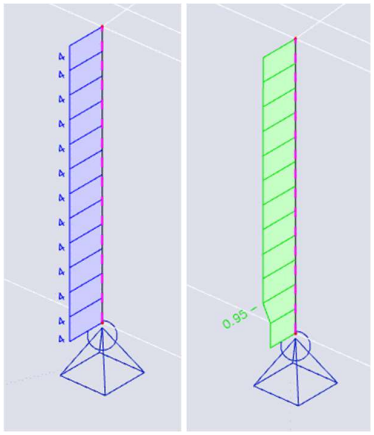
The iterative approach increases a set of internal forces (N, M_{yEd}, M_{zEd}) simultaneously in small steps until the point is reached where one of the section checks reaches a unity check of 1. The increase of internal forces happens by means of dividing the set of internal forces (N, M_{yEd}, M_{zEd}) with the same utilisation factor η.

Example: Industrial hall.esa

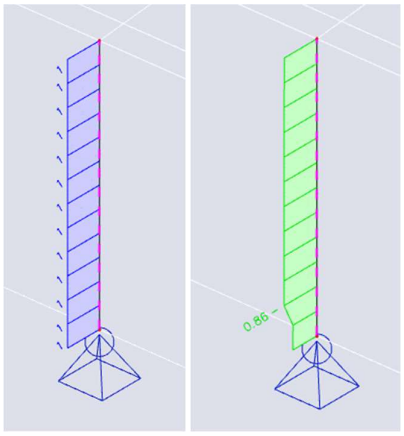
Consider column B28 (for combination CO1-ULS). When we have a look at the section class, we can see that a part of the column is classified as class 1, a part as class 3 and a part as class 4.



When we use **Maximum class along the member** as **stability classification method**, the worst section class (class 4) is taken as the stability class. This leads to an overall unity check of 0.95



We can calculate more economic by taking the **stability classification method** as **Utilisation factor η** in the **steel settings**. The decisive position for the stability classification is now based on the utilisation factor. The decisive position for stability classification is at 6.9 m (at the top of the column) and the decisive utilisation factor is 0.59. This leads to a stability class 1 and a more economic unity check of 0.86



5.2. Flexural buckling

The flexural buckling check will be executed following EN 1993-1-1 art. 6.3.1 formula (6.46):

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

$N_{b,Rd}$ is calculated with a reduction factor χ, which is depending on the buckling factor k. Since this buckling factor is playing an important role in this calculation, we will start with explaining this factor and we will focus on the flexural buckling check itself further in this chapter.

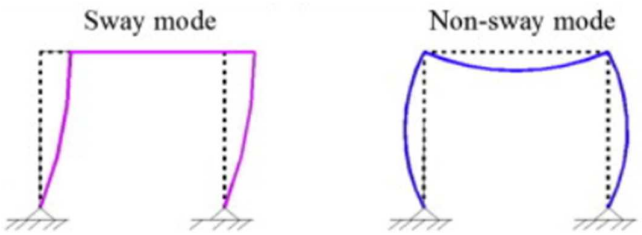
5.2.1. Buckling factors

In SCIA Engineer the calculation of the buckling coefficient k_y or k_z can be done automatically or you can set it manually.

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.

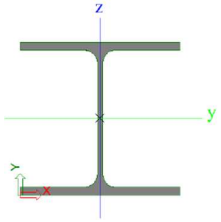
General method

The buckling coefficients ky and kz are, by default, automatically calculated by SCIA on the base of two approximate formulas respectively for sway and non-sway structures. These formulas produce buckling coefficients which are, respectively, larger (sway) and smaller (non-sway) than 1.

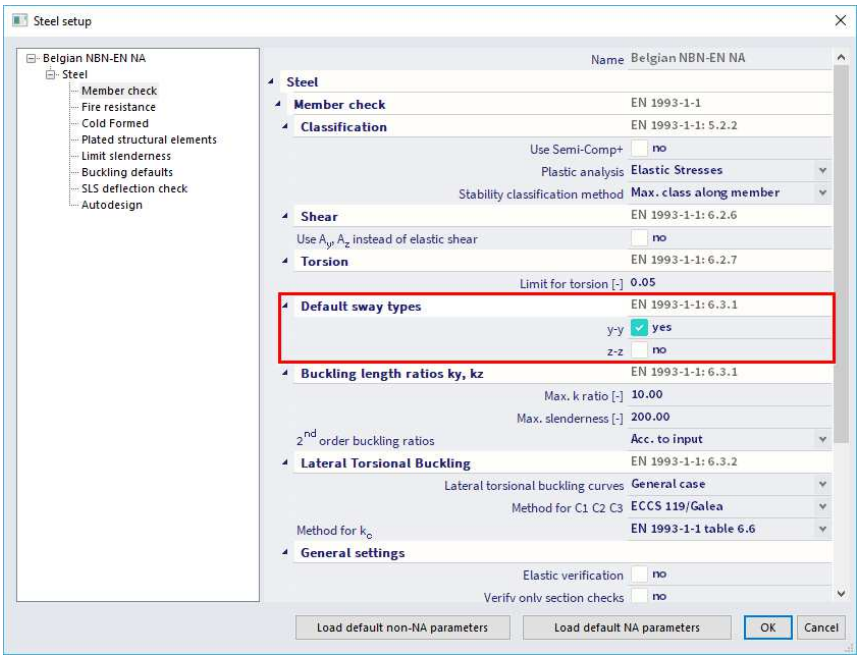


So it is important for this method that you choose the correct option sway or non-sway for the two local directions:

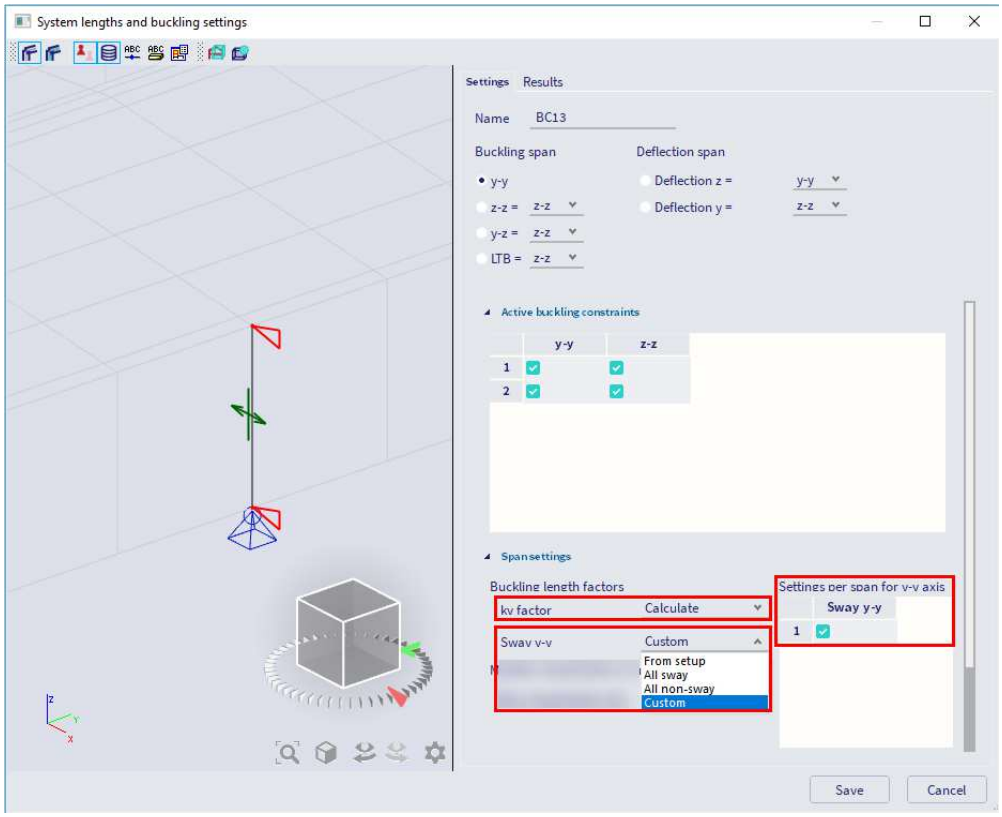
- y-y: buckling around the local y-axis (so deformation in the direction of the local z-axis)
- z-z: buckling around the local z-axis (so deformation in the direction of the local y-axis)



The option **sway** or **non-sway** can be chosen for the whole structure in the **steel settings**:



This can also be changed for each element separately, by using the system lengths and buckling settings. This can be found for each element via **Property panel > 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:

- **From 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 ≤ 1):

$$\frac{1}{L} = k = \frac{(\rho_1\rho_2 + 5\rho_1 + 5\rho_2 + 24)(\rho_1\rho_2 + 4\rho_1 + 4\rho_2 + 12)2}{(2\rho_1\rho_2 + 11\rho_1 + 5\rho_2 + 24)(2\rho_1\rho_2 + 5\rho_1 + 11\rho_2 + 24)}$$

- for a sway (flexible) structure (k > 1):

$$\frac{1}{L} = k = x\sqrt{\frac{\pi^2}{\rho_1 x} + 4}$$

With:

- $x = \frac{4\rho_1\rho_2 + \pi^2\rho_1}{\pi^2(\rho_1+\rho_2)+8\rho_1\rho_2}$
- $\rho_i = \frac{C_i L}{EI}$
- k buckling factor
- L system length
- E modulus of Young
- I moment of inertia
- $C_i = \frac{M_i}{\phi_i}$ stiffness in node i
- M_i moment in node i
- ϕ_i rotation in node 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 nonlinear, you 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:
  - local distributed loads qy=1 N/m and qz=-100 N/m on the beams
  - global distributed loads Qx =10000 N/m and Qy =10000 N/m on the columns
- load case 2:
  - local distributed loads qy=-1 N/m and qz=-100 N/m on the beams
  - global distributed loads Qx =-10000 N/m and Qy=-10000 N/m on the columns

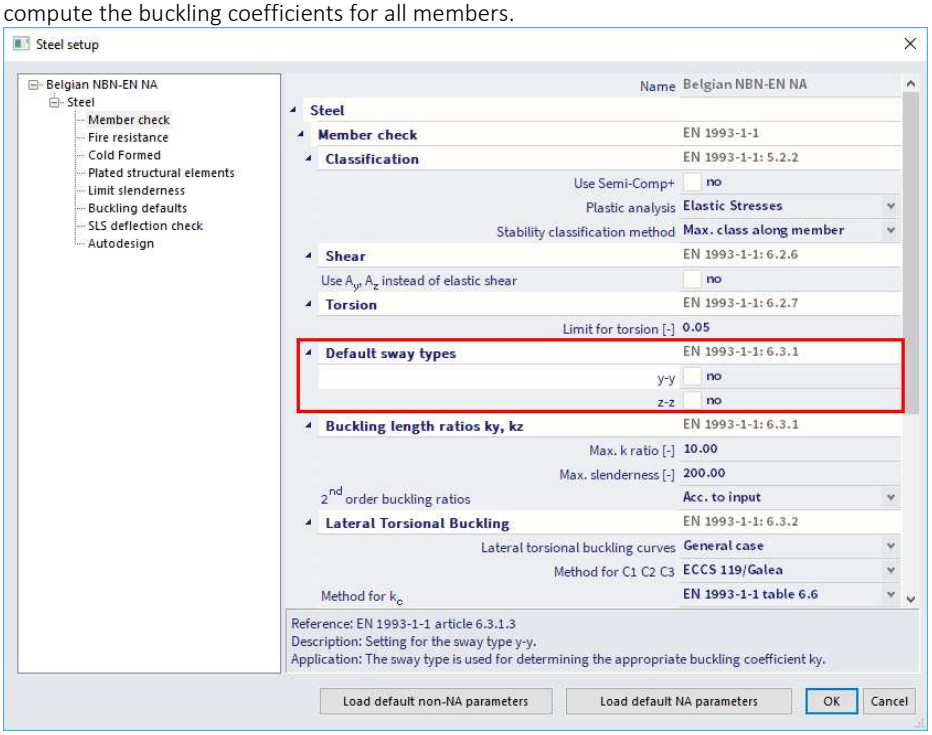
The used approach gives good results for frame structures with **perpendicular rigid or semi-rigid beam** connections. **For other cases, you must evaluate the presented bucking ratios**.

The concept of sway or non-sway structure is directly linked to the critical coefficient α_{cr} issue of the stability analysis (see for reference the ECCS 119):

- When $\alpha_{cr} \geq 10$, the structure is non-sway, therefore the buckling coefficients will be smaller than 1;
- When $\alpha_{cr} < 10$, the structure is sway, therefore the buckling coefficients will be larger than 1.

Hence, you should perform a stability analysis prior to setup the sway / non-sway parameter. In that way you can optimise better the buckling coefficients ky and kz than only with the linear analysis.

- If $\alpha_{cr} \geq 10$, you can simply perform a **linear analysis** specifying, in the **steel settings** window, the **non-sway** behaviour of the structure for the y-y and z-z directions. Then the non-sway formula will be used to compute the buckling coefficients for all members.



- In case $\alpha_{cr} < 10$, you can choose to perform a **linear analysis** using **sway buckling factors** (which are always larger than 1). This method results in a much simpler analysis with respect to a second order analysis accounting for global and local imperfections. However, you should carefully check that this possibility is allowed in the used design norm (for instance, the Belgian national annex of Eurocode currently does not allow this method, thus requiring a second order analysis). Also, note that this method is **more conservative** with respect to a second order calculation accounting for global and local imperfections.

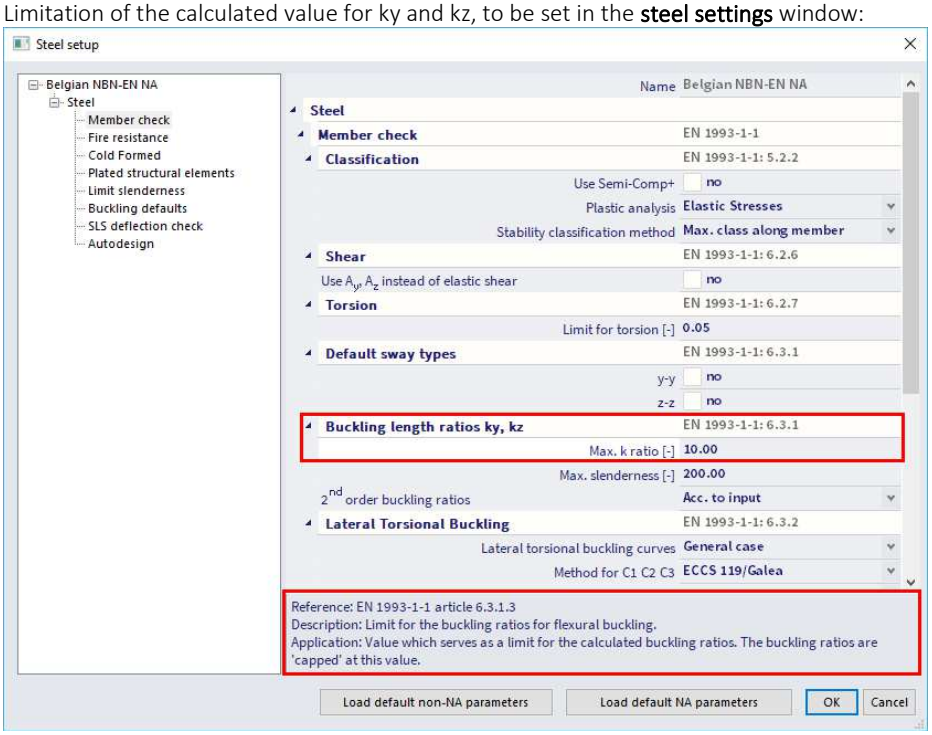
Note: sway and non-sway is not the same as unbraced and braced:

- If $\Psi_{br} > 0{,}2\ \Psi_{unbr}$: the frame is unbraced;
- If $\Psi_{br} \leq 0{,}2\ \Psi_{unbr}$: the frame is braced.

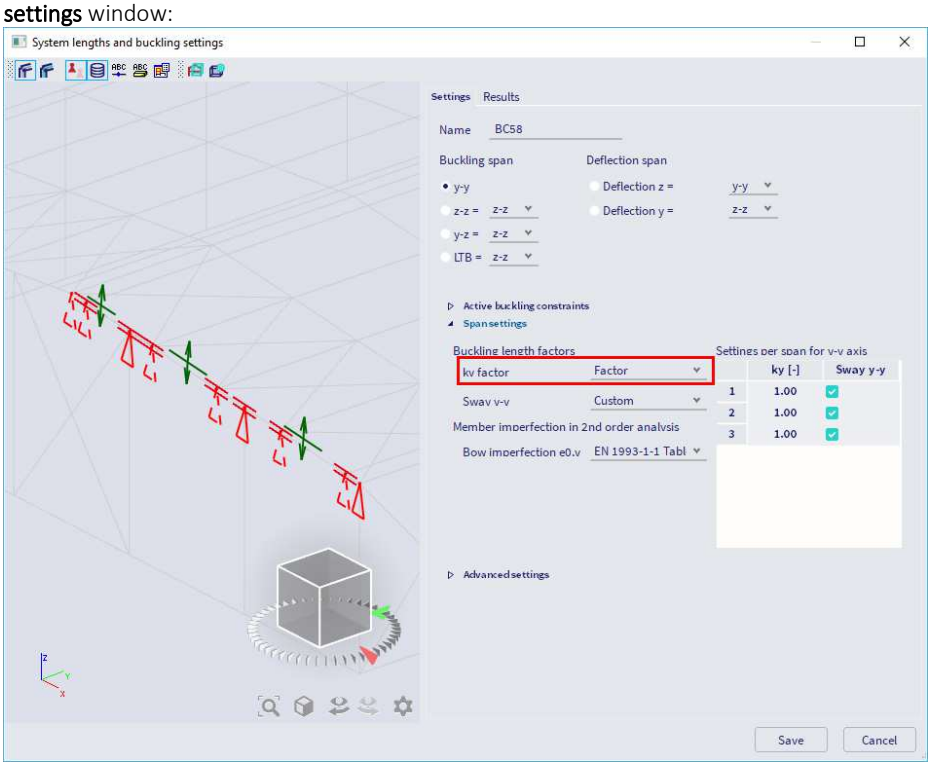
Ψ_{br} is the lateral flexibility of the structure with bracing system.

As said before, the formulas used for the computation of ky and kz are valid only in case of rigid and/or semi-rigid frame structures. This limitation implies that the ky and kz values automatically computed by SCIA should be critically checked when the application case is different from the one specified above. In such cases you have several possibilities:

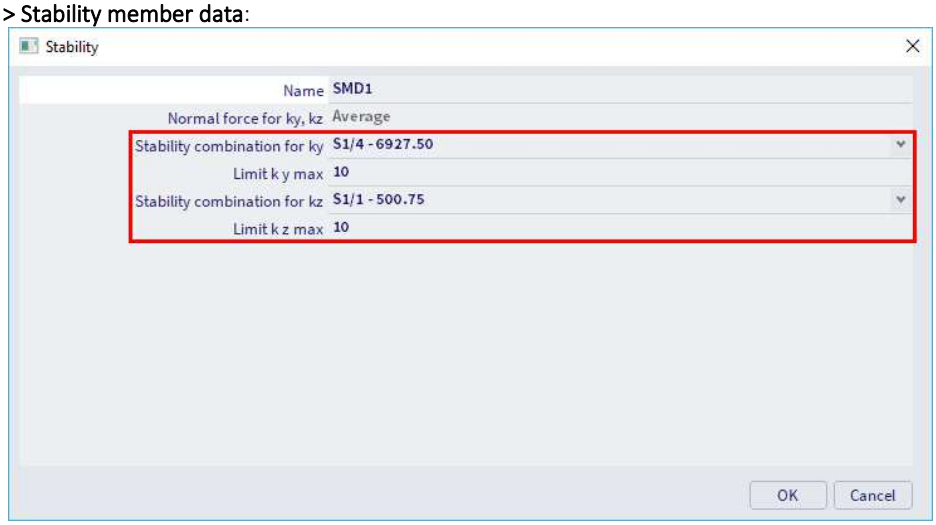
- Limitation of the calculated value for ky and kz, to be set in the **steel settings** window:



- Manual input of the buckling coefficient or buckling length, to be set in the **System lengths and buckling settings** window:



- Numerical computation of the buckling coefficient via a stability analysis.
  In this last case, a stability analysis should be performed, and the instability mode of the element for which the ky and kz are looked for should be retrieved. Once this has been done, these stability modes can be affected to the element via **Input panel > Steel > Stability member data** or **Process toolbar > Steel > Stability member data**:



Example: **Buckling Factor.esa**

Consider column B1:

- L = 4000mm
- Set as sway
- In node N1: M_y = 0 kNm → $C_2 = \rho_2 = 0$
- This node N1 defines ρ_2 because ρ_2 is always the smallest of the two.
- In node N2 for load case LC1:
  - M_{y1} = 79883 kNm
  - $\phi_1 = \phi_y$ = 1523,3 mrad
  - $C_1 = M_{y1}/\phi_1$ = 79883 kNm / 1523,3 mrad = 52,44 kNm/mrad = 5,244 x 10^{10} Nmm/rad
  - E = 210000 N/mm²
  - I_y = 162700000 mm⁴
  - $\rho_1 = \frac{C_i L}{EI} = \frac{5{,}244 \cdot \frac{10^{10}\,\text{Nmm}}{\text{rad}} \cdot 4000\,\text{mm}}{210000 \frac{\text{N}}{\text{mm}^2} \cdot 162700000\,\text{mm}^4} = 6{,}139$
  - $x = \frac{4\rho_1\rho_2 + \pi^2\rho_1}{\pi^2(\rho_1+\rho_2)+8\rho_1\rho_2} = \frac{4 \cdot 6{,}139 \cdot 0 + \pi^2 \cdot 6{,}139}{\pi^2(6{,}139+0)+8 \cdot 6{,}139 \cdot 0} = 1{,}0$
  - $k = x\sqrt{\frac{\pi^2}{\rho_1 x} + 4} = 1{,}0\sqrt{\frac{\pi^2}{6{,}369 \cdot 1{,}00} + 4} = 2{,}368$
  - $N_{cr} = \frac{\pi^2 EI}{k^2 L^2} = \frac{\pi^2 \cdot 210000\,\text{N/mm}^2 \cdot 162700000\,\text{mm}^4}{(2{,}368)^2 (4000)^2} = 3758575\,\text{N} = 3759\,\text{kN}$

Those values can also be found in SCIA Engineer: via **Menu bar > Design > Steel members > Slenderness** or **Process toolbar > Steel > Steel slenderness** the buckling length ky will be found:

Steel slenderness

Linear calculation

Member	CS Name	Part	Sway y / Sway z	Ly [m] / Lz [m]	ky [-] / kz [-]	Iy [m] / Iz [m]	Lam y [-] / Lam z [-]	Iyz [m]	l LTB [m]
B1	CS1	1	Yes	4.000	2.37	9.472	63.31	4.000	4.000
			Yes	4.000	1.00	4.000	105.61		

This value can also be found in the stability check through **Menu bar > Design > Steel members** > **ULS Check** or **Process toolbar > Steel > Steel ULS check** under the buckling parameters. And here also the critical normal force N_{cr} can be found:

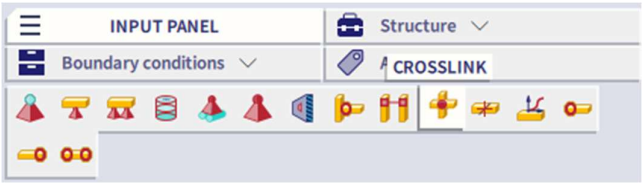
Buckling parameters		yy	zz	
Sway type		sway	sway	
System length	L	4.000	4.000	m
Buckling factor	k	2.37	1.00	
Buckling length	l_{cr}	9.472	4.000	m
Critical Euler load	N_{cr}	3758.82	1351.09	kN
Slenderness	λ	63.31	105.61	
Relative slenderness	λ_{rel}	0.67	1.12	
Limit slenderness	$\lambda_{rel,0}$	0.20	0.20	

Calculation of buckling factors for crossing diagonals

As the previous default method is only valid for perpendicular connections, this method cannot 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.

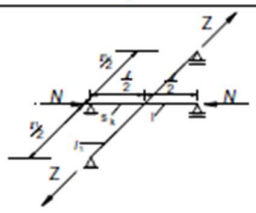
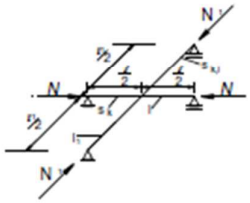
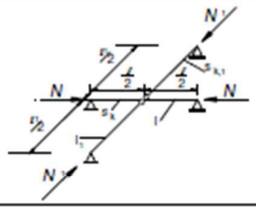
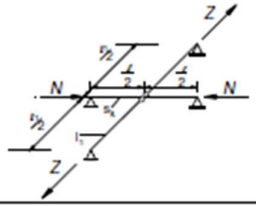
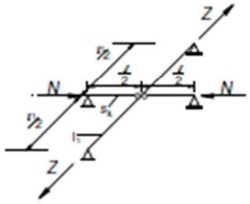
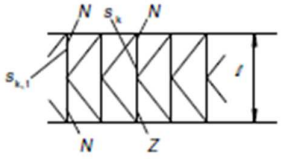
This method is only applicable for 2 diagonals with a hinged or rigid connection in the middle. To use this functionality in SCIA Engineer, you must connect the two diagonals with the option **Input panel > Structure > Boundary conditions > Crosslink**:



When connecting two beams with this option, in the **System lengths and buckling settings** window of the two elements you should set **Crossing diagonals** as special buckling system type (this option can be found under **Advanced settings**):



If this option is used, SCIA Engineer will use the method from EN1993-2 to calculate the buckling factors for the diagonals:

	1	2	3
1		$\beta = \sqrt{\dfrac{1-\dfrac{3}{4}\dfrac{Z\ell}{N\ell_1}}{1+\dfrac{I_1\ell^3}{I\ell_1^3}}}$ but $\beta \geq 0,5$	
2		$\beta = \sqrt{\dfrac{1+\dfrac{N_1\ell}{N\ell_1}}{1+\dfrac{I_1\ell^3}{I\ell_1^3}}}$ but $\beta \geq 0,5$	$\beta_1 = \sqrt{\dfrac{1+\dfrac{N\ell_1}{N_1\ell}}{1+\dfrac{I\ell_1^3}{I_1\ell^3}}}$ but $\beta_1 \geq 0,5$
3		continuous compression members $\beta = \sqrt{1+\dfrac{\pi^2}{12}\dfrac{N_1\ell}{N\ell_1}}$	hinged compression members $\beta_1 = 0,5$ when $EI \geq \dfrac{N_1\ell^3}{\pi^2\ell_1}\left(\dfrac{\pi^2}{12}+\dfrac{N\ell_1}{N_1\ell}\right)$
4		$\beta = \sqrt{1-0,75\dfrac{Z\ell}{N\ell_1}}$ but $\beta \geq 0,5$	
5		$\beta = 0,5$ when $\dfrac{N\ell_1}{Z\ell} \leq 1$ or when $EI_1 \geq \dfrac{3Z\ell_1^2}{4\pi^2}\left(\dfrac{N\ell_1}{Z\ell}-1\right)$	
6		$\beta = \left(0,75-0,25\left\lvert\dfrac{Z}{N}\right\rvert\right)$ but $\beta \geq 0,5$	$\beta_1 = \left(0,75-0,25\dfrac{N_1}{N}\right)$ $N_1 < N$

With:

- β buckling length factor
- 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 modules of Young (elastic modulus)

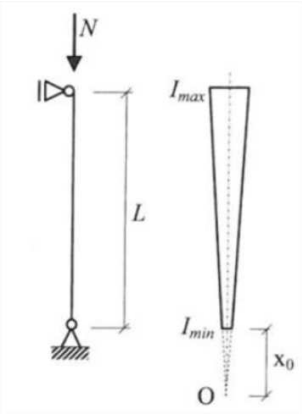
Calculation of the buckling length for a VARH element

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 y-y axis (strong axis), is equal to the member length.

For a VARH element we can define

- k_y buckling coefficient around the y-y-axis
- L_y system length around the y-y-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 modules of Young (elastic modulus)
- $N_{cr,y}$ critical Euler force around the y-axis



Hirt and Crisinel (Ref. [17]) presents expressions for the elastic critical load of axially loaded non-prismatic members of double symmetric cross-sections. Flexural buckling around the strong axis of the cross-sections occurs for:

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

Where:

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

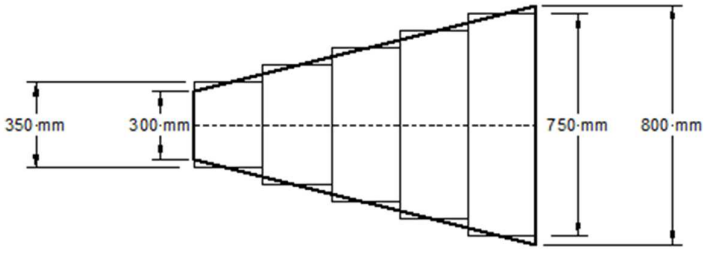
And C is a coefficient that depends on the parameter r, defined as the ratio between the minimum and the maximum moments of inertia.

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

For a tapered member, C can be calculated as:

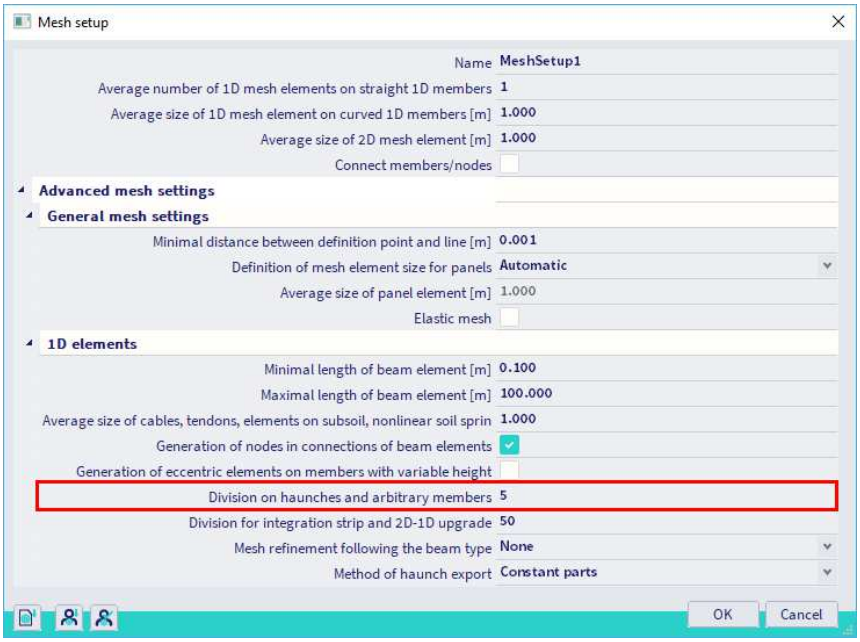
$$c = 0{,}08 + 0{,}92 \cdot r$$

Example: **VARH.esa**



Consider column B1: the VARH element going from 800 mm to 300 mm

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** window (**Menu bar > Tools > Calculation & mesh > Mesh settings**) :



$I_{y,min} = 1{,}7041 \cdot 10^8 mm^4$

$I_{y,max} = 1{,}6989 \cdot 10^9 mm^4$

$$r = \sqrt{\frac{I_{y,min}}{I_{y,max}}} = \sqrt{\frac{1{,}7041 \cdot 10^8 mm^4}{1{,}6989 \cdot 10^9 mm^4}} = 0{,}316711$$

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

$$I_{y,eq} = C * I_{y,max} = 0{,}3713 * 1{,}6989e^9 = 6{,}3093 \cdot 10^8 mm^4$$

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

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	4.27	1.00	
Buckling length	l_{cr}	17.075	4.000	m
Critical Euler load	N_{cr}	4485.32	7000.23	kN
Slenderness	λ	134.11	55.79	
Relative slenderness	λ_{rel}	1.43	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	$N_{b,Rd}$	914.27	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	$N_{b,Rd}$	914.27	kN
Unity check		0.77	-

5.2.2. 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 buckling length
- k buckling factor
- L system length

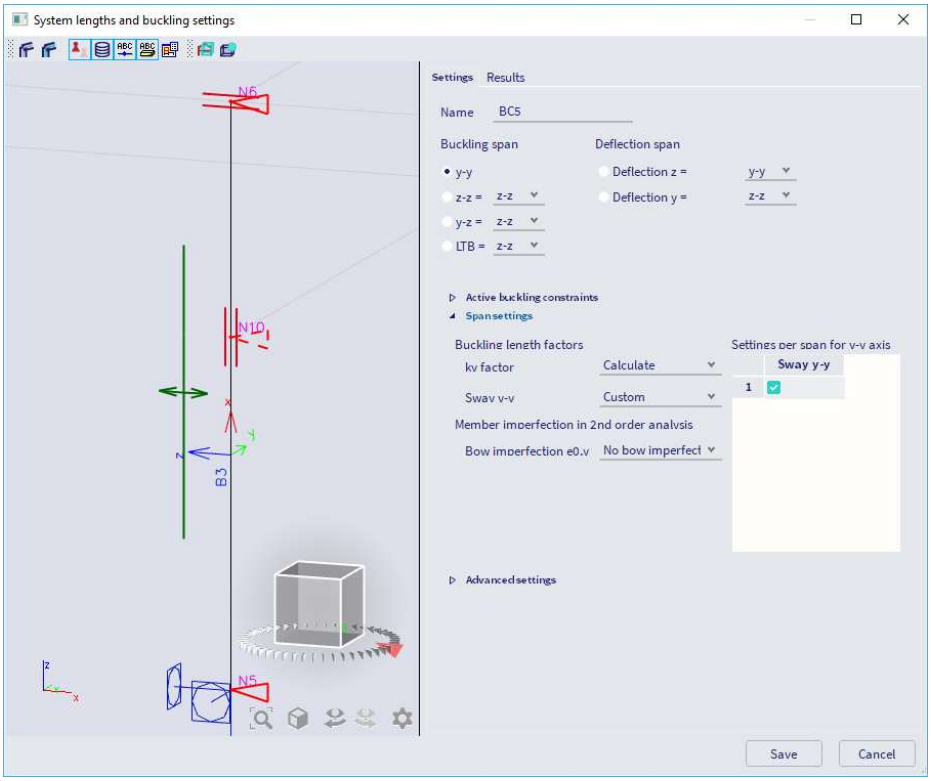
The system length of an element is defined by the property 'System lengths and buckling settings' of the element. It is the distance between the buckling supports.

The buckling length can be calculated with the buckling factor (which can be calculated automatically or manually inputted) or it can be given manually.

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 property panel for this column, you can choose for **System lengths and buckling settings**. With this option, the system length of the beam can be inputted. By 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.
- y-y 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 (y-y) 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
- z-z direction:
  - 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 (z-z) 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 (z-z) 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 3 m
  - Around the z-axis: the length between node N5 and N10 for the first part of the column (1,8 m) and the length between N10 and N6 for the second part of the column (1,2 m)
  - This can also be found via **Menu bar > Design > Steel members > Slenderness** or **Process toolbar > Steel > Steel slenderness**:

Steel slenderness

Linear calculation

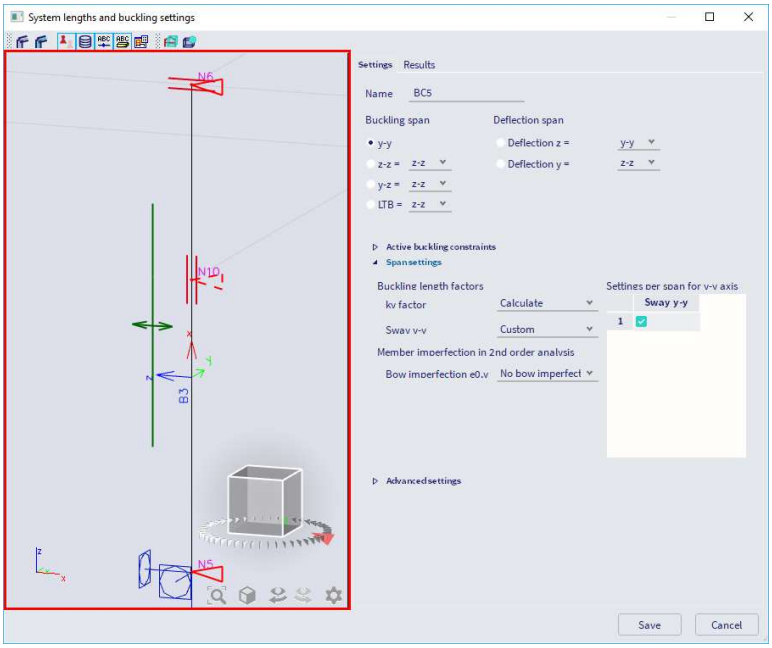
Member	CS Name	Part	Sway y / Sway z	Ly [m] / Lz [m]	ky [-] / kz [-]	ly [m] / lz [m]	Lam y [-] / Lam z [-]	lyz [m]	l LTB [m]
B3	CS3	1	Yes	3.000	1.09	3.270	18.51	1.800	1.800
			No	1.800	0.51	0.919	26.78		
B3	CS3	2	Yes	3.000	1.09	3.270	18.51	1.200	1.200
			No	1.200	0.57	0.680	19.83		

In this window you 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 **Menu bar > Libraries > Structure and analysis > Buckling groups** and click on **<New>** for creating a new buckling group or click on **<Edit>** to modify an existing buckling group.
- via the properties of a 1D-member: **Property panel > System lengths and buckling settings**.

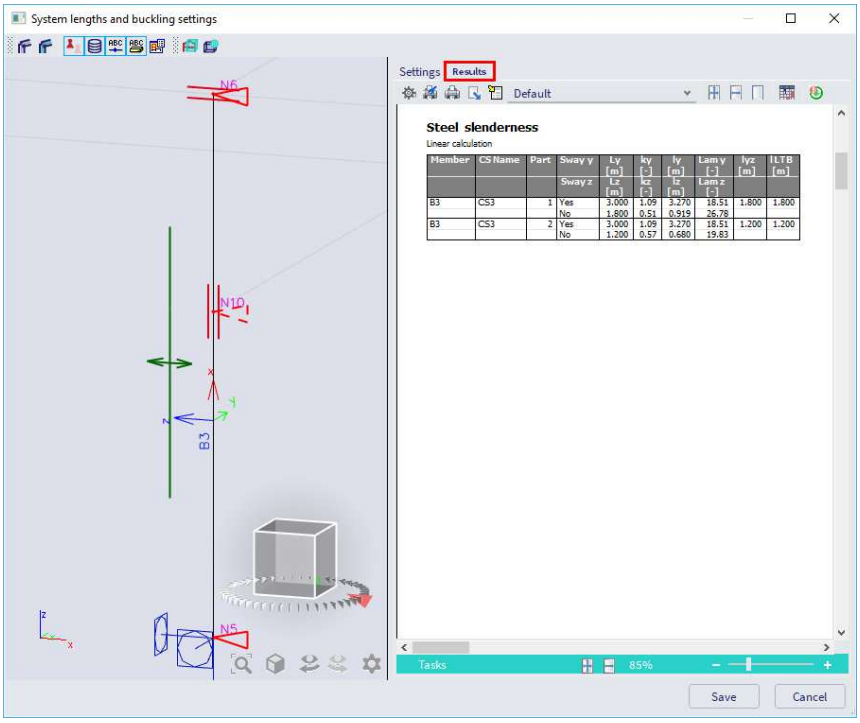


The left part of the dialog gives you a graphical representation of the 1D-members in the buckling system with their buckling constraints and information about the sway settings per span. It is not only a representation of the above mentioned settings, but it also allows **editing directly in that graphical 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, you 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 for the deflection spans deflection y, deflection z)
- Rectangular symbol (buckling span y-z, LTB (Lateral Torsional Buckling))

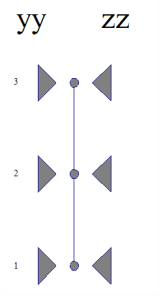
Besides this, with this new improved buckling settings, it is easier to access the results. You can see them by clicking on the **Results** tab of the system lengths and buckling settings window.



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

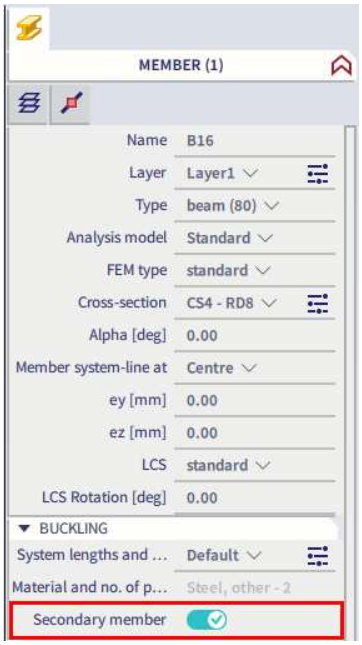
- When the local axes are exactly in the same direction (so in this case the local x-axis is in the same direction and the angle between the beams is exactly 180°).
- If no hinge has been inputted between the two beams.

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

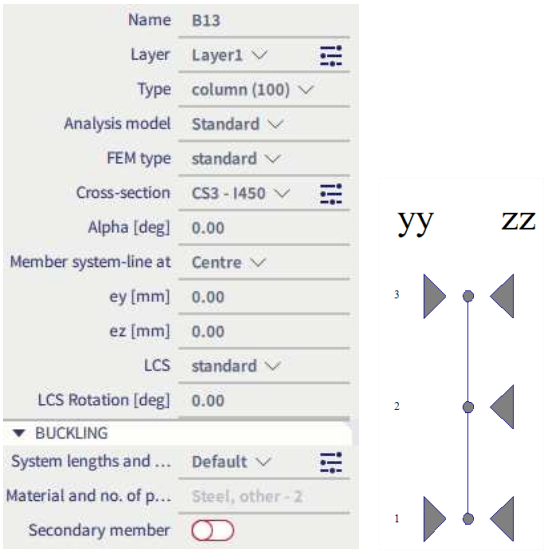


When looking at the rendered view, it will be clear that beam B16 is too weak to have an influence on the system length of beam B13. In SCIA Engineer there is possibility to exclude a beam from a buckling system.

Select beam B16 and go to 'Buckling' in its properties window. It is possible to indicate that beam B16 is a **secondary beam** and should not be considered 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:



5.2.3. Flexural buckling check

The flexural buckling check will be executed following EN 1993-1-1 art. 6.3.1 formula (6.46):

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

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:

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

With:

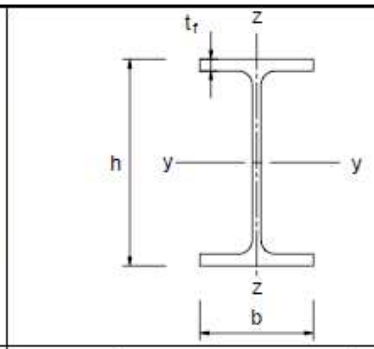
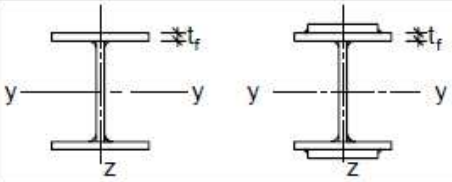
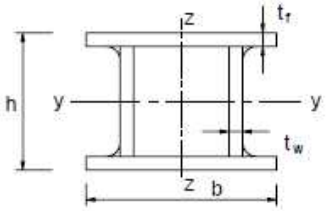
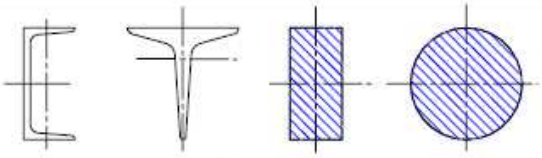
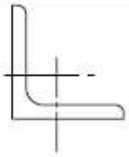
- $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$
- $\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$ for class 1, 2 or 3 cross-sections

  $\bar{\lambda} = \sqrt{\frac{A_{eff} \cdot f_y}{N_{cr}}}$ for class 4 cross-sections
- $N_{cr} = \frac{\pi^2 EI}{k^2 L^2}$ critical normal force (Euler force)
- α imperfection factor depending on the buckling curve:

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

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

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

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

Example: **Buckling Curves.esa**

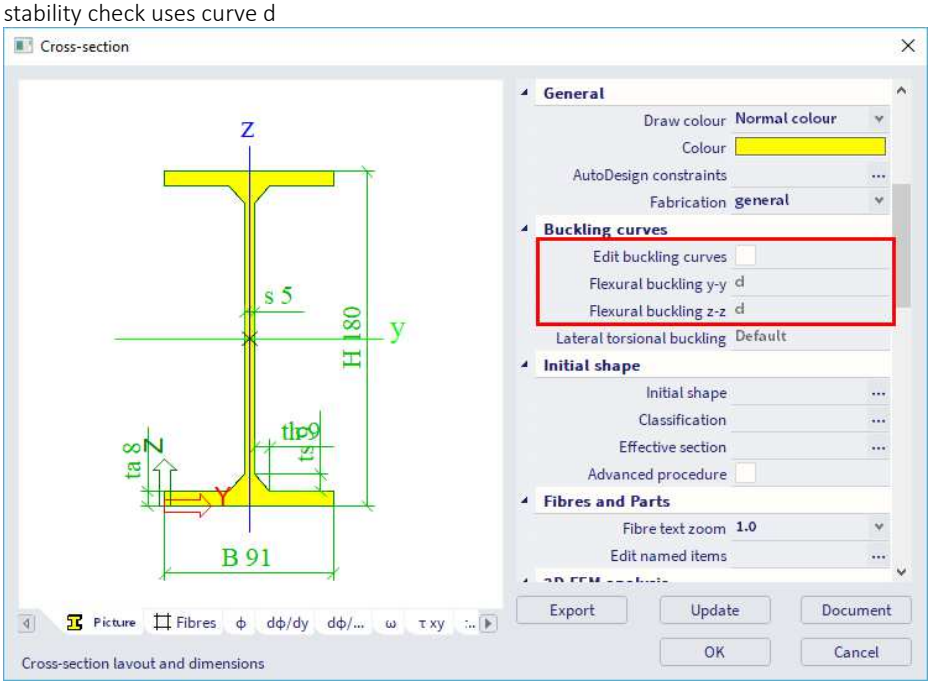
In Table 6.2 general buckling curves are given for the most common profiles. For some cross-section types, SCIA Engineer will automatically use these curves. The cross-section groups that are supported are:

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

For the other cross-section groups, 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 – IPE180
  - buckling curve a for y-y according to code
  - stability check uses curve a, according to code
- B2: CS2 – I form (Geometric shapes group)
  - non-standard cross-section: no buckling curve according to code. Buckling curve d is used.
  - stability check uses curve d



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. SCIA Engineer will first show the buckling parameters of this beam:

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

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	6,900	6,900	m
Buckling factor	k	3,58	0,87	
Buckling length	l_{cr}	24,716	5,978	m
Critical Euler load	N_{cr}	5635,44	3067,01	kN
Slenderness	λ	83,15	112,71	
Relative slenderness	λ_{rel}	0,89	1,20	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		a	b	
Imperfection	α	0,21	0,34	
Reduction factor	χ	0,74	0,48	
Buckling resistance	$N_{b,Rd}$	3284,63	2111,87	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 6,900 m, so the buckling length = 3,58 x 6,900 m = 24,716 m

With this buckling length the critical Euler load N_{cr} can be calculated. Afterwards the slenderness and the relative slenderness 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 be used in the following formula:

$$N_{b,y,Rd} = \frac{\chi_y \times A \times f_y}{\gamma_{M1}} = \frac{0,74 \times 1,8800 \cdot 10^4[mm^2] \times 235,0[MPa]}{1,00} = 3284,63[kN]$$

This result in a buckling resistance $N_{b,Rd}$ = 3284,63 kN for flexural buckling around the local y-axis.

The same principle can be repeated for flexural buckling around the local z-axis.

This will result in a lower buckling resistance: $N_{b,Rd}$ = 2111,87 kN.

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

Flexural Buckling verification			
Cross-section area	A	1,8800e+04	mm^2
Buckling resistance	$N_{b,Rd}$	2111,87	kN
Unity check		0,08	-

5.3. 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 torsional (-flexural) buckling check is executed according to EN 1993-1-1, art 6.3.1.4. In case of an RHS section or CHS section the check will not be executed and a note will be shown instead. In case of an I-section, when the check is not limiting it will not be printed and a note is shown instead. Not limiting is defined here as a unity check lower than the unity check for Flexural Buckling. In case however Flexural buckling can be ignored (due to low compression force or low slenderness) the comparison is done with the unity check of the compression check. The buckling curve for torsional (-flexural) buckling is taken as the z-z buckling curve. The value of the elastic critical load N_{cr} is taken as the smallest of $N_{cr,T}$ (Torsional buckling) and $N_{cr,TF}$ (Torsional-Flexural buckling). For doubly symmetric sections the elastic critical load N_{cr} is taken equal to $N_{cr,T}$.

The elastic critical load $N_{cr,T}$ for torsional buckling is calculated as Ref.[37]:

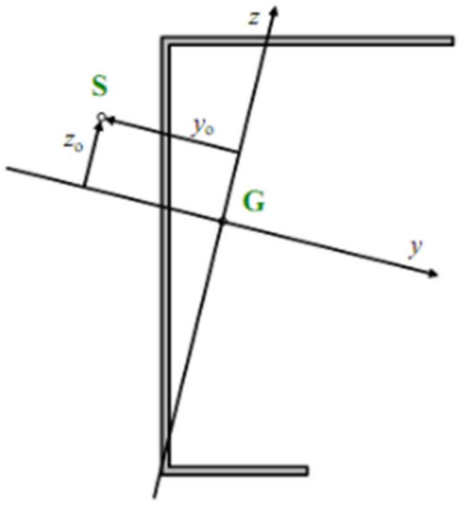
$$N_{cr,T} = \frac{1}{i_0^2}\left(GI_t + \frac{\pi^2 EI_w}{I_t^2}\right)$$

$$i_0^2 = i_y^2 + i_z^2 + y_0^2 + z_0^2$$

$$i_0^2(N - N_{cr,y})(N - N_{cr,z})(N - N_{cr,T}) - N^2 y_0^2(N - N_{cr,z}) - N^2 z_0^2(N - N_{cr,y}) = 0$$

With

- G shear modulus
- E modulus of elasticity
- i_y radius of gyration about the strong axis
- i_z radius of gyration about weak axis
- I_t torsion constant
- I_w warping constant
- I_T buckling length for the torsional buckling mode
- y_0 and z_0 coordinates of the shear centre with respect to the centroid
- $N_{cr,y}$ critical axial load for flexural buckling around the y-y axis
- $N_{cr,z}$ critical axial load for flexural buckling around the y-y axis
- $N_{cr,T}$ critical axial load for torsional buckling



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	-

$$N_{cr,T} = \frac{1}{i_0^2} \times \left(G \times I_t + \frac{\pi^2 \times E \times I_w}{l_{cr}^2} \right) = \frac{1}{125[mm]^2} \times \left(80769,2[MPa] \times 3,1000 \cdot 10^5[mm^4] + \frac{\pi^2 \times 210000,0[MPa] \times 5,5659 \cdot 10^{10}[mm^6]}{3,600[m]^2} \right)$$

$$= 2156,75[kN]$$

$$N_{cr,TF} = 1302,24[kN]$$

$$\lambda_{rel,T} = \sqrt{\frac{A \times f_y}{N_{cr}}} = \sqrt{\frac{5,3300 \cdot 10^3[mm^2] \times 235,0[MPa]}{1302,24[kN]}} = 0,98$$

$$\chi = \min\left(\frac{1}{\varphi + \sqrt{\varphi^2 - \lambda_{rel,T}^2}}; 1 \right) = \min\left(\frac{1}{1,17 + \sqrt{1,17^2 - 0,98^2}}; 1 \right) = \min(0,55; 1) = 0,55$$

$$N_{b,Rd} = \frac{\chi \times A \times f_y}{\gamma_{M1}} = \frac{0,55 \times 5,3300 \cdot 10^3[mm^2] \times 235,0[MPa]}{1,00} = 690,40[kN] \quad \text{(EC3-1-1: 6.47)}$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{b,Rd}} = \frac{|-100,00[kN]|}{690,40[kN]} = \mathbf{0,14 \leq 1,00} \quad \text{(EC3-1-1: 6.46)}$$

5.4. Lateral torsional buckling

5.4.1. Calculation of Mcr

General formula

The elastic critical moment for lateral torsional buckling M_{cr} is calculated by the following formula:

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

Where

- E — Young modulus of elasticity (E = 210000 N/mm² for steel)
- G — shear modulus (G = 80770 N/mm² for steel)
- k_{LT}*L — lateral torsional buckling length of the beam between points which have lateral restraint (= l_{LTB})
- I_w — warping constant
- I_t — torsional constant
- I_z — moment of inertia about the minor axis
- z_g — 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 — coordinate of the shear center

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

In SCIA Engineer there are different methods implemented for the calculation of those C-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 M_{cr} uses the following parameters:

- C_1, C_2, C_3: calculated according to ENV, ECCS or Lopez
- LTB length: k_{LT}*L
- k and k_w: factors related to the end fixity
- z_g: load position
- z_j: asymmetry of the section

More details about each parameter are given in separate chapters.

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

Calculation of C_1, C_2 and C_3 factors

Three C-parameters are used in the general formula for M_{cr}:

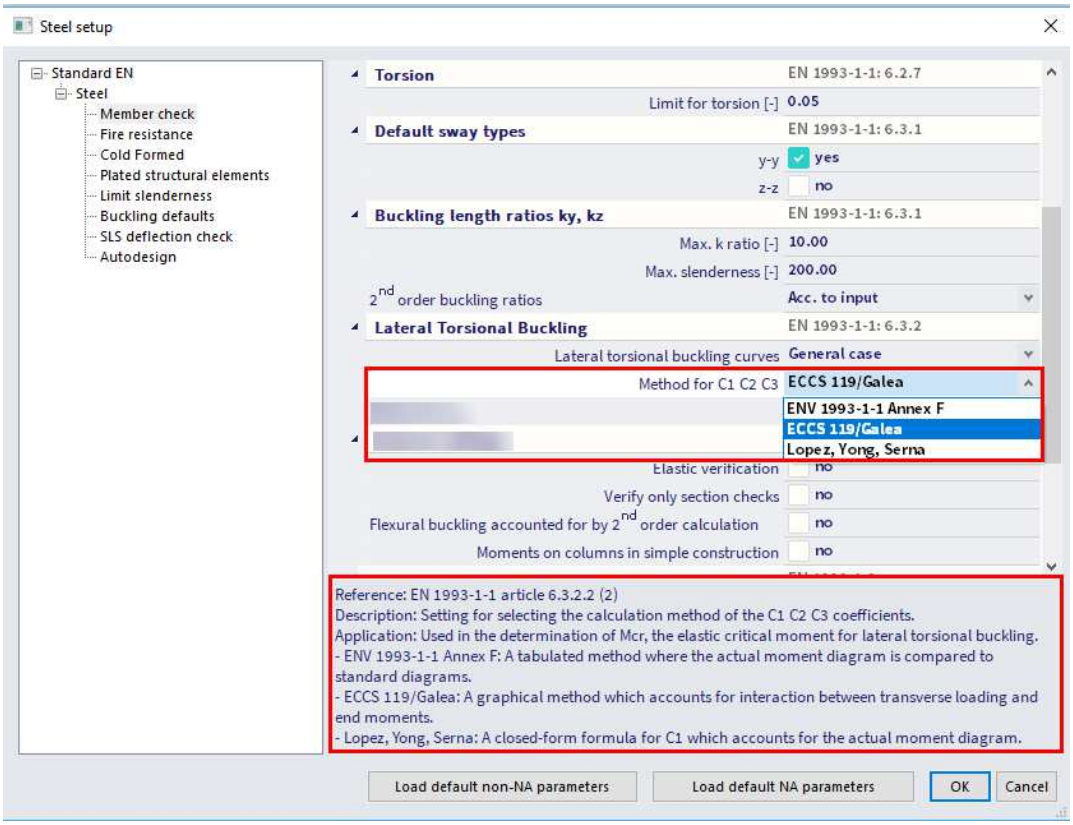
- C_1 takes into account the shape of the moment diagram
- C_2 takes into account the position of the loading
- C_3 takes into account the asymmetry of the cross-section

The coefficients **C_1**, **C_2** and **C_3** can be calculated in SCIA Engineer according to three different methods:

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

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

You can choose between those 3 methods in the steel settings:



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: C_1, C_2 and C_3) 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 reaches 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 C_1, C_2 and C_3.

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 C_1 and C_2 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. 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]).

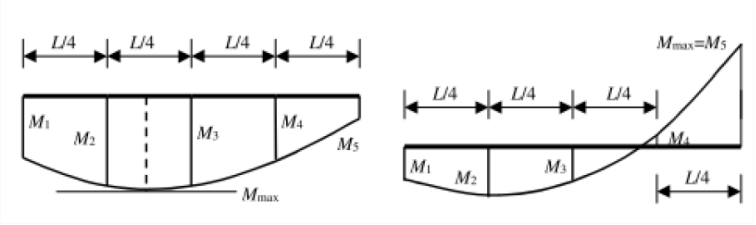
When using this method, the coefficients **C_2** and **C_3** are set to **zero**.

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
- $M_1 \rightarrow M_5$ 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} 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 settings 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_{LT}	12.000	m
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 centre distance	d_z	0.00	mm
Distance of load application	z_g	-245.00	mm
Mono-symmetry constant	β_y	0.00	mm
Mono-symmetry constant	z_j	0.00	mm

M_{cr} = 1576,03 kNm

ECCS 119/Galea

Mcr parameters			
LTB length	l_{LT}	12.000	m
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 centre distance	d_z	0.00	mm
Distance of load application	z_g	-245.00	mm
Mono-symmetry constant	β_y	0.00	mm
Mono-symmetry constant	z_j	0.00	mm

M_{cr} = 1118,50 kNm

Lopez, Yong, Serna

Mcr parameters			
LTB length	l_{LT}	12.000	m
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 centre distance	d_z	0.00	mm
Distance of load application	z_g	-245.00	mm
Mono-symmetry constant	β_y	0.00	mm
Mono-symmetry constant	z_j	0.00	mm

M_{cr} = 842,64 kNm

As you can see, there can be a big difference between the three calculation models.

In the method following 'Lopez, Yong, Serna' the values for C_2 and C_3 are always taken equal to zero.

When comparing the C_1 factors, the method following 'ECCS 119/Galea' and 'Lopez, Yong, Serna' are approximately the same (1,26 and 1,20 respectively), but the C_1 factor following the 'ENV 1993-1-1 Annex F' results in a total different value: 1,89.

When we would calculate M_{cr} more detailed with a lateral torsional buckling method which takes into account second order behaviour, we have 1169,66 kNm as result for M_{cr} on element B2. So the default choice for using ECCS 119/Galea to calculate the C-factors makes sense. Note that lateral torsional buckling method which takes into account second order behaviour is only supported for SCIA Engineer 22.0 or older (32-bit!).

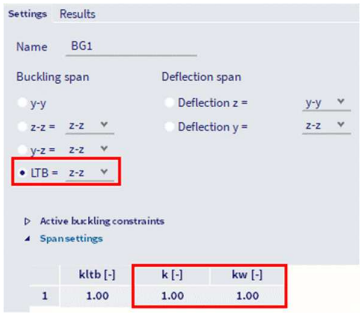
LTB parameters			
Method for LTB curve		General case	
Plastic section modulus	$W_{pl,y}$	3.9490e+06	mm^3
Elastic critical moment	M_{cr}	1169.66	kNm
Relative slenderness	$\lambda_{rel,LT}$	0.89	
Limit slenderness	$\lambda_{rel,LT,0}$	0.20	
LTB curve		a	
Imperfection	α_{LT}	0.21	
Reduction factor	χ_{LT}	0.74	
Design buckling resistance	$M_{b,Rd}$	686.74	kNm
Unity check		0.70	-

Note: The elastic critical moment M_{cr} is calculated using the FriLo BTII Solver.

k and kw factors

It is generally assumed that k = k_w = 1, which means that the ends of the element are not fixed. If the ends are fixed, values lower than one can be used (usually 0,5 for full fixity or 0,7 for one end fixed and one end free) and this would lead to bigger values of M_{cr}. You can adapt the values of k and k_w within the system lengths and buckling settings.

Select the member, then open the **System lengths and buckling settings** window from the property panel. Select **LTB** and in **Span settings** you can modify the values for k and k_w:



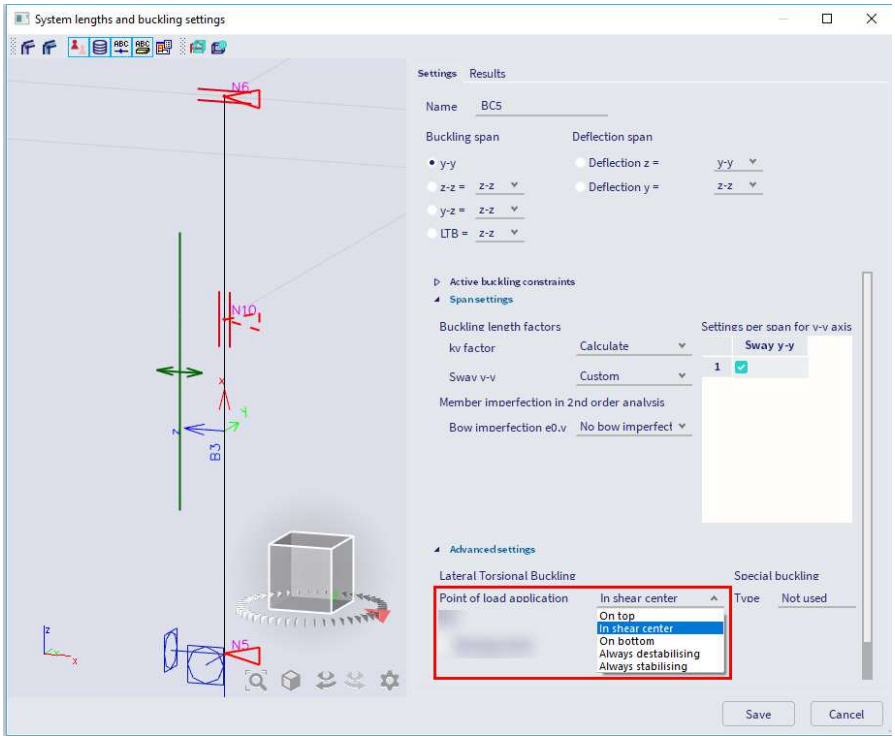
When using cantilever elements, these values are automatically changed by SCIA Engineer.

Load position

The load position has an influence on the calculation of M_{cr} through the value of z_g. You can choose among five load positions.

If you open system lengths and buckling settings, you can see an option called **Point of load application** under the **Advanced settings**. 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

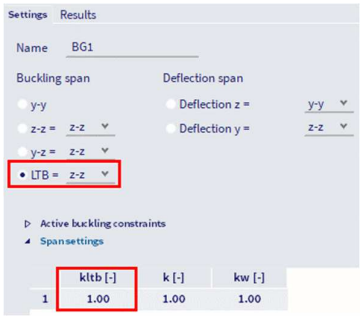


This value is set to **In shear center** by default and can be adapted to influence the value of M_{cr}.

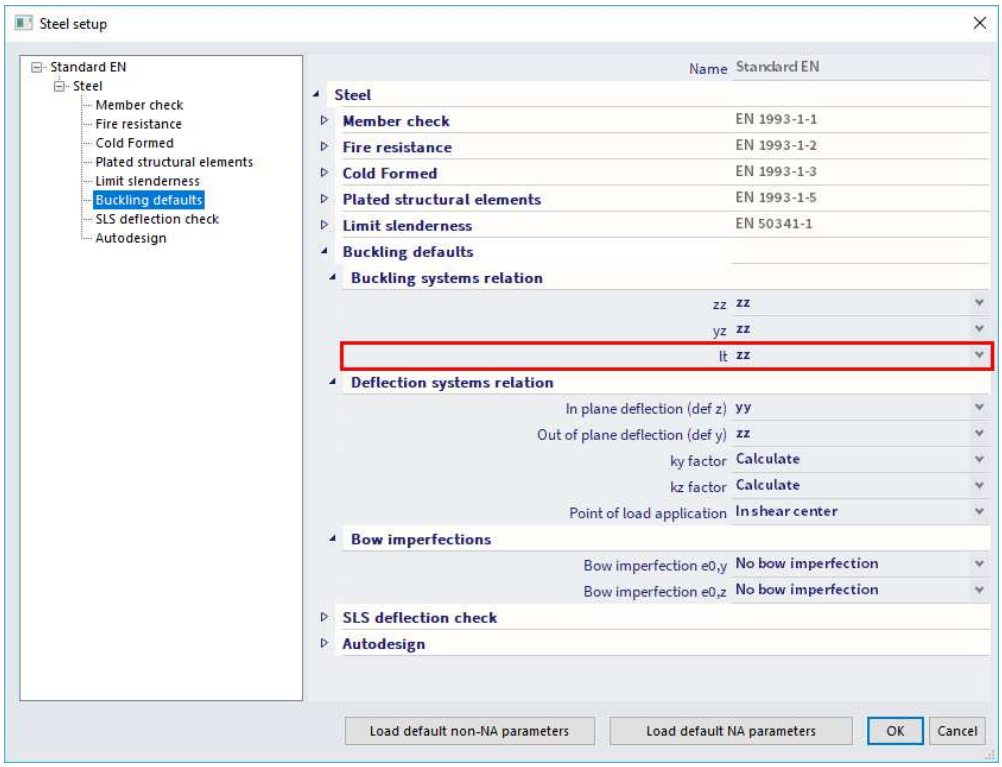
LTB length

LTB length is calculated as $l_{LTB} = k_{ltb} * L_{LTB}$

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:



L_{LTB} 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 the steel settings window:



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.

Dutch National Annex

The Dutch National Annex of EN 1993-1-1 describes another formula (NB.NB.2) to calculate M_{cr}.

$$M_{cr} = k_{red} \cdot \frac{C}{L_g} \cdot \sqrt{E \cdot I_z \cdot G \cdot I_t}$$

SCIA Engineer follows this method when the Dutch National Annex is used.

Example: Cfactors.esa

Open the project and select the Dutch National Annex in the Manager for National annexes and recalculate the project.

The steel check for beam B1 shows now the lateral torsional buckling check with M_{cr} calculated according to the Dutch National Annex.

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	$W_{pl,y}$	3.9490e+06	mm^3
Elastic critical moment	M_{cr}	1143.93	kNm
Relative slenderness	$\lambda_{rel,LT}$	0.90	
Limit slenderness	$\lambda_{rel,LT,0}$	0.20	
LTB curve		a	
Imperfection	α_{LT}	0.21	
Reduction factor	χ_{LT}	0.73	
Design buckling resistance	$M_{b,Rd}$	680.68	kNm
Unity check		0.71	-

Mcr parameters			
LTB length	l_{LT}	12.000	m
Fork length	L_g	12.000	m
Influence of load position		stabilising	
Factor	a	575.00	
Reduction factor	k_{red}	1.00	
Coefficient	C	5.89	
Factor	S	2177.97	mm
LTB moment factor	C_1	1.27	
LTB moment factor	C_2	0.51	

Note: M_{cr} has been calculated according to the Dutch NEN-EN NA.

The Dutch National Annex also mentions conditions, which need to be fulfilled to follow this method:

Bijlage NB.NB (normatief) Het kritisch elastische kipmoment

NB.NB.1 Toepassingsgebied

(1) De in deze bijlage gegeven rekenregels voor het kritieke elastische kipmoment zijn van toepassing voor:

— dubbel-symmetrische I-vormige doorsneden waarbij de belasting niet hoger dan 0,1 maal de hoogte van het profiel boven de bovenflens aangrijpt;

— buisprofielen waarvoor geldt: $h / b > 3$, waarbij de belasting niet hoger dan 0,1 maal de hoogte van het profiel boven de bovenflens aangrijpt.

OPMERKING Voor buisprofielen waarvoor geldt $h / b \leq 3$ mag zijn aangenomen dat ze niet gevoelig zijn voor kipinstabiliteit.

(2) Het toepassingsgebied van de in deze bijlage gegeven rekenregels is beperkt tot staven die zijn begrensd door gaffels of zijn opgelegd met onderflensinklemmingen die voldoen aan het gestelde in 6.3.2.5 en waarbij de verhouding van de lengte van de staaf tussen de gaffels L_s en de hoogte h van het profiel groter is dan of gelijk is aan 5 ofwel: $L_s / h \geq 5$.

(3) Als geldt $L_s / h < 5$, dan moet dezelfde toetsingsprocedure zijn gevolgd als wanneer $\alpha > 5\,000$. Hiervoor geldt NB.NB.4.2(3).

When one of the conditions is not fulfilled, SCIA will give a warning message (warning W30) and the default Eurocode method will be executed.

5.4.2. Lateral torsional buckling check

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:

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

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
- α imperfection factor 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:

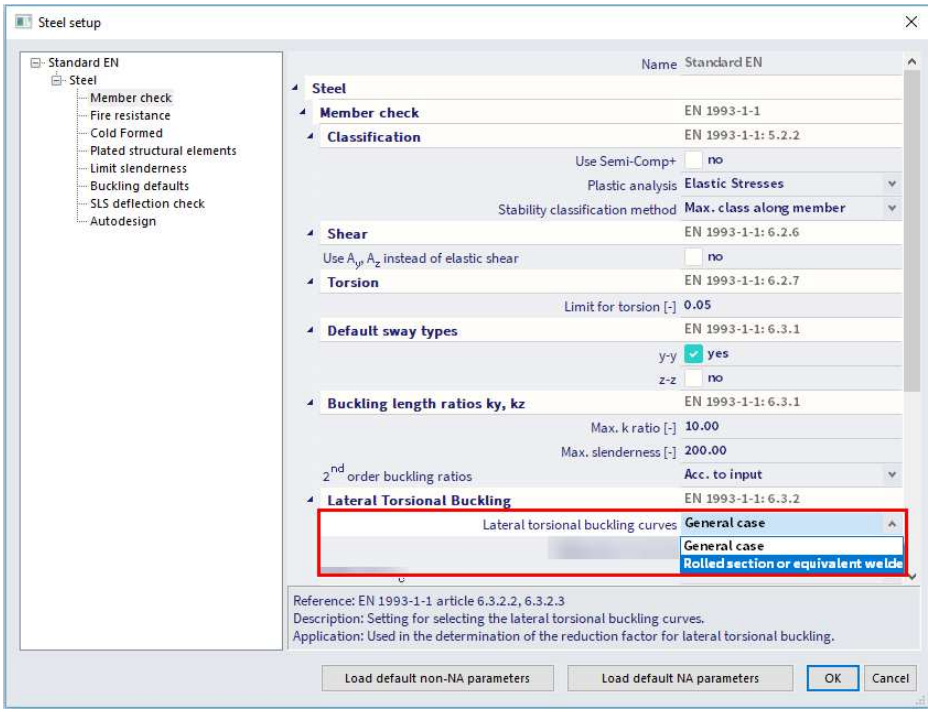
Cross-section	Limits	Buckling curve
Rolled I-sections	h/b ≤ 2 h/b > 2	**a** **b**
Welded I-sections	h/b ≤ 2 h/b > 2	**c** **d**
Other cross-sections	-	**d**

When slenderness $\bar{\lambda}_{LT} \leq 0{,}2$ or 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. The theory in ECCS - N° 119 clearly specifies that it is valid only for I-sections or sections with comparable shape.

This distinction can also be chosen in SCIA Engineer through the steel settings and is set for all the members in the project:



With this option, the reduction factor for lateral torsional buckling is calculated a bit differently:

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

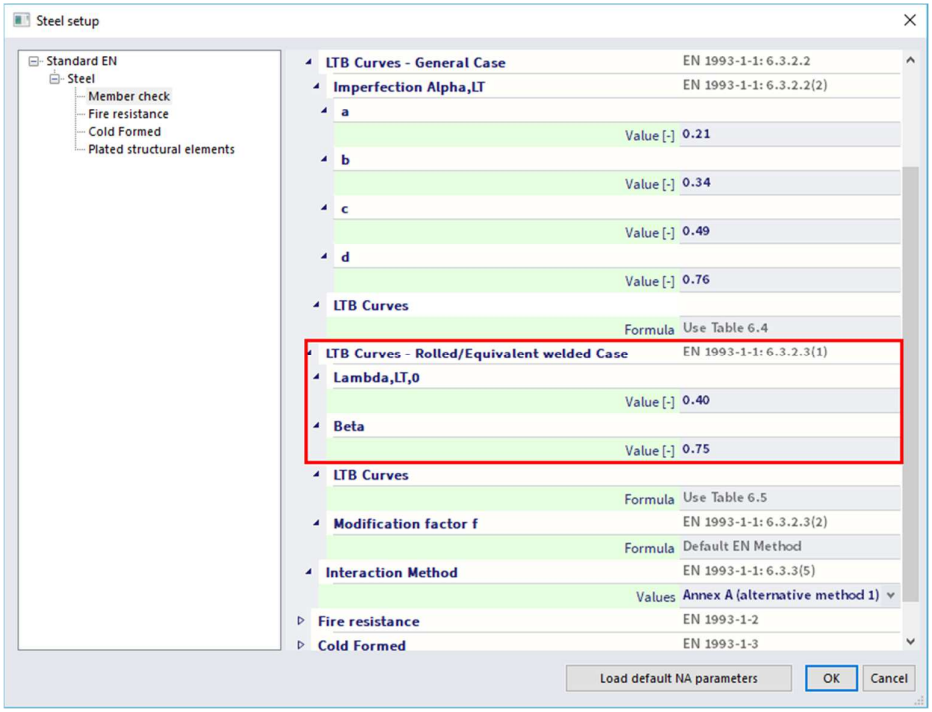
With

- $\Phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}\right) + \beta\bar{\lambda}_{LT}^2\right]$
- $\bar{\lambda}_{LT,O}$= 0,4 (maximum value)
- β= 0,75 (minimum value)
-

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

When slenderness $\bar{\lambda}_{LT} \leq \bar{\lambda}_{LT,O}$ or for $\frac{M_{Ed}}{M_{cr}} \leq \bar{\lambda}_{LT,O}^2$ lateral torsional buckling effects may be ignored.

Following EN 1993-1-1 article 6.3.2.3: $\bar{\lambda}_{LT,0}$= 0,4 but this value can be adapted in the national annex of a country in SCIA Engineer (also the value for β can be changed here):



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

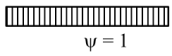
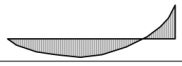
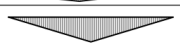
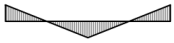
$$\chi_{LT,mod} = \frac{\chi_{LT}}{f}$$

Parameter 'f' may be defined in the National Annex. The following minimum values are recommended:

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

With k_c by default taken from the next table:

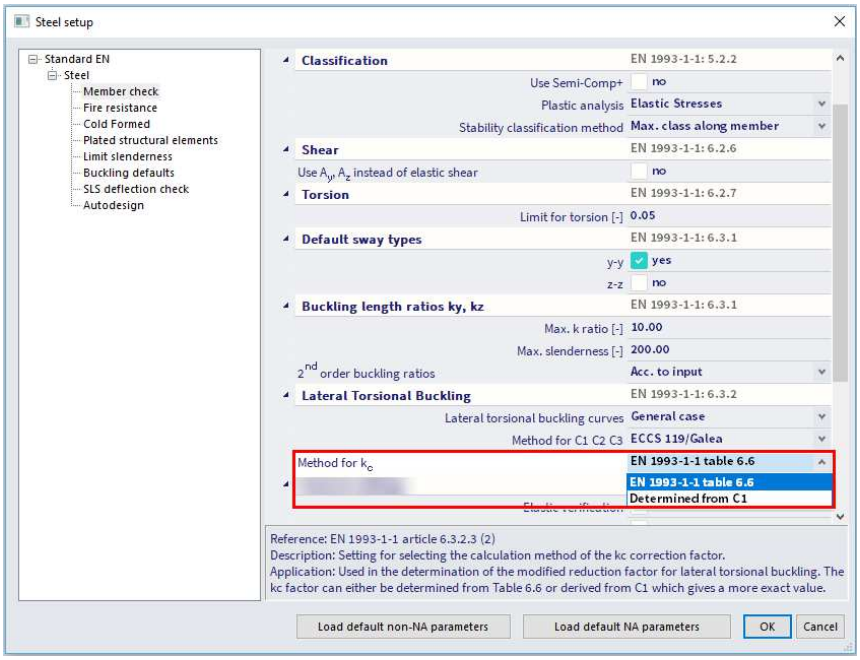
Table 6.6: Correction factors k_c

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

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

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

In SCIA Engineer you 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) via the steel settings:



We will demonstrate this LTB check in SCIA Engineer with following example.

Example: **Industrial hall.esa**

Consider beam B141.

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		0,15	-

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.

M_{cr} parameters			
LTB length	l_{LT}	6,000	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 centre 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

$$M_{cr} = C_1 \times \frac{\pi^2 \times E \times I_z}{l_{LT}^2} \times \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \times \frac{I_w}{I_z} + \frac{l_{LT}^2 \times G \times I_t}{\pi^2 \times E \times I_z} + (C_2 \times z_g - C_3 \times z_j)^2} - (C_2 \times z_g - C_3 \times z_j) \right] = 1,13$$

$$\times \frac{\pi^2 \times 210000,0[MPa] \times 2,4800 \cdot 10^6[mm^4]}{6,000[m]^2}$$

$$\times \left[\sqrt{\left(\frac{1,00}{1,00}\right)^2 \times \frac{2,5514 \cdot 10^{10}[mm^6]}{2,4800 \cdot 10^6[mm^4]} + \frac{6,000[m]^2 \times 80769,2[MPa] \times 1,9700 \cdot 10^5[mm^4]}{\pi^2 \times 210000,0[MPa] \times 2,4800 \cdot 10^6[mm^4]} + (0,45 \times 0,00[mm] - 0,53 \times 0,00[mm])^2} - (0,45 \times 0,00[mm] - 0,53 \times 0,00[mm]) \right]$$

$$= 56,14[kNm]$$

$$\lambda_{rel,LT} = \sqrt{\frac{W_{pl,y} \times f_y}{M_{cr}}} = \sqrt{\frac{3,6380 \cdot 10^5[mm^3] \times 235,0[MPa]}{56,14[kNm]}} = 1,23$$

$$\lambda_{rel,T} = 0,43 - 0,29 \times \lambda_{rel,LT} = 0,43 - 0,29 \times 1,23 = 0,07$$

$$\lambda_{rel,EXTRA} = \lambda_{rel,LT} + \lambda_{rel,T} = 1,23 + 0,07 = 1,31$$

$$\chi_{LT} = \min\left(\frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^2 - \lambda_{rel,EXTRA}^2}}; 1\right) = \min\left(\frac{1}{1,47 + \sqrt{1,47^2 - 1,31^2}}; 1\right) = \min(0,47; 1) = 0,47 \quad \text{(EC3-1-1: 6.56)}$$

$$M_{b,Rd} = \chi_{LT} \times W_{pl,y} \times \frac{f_y}{\gamma_{M1}} = 0,47 \times 3,6380 \cdot 10^5[mm^3] \times \frac{235,0[MPa]}{1,00} = 39,92[kNm] \quad \text{(EC3-1-1: 6.55)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{b,Rd}} = \frac{|6,01[kNm]|}{39,92[kNm]} = \mathbf{0,15 \leq 1,00} \quad \text{(EC3-1-1: 6.54)}$$

Note that at the end of the output for this check, the calculation method for the C-parameters will be indicated.

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 M_{cr} has to be adapted.

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

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

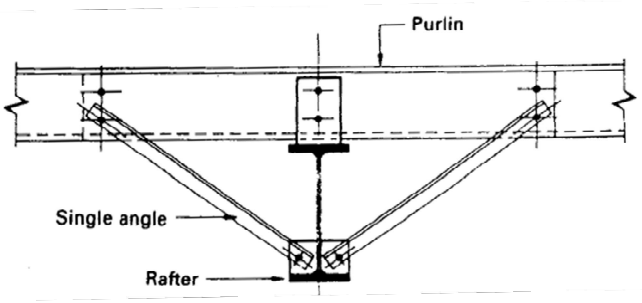
$\lambda_{LT} = \sqrt{\dfrac{M_{pl,y}}{M_{cr,y}}}$

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

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

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

5.4.3. 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}:

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

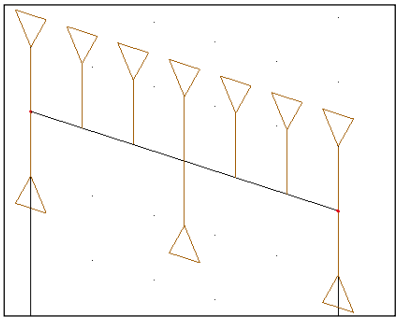
Where:

- k_{LT}*L: 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 the top flange or on the bottom flange. The LTB lengths for the compressed flange are taken as distances between these restraints. The LTB moments factors are calculated between these restraints.

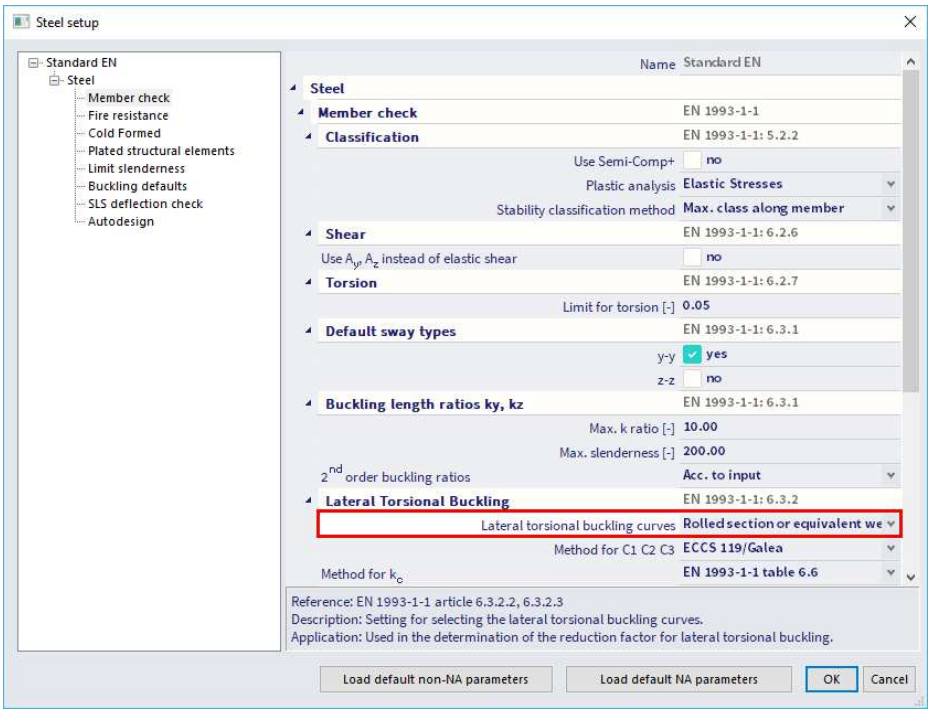
The restraints can be inputted via **Input panel > Steel > Steel LTB restraints** or via **Process toolbar > Steel > Steel LTB restraints**.



Note: only the restraints on the compressed side are taken into account.

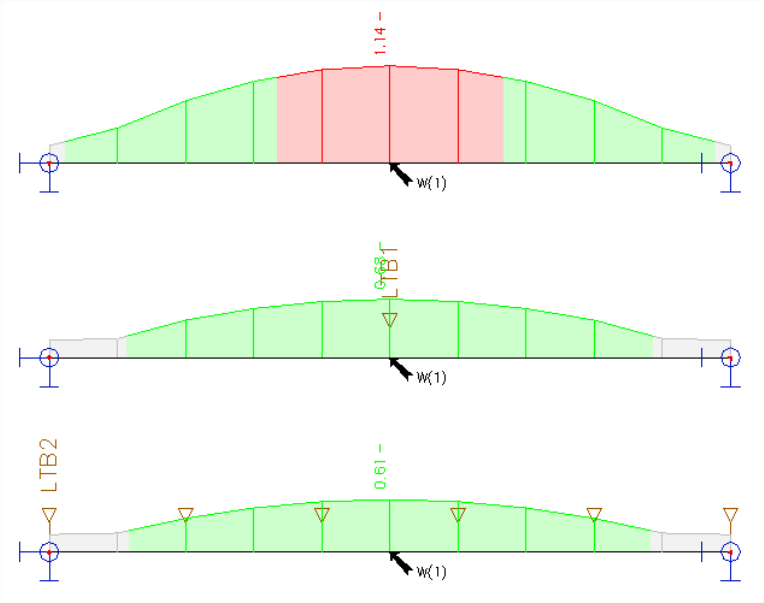
Example: **LTB_Restraints.esa**

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 settings:



Note: the default setting for LTB curves is **General case**.

The results for LC1 are:



B1

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

LTB parameters			
Method for LTB curve		Alternative case	
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	
LTB parameters			
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_{LT}	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 centre 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	
LTB parameters			
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_{LT}	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 centre 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:

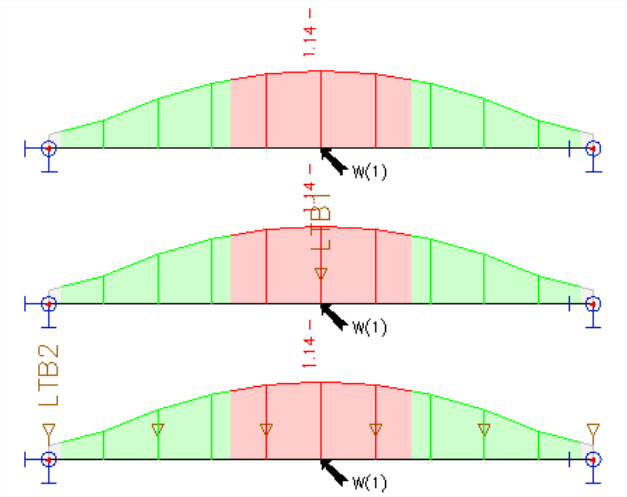
Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

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).

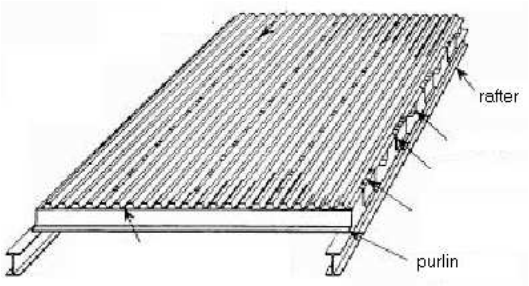
Mcr parameters			
LTB length	l_{LT}	1.400	m
Influence of load position		no influence	
Correction factor	k	1.00	
Correction factor	k_w	1.00	
LTB moment factor	C_1	1.01	
LTB moment factor	C_2	0.02	
LTB moment factor	C_3	1.00	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry 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:

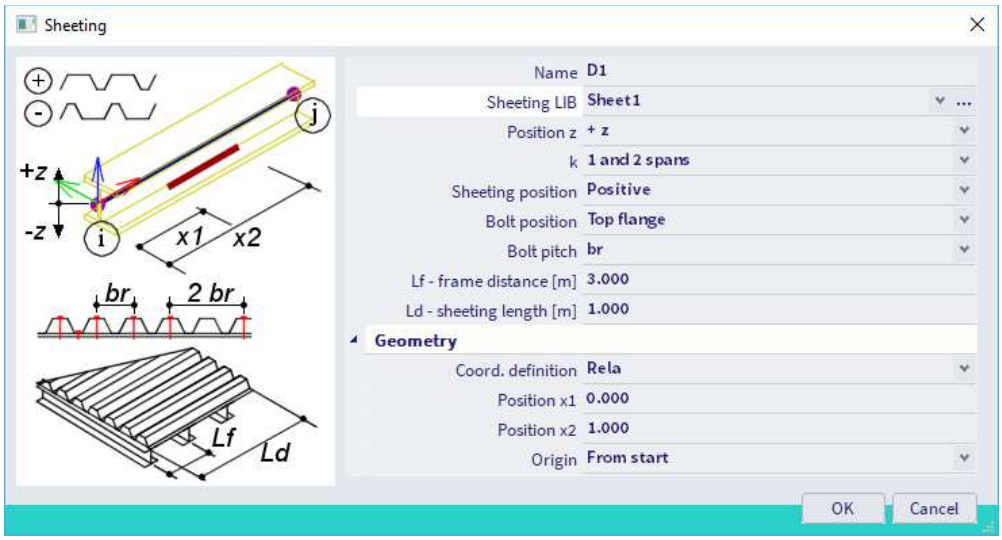


5.4.4. Sheeting

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



A sheeting can be inputted via **Input panel > Steel > Steel sheeting** or **Process toolbar > Steel > Steel sheeting**:



The settings for the diaphragm are:

- Sheeting LIB — with this option you can choose between sheeting types of the library of SCIA Engineer or input his own sheeting.
- Position z — position of the sheeting according to the LCS of the beam
- k — 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
- Bolt pitch — bolts may be either:
  in every rib (i.e. 'br')
  in each second rib (i.e. '2 br')
- Frame distance — distance of frames
- Sheeting length — 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 I_t is adapted with the stiffness of the sheeting.

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:

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

Where:

- $\frac{1}{\text{vorh } C_\vartheta} = \frac{1}{C_{\vartheta M,k}} + \frac{1}{C_{\vartheta A,k}} + \frac{1}{C_{\vartheta P,k}}$
- $C_{\vartheta M,k} = k \frac{E\, I_{eff}}{s}$
- $C_{\vartheta A,k} = C_{100} \left[\frac{b_a}{100}\right]^2 \quad \text{if } b_a \leq 125$
- $C_{\vartheta A,k} = 1{,}25\, C_{100} \left[\frac{b_a}{100}\right] \quad \text{if } 125 < b_a < 200$
- $C_{\vartheta P,k} \approx \frac{3 \cdot E \cdot I_s}{(h-t)}$
- $I_s = \frac{s^3}{12}$

With:

- l LTB length
- G shear modulus
- vorhC_ϑ actual rotational stiffness of the sheeting
- $C_{\vartheta M,k}$ rotational stiffness of the sheeting
- $C_{\vartheta A,k}$ rotational stiffness of the connection between the sheeting and 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 width of the beam flange (in mm)
- C_{100} rotation coefficient – see table below
- h beam height
- t thickness beam flange
- s thickness beam web

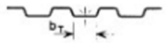
Positioning of sheeting		Sheet fastened through		Pitch of fasteners		Washer diameter	C_{100}	$b_{T,max}$
Positive	Negative	Trough	Crest	$e = b_R$	$e = 2b_R$	[mm]	[kNm/m]	[mm]
For gravity loading:								
×		×		×		22	5,2	40
×		×			×	22	3,1	40
	×		×	×		K_a	10,0	40
	×		×		×	K_a	5,2	40
	×	×		×		22	3,1	120
	×	×			×	22	2,0	120
For uplift loading:								
×		×		×		16	2,6	40
×		×			×	16	1,7	40

Key:
b_R is the corrugation width [185 mm maximum];
b_T is the width of the sheeting flange through which it is fastened to the purlin.

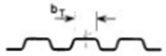
K_a indicates a steel saddle washer as shown below with $t \geq 0{,}75$ mm



Sheet fastened:
- through the trough:



- through the crest:



The values in this table are valid for:
- sheet fastener screws of diameter: $\phi = 6{,}3$ mm;
- steel washers of thickness: $t_w \geq 1{,}0$ mm;
- sheeting of nominal core thickness: $t \geq 0{,}66$ mm.

Example: **Diaphragm.esa**

Consider member B1:

- LTB length = 7 m
- C_1 = 1,13, C_2= 0,45, C_3 = 0,53
- M_{cr} = 281 kNm

Consider member B2

- $I_{t,id}$ = 1840207 mm4
- LTB length = 7 m
- C_1 = 1,13, C_2= 0,45, C_3 = 0,53
- M_{cr} = 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	m⁴/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	K_{1-}	0.167	m/kN
Sheeting factor	K_{2-}	15.700	m²/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	$vorhC_\theta$	19.06	kNm/m
LTB length	l_{LT}	7.000	m
Cross-section torsional constant	I_t	6.6900e-07	m⁴
Additional torsional constant	$I_{t,add}$	1.1715e-06	m⁴
Adapted torsional constant	$I_{t,id}$	1.8405e-06	m⁴

$$S = \frac{L_f \times 10000}{K_{1-} + \frac{K_{2-}}{L_d}} = \frac{3.000[m] \times 10000}{0.167[m/kN] + \frac{15.700[m^2/kN]}{3.000[m]}} = 5555.21[kN]$$

$$S_{erf} = \frac{\left(\frac{E \times I_w \times \pi^2}{l_{LT}^2} + G \times I_t + \frac{E \times I_z \times \pi^2}{l_{LT}^2} \times 0.25 \times h^2\right) \times 70}{h^2}$$

$$= \frac{\left(\frac{210000.0[MPa] \times 7.9100 \cdot 10^{-7}[m^6] \times \pi^2}{7.000[m]^2} + 80769.2[MPa] \times 6.6900 \cdot 10^{-7}[m^4] + \frac{210000.0[MPa] \times 1.6760 \cdot 10^{-5}[m^4] \times \pi^2}{7.000[m]^2} \times 0.25 \times 450[mm]^2\right) \times 70}{450[mm]^2}$$

$$= 42650.44[kN]$$

$$C_{\theta M,k} = \frac{k \times E \times I}{L_f} = \frac{4.00 \times 210000.0[MPa] \times 0.00[m^4/m]}{3.000[m]} = 2637.60[kNm/m]$$

$$C_{\theta P,k} = \frac{\frac{3 \times t_w^3}{12} \times E}{h - t_f} = \frac{\frac{3 \times 9[mm]^3}{12} \times 210000.0[MPa]}{450[mm] - 15[mm]} = 100.15[kNm/m]$$

$$C_{\theta A,k} = C_{100} \times \frac{b}{0.1} \times 1.25 = 10.00[kNm/m] \times \frac{190[mm]}{0.1} \times 1.25 = 23.75[kNm/m]$$

$$C_{100} = 10.00[kNm/m]$$

$$vorhC_\theta = \frac{1}{\frac{1}{C_{\theta P,k}} + \frac{1}{C_{\theta A,k}} + \frac{1}{C_{\theta M,k}}} = \frac{1}{\frac{1}{100.15[kNm/m]} + \frac{1}{23.75[kNm/m]} + \frac{1}{2637.60[kNm/m]}} = 19.06[kNm/m]$$

$$I_{t,add} = \frac{vorhC_\theta \times l_{LT}^2}{\frac{\pi^2}{G}} = \frac{19.06[kNm/m] \times 7.000[m]^2}{\frac{\pi^2}{80769.2[MPa]}} = 1.1715 \cdot 10^{-6}[m^4]$$

$$I_{t,id} = I_t + I_{t,add} = 6.6900 \cdot 10^{-7}[m^4] + 1.1715 \cdot 10^{-6}[m^4] = 1.8405 \cdot 10^{-6}[m^4]$$

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_{LT}	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 centre 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

$$M_{cr} = C_1 \times \frac{\pi^2 \times E \times I_z}{l_{LT}^2} \times \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \times \frac{I_w}{I_z} + \frac{l_{LT}^2 \times G \times I_{t,id}}{\pi^2 \times E \times I_z} + (C_2 \times z_g - C_3 \times z_j)^2} - (C_2 \times z_g - C_3 \times z_j) \right] = 1.13$$

$$\times \frac{\pi^2 \times 210000.0[\text{MPa}] \times 1.6760 \cdot 10^{-5}[\text{m}^4]}{7.000[\text{m}]^2}$$

$$\times \left[\sqrt{\left(\frac{1.00}{1.00}\right)^2 \times \frac{7.9100 \cdot 10^{-7}[\text{m}^6]}{1.6760 \cdot 10^{-5}[\text{m}^4]} + \frac{7.000[\text{m}]^2 \times 80769.2[\text{MPa}] \times 1.8405 \cdot 10^{-6}[\text{m}^4]}{\pi^2 \times 210000.0[\text{MPa}] \times 1.6760 \cdot 10^{-5}[\text{m}^4]} + (0.45 \times 0[\text{mm}] - 0.53 \times 0[\text{mm}])^2} - (0.45 \times 0[\text{mm}] - 0.53 \times 0[\text{mm}]) \right]$$

$$= 404.94[\text{kNm}]$$

$$\lambda_{rel,LT} = \sqrt{\frac{W_{pl,y} \times f_y}{M_{cr}}} = \sqrt{\frac{1.7020 \cdot 10^{-3}[\text{m}^3] \times 235.0[\text{MPa}]}{404.94[\text{kNm}]}} = 0.99$$

$$\chi_{LT} = \min\left(\frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^2 - \lambda_{rel,LT}^2}}, 1\right) = \min\left(\frac{1}{1.13 + \sqrt{1.13^2 - 0.99^2}}, 1\right) = \min(0.60, 1) = 0.60 \quad \text{(EC3-1-1: 6.56)}$$

$$M_{b,Rd} = \chi_{LT} \times W_{pl,y} \times \frac{f_y}{\gamma_{M1}} = 0.60 \times 1.7020 \cdot 10^{-3}[\text{m}^3] \times \frac{235.0[\text{MPa}]}{1.00} = 240.36[\text{kNm}] \quad \text{(EC3-1-1: 6.55)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{b,Rd}} = \frac{|245.00[\text{kNm}]|}{240.36[\text{kNm}]} = \mathbf{1.02 > 1.00} \quad \text{(EC3-1-1: 6.54)}$$

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

5.5. 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 formulas (6.61) and (6.62):

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

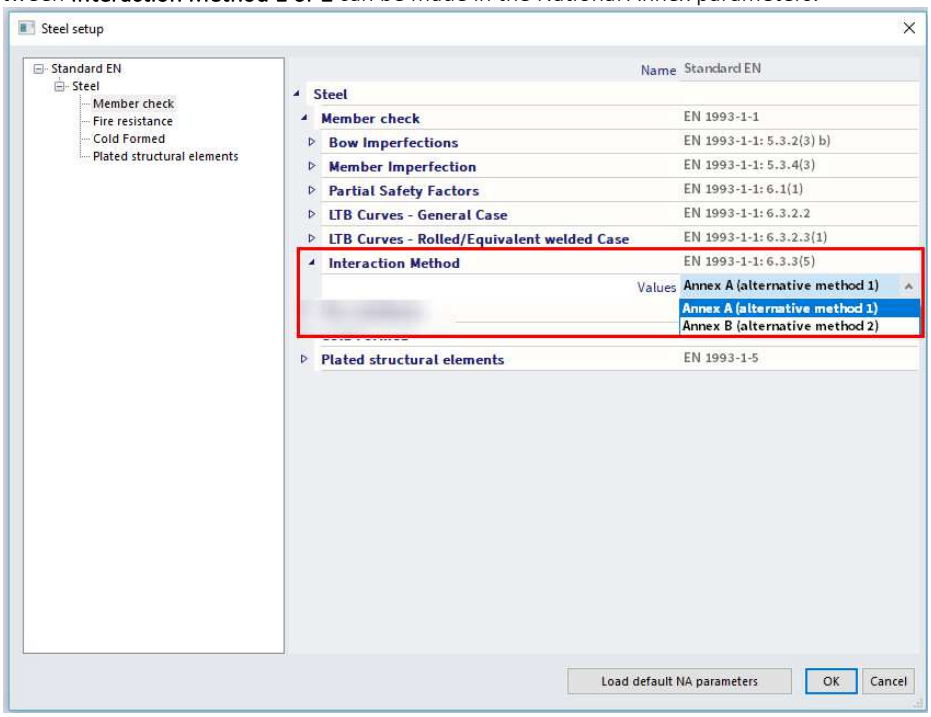
$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

- N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ are the design values of the compression force and the **MAXIMUM** moments about the y-y and z-z axis along the member respectively
  Note: the word 'MAXIMUM' was forgotten in the translation of the Belgian National Annex.
- χ_y and χ_z are the reduction factors due to flexural buckling
- χ_{LT} is the reduction factor due to lateral torsional buckling
-

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

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

The choice between **Interaction Method 1 or 2** can be made 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 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.

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 area	A	1,8800e+04	mm^2
Plastic section modulus	$W_{pl,y}$	5,1100e+06	mm^3
Plastic section modulus	$W_{pl,z}$	6,3100e+05	mm^3
Design compression force	N_{Ed}	158,70	kN
Design bending moment (maximum)	$M_{y,Ed}$	-699,29	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-0,37	kNm
Characteristic compression resistance	N_{Rk}	4418,00	kN
Characteristic moment resistance	$M_{y,Rk}$	1200,85	kNm
Characteristic moment resistance	$M_{z,Rk}$	148,28	kNm
Reduction factor	χ_y	0,74	
Reduction factor	χ_z	0,48	
Reduction factor	χ_{LT}	0,71	
Interaction factor	k_{yy}	0,99	
Interaction factor	k_{yz}	1,30	
Interaction factor	k_{zy}	0,53	
Interaction factor	k_{zz}	0,81	

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	$N_{cr,y}$	5635,44	kN
Critical Euler load	$N_{cr,z}$	3067,01	kN
Elastic critical load	$N_{cr,T}$	4845,28	kN
Plastic section modulus	$W_{pl,y}$	5,1100e+06	mm^3
Elastic section modulus	$W_{el,y}$	4,4110e+06	mm^3
Plastic section modulus	$W_{pl,z}$	6,3100e+05	mm^3
Elastic section modulus	$W_{el,z}$	3,9900e+05	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)	

Interaction method 1 parameters			
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	0,97	
Factor	ε_y	18,78	
Factor	a_{LT}	1,00	
Critical moment for uniform bending	$M_{cr,0}$	1008,48	kNm
Relative slenderness	$\lambda_{rel,0}$	1,09	
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	
Factor	b_{LT}	0,00	
Factor	c_{LT}	1,43	
Factor	d_{LT}	0,00	
Factor	e_{LT}	0,73	
Factor	w_y	1,16	
Factor	w_z	1,50	
Factor	n_{pl}	0,04	
Maximum relative slenderness	$\lambda_{rel,max}$	1,20	
Factor	C_{yy}	0,99	
Factor	C_{yz}	0,43	
Factor	C_{zy}	0,96	
Factor	C_{zz}	0,99	

Unity check (6.61) = 0,05 + 0,81 + 0,00 = 0,86 -
Unity check (6.62) = 0,08 + 0,43 + 0,00 = 0,51 -

When turning on the formulas in the detailed output, the calculation of each parameter will be shown.

5.6. Shear buckling check – EN 1993-1-5

5.6.1. General

The shear buckling check will check if the web of the cross-section can be subject to local buckling.

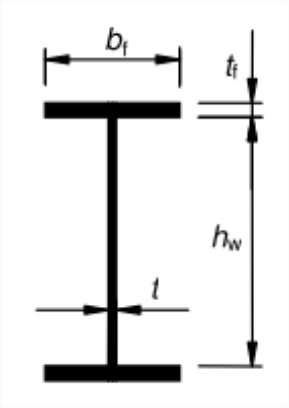
This check is not included in EN 1993-1-1, but in EN 1993-1-5: Design of steel structures – Part 1-5: Plated structural elements.

First, there will be checked if the value for the slenderness is smaller 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:

- $\epsilon = \sqrt{\frac{235}{f_y\left[\frac{N}{mm^2}\right]}}$
- 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 IPE160 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: 72ε/η = 72 x 1,00 / 1,2 = 60

- h_w / t < 72ε/η → 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: 72ε/η = 72 x 1,00 / 1,2 = 60

- h_w / t > 72ε/η
- 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)

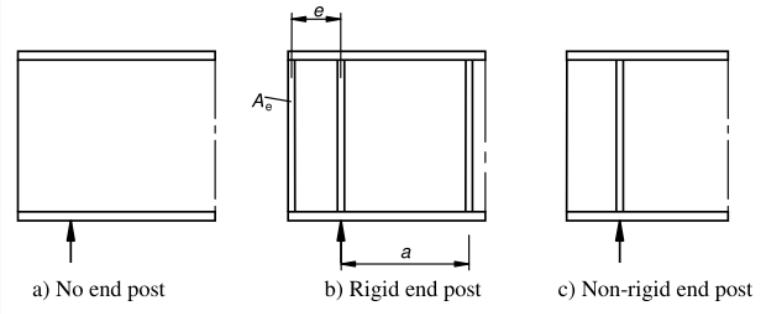
Shear Buckling parameters			
Buckling field length	a	6.000	m
Web		unstiffened	
End post		non-rigid	
Web height	h_w	582	mm
Web thickness	t	6	mm
Yield strength	f_{yw}	235.0	MPa
Flange width	b_f	276	mm
Flange thickness	t_f	9	mm
Yield strength	f_{yf}	235.0	MPa
Material coefficient	ε	1.00	
Shear correction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

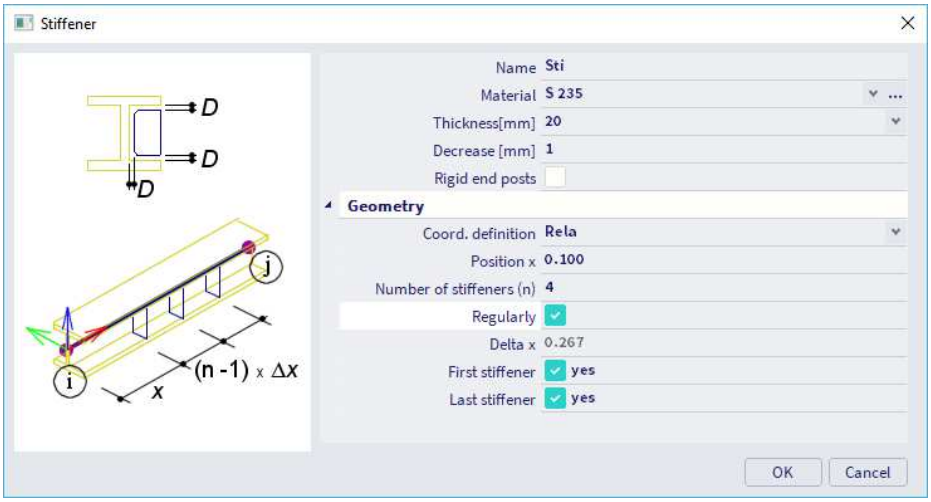
5.6.2. Stiffeners

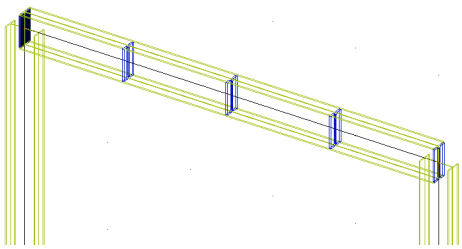
In EN 1993-1-5, figure 5.1 the definitions of stiffeners on a member are given:



Those stiffeners will influence the total length for shear buckling. In SCIA Engineer those stiffeners can be inputted on a beam and will **ONLY** have an influence on the shear buckling check in the steel service.

Those stiffeners can be inputted via **Input panel > Steel > Steel stiffeners** or **Process toolbar > Steel > Steel stiffeners**.





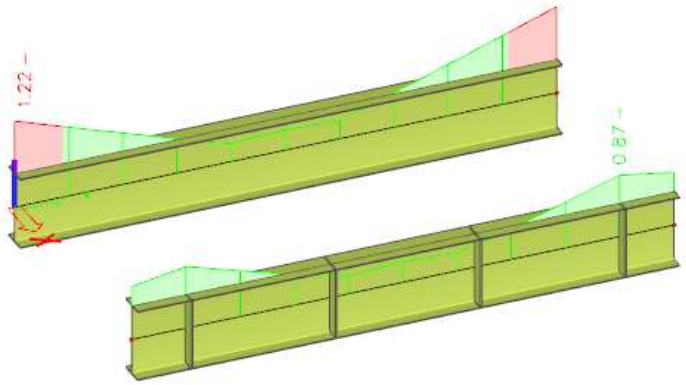
The stiffeners define the field dimensions which are only relevant for the shear buckling check. When no stiffeners are defined, the value for buckling field length 'a' is taken equal to the member length.

If you want to take stiffeners into account in the analytical model, you need to model them outside the steel service. You could create a new cross-section, in which the stiffener is considered, and apply that cross-section to the 1D member, each time over a length equal to the thickness of the stiffener. When calculating the model, the stiffener will be considered in the results of the deformations, stresses, …

Example: **Stiffeners.esa**

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

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



B1 - without stiffeners

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	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** -

B2 - with stiffeners

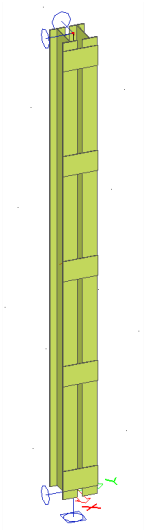
Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters			
Buckling field length	a	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_τ	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 -

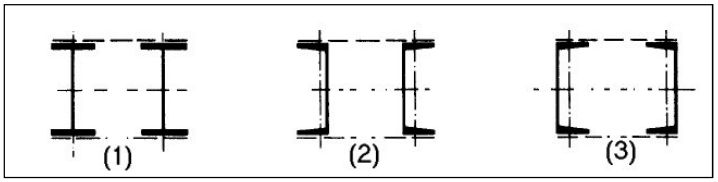
Chapter 6: ULS check for battened compression members



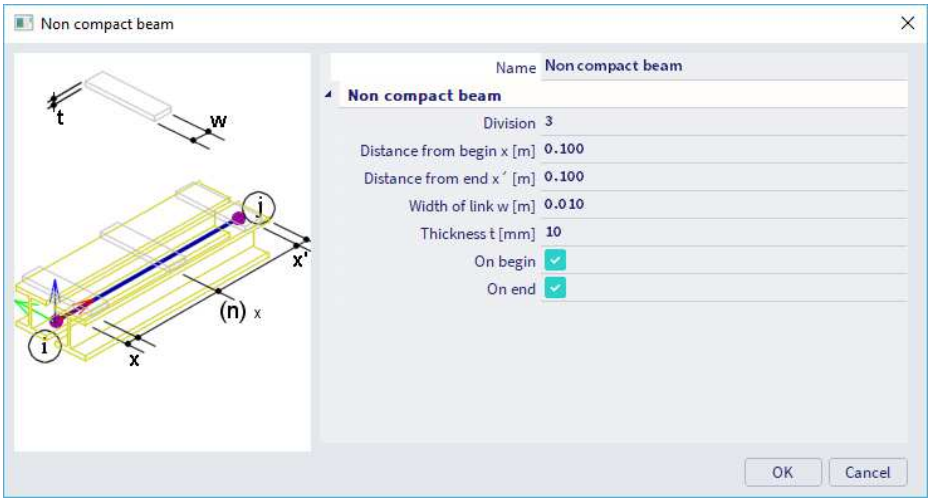
This check will be executed following EC-EN 1993-1-1 art.6.4.1 and 6.4.3.

The following cross-section pairs are supported as battened compression member:

(1) 2I
(2) 2Uo
(3) 2Uc

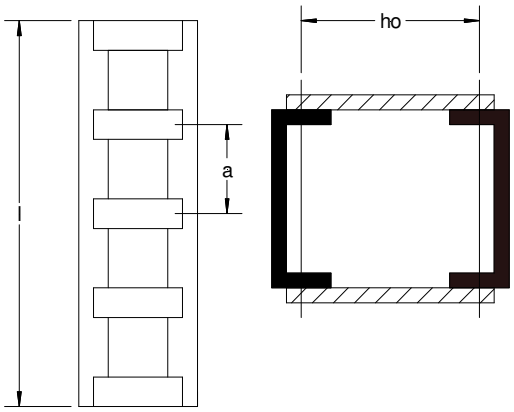


You can add the links (battens) via **Input panel > Steel > Links** or **Process toolbar > Steel > Links**.



The following additional checks are performed when performing the ULS check:

- 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: **EC_EN_Battened_Compression_Members_I.esa**

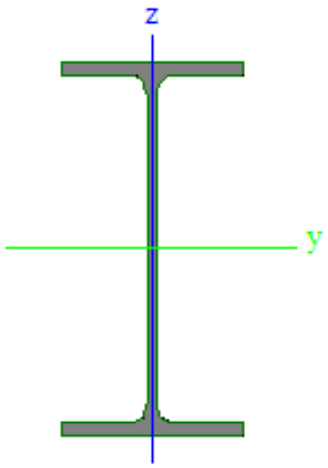
Consider member B1.

On this beam, only a compression force of 500 kN 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 IPE330 cross-section:



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)
  = 5000 mm – 200 mm – 200 mm = 4600 mm
- a — distance between battens = l / (number of divisions) = 4600 mm / 4 = 1150 mm
- h_0 — distance between centroids of chords = 210 mm
- I_{ch} — I_z of the IPE330 profile
- A_{ch} — area if one I-profile, so A of IPE330
- λ — = L / i_0 = 4600 mm / 110,833 mm = 41,5

  $$I_1 = 0{,}5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot I_{ch}$$
  $$= 0{,}5 \cdot (210\ mm)^2 \cdot (6260\ mm^2) + 2 \cdot (7881000\ mm^4) = 1{,}538 \times 10^8\ mm^4$$
  $$i_0 = \sqrt{\frac{I_1}{2 \cdot A_{ch}}} = \sqrt{\frac{1{,}538 \times 10^8 mm^4}{2 \cdot 6260 mm^2}} = 110{,}833\ mm$$
- μ — = 0 if λ ≥ 150
  $= 2 - \frac{\lambda}{75}$ if 75 < λ < 150
  = 1,0 if λ ≤ 75
  for this example λ = 41,5 < 75 → $\mu = 1{,}00$
- I_{eff} $= 0{,}5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot \mu \cdot I_{ch}$
  $= 0{,}5 \cdot (210\ mm)^2 \cdot (6260\ mm^2) + 2 \cdot (1{,}00) \cdot (7881000\ mm^4) = 1{,}538 \times 10^8\ mm^4$
- S_v $= \frac{24 \cdot E \cdot Ich}{a^2 \cdot \left(1 + \frac{2 \cdot Ich}{n \cdot Ib} \cdot \frac{h0}{a}\right)}$ but $Sv \leq \frac{2 \cdot \pi^2 \cdot E \cdot Ich}{a^2}$
  n = number of planes of battens: n = 2
  I_b = (thickness batten) x (width of batten)³ / 12 = (7 mm) x (220 mm)³ / 12
  = 6,211 x 10^6 mm^4

  $$\frac{24 \cdot E \cdot Ich}{a^2 \cdot \left(1 + \frac{2 \cdot Ich}{n \cdot Ib} \cdot \frac{h0}{a}\right)} = \frac{24 \cdot \left(\frac{210000N}{mm^2}\right) \cdot (7881000 mm^4)}{(1150mm)^2 \cdot \left(1 + \frac{2 \cdot (7881000 mm^4)}{2 \cdot (6{,}211 \times 10^6\ mm^4)} \cdot \frac{210mm}{1150mm}\right)} = 2{,}438 \times 10^4\ kN$$
  $$\frac{2 \cdot \pi^2 \cdot E \cdot Ich}{a^2} = \frac{2 \cdot \pi^2 \cdot \left(\frac{210000N}{mm^2}\right) \cdot (7881000 mm^4)}{(1150)^2} = 2{,}470 \times 10^4\ kN$$
  $$\rightarrow Sv = 2{,}438 \times 10^4\ kN$$
- e_0 — = l/500 = 4600 mm / 500 = 9,2 mm
- $M_{Ed,I}$ — M_z in the cross-section under consideration = 0 kNm
- M_{Ed} — if $\left(1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}\right) > 0$ → $M_{Ed} = \frac{N_{Ed} \cdot e_0 + M_{Ed,I}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}}$

  otherwise: → M_{Ed} = 1 x 10^6 kNm

  $$N_{cr} = \frac{\pi^2 \cdot E \cdot I_{eff}}{L^2} = \frac{\pi^2 \cdot \left(\frac{210000N}{mm^2}\right) \cdot (1{,}538 \times 10^8 mm^4)}{(4600mm)^2} = 15064\ kN$$
  $$\rightarrow \left(1 - \frac{500kN}{15064kN} - \frac{500kN}{24700\ kN}\right) = 0{,}9466 > 0$$
  $$\rightarrow M_{Ed} = \frac{N_{Ed} \cdot e_0 + M_{Ed,I}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}} = \frac{500kN \cdot 9{,}2mm + 0kNm}{0{,}9466} = 4860\ kNmm = 4{,}860\ kNm$$
- V_{Ed} $= \frac{\pi \cdot M_{Ed}}{l} = \frac{\pi \cdot 4860 kNmm}{4600mm} = 3{,}32\ kN$

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:

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	-

- $N_G = N_{Chord} = 0{,}5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}} = 0{,}5 \cdot 500kN + \frac{(4860kNmm) \cdot (210mm) \cdot (6260mm^2)}{2 \cdot 1{,}538 \times 10^8 mm^4} = 270{,}78\ kN$
- $V_G = V_{Ed}/2$ = 3,32kN / 2 = 1,66 kN
- $M_G = \frac{V_{Ed} \cdot a}{4} = \frac{3{,}32kN \cdot 1150mm}{4} = 0{,}95\ kNm$

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

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 'Check of chord as beam in field between battens')
- l_{cr} buckling length
- λ slenderness = $\frac{l_{cr}}{\sqrt{\frac{I_{cg}}{A_{cg}}}} = \frac{1150mm}{\sqrt{\frac{7881000mm^4}{6260mm^2}}} = 32{,}411$
- λ_{rel} relative slenderness = $\sqrt{\frac{Ach \cdot fy}{N_{cr,z}}} = \sqrt{\frac{6260mm^2 \cdot \frac{275N}{mm^2}}{12351000\,N}} = 0.373$

  with $N_{cr,z} = \frac{\pi^2 \cdot E \cdot I_{ch}}{(l_{cr})^2} = \frac{\pi^2 \cdot \left(\frac{210000N}{mm^2}\right) \cdot (7881000mm^4)}{(1150mm)^2} = 12351\ kN$
- Buckling curve = b, this is the buckling curve for the IPE330 around the z-axis
- α imperfection factor for buckling curve b = 0,34
- χ reduction factor, can be calculated as explained in the buckling check
- UC unity check = $\frac{N_{ch,Ed}}{N_{b,Rd}} = \frac{270{,}78kN}{1618{,}2\ kN} = 0{,}16$

  with $N_{b,Rd} = \chi \cdot \frac{A_{ch} \cdot f_y}{\gamma_{M1}} = 0{,}94 \cdot \frac{6260mm^2 \cdot 275N/mm^2}{1{,}00} = 1618{,}21\ kN$

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 $= \frac{V_{Ed} \cdot a}{h_0 \cdot 2} = \frac{3{,}32kN \cdot 1150mm}{210mm \cdot 2} = 9{,}09kN$
- M_G = 0,954 kNm (see previous check)
- σ $= \frac{M_G \cdot b/2}{Ib} = \frac{954000Nmm \cdot 220mm/2}{6.211 \cdot 10^6 mm^4} = 16{,}9\ MPa$

  calculation of I_b: see calculation of S_v above
- UC σ $= \frac{\sigma}{\frac{f_y}{\gamma_{M0}}} = \frac{16{,}9MPa}{\frac{275N/mm^2}{1{,}00}} = 0{,}06$
- τ $= \frac{T}{b \cdot t} = \frac{9090N}{220mm \cdot 7mm} = 5{,}9\ MPa$
- UC τ $= \frac{\tau}{\frac{f_y/\sqrt{3}}{\gamma_{M0}}} = \frac{5{,}9MPa}{\frac{275N/mm^2/\sqrt{3}}{1{,}00}} = 0{,}04$

Chapter 7: Optimisation

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

- cross-section optimisation: optimisation of a chosen cross-section;
- overall optimisation: optimisation of one or more (or all) cross-sections at the same time.

Both options are explained in the next two paragraphs.

7.1. Cross-section optimisation

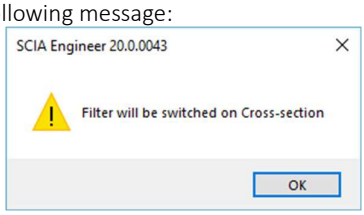
With this option you can optimise a cross-section for the steel code check. This will be explained in the example below.

Example: **Optimisation.esa**

The optimisation can be found via the steel check command. First the steel code check has to be executed, so first click on **Refresh** in the property panel. Then you can choose for the action **Autodesign**:

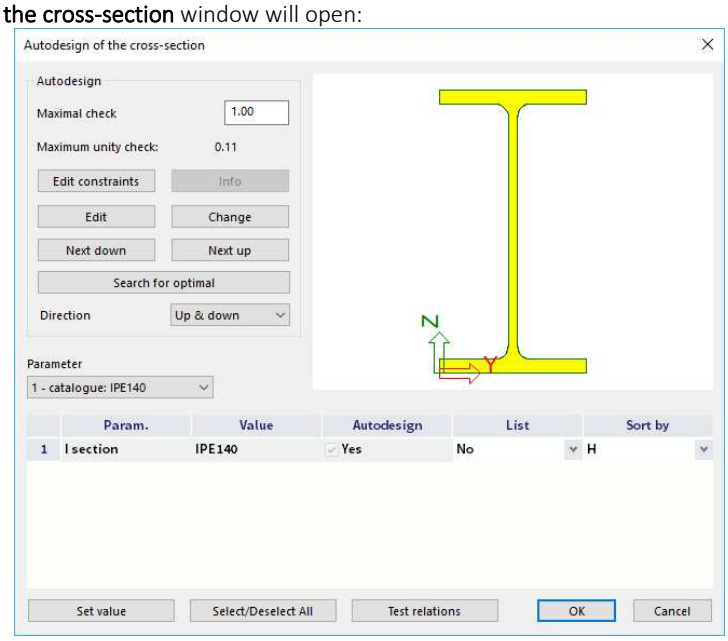


It's possible that SCIA Engineer will give following message:



In the property panel the option for **Filter** has been changed into **Cross-section**. Now choose here for the cross-section IPE140 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.

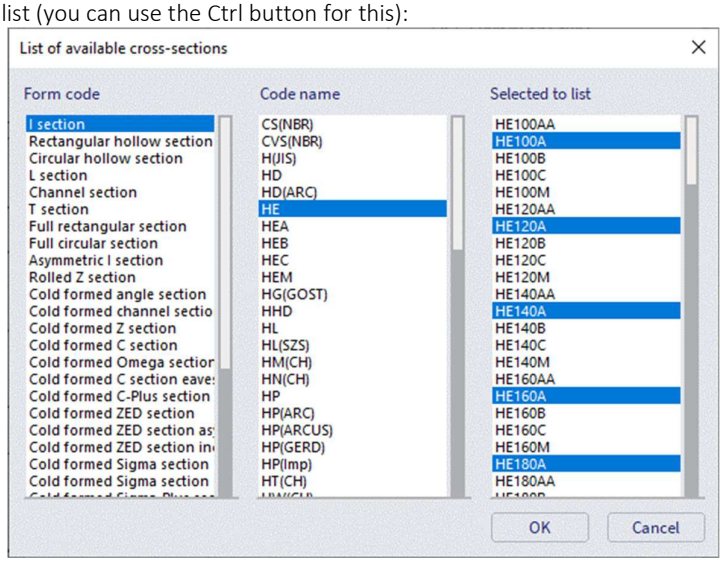
The **Maximum unity check** displayed below is the maximum unity check for the IPE140, found in this project.

When clicking on **<Search for optimal>**, SCIA Engineer will suggest the smallest IPE cross-section which will resist the calculated internal forces. In this example an IPE80AA will be found. When clicking on **<OK>** now, SCIA Engineer will replace this IPE140 profile by an IPE80AA profile automatically.

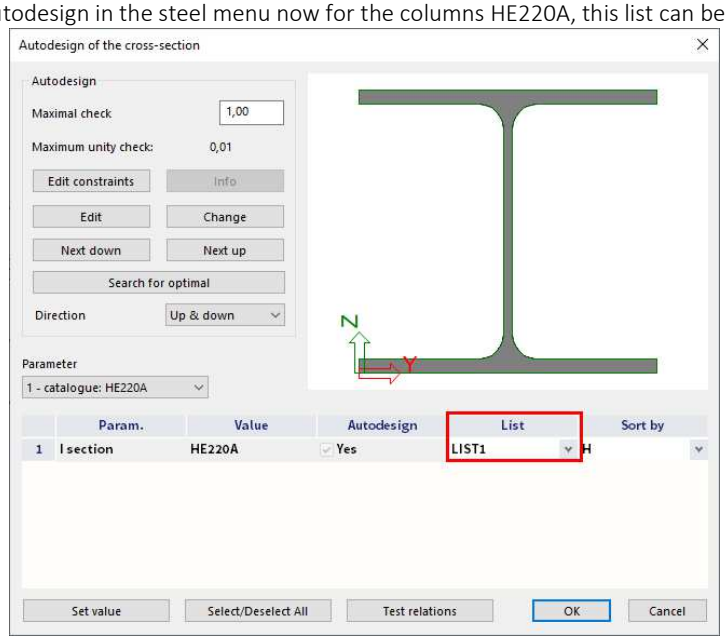
Now the steel code check of this IPE80AA profile is executed with the internal forces calculated with the properties of the IPE140 profile. For example, the self weight will not be taken into account correctly. So, the project should be recalculated, before accepting those new unity check results.

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 us make a list for the columns. This can be done through **Menu bar > Libraries > Structure and analysis > Cross section list** and choose for **Library cross-section of one type**. Now select the profiles that should be added to this list (you can use the Ctrl button for this):



When performing an Autodesign in the steel menu now for the columns HE220A, this list can be chosen:



For the moment this profile has a maximum unity check of 0,01, so SCIA Engineer will search for the smallest profile from this 'LIST1' which will pass the unity check.

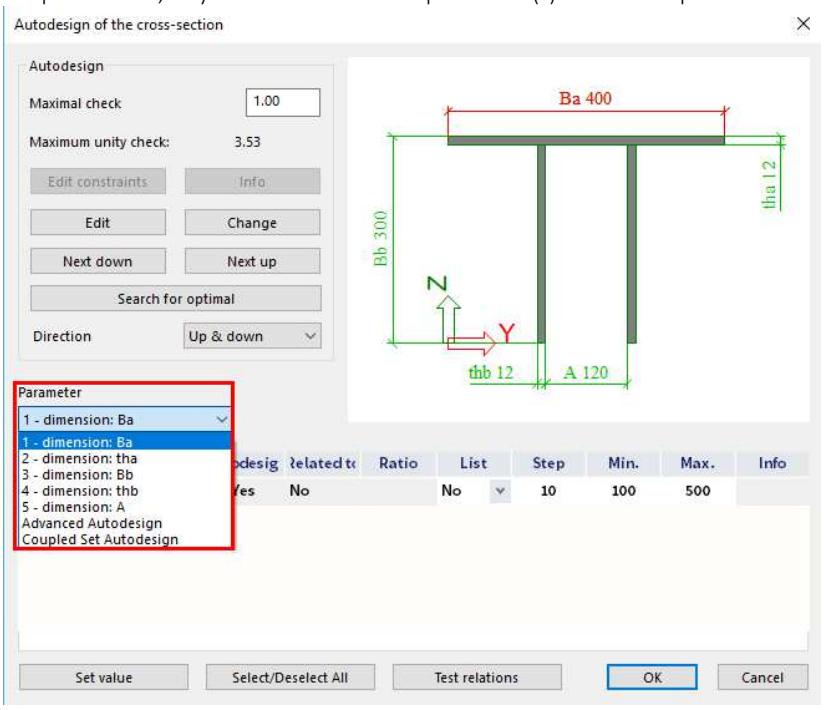
Close this example without saving! It will be used in the next paragraph again!

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

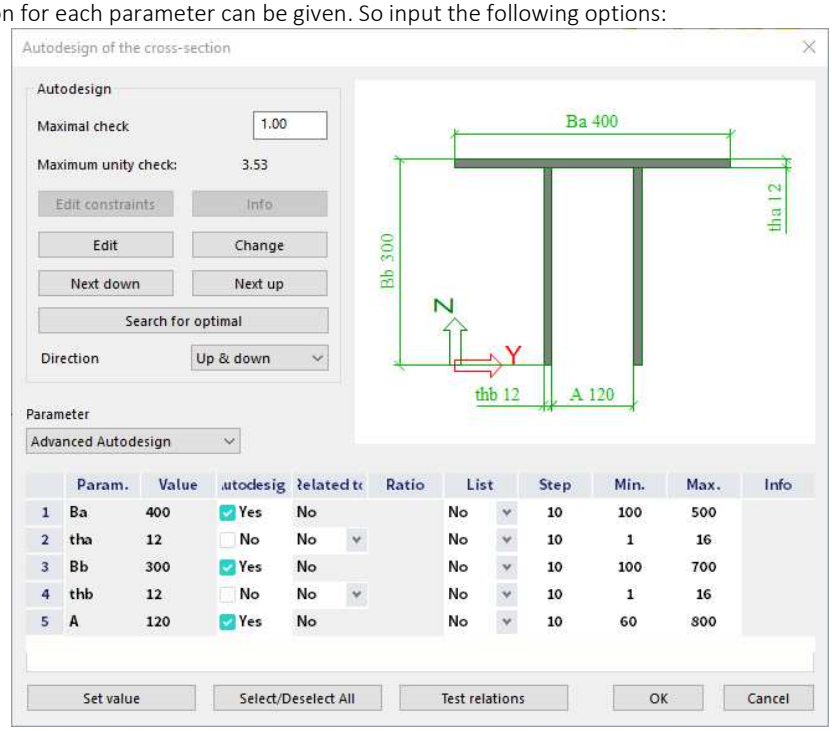
Example: **Optimisation2.esa**

Go to the steel check command and execute Autodesign for this beam.

This profile has a lot of parameters, so you can choose which parameter(s) should be updated:

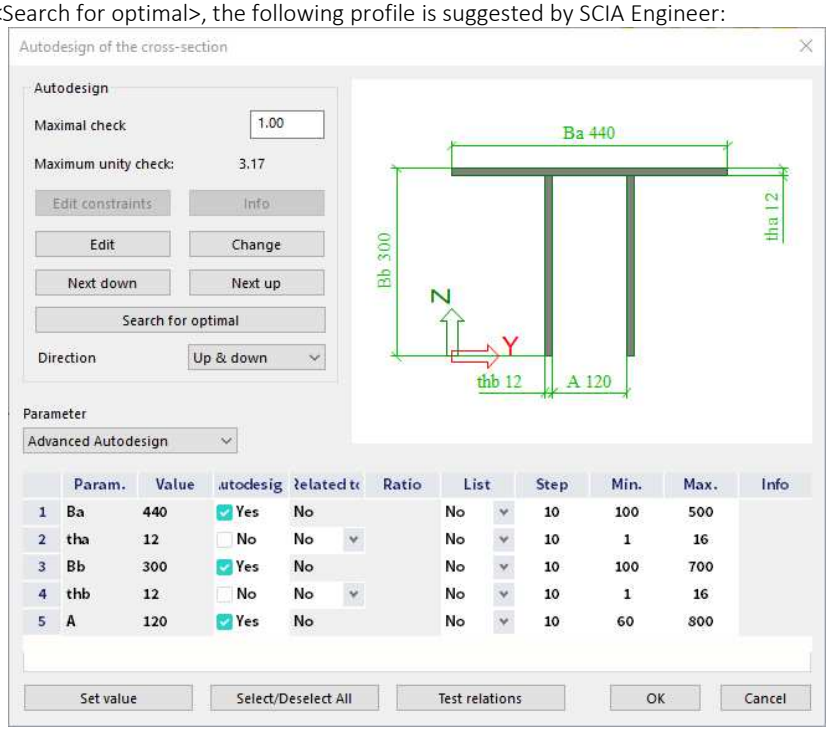


Also the option **Advanced Autodesign** will appear. With this option several parameters can be optimised at the same time and restriction for each parameter can be given. So input the following options:



With these settings, thicknesses thb and tha are not autodesigned. If you want to autodesign these as well, the step should be chosen wisely (for example a step of 1 instead of 10). 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 500 mm can be inputted. And the **Maximum unity check** for this profile will be adapted automatically.

7.2. 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:

- stop because all profiles do 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 you.

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**

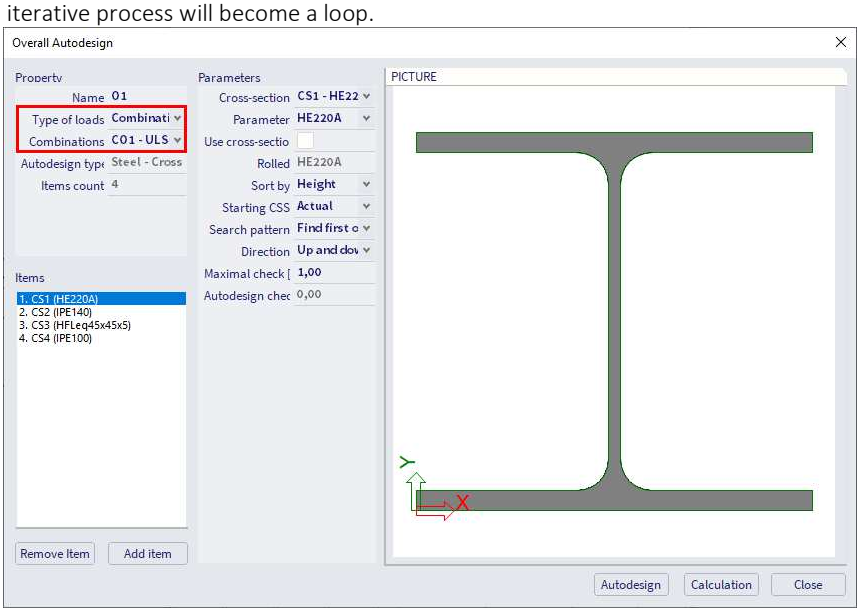
If this example is still open from the previous chapter, please close it and reopen it without saving.

Calculate the project and go to **Menu bar > Tools > Calculation & Mesh > Autodesign**.

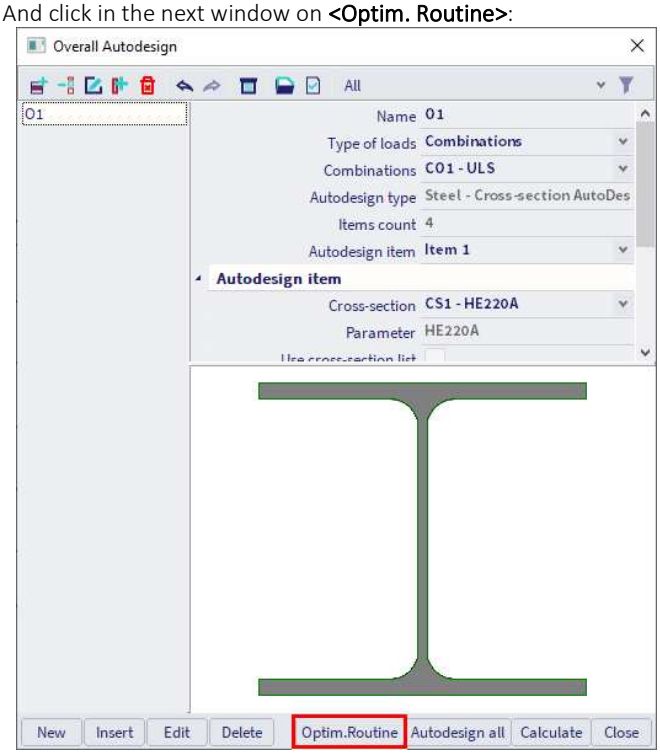
Click here on **Add item** and choose for **Steel > Cross-section AutoDesign** and add all cross-sections in this autodesign process.

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

Also for each profile, you can indicate if this profile has to be auto designed 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 **<Optim. Routine>**:



Now you 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 **6** 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 6 iteration steps. In this example SCIA Engineer will stop after 5 iterations steps, because all cross-sections remain the same after the fifth step.

After this process, an information window about this iteration will be displayed. The first two columns (Cross-section and Parameter) don't provide a lot of information: they both mention the final cross-section.

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 - HE140C	HE140C	Height	CS1 - HE220A	CS1 - HE180C	0,51
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: HE220A → HE180C
- 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 - HE140C	HE140C	Height	CS1 - HE180C	CS1 - HE140M	0,48
CS2 - IPE200O	IPE200O	Height	CS2 - IPE200O	CS2 - IPE200	0,99
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq80x80x8	CS3 - HFLeq90x90x9	0,80
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140AA	CS4 - IPE140A	0,97

Changed profiles:

- CS1: HE180C → HE140M
- 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 - HE140C	HE140C	Height	CS1 - HE140M	CS1 - HE140C	0,60
CS2 - IPE200O	IPE200O	Height	CS2 - IPE200	CS2 - IPE200	0,99
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq90x90x9	CS3 - HFLeq90x90x9	0,96
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140A	CS4 - IPE140A	0,97

Changed profiles:

- CS1: HE140M → HE140C

ITERATION STEP 4:

4. Routine step: 4

Cross-section	Parameter	Sort by	Original cross-section	Autodesign of cross-section	Autodesign check [-]
CS1 - HE140C	HE140C	Height	CS1 - HE140C	CS1 - HE140C	0,54
CS2 - IPE200O	IPE200O	Height	CS2 - IPE200	CS2 - IPE200O	0,85
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq90x90x9	CS3 - HFLeq90x90x9	0,98
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140A	CS4 - IPE140A	0,97

Changed profiles:

- CS2: IPE200 → IPE200O

ITERATION STEP 5:

5. Routine step: 5

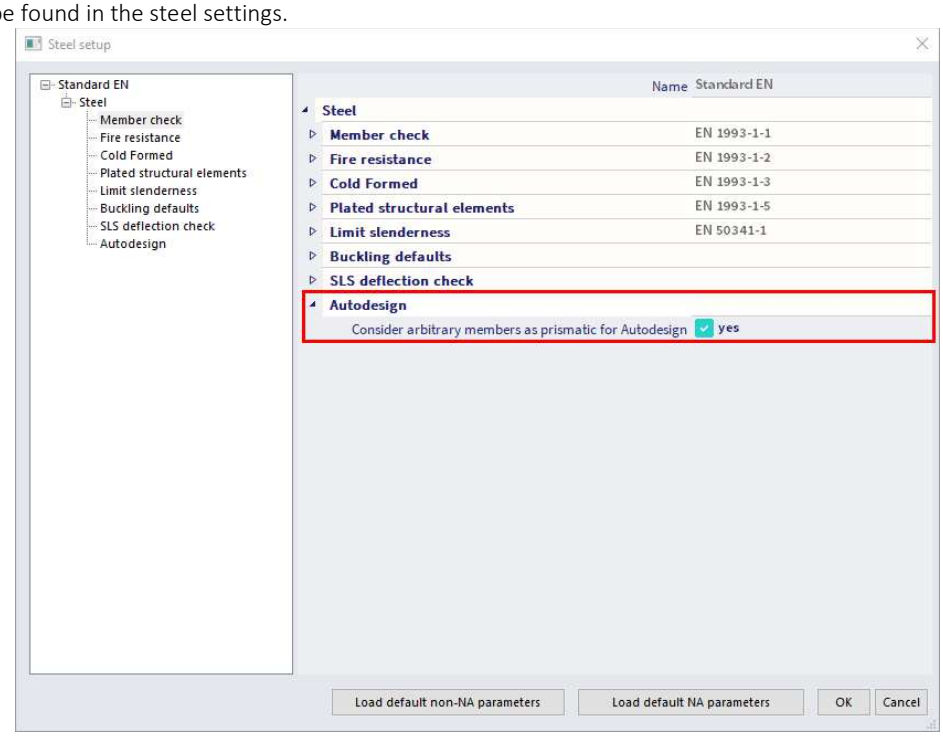
Cross-section	Parameter	Sort by	Original cross-section	Autodesign of cross-section	Autodesign check [-]
CS1 - HE140C	HE140C	Height	CS1 - HE140C	CS1 - HE140C	0,53
CS2 - IPE200O	IPE200O	Height	CS2 - IPE200O	CS2 - IPE200O	0,85
CS3 - HFLeq90x90x9	HFLeq90x90x9	Height	CS3 - HFLeq90x90x9	CS3 - HFLeq90x90x9	0,96
CS4 - IPE140A	IPE140A	Height	CS4 - IPE140A	CS4 - IPE140A	0,97

As a limit number we put 6 iteration steps, but since the cross-sections are the same as in the previous iteration step, the iteration process is completed after 5 iterations.

Note that also here you can work with a list of cross-sections from which the autodesign routine can choose.

7.3. 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 the steel settings.



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.

Chapter 8: 2nd order calculation and imperfections

8.1. Overview

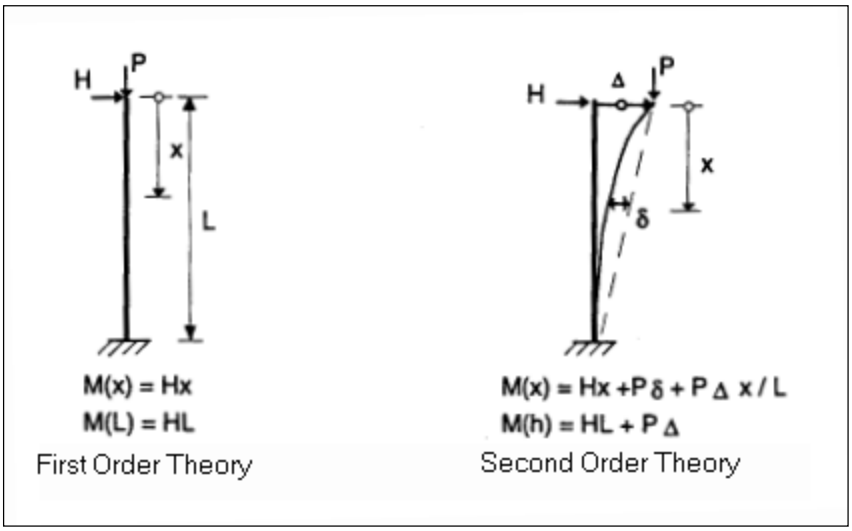
Global analysis aims at determining the distribution of the internal forces and moments and the corresponding displacements in a structure subjected to a specified loading.

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 applied in all cases, while first-order theory may be used only when the displacement effects on the structural behaviour are negligible.

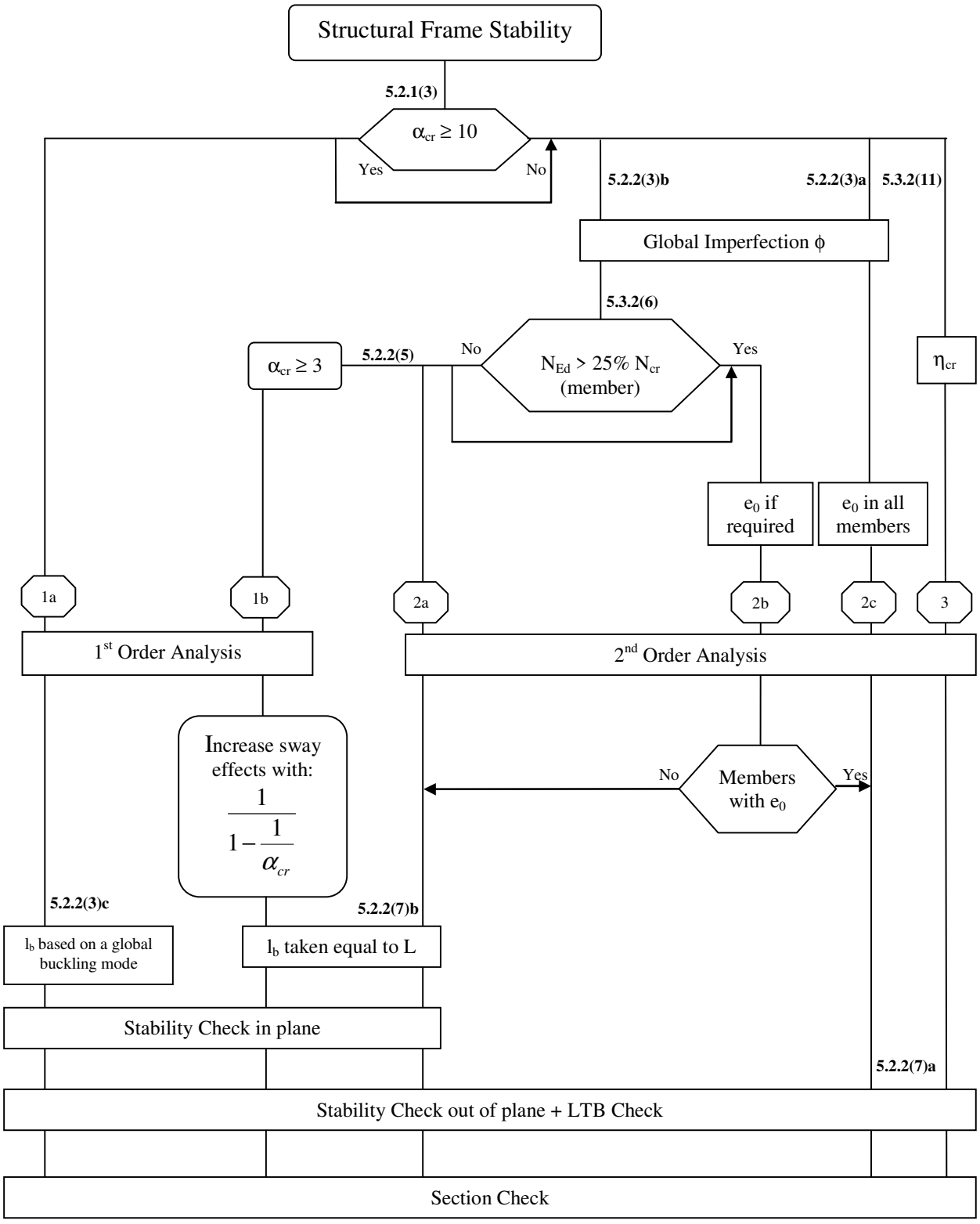
The second-order effects are made up of a local or member second-order effects, referred to as the P-δ effect, and a global second-order effect, referred to as the P-Δ effect.



On the next page an overview of the global analysis following the EN 1993-1-1, chapter 5, is 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 (α_{cr} > 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 if necessary local/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 behaviour are negligible.
- In the next paragraphs the rules in this overview will be explained.

To take into account all non-linearities in the model, non-linear load combinations are made.



With:

- η_{cr} elastic critical buckling mode
- L member system length
- l_b buckling length

8.2. Alpha critical

The calculation of alpha critical is done by a stability calculation in SCIA Engineer.

According to 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:

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

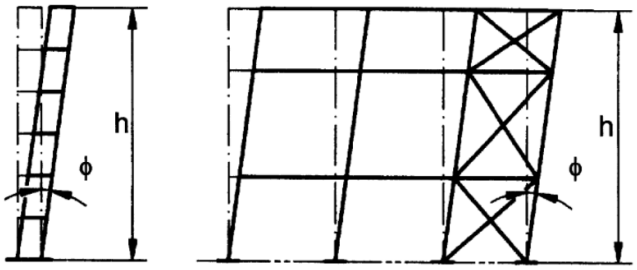
With:

- α_{cr}: factor by which the design loading must be increased to cause elastic instability in a global mode
- F_{Ed}: design loading on the structure
- F_{cr}: 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.

8.3. 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):

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

With:

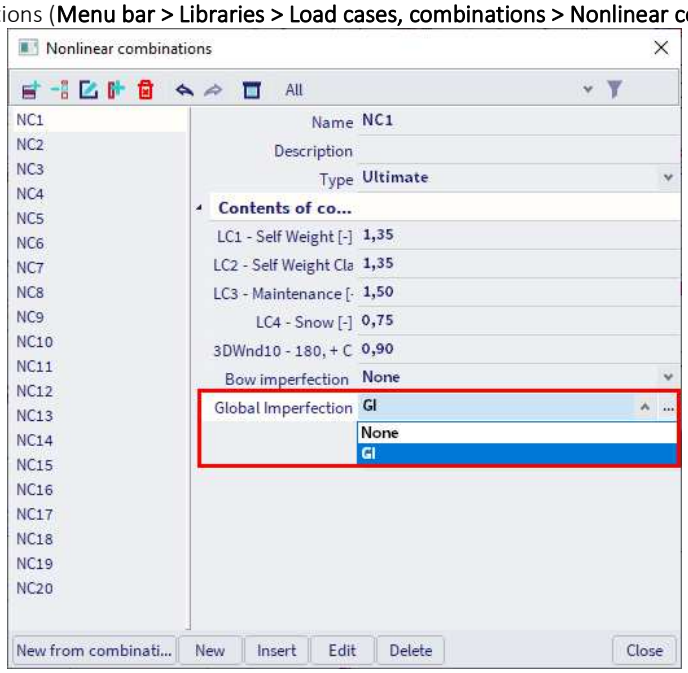
- $\alpha_h = \frac{2}{\sqrt{h}}$ but $\frac{2}{3} \leq \alpha_h \leq 1{,}0$
- $\alpha_m = \sqrt{0{,}5\left(1 + \frac{1}{m}\right)}$
- h height of the structure in meters
- mnumber of columns in a row including only those columns which carry a vertical load N_{Ed} not less than 50% of the average value 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**

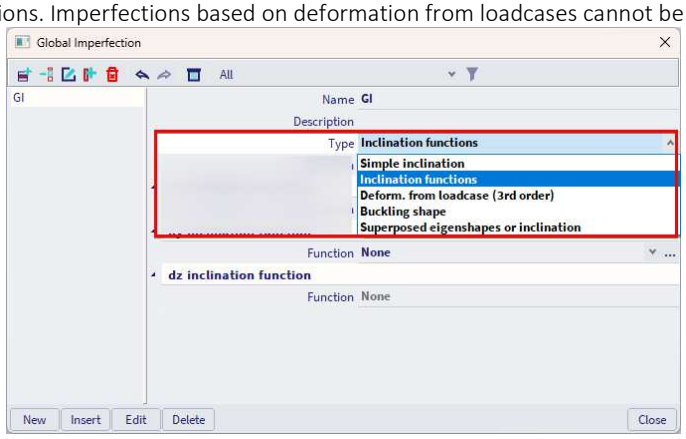
In this example, two global imperfections functions will be used: one according to global X-direction and one according to the global Y direction. It is not necessary to combine both imperfections in the same combination.

In SCIA Engineer you can choose to set no **global imperfection** (None) or you can define a Global Imperfection (here GI) in the nonlinear combinations (**Menu bar > Libraries > Load cases, combinations > Nonlinear combinations**).



There are four options for the **global imperfections** (**Menu bar > Libraries > Load cases, combinations > Global Imperfections**):

- **Simple inclination**: the imperfection is defined as a simple inclination. The inclination is defined in mm per m height of the structure. More specifically a horizontal displacement is given in the global X and/or Y direction which has a linear relation to the height (global Z direction).
- **Inclination functions**: the imperfection is defined by a deformation-to-height curve, similar to the simple inclination. The curve can then be assigned to an appropriate non-linear combination. These inclination functions are entered through Initial deformations (this option is explained below)
- **Deformation from loadcase**: with this option you can choose for a calculated loadcase and the deformation of that loadcase will be used as initial global imperfection. . This option can be used to take into account for example the imperfections due to the self-weight. Especially for slender beams this can be important.
- **Buckling shape**: the imperfection is derived from the buckling data
- **Superposed eigenschapes or inclination:** the imperfection is derived as a combination of global imperfections. Imperfections based on deformation from loadcases cannot be used to superpose.



In this example the option **Inclination functions** has been chosen.

These inclination functions are entered through **Menu bar > Libraries > Structure and analysis > Initial deformations**.

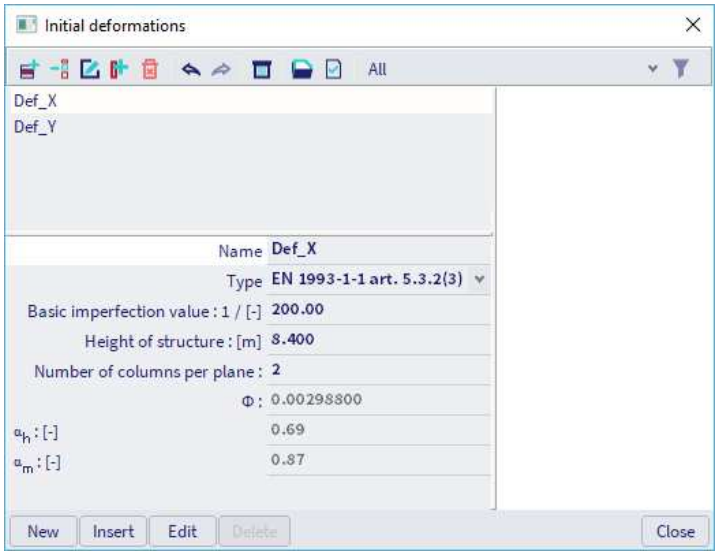
The **Type** is chosen as **EN 1993-1-1 art. 5.3.2(3)**, with a **basic imperfection** of **1/200**. In this way the inclination function is calculated according to the code EN 1993-1-1.

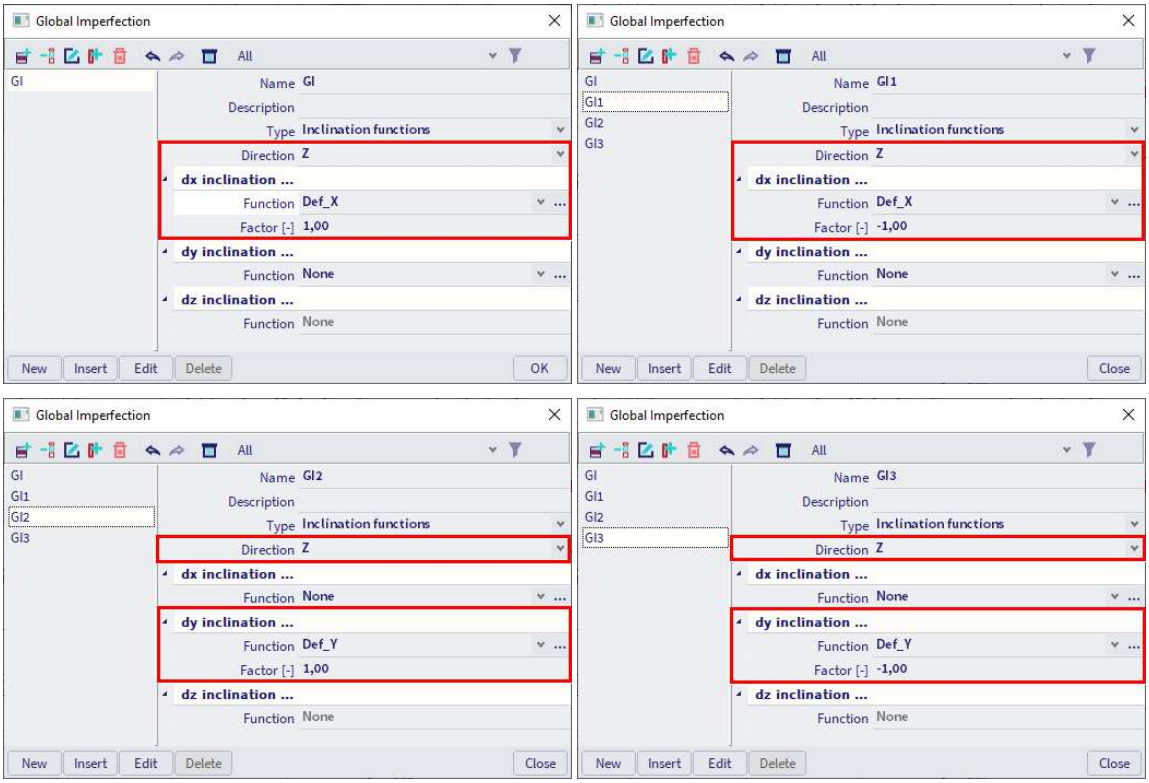
The **height** of the construction is **8,4 m** for both inclination functions.

There are 6 columns in the X-direction, but in the middle spans 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 (GI with factor 1)
- NC6-NC10: inclination in the negative X-direction (GI1 with factor -1)
- NC11-NC15: inclination in the positive Y-direction (GI2 with factor 1)
- NC16-NC20: inclination in the negative Y-direction (GI3 with factor -1)

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, the linear calculation first needs to be executed before the nonlinear combinations can be created.

8.4. Bow imperfection

8.4.1. N_{Ed} > 25% N_{cr}

The relative initial local bow imperfection 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):

> (6) When performing the global analysis for determining end forces and end moments to be used in member checks according to 6.3 local bow imperfections may be neglected. However for frames sensitive to second order effects local bow imperfections of members additionally to global sway imperfections (see 5.2.1(3)) should be introduced in the structural analysis of the frame for each compressed member where the following conditions are met:
>
> – at least one moment resistant joint at one member end
>
> – $\bar{\lambda} > 0{,}5\sqrt{\frac{A\,f_y}{N_{Ed}}}$ (5.8)
>
> where N_{Ed} is the design value of the compression force
>
> and $\bar{\lambda}$ is the in-plane non-dimensional slenderness calculated for the member considered as hinged at its ends
>
> **NOTE** Local bow imperfections are taken into account in member checks, see 5.2.2 (3) and 5.3.4.

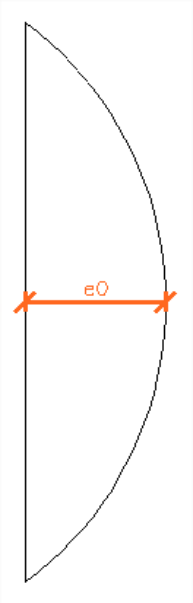
So:

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} > 0{,}5 \sqrt{\frac{A \cdot f_y}{N_{Ed}}} \rightarrow \sqrt{\frac{A \cdot f_y}{N_{cr}}} > 0{,}5 \sqrt{\frac{A \cdot f_y}{N_{Ed}}} \text{ or } \frac{1}{N_{cr}} > 0{,}25 \frac{1}{N_{Ed}} \text{ or } \mathbf{N_{Ed} > 0{,}25\, N_{cr}}$$

So, the bow imperfection must be applied when the normal force N_{Ed} in a member is higher than 25% of the member's critical load N_{cr}.

8.4.2. 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

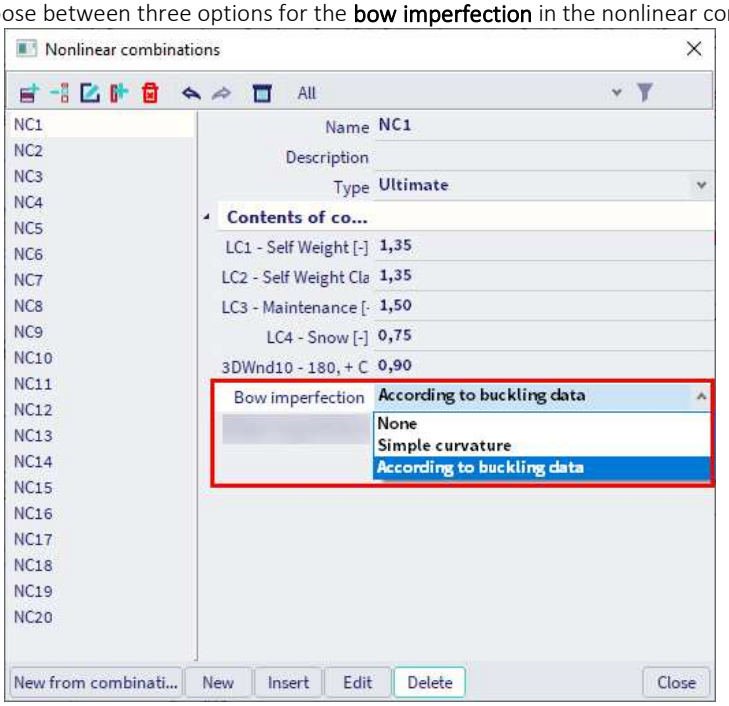
Where L is the member length.

As explained in the previous chapter, the bow imperfection must 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}, you 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) you need to input the buckling curve manually.

Example: **Industrial hall.esa**

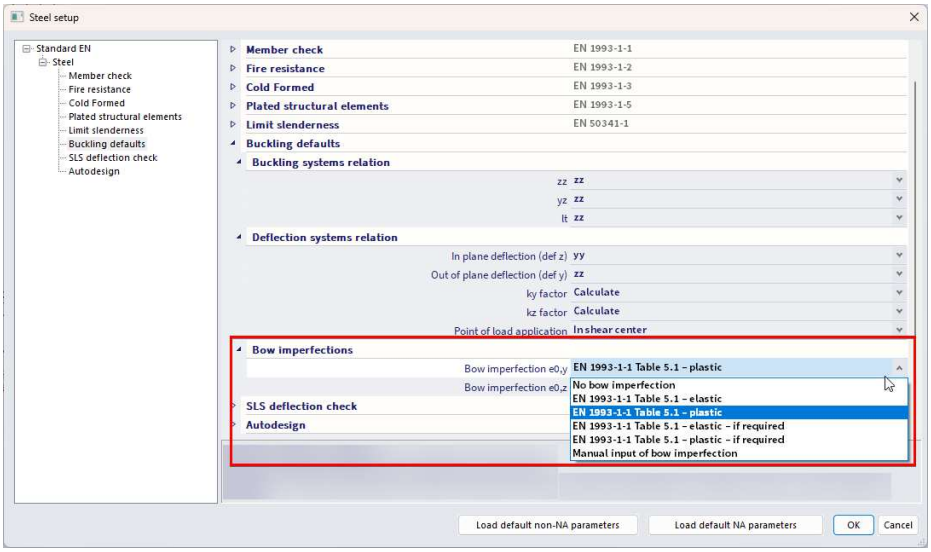
In SCIA Engineer you can choose between three options for the **bow imperfection** in the nonlinear combinations:



- **None**: no bow imperfection will be taken into account
- **Simple curvature**: you will input a deformation 'f' or '1/f' for all the members, where '1/f' corresponds with the value 'e_0/L' as explained earlier.
- **According to buckling data**: with this option you 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.

Since SCIA Engineer 21.0 the buckling data can be defined via the **steel settings**:



It is also possible to define the bow imperfections on buckling groups level (via Buckling groups or System lengths and buckling groups).

You can choose between 6 options:

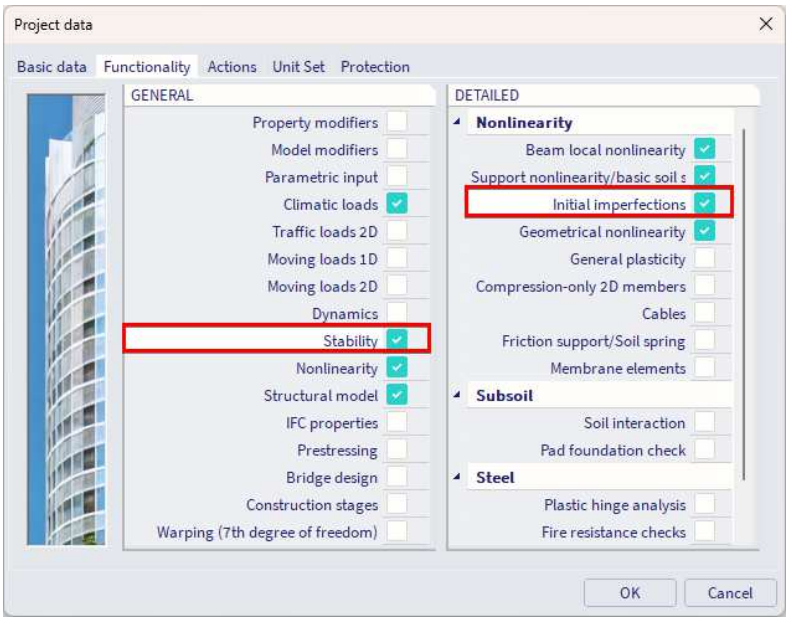
- **EN 1993-1-1 Table 5.1 – elastic**: the elastic value following the buckling curve of the cross-section will be used
- **EN 1993-1-1 Table 5.1 – plastic**: the plastic value following the buckling curve of the cross-section will be used
- **EN 1993-1-1 Table 5.1 – elastic – if required**: the elastic value following the buckling curve of the cross-section will be used if $N_{Ed} > 25\% N_{cr}$
- **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 $N_{Ed} > 25\% N_{cr}$
- **Manual input of bow imperfection**: you 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 **EN 1993-1-1 Table 5.1 – plastic** has been inputted.

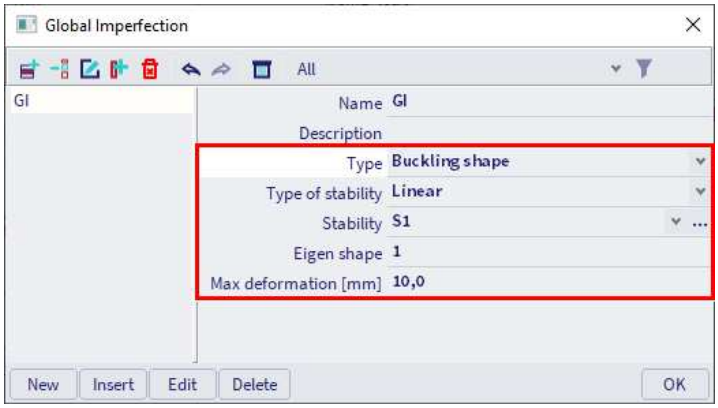
8.5. Buckling shape as imperfection - η_{cr}

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 functionalities **Nonlinearity > Initial imperfections** and **Stability** must be activated:



So first, the stability calculation (linear or nonlinear) is calculated. Afterwards in the global imperfection dialog, you can choose the buckling shape you want to be considered. First you can choose for the stability combination and just below for the calculated mode:



Then you can assign this global imperfection to the desired nonlinear combination.

The last option that must 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 the 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 **IPE300**, is fabricated from **S 235** and has the following relevant properties:



$E = 210000\ N/mm^2$
$f_y = 235\ N/mm^2$
$\gamma_{M1} = 1{,}00$
$L = 5000\ mm$
$A = 5380\ mm^2$
$I_y = 83560000\ mm^4$
$I_z = 6040000\ mm^4$
$W_{pl,y} = 628400\ mm^3$
$W_{pl,z} = 125000\ mm^3$

Calculation of the buckling shape

First a **stability calculation** is done using a load of 1 kN. This way, the elastic critical buckling load N_{cr} is obtained.

To obtain precise results, the **Number of 1D elements** is set to **10**. This can be done via **Menu bar > Tools > Calculation & Mesh > Mesh settings**. 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 for the critical load coefficients (**Menu bar > Results > Critical load coefficients**):

Critical load coefficients

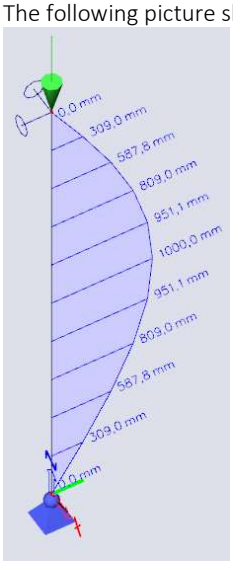
N -	f []
Stability combination : S1	
1	500.75
2	2003.39
3	4511.38
4	6927.50

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

$$N_{cr,y} = \frac{\pi^2 EI_y}{l^2} = \frac{\pi^2 \cdot 210000 N/mm^2 \cdot 8{,}3560 \cdot 10^7 mm^4}{(5000mm)^2} = 6927{,}515\ kN$$

For the weak axis we can calculate $N_{cr,z}$ with I_z to become 500,744 kN.

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



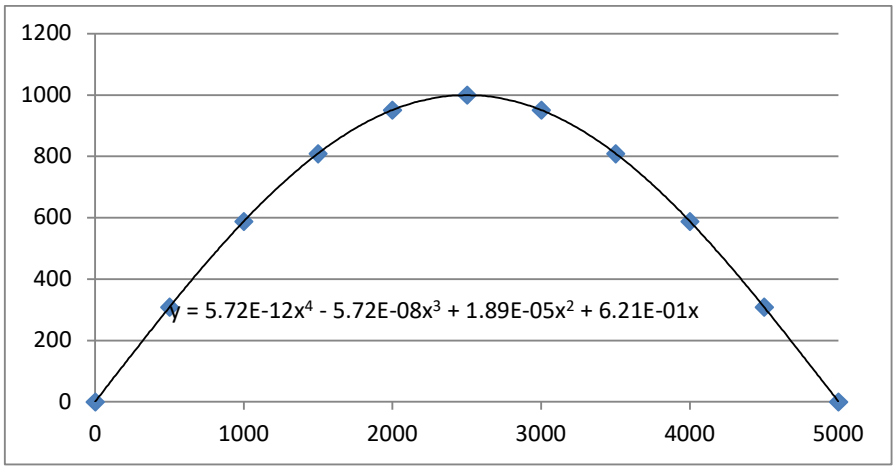
RESULTS TABLE — 1D deformations; Stability calculation. Buckling shapes are normali...

	Name	dx [m]	Case	ux [mm]	uy [mm]	uz [mm]	φx [mrad]	φy [mrad]	φz [mrad]	Utotal [mm]
1	B1	0,000	S1/1 - 500,75	0,0	0,0	0,0	-628,3	0,0	0,0	0,0
2	B1	0,500-	S1/1 - 500,75	0,0	309,0	0,0	-597,6	0,0	0,0	309,0
3	B1	0,500+	S1/1 - 500,75	0,0	309,0	0,0	-597,6	0,0	0,0	309,0
4	B1	1,000-	S1/1 - 500,75	0,0	587,8	0,0	-508,3	0,0	0,0	587,8
5	B1	1,000+	S1/1 - 500,75	0,0	587,8	0,0	-508,3	0,0	0,0	587,8
6	B1	1,500-	S1/1 - 500,75	0,0	809,0	0,0	-369,3	0,0	0,0	809,0
7	B1	1,500+	S1/1 - 500,75	0,0	809,0	0,0	-369,3	0,0	0,0	809,0
8	B1	2,000-	S1/1 - 500,75	0,0	951,1	0,0	-194,2	0,0	0,0	951,1
9	B1	2,000+	S1/1 - 500,75	0,0	951,1	0,0	-194,2	0,0	0,0	951,1
10	B1	2,500-	S1/1 - 500,75	0,0	1000,0	0,0	0,0	0,0	0,0	1000,0
11	B1	2,500+	S1/1 - 500,75	0,0	1000,0	0,0	0,0	0,0	0,0	1000,0
12	B1	3,000-	S1/1 - 500,75	0,0	951,1	0,0	194,2	0,0	0,0	951,1
13	B1	3,000+	S1/1 - 500,75	0,0	951,1	0,0	194,2	0,0	0,0	951,1
14	B1	3,500-	S1/1 - 500,75	0,0	809,0	0,0	369,3	0,0	0,0	809,0
15	B1	3,500+	S1/1 - 500,75	0,0	809,0	0,0	369,3	0,0	0,0	809,0
16	B1	4,000-	S1/1 - 500,75	0,0	587,8	0,0	508,3	0,0	0,0	587,8
17	B1	4,000+	S1/1 - 500,75	0,0	587,8	0,0	508,3	0,0	0,0	587,8
18	B1	4,500-	S1/1 - 500,75	0,0	309,0	0,0	597,6	0,0	0,0	309,0
19	B1	4,500+	S1/1 - 500,75	0,0	309,0	0,0	597,6	0,0	0,0	309,0
20	B1	5,000	S1/1 - 500,75	0,0	0,0	0,0	628,3	0,0	0,0	0,0

1D deformations

You can ask for this result via **Menu bar > Results > 3D deformations** or **Menu bar > Results > 1D members > deformations**.

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.

- $\eta_{cr} = 5{,}72 \cdot 10^{-12} \cdot x^4 - 5{,}72 \cdot 10^{-8} \cdot x^3 + 1{,}89 \cdot 10^{-5} \cdot x^2 + 6{,}21 \cdot 10^{-1} \cdot x$
- $\eta''_{cr,max} = 6{,}86 \cdot 10^{-11} \cdot x^2 - 3{,}43 \cdot 10^{-7} \cdot x + 3{,}78 \cdot 10^{-5}$

<u>Calculation of e_0</u>

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

With:

- $N_{Rk} = f_y \cdot A = 235 \frac{N}{mm^2} \cdot 5380\ mm^2 = 1\ 264\ 300\ N$
- $M_{Rk,y} = f_y \cdot W_{pl,y} = 235 \frac{N}{mm^2} \cdot 628\ 000\ mm^3 = 147\ 580\ 000\ Nmm$ (class 2)
- $M_{Rk,z} = f_y \cdot W_{pl,z} = 235 \frac{N}{mm^2} \cdot 125\ 000\ mm^3 = 29\ 375\ 000\ Nmm$ (class 2)
- $\overline{\lambda_y} = \sqrt{N_{Rk}/N_{cr,y}} = \sqrt{1\ 264\ 300\ N/6\ 927\ 515\ N} = 0{,}43$
- $\overline{\lambda_z} = \sqrt{N_{Rk}/N_{cr,z}} = \sqrt{1\ 264\ 300\ N/500\ 744\ N} = 1{,}59$
- $\alpha = 0{,}21$ for buckling curve a (y-y)
- $\alpha = 0{,}34$ for buckling curve b (z-z)

These intermediate results can be verified through SCIA Engineer when performing a steel code check on the column:

Flexural Buckling check

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

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	5,000	5,000	m
Buckling factor	k	1,00	1,00	
Buckling length	l_{cr}	5,000	5,000	m
Critical Euler load	N_{cr}	6927,50	500,75	kN
Slenderness	λ	40,12	149,22	
Relative slenderness	λ_{rel}	0,43	1,59	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	

$$e_{0,y} = 0{,}21 \cdot (0{,}43 - 0{,}2) \cdot \frac{147\,580\,000\ \text{Nmm}}{1\,264\,300\ \text{N}} = \mathbf{5{,}569\ mm}$$

$$e_{0,z} = 0{,}34 \cdot (1{,}59 - 0{,}2) \cdot \frac{29\,375\,000\ \text{Nmm}}{1\,264\,300\ \text{N}} = \mathbf{10{,}972\ mm}$$

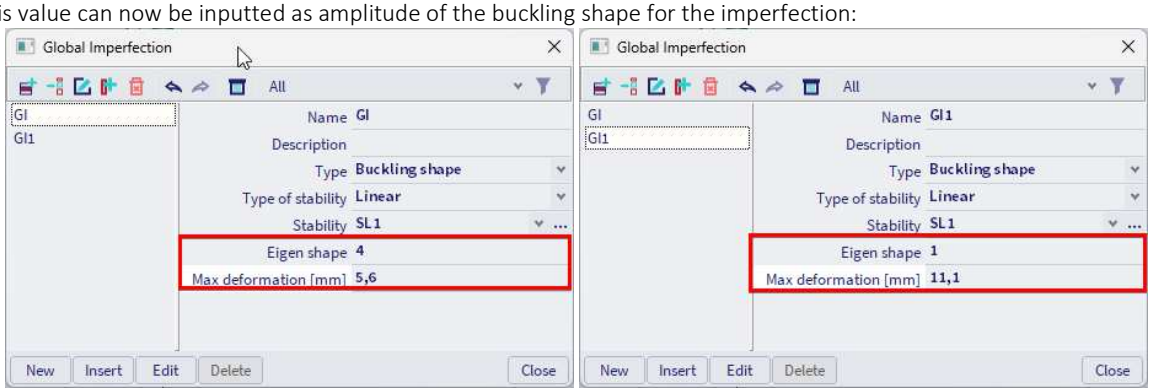
Calculation of η_{init}

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

The section in the middle of the column is decisive → x = 2500

- η_{cr} at mid section = 1000,31
- $\eta''_{cr,max}$ at mid section = $-3{,}912 \cdot 10^{-4} \cdot 1/\text{mm}^2$
- $\eta_{init,y} = e_{0,y} \cdot \frac{N_{cr,y}}{E \cdot I_y \cdot \eta''_{cr,max}} \cdot \eta_{cr} = 5{,}569\ \text{mm} \frac{6\,927\,515\ \text{N}}{\frac{210000\text{N}}{\text{mm}^2} \cdot 83\,560\,000\ \text{mm}^4 \cdot 3{,}912 \cdot 10^{-4} \cdot \frac{1}{\text{mm}^2}} \cdot 1000 = \mathbf{5{,}621\ mm}$
- $\eta_{init,z} = e_{0,z} \cdot \frac{N_{cr,z}}{E \cdot I_z \cdot \eta''_{cr,max}} \cdot \eta_{cr} = 10{,}972\ \text{mm} \frac{500\,744\ \text{N}}{\frac{210000\text{N}}{\text{mm}^2} \cdot 6\,040\,000\ \text{mm}^4 \cdot 3{,}912 \cdot 10^{-4} \cdot \frac{1}{\text{mm}^2}} \cdot 1000 = \mathbf{11{,}073\ mm}$

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

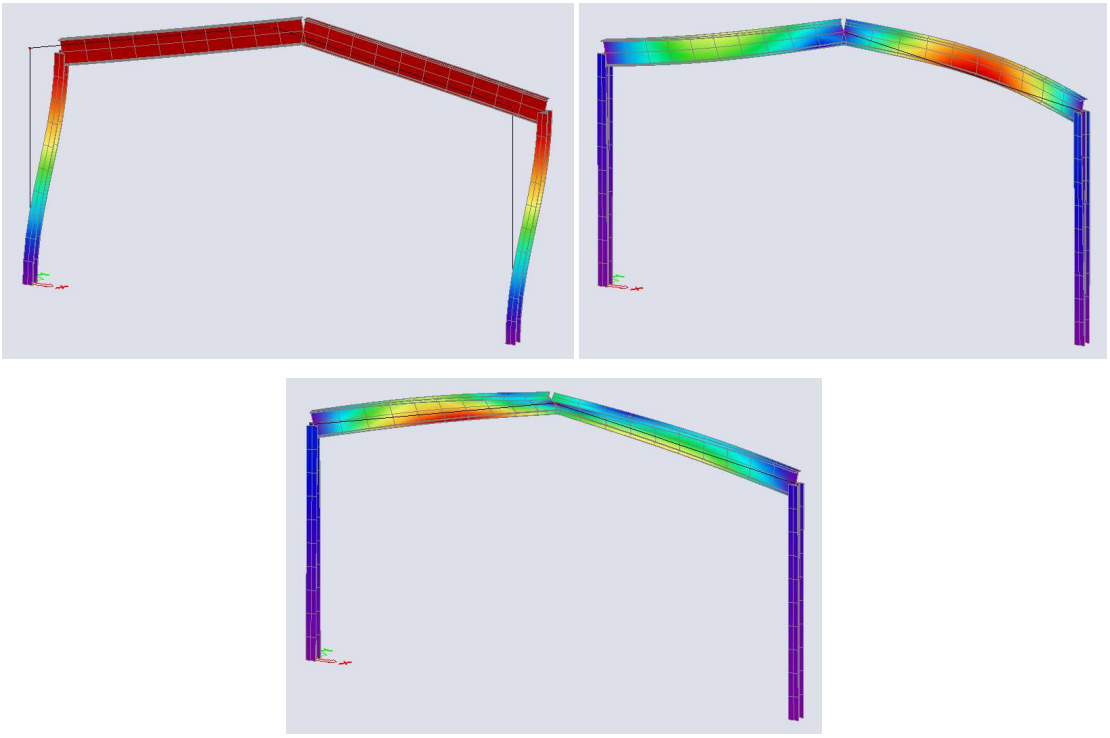


NC1 (with GI and load factor 400) will lead to a unity check of 0,48 while NC2 (with GI1 and load factor 400) will lead to a unity check of 1,04.

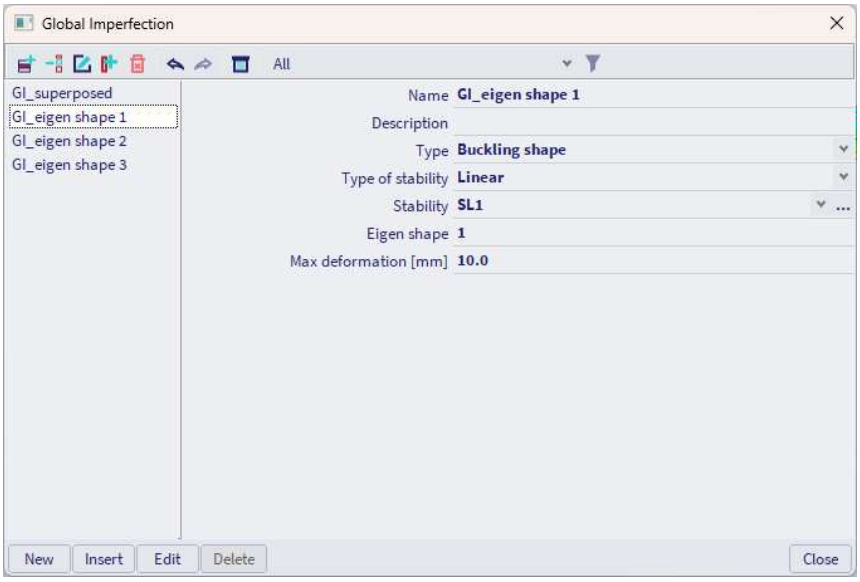
8.6. Superposed eigenshapes or inclination

Since SCIA Engineer 2027 it's possible to define a global imperfection via superposed eigenshapes or inclination.

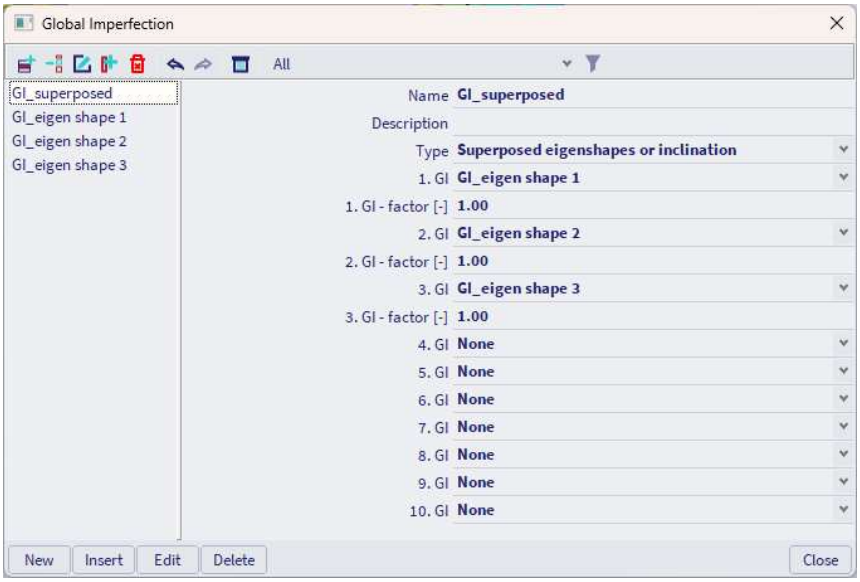
With the stability analysis multiple eigen shapes can be calculated, below three eigen shapes on a portal frame are illustrated:



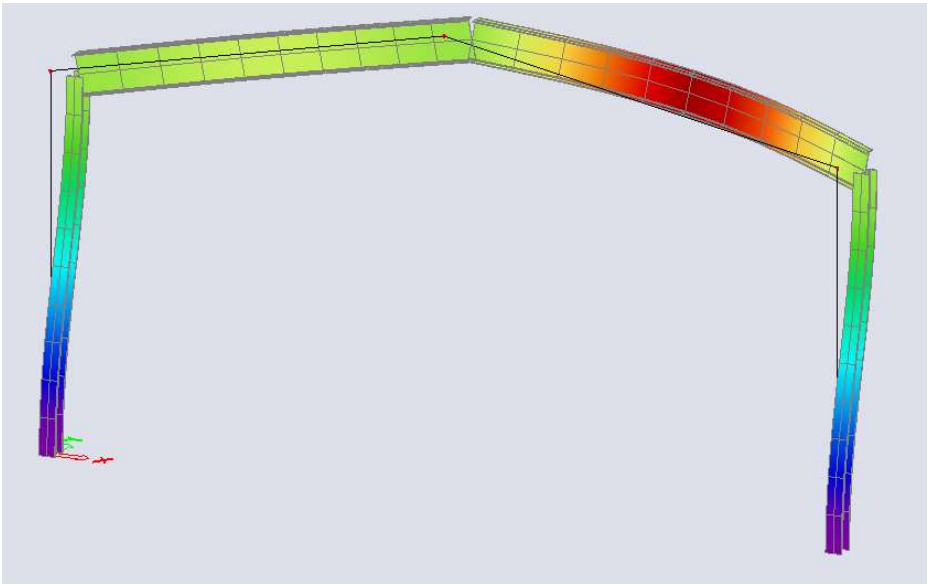
In the Global imperfection dialog an imperfection based on each individual eigen shape can be created:



Then with the Type Superposed eigenshapes or inclination these eigen shapes can be combined with each other into one global imperfection. The factor field can be used to give different weights to each shape:



As a result the global imperfection is a combination of the three shapes above:



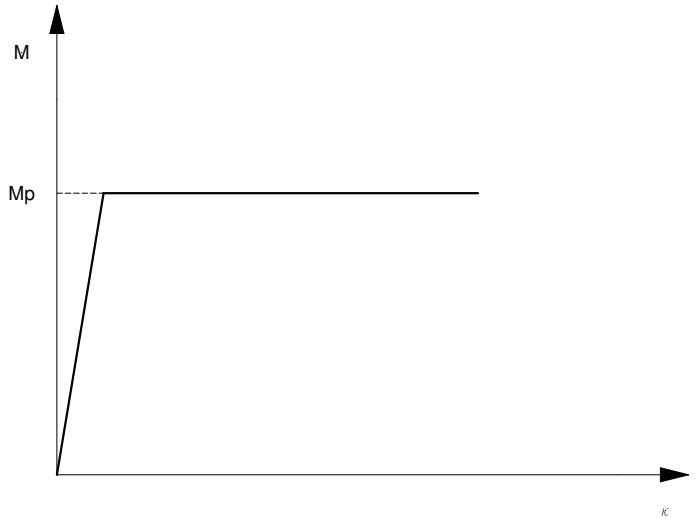
Chapter 9: Physical non-linearity

When the stresses are dependent on the strains in a nonlinear way, the nonlinearity is called a physical nonlinearity.

9.1. Plastic hinges

When a normal linear calculation is performed and limit stress is achieved in any part of the structure, the dimension of critical elements must be increased. If however, plastic hinges are taken into account, the achievement of limit stress causes the formation of plastic hinges at appropriate joints and the calculation can continue with another iteration step. The stress is redistributed to other parts of the structure and better utilization of overall load bearing capacity of the structure is obtained.

The material behaves linear elastic until the plastic limit is reached after which it behaves fully plastic. The σ-ε diagram thus has the same shape as the Moment-Curvature diagram:



The full plastic moment is given as **M_p**, the curvature as **k**.

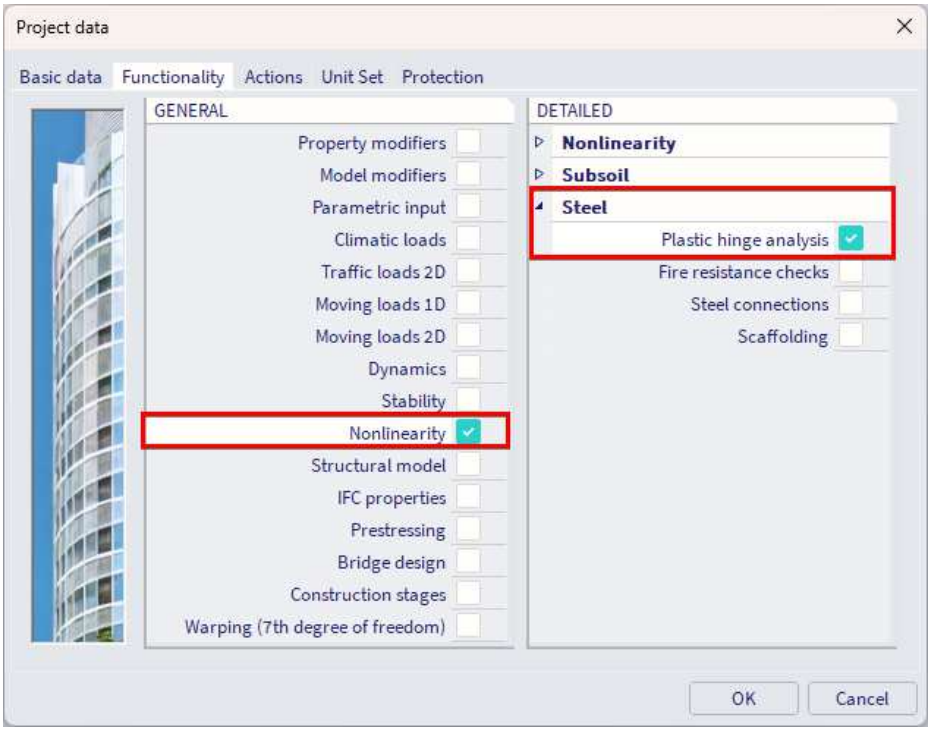
In SCIA Engineer, a plastic moment can only occur in a **mesh-node**. This implies that the mesh needs to be refined if a plastic hinge is expected to occur at another location than the member ends.

The reduction of the plastic moment has been implemented according to the following codes: **EC3**, **DIN 18800** and **NEN 6770**.

There is off course a risk when taking plastic hinges into account. If a hinge is added to the structure, the statically indeterminateness is reduced. If other hinges are added, it may happen that the structure becomes a mechanism. This would lead to a collapse of the structure and the (nonlinear) calculation is stopped.

On the other hand, plastic hinges can be used to calculate the plastic reliability margin of the structure. The applied load can be increased little by little (e.g. by increasing the load case coefficients in a combination) until the structure collapses. This approach can be used to determine the maximum load multiple that the structure can sustain.

To take into account plastic hinges for steel structures, the functionality **Steel > Plastic Hinge analysis** must be activated in the functionality tab of the project settings.



The choice of code which needs to be applied can be specified through **Menu bar > Tools > Calculation & Mesh > Solver settings.** The no code option simply follows the EC-EN logic.



It is not possible to choose on what members or in which nodes plastic hinges can be formed. If the functionality is activated, a plastic hinge will form wherever the moment resistance is exceeded regardless of the element's plastic capacity.

Example: **Plastic_Hinges.esa**

In this project, a continuous beam is considered. The beam has a cross-section type IPE300 and is fabricated in S 235.

According to Eurocode 3, the plastic moment around the y-axis is given by:

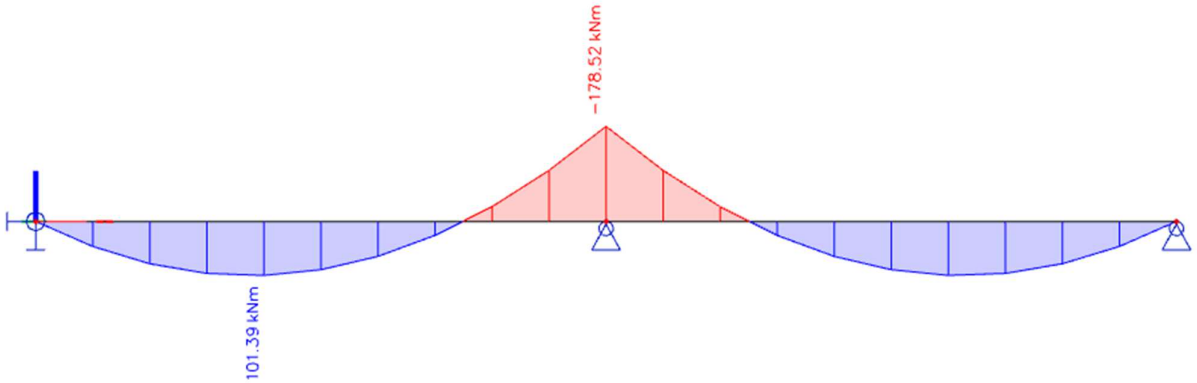
Axis system	Plastic bending moment resistance	Reduced plastic bending moment resistance for high normal force	Reduced plastic bending moment resistance for high shear force	Reduced plastic bending moment resistance for high normal force and high shear force
y-y	$M_{y,pl,Rd} = \frac{W_{pl,y} * f_y}{\gamma_{M0}}$	$M_{y,N,Rd} = M_{y,pl,Rd} * \frac{1-n}{1-0,5*a}$	$M_{y,V,Rd} = M_{y,pl,Rd} * (1-\rho_z)$	$M_{y,N,V,Rd} = M_{y,N,Rd} * (1-\rho_z)$
z-z	$M_{z,pl,Rd} = \frac{W_{pl,z} * f_y}{\gamma_{M0}}$	$M_{z,N,Rd} = M_{z,pl,Rd} * \left[1-\left(\frac{n-a}{1-a}\right)^2\right]$	$M_{z,V,Rd} = M_{z,pl,Rd} * (1-\rho_y)$	$M_{z,N,V,Rd} = M_{z,N,Rd} * (1-\rho_y)$

For the beam (no normal or high shear force present) considered this gives the following:

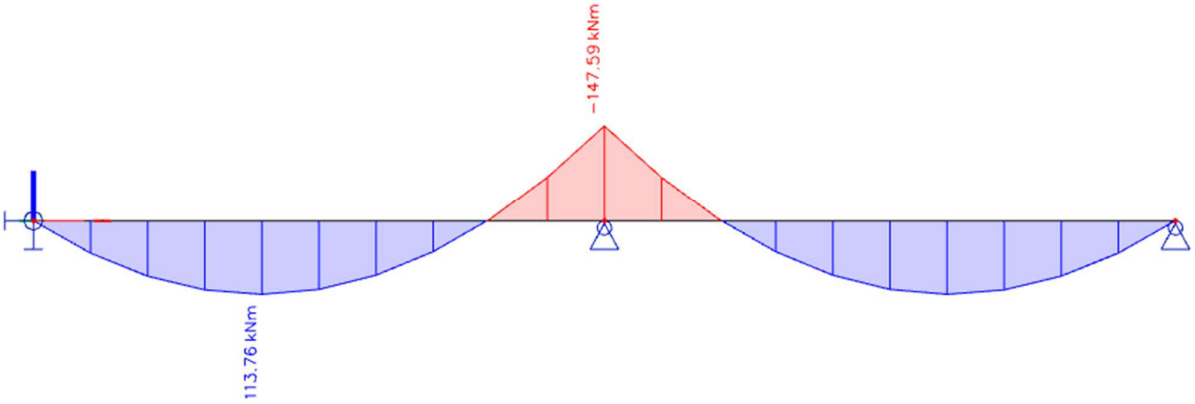
- f_y = 235 N/mm²
- $W_{pl,y}$ = 6,28 10^5 mm³
- γ_{M0} = 1,0

$$\Rightarrow M_{pl,y,d} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = 147{,}59 \text{ kNm}$$

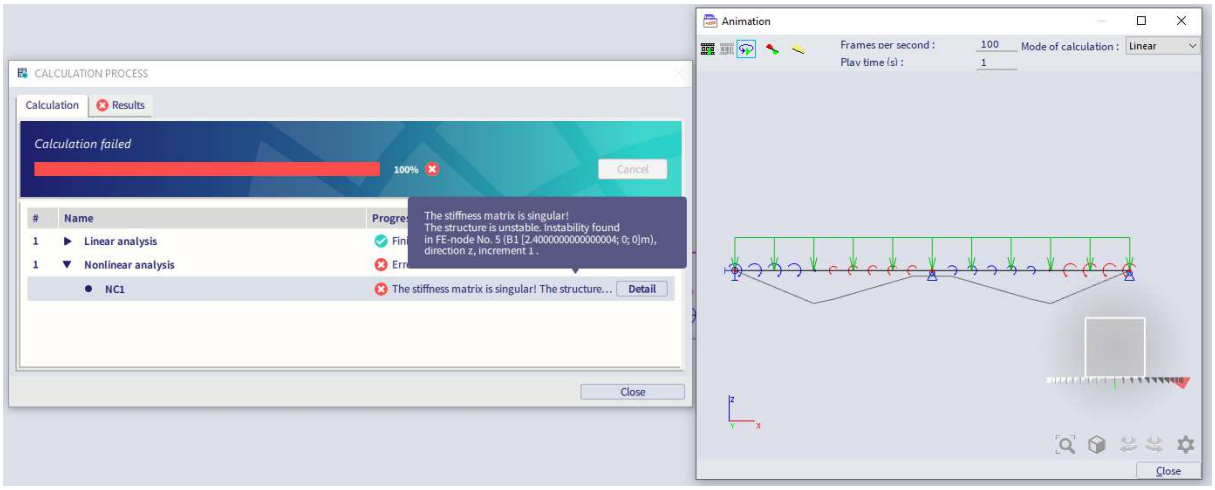
A **linear analysis** shows the following moment-diagram:



A **nonlinear analysis** taking into account plastic hinges gives the following result:



When the load is increased further, another plastic hinge will form in the middle of a span thus creating a mechanism. The nonlinear calculation will fail and a singularity message will be given. You can click on Detail to open an animation window which shows the expected scene, where an additional plastic hinge in the middle of the span leads to a mechanism



9.2. General plastic analysis

A general plastic analysis can be carried out for any 2D members (plates, walls and shells). The von Mises yield condition is currently available, which is suitable for ductile materials in general, such as metals (steel, aluminium, ...). It is a symmetric behaviour acting in the same way in tension and compression, with or without hardening in the plastic branch.

The plastic behaviour of materials may be combined with other types of nonlinearities in SCIA Engineer.

Note: plasticity is not supported yet for 1D members. The 1D members that are present in the model will be considered as elastic.

9.2.1. Von Mises yield criterion

In SCIA Engineer, the **von Mises yield criterion** is implemented.

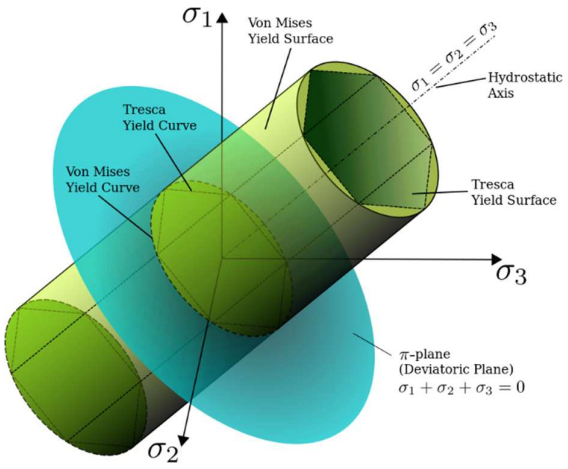
This criterion suggests that the yielding of materials begins when the second deviatoric stress invariant J2 reaches a critical value. For this reason, it is sometimes called the J2-plasticity or J2 flow theory. It is part of a plasticity theory that applies best to ductile materials, such as metals. Prior to yield, material response is assumed to be elastic.

In materials science and engineering the von Mises yield criterion can be also formulated in terms of the von Mises stress or equivalent tensile stress, σ_E, a scalar stress value that can be computed from the Cauchy stress tensor. In this case, a material is said to start yielding when its von Mises stress reaches a critical value known as the yield strength, σ_y. The von Mises stress is used to predict yielding of materials under any loading condition from results of simple uniaxial tensile tests. The von Mises stress satisfies the property that two stress states with equal distortion energy have equal von Mises stress.

The Von Mises stress is expressed as:

$$\sigma_E = \sqrt{\frac{1}{2} \cdot [(\sigma_{11} - \sigma_{22})^2 + (\sigma_{22} - \sigma_{33})^2 + (\sigma_{33} - \sigma_{11})^2 + 6 \cdot (\sigma_{12}^2 + \sigma_{23}^2 + \sigma_{31}^2)]}$$

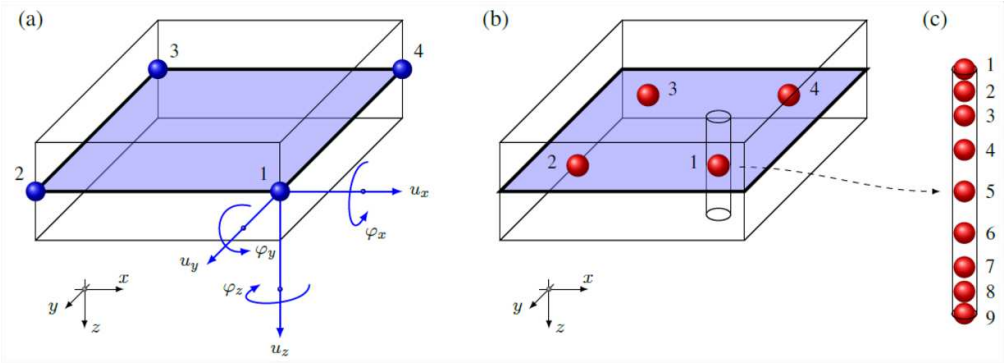
Because the von Mises yield criterion is independent of the first stress invariant, I1, it is applicable for the analysis of plastic deformation for ductile materials such as metals, as the onset of yield for these materials does not depend on the hydrostatic component of the stress tensor, just like the Tresca criterion.



9.2.2. Finite element model

Drilling rotations at each node is used for in-plane loading. This means that element has six degrees of freedom at each node and is therefore compatible with other types of elements (beam/solid elements).

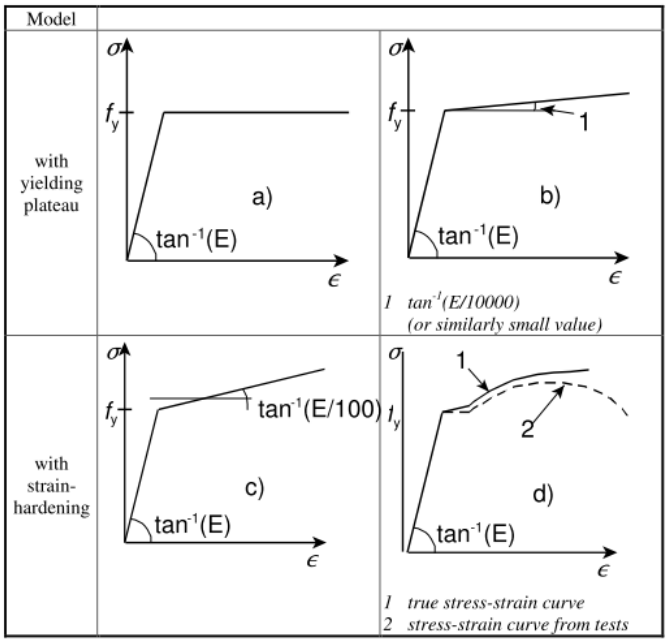
Within the element area the Gauss 2x2 quadrature points are used. Each of these Gauss quadrature points is realized by nine Gauss-Lobatto quadrature points throughout the thickness, so the four-node element has 2x2x9=36 quadrature points in total.



Due to these Gauss-Lobatto points the element can handle bending loading with high accuracy. In all of these points the nonlinear model is computed independently using the plane stress formulation. Linear transversal shear stiffness is assumed.

9.2.3. Material properties

Figure C.2 from EN 1993-1-5 is used for the material behaviour:



The different models are:

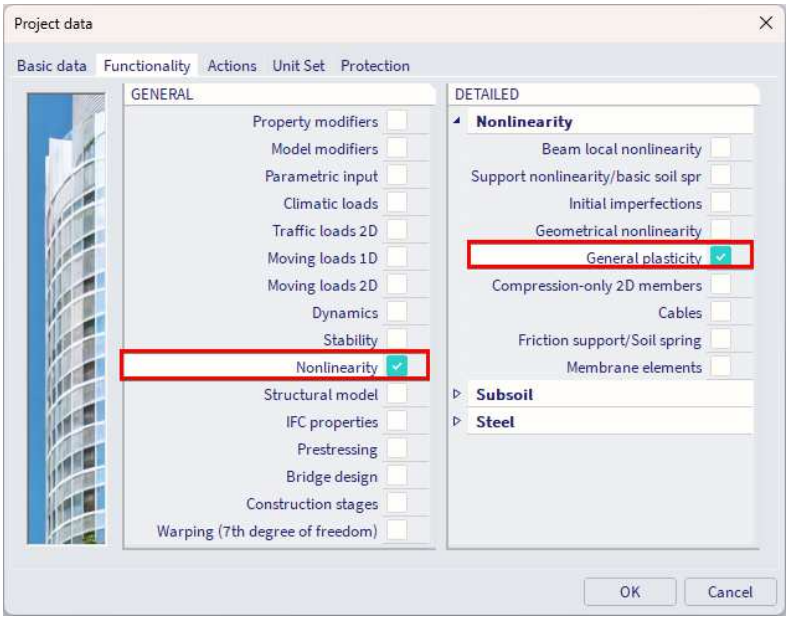
a) elastic-plastic without strain hardening
b) elastic-plastic with a nominal plateau slope
c) elastic-plastic with linear strain hardening
d) true stress-strain curve modified from the test results as follows:

$$\sigma_{true} = \sigma\,(1+\varepsilon)$$
$$\varepsilon_{true} = \ell n\,(1+\varepsilon)$$

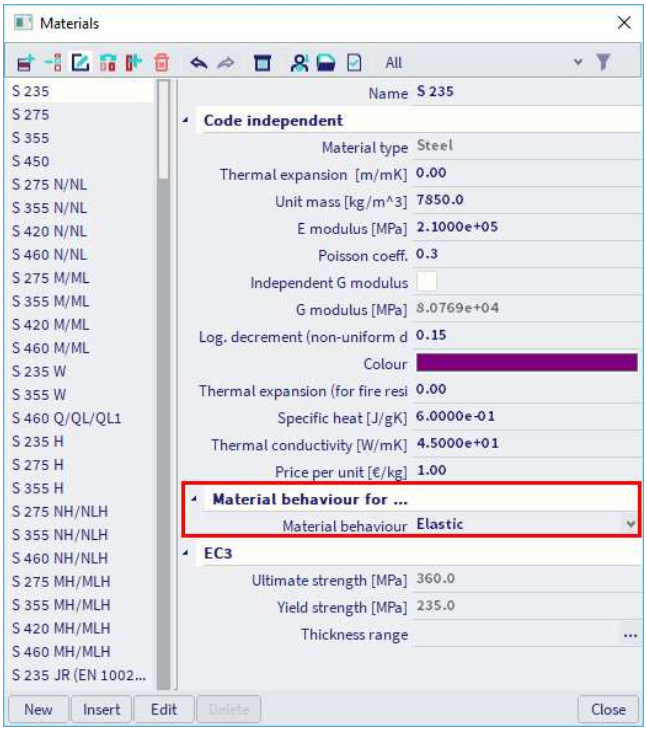
In SCIA Engineer, a), b) and c) are implemented.

9.2.4. General plasticity in SCIA Engineer

General plasticity is a specific type of nonlinearity in SCIA Engineer. This means that **General plasticity** is a subfunctionality of the nonlinear analysis.



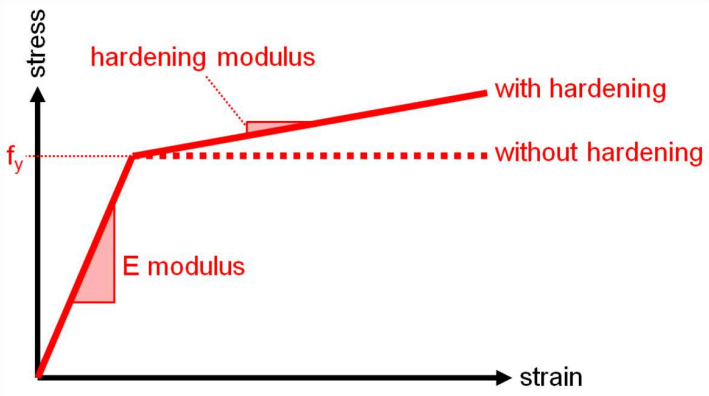
The nonlinearity of materials is defined directly in the material library. See the property group **Material behaviour for nonlinear analysis**.



By default, all materials in the library are set as elastic. This means, that the selected material will behave elastically during a nonlinear analysis. The plastic properties of materials are generic, code independent in SCIA Engineer and are therefore available for any material, regardless of the selected design code.

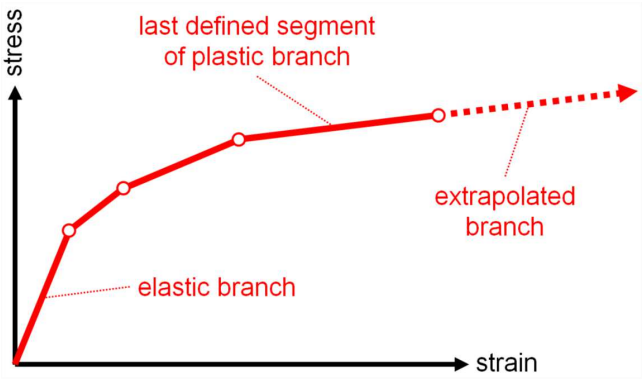
Plasticity can be enabled by selecting a type of plastic behaviour. For steel we can use **Isotropic elasto-plastic von Mises**. It corresponds to a bilinear stress-strain relationship, identical in tension and compression. The plastic branch may have a slope (hardening modulus) or not.

The stress-strain relationship is automatically generated from 3 parameters: Young's modulus (elastic part), yield stress for uniaxial tension and, optionally, hardening modulus (slope of the plastic branch).

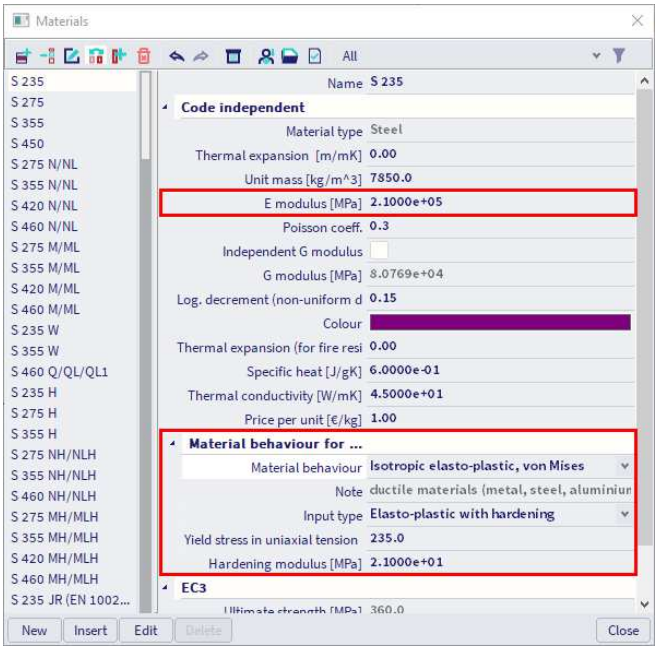


Only the tension part of the diagram is defined, as it is related to a plastification condition in general 3D stress state in principal stress directions. Some plastification models allow for a different yield stress in compression, which is defined separately. There is no limit (ultimate) strain value for the analysis.

When the actual strain value in the structure exceeds the defined diagram, the diagram is extrapolated, tangent to the last defined segment of the stress-strain relationship. The reason for that is, that the analysis would then fail and it would be impossible to find where the problem is located in the structure. It is therefore preferable, that the analysis continues and that you check the obtain strain values after the analysis.



The following parameters define the nonlinear behaviour of the material in the material library:



- **E modulus:** Young's modulus of the material which defines the slope of the elastic part of the stress-strain diagram
- **Material behaviour: t**he type of material behaviour for the nonlinear analysis has to be selected
  - *Elastic*
  - *Isotropic elasto-plastic, Tresca (ductile materials such as metal, steel, aluminium)*
  - *Isotropic elasto-plastic, von Mises (ductile materials such as metal, steel, aluminium)*
  - *Isotropic elasto-plastic, Drucker-Prager (materials with difference in compressive en tensile strength such as concrete and soil)*
  - *Isotropic elasto-plastic, Mohr-Coulomb (materials with difference in compressive en tensile strength such as concrete and soil)*
- **Input type: d**efines the definition of the plastic branch of the stress-strain diagram
  - *Elasto-plastic*: in the plastic domain, the stress remains constant when the strain increases
  - *Elasto-plastic*: in the plastic domain, the stress increases with the strain
- **Yield strength: e**lastic limit for plastification due to shear
- **Hardening modulus:** slope of the plastic branch of the stress-strain diagram

Note: Various types of nonlinearity may be combined in the same project. However it is not possible to cumulate several types of nonlinearity on the same 2D member. The property *FEM non-linear model* will behave as follows, when combined with a plastic material:

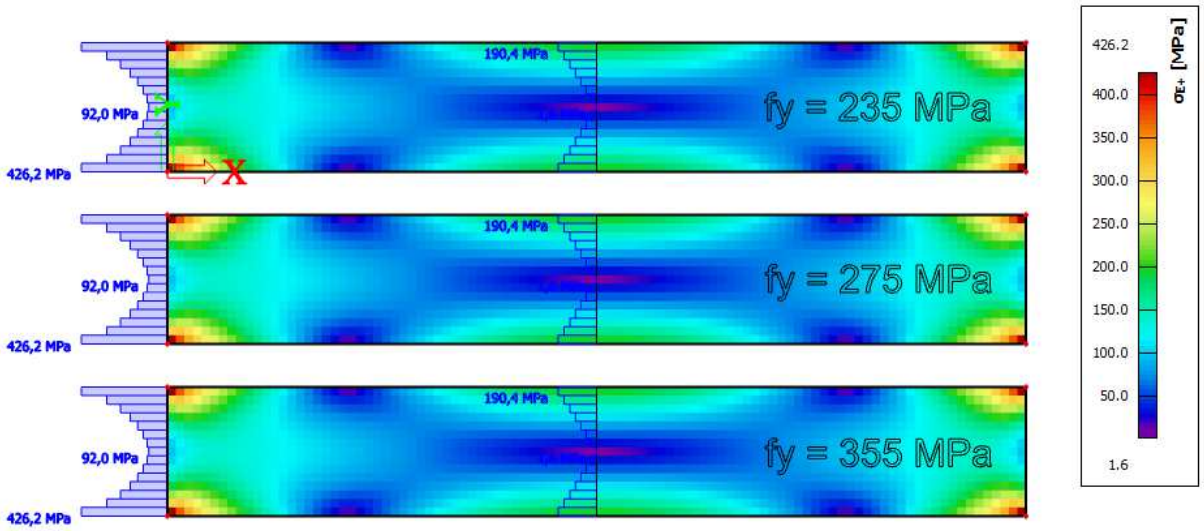
- Plastic material and *2D press-only* behaviour: the press-only behaviour will be ignored and the 2D member will behave as plastic.
- Plastic material and *membrane* behaviour: the plastic behaviour will be ignored and the 2D member will behave as an elastic membrane element.

When starting the analysis, a warning message will be displayed giving the same information about functionality conflicts.

Example: **Plastic_Plate_Stresses.esa**

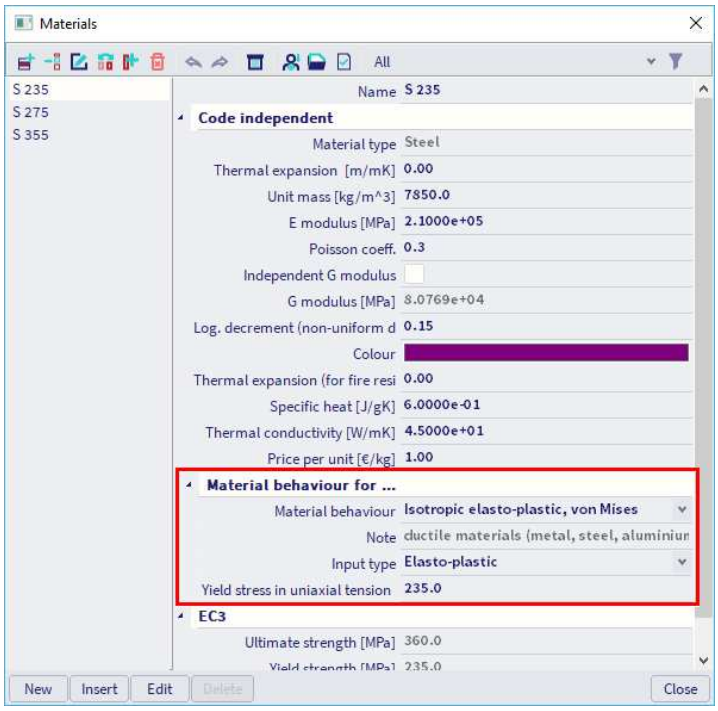
In this project, 3 vertical walls of different steel materials are loaded each time by the same vertical surface load of 242,90 kN/m². The value of the load is high enough to make sure that the Von Mises stresses in every wall are higher than the allowed yield strength fy of the steel materials.

A **linear analysis** shows the following results for the Von Mises stresses (Main menu > Results > 2D members > Stresses/strains):

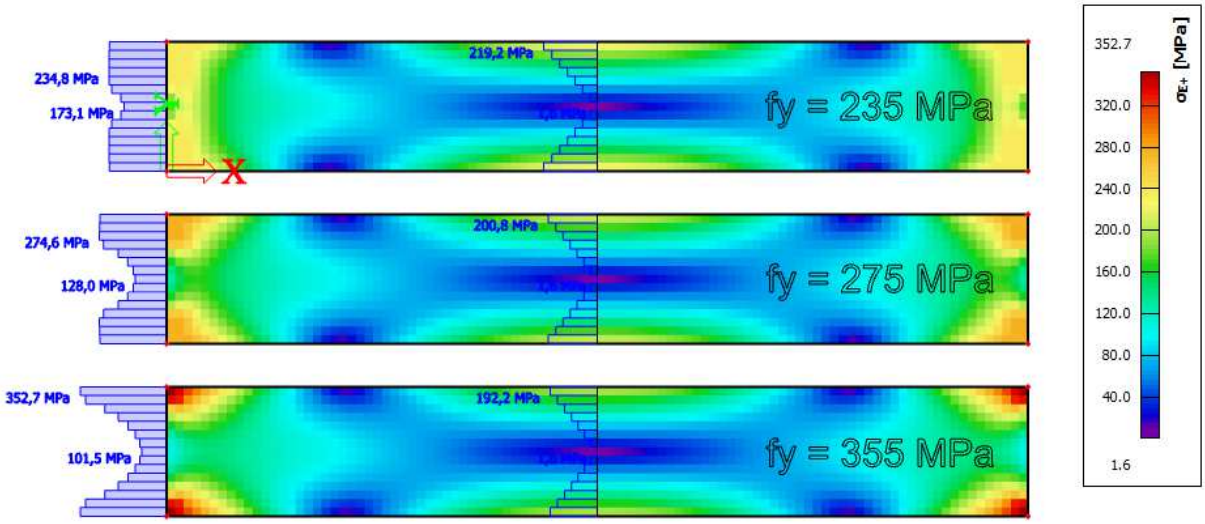


As expected, the Von Mises stresses for every wall are exactly the same and are higher than the yield strength fy.

For every used material, the following properties for the non-linear analysis are inserted:



The **nonlinear analysis** shows the following results for the Von Mises stresses:

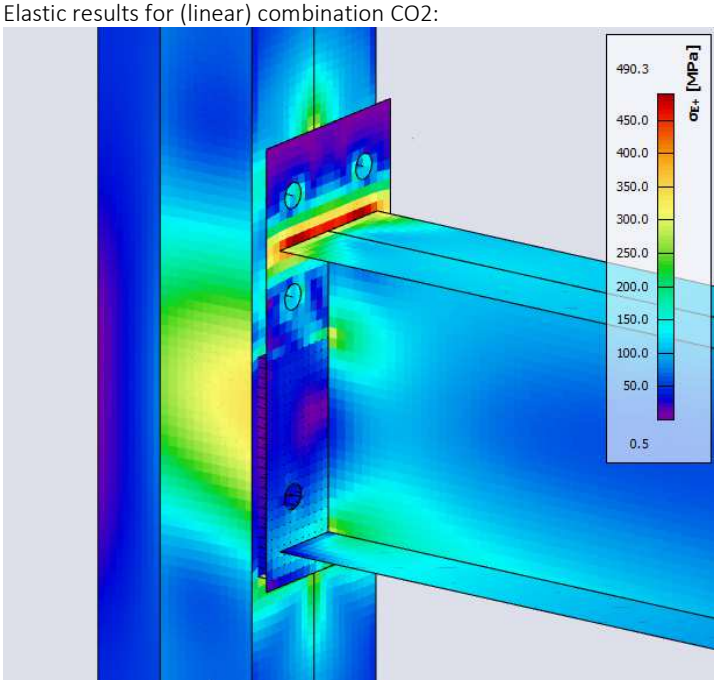


Example: **Beam_Column_Connection.esa**

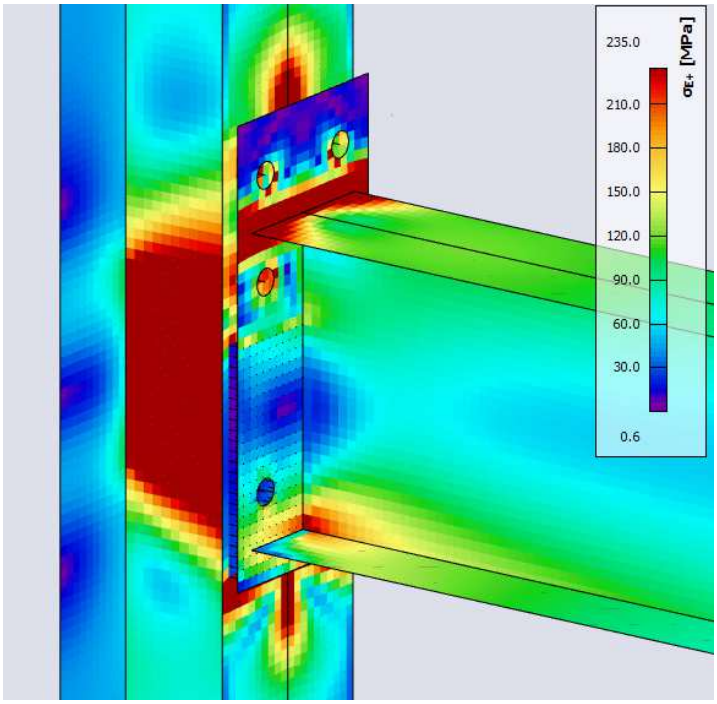
The functionality **General Plasticity** can also be used to model steel connections using finite elements to perform a plastic stress check.

In this example, a bolted column-beam connection is modelled using 2D finite elements in SCIA Engineer. That way the plastic stresses can be calculated by performing a nonlinear analysis.

Elastic results for (linear) combination CO2:



Plastic results for nonlinear combination NC2:



Chapter 10: SLS check

10.1. 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 **Menu bar > Results > Nodal displacement** and look at the combination **CO2 – SLS**.

Horizontal deformation

The maximum displacement in the X direction is 26,6 mm on a height of 6,9 m.

And in the Y-direction 27,3 mm on a height of 8,1 m.

Limit for Limit for horizontal deflection δ is h/150

- 6900/150 = 46 mm → 26,6 mm < 46 mm → OK
- 8100/150 = 54 mm → 27,3 mm < 54 mm → OK

Vertical deformation

The maximum displacement in the Z direction is 58,7 mm

Limit for Limit for vertical deflection δ is L/200

- 30000/200 = 150 mm → 58,7 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	**-26,6**	-0,5	0,0	-1,4	0,2	0,0	26,6
N114	CO2/2	**26,6**	-0,5	0,0	-1,4	-0,2	0,0	26,6
N70	CO2/3	0,0	**-27,3**	0,0	1,5	0,0	1,3	27,3
N60	CO2/4	0,0	**27,3**	0,0	-1,5	0,0	1,3	27,3
N109	CO2/5	13,4	0,0	**-58,7**	-0,2	-0,3	0,0	**60,2**
N82	CO2/6	10,0	-9,3	**14,5**	-26,8	-0,2	1,9	19,9
N80	CO2/6	9,7	-10,3	0,0	**-29,7**	0,0	1,3	14,2
N179	CO2/7	-9,7	10,3	0,0	**29,7**	0,0	1,3	14,2
N116	CO2/8	-9,9	-0,2	-21,5	-0,5	**-5,9**	0,0	23,7
N112	CO2/5	9,9	-0,2	-21,5	-0,5	**5,9**	0,0	23,7
N95	CO2/7	-9,8	-9,5	0,0	-27,7	0,0	**-4,3**	13,7
N87	CO2/6	9,8	-9,5	0,0	-27,7	0,0	**4,3**	13,7

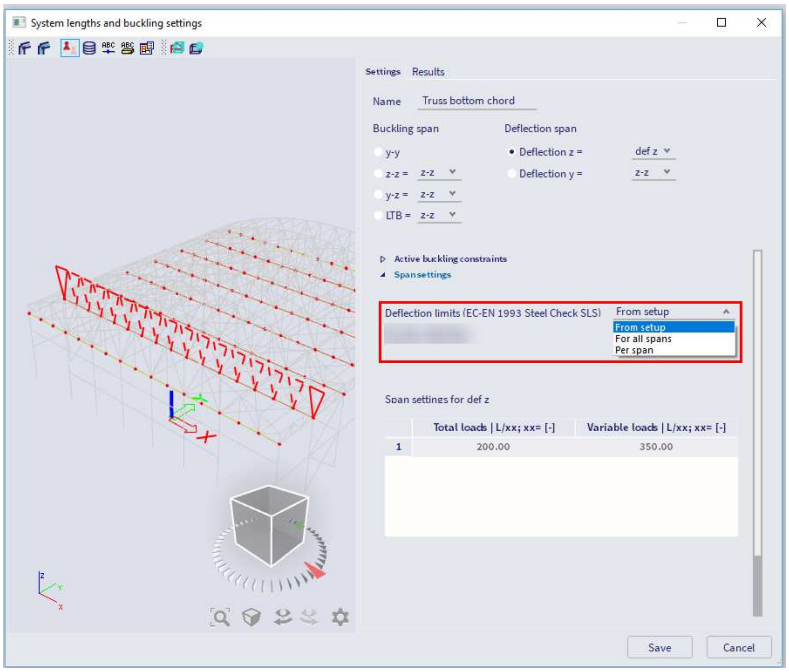
10.2. Relative deformation – SLS check

To perform a steel SLS check, the following data needs to be specified:

- the span of the elements to be checked;
- the deflection limits of the elements to be checked;
- the camber attributed to the elements (optional).

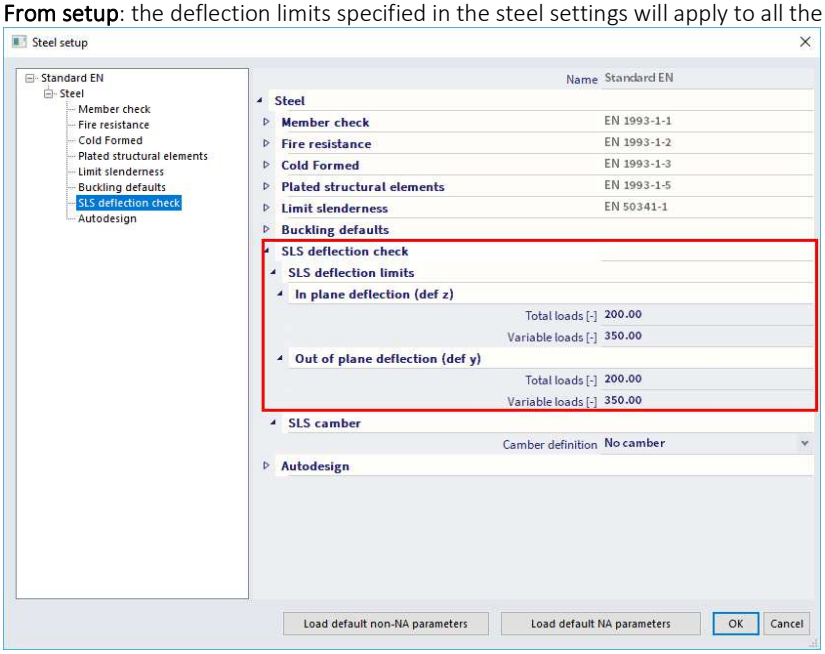
The span is defined in the dialog **System lengths and buckling groups**. There can also overwrite the deflection limits and camber values of the steel setup for a certain buckling group.

The deflection limits can be set in the **Span settings** part of the dialog. The deflection limits can be set independently for the local y and z directions (based on the active deflection option, defy or defz), and independently for total loads and variable loads.

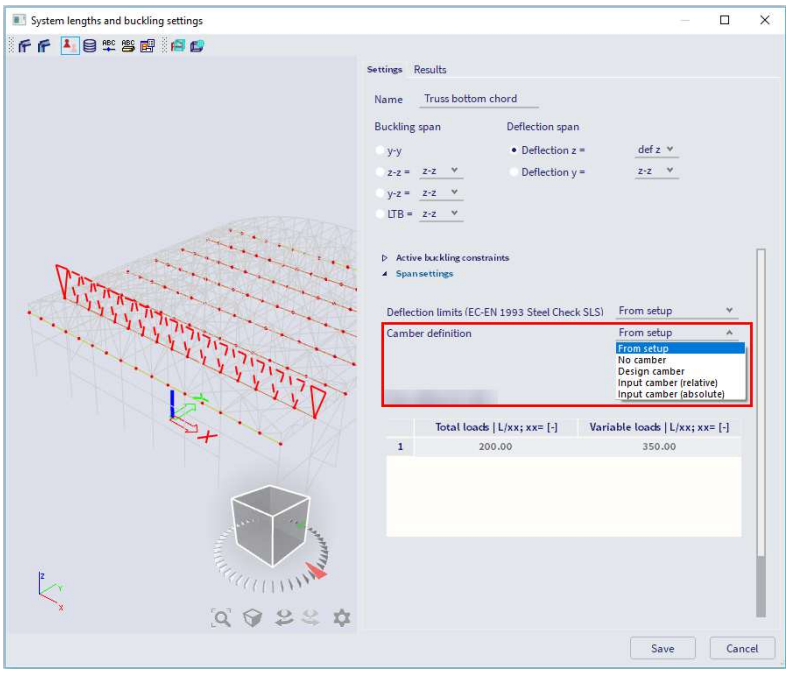


Three options are available:

- **For all spans**: the same deflection limits will apply to all the spans of the selected element(s)
- **Per span**: different deflection limits will apply to different spans
- **From setup**: the deflection limits specified in the steel settings will apply to all the spans



The camber can be set independently for the local z and y direction (based on the active deflection option, defy or defz) via the combo box:



Five options are available:

- **No camber**: no camber will be applied
- **Design camber**: the camber can be designed based on the deflection limits. You can define certain boundary conditions (camber shape, minimum camber, camber limit, maximum camber and rounding) This option is available since SCIA 19.1
- **Input camber (absolute)**: the camber value will be input, span by span, in absolute dimensions (length units)
- **Input camber (relative)**: the camber value will be input, span by span, as a relative coordinate (1/length units)
- **From setup:** the camber values specified in the steel setup will be applied to all spans

Note: The camber is defined with respect to the local z axis. If the local z axis is directed downward, you must input a negative value in order to have an upward camber.

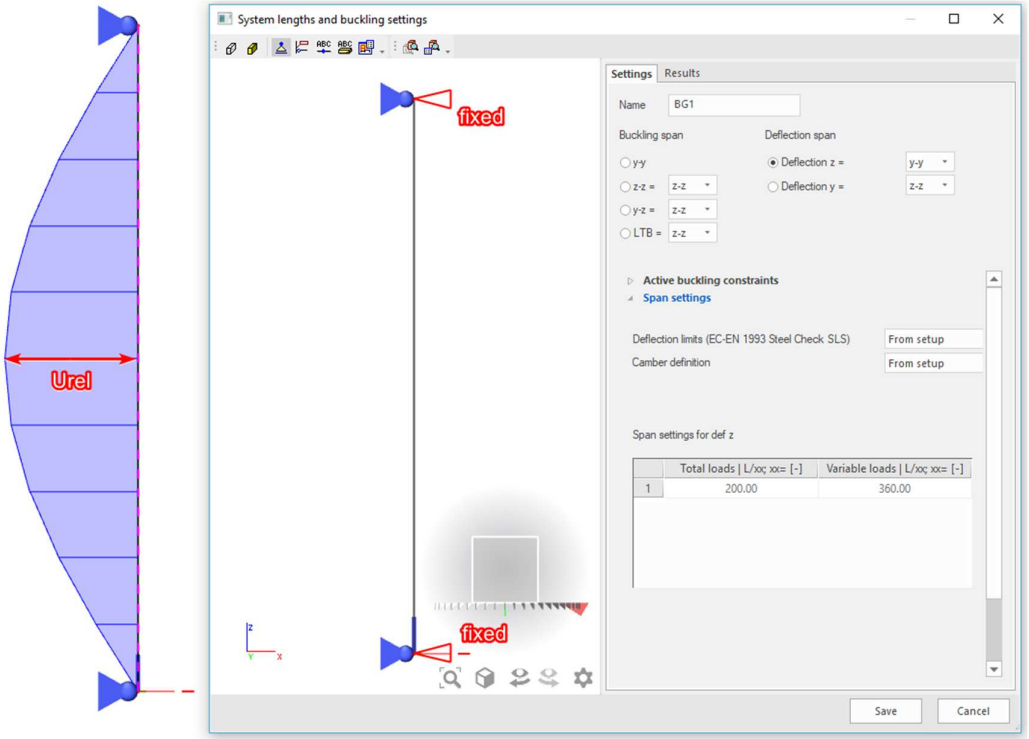
With the option **Menu bar > Design > Steel members > SLS check** or **Process toolbar > Steel > Steel SLS check**, the relative deformations can be checked. The relative deformations are given as absolute value, relative value related to the span, or as unity check related to the limit for the relative value to the span.

It is also possible to visualise the camber via the SLS check.

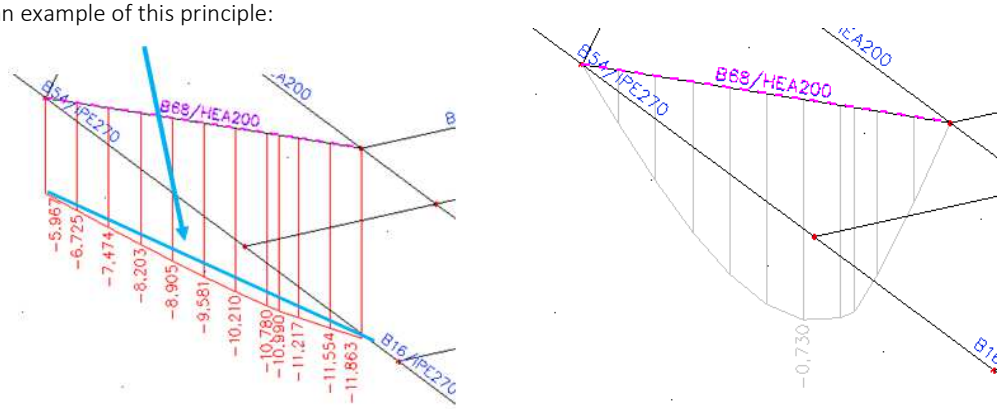
Two options for this span are possible and explained below.

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 of this principle:



Calculation for this relative deformation:

Deformation in the beginning of the beam = 5,967 mm and at the end = 11,863 mm.

Maximum deformation is at 0,979 m from the beginning at the beam.

So this point has already displaced 9,5 mm (see blue line at the picture):

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

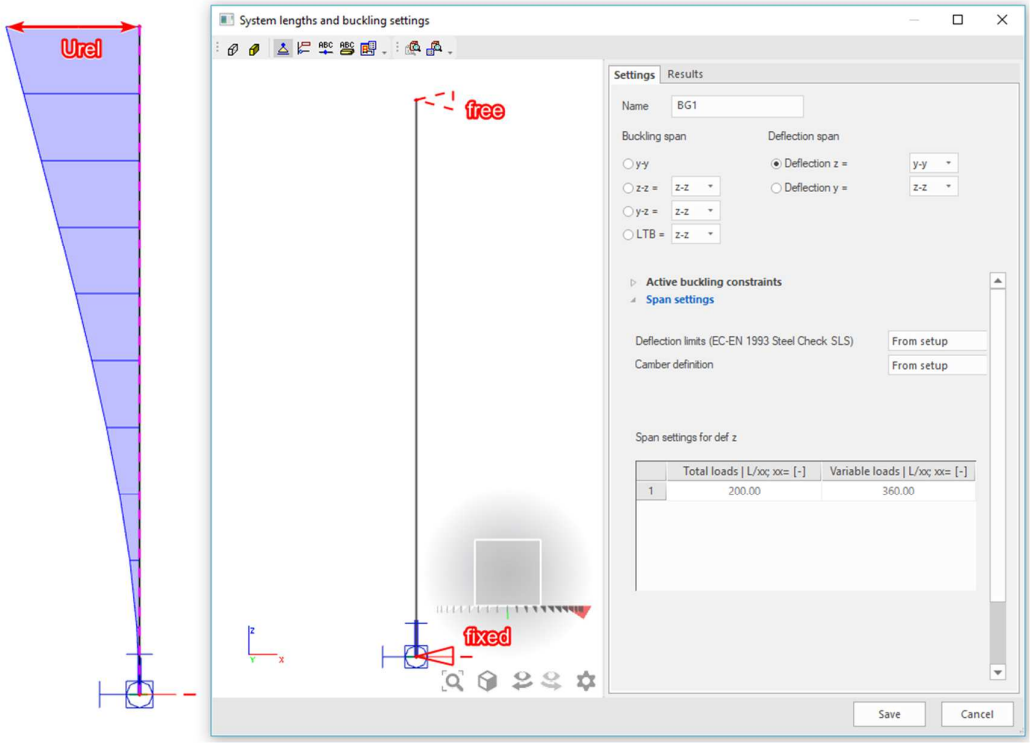
And $u_{z,\ \text{relative}}$ = 10,2 mm – 9,5 mm = 0,7 mm

And suppose the length of this beam = 1632 m:

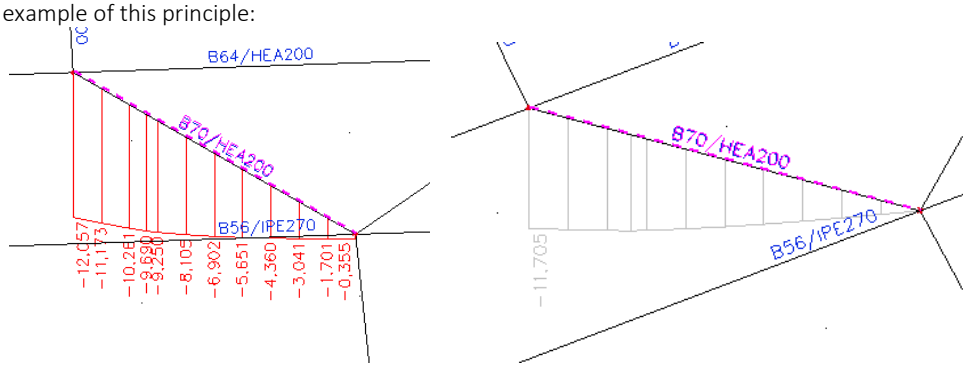
$$\text{rel uz} = \frac{0{,}73\ \text{mm}}{1632\ \text{mm}} = 1/2236$$

<u>1 node supported and the other node free</u> (for example cantilever beams)

This is the case if one end node is free:



In below an example of this principle:



Now the maximal relative deformation is taken as the deformation minus the deformation of the fixed node.

And $u_{z,relative}$ = 12,057 mm – 0,355 mm = 11,7 mm

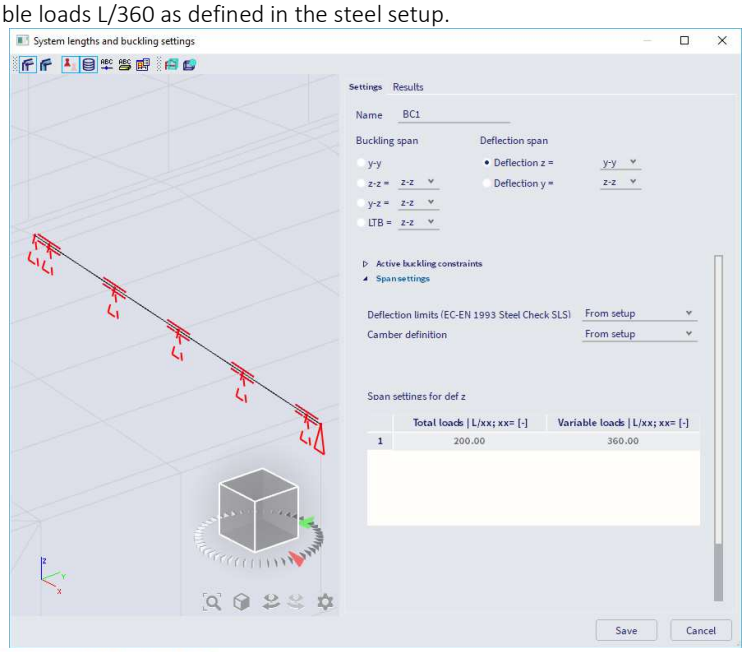
And suppose the length of this beam = 1632 mm:

$$\text{rel uz} = \frac{11{,}7\ \text{mm}}{1632\ \text{mm}} = 1/139$$

Now this principle is shown at the example of the industrial hall.

Example: **Industrial Hall.esa**

Consider beam B10. In the **System lengths and buckling settings** dialog the deflection limit for the total loads is taken as L/200 and for the variable loads L/360 as defined in the steel setup.



EC-EN 1993 Steel Check SLS

Linear calculation
Combination: CO2
Coordinate system: Principal
Extreme 1D: Global
Selection: B10
Overall Unity Check

Name	dx [m]	Case	$u_{y,max}$ [mm] $u_{z,max}$ [mm]	$u_{y,var}$ [mm] $u_{z,var}$ [mm]	Lim. $u_{y,max}$ [mm] Lim. $u_{z,max}$ [mm]	Lim. $u_{y,var}$ [mm] Lim. $u_{z,var}$ [mm]	Check $u_{y,max}$ [-] Check $u_{z,max}$ [-]	Check $u_{y,var}$ [-] Check $u_{z,var}$ [-]	Camber dx u_z [mm] Camber [mm]	Check Overall [-]
B10	14,325-	CO2/1	0,0 13,3	0,0 46,2	17,2 150,7	9,6 83,7	0,00 0,09	0,00 0,55	- -	**0,55**

Name	Combination key
CO2/1	LC1 + LC2 + 3DWnd3

Note: since SCIA Engineer 24.0 the output can also be displayed as relative values (L/xx) via property “Type of values = Overall Unity Check rel” in the property panel.

Around the y-y axis (in the z-direction), only the first node has been supported, so the beam behaves as a cantilever and the deflection limit will be adapted with a factor 2.

Length of the element = 15,075 m

Deformation $u_{z,max}$ = 13,3 mm (and 0 mm at the beginning of the beam)

→ $u_{z,max,relative}$ = 13,3 mm – 0,0 mm = 13,3 mm

→ $u_{z,max,relative}$ / length = 13,3 mm / 15075 mm = 1/1129 → check = $\frac{1/1129}{1/100} = 0{,}09$

Deformation $u_{z,var}$ = 46,2 mm (and 0 mm at the beginning of the beam)

→ $u_{z,var,relative}$ = 46,2 mm – 0,0 mm = 46,2 mm

→ $u_{z,max,relative}$ / length = 46,2 mm / 15075 mm = 1/326 → check = $\frac{1/326}{1/180} = 0{,}55$

Chapter 11: Fire resistance check

For the fire resistance calculation, a Professional or an Ultimate edition is necessary. The fire resistance check has been inputted in module sensd.05.xx (for example sensd.05.01 for the code EC-EN).

11.1. 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.

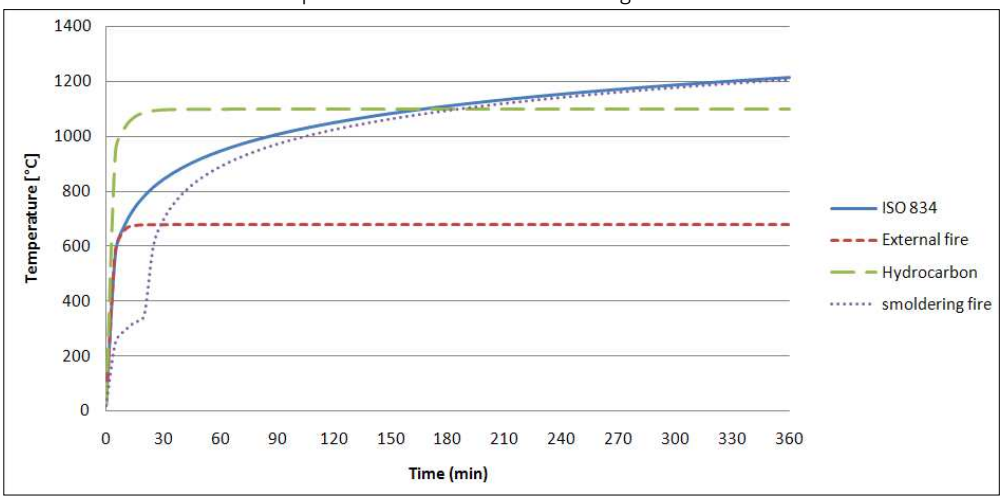
The required levels of fire safety depend on factors such as:

- type of occupancy;
- height and size of the building;
- effectiveness of fire brigade action;
- active measures such as vents and sprinklers.

11.1.1. Temperature-time curves

This is the temperature of the air during the time.

You can choose between 4 nominal temperature-time curves in SCIA Engineer:



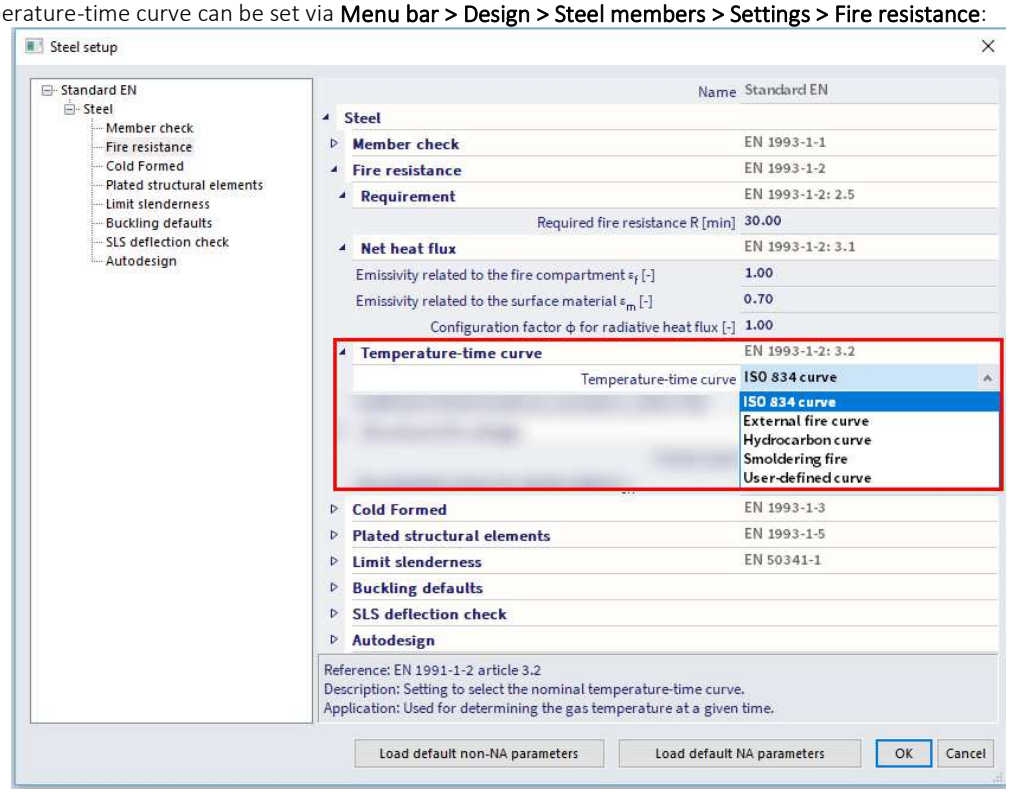
Standard ISO 834 curve: $\Theta_g = 20 + 345 \log_{10}(8\,t + 1)$ [°C]
$\alpha_c = 25\ W/m^2K$

External fire curve: $\Theta_g = 660\ (1 - 0{,}687\ e^{-0{,}32\,t} - 0{,}313\ e^{-3{,}8\,t}) + 20$ [°C]
$\alpha_c = 25\ W/m^2K$

Hydroncarbon curve: $\Theta_g = 1080\ (1 - 0{,}325\ e^{-0{,}167\,t} - 0{,}675\ e^{-2{,}5\,t}) + 20$ [°C]
$\alpha_c = 50\ W/m^2K$

Smoldering fire curve: $\Theta_g = 154\sqrt[4]{t} + 20$ [°C]
$\alpha_c = 50\ W/m^2K$
After 21 minutes followed by the standard ISO 834 curve

The temperature-time curve can be set via **Menu bar > Design > Steel members > Settings > Fire resistance**:



11.1.2. 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: $k_{sh} = 0.9\frac{\left[\frac{A_m}{V}\right]_b}{\left[\frac{A_m}{V}\right]}$
  → 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.
- $[A_m/V]_b$ box value for the section factor
- $\dot{h}_{net}$ design value on the net heat flux per unit area [W/m²]
  → this value should be obtained from EN 1991-1-2 with □f = 1,0 and □m = 0,7.
- $\phi = \frac{c_p\rho_p}{c_a\rho_a}d_p\cdot A_p/V$
- 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:

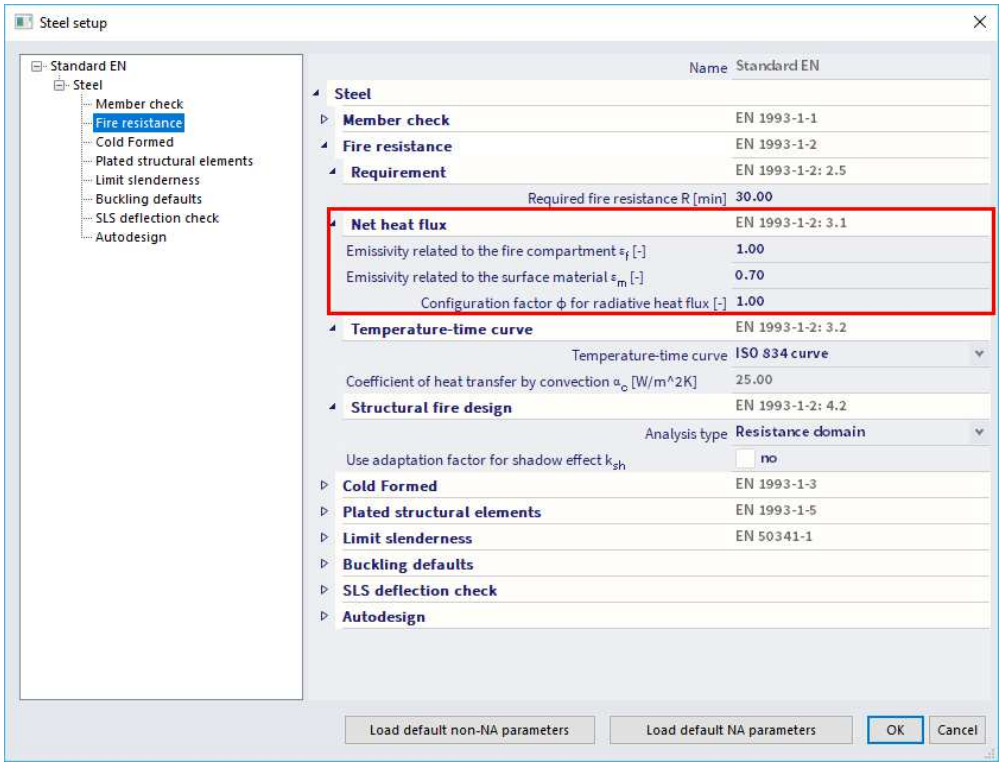
$$\dot{h}_{net} = \dot{h}_{net,r} + \dot{h}_{net,c}$$

- $\dot{h}_{net,c}$ heat transfer by convection [W/m²]
  $= \alpha_c(\Theta_g - \Theta_m)$
- $\dot{h}_{net,r}$ heat transfer by radiation [W/m²]
  $= \Phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma \cdot [(\Theta_r + 273)^4 - (\Theta_m + 273)^4]$

With:

- α_c coefficient of heat transfer by convection [W/m²K]
- Θ_g 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 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 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.

The parameters of the previous formulas can be adapted in the steel settings:



11.1.3. Steel properties

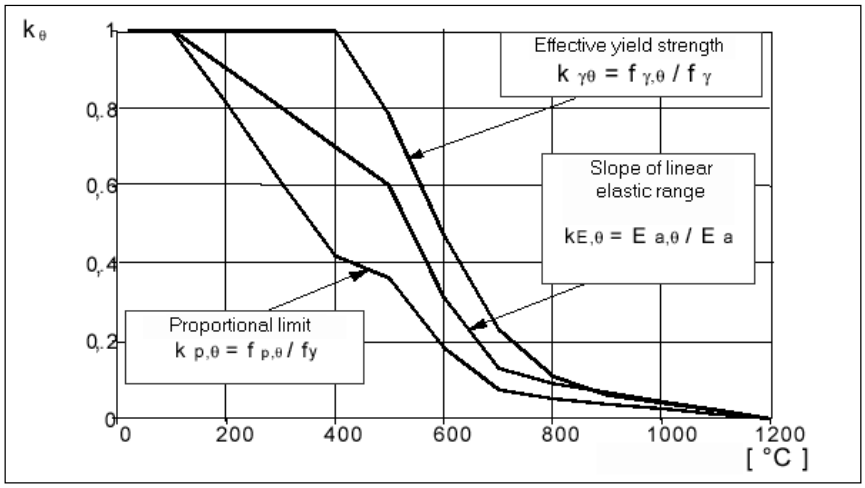
Most of the steel properties will change by a different temperature, so once the steel temperature is known, the steel properties can be calculated. In below the properties for carbon steel from EN 1993-1-2, art.3 are used. The properties for stainless steel can be found in EN 1993-1-2 Annex C. There are also reduction factors for class 4 cross-sections according to table E.1 of EN 1993-1-2 Annex E.

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: $k_{P,\theta} = f_{P,\theta}/f_y$

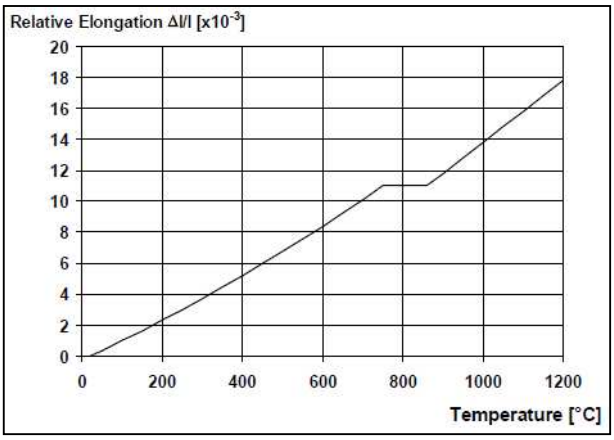
Slope of linear elastic range, relative to slope at 20°C: $k_{E,\theta} = E_{a,\theta}/E_a$



Thermal elongation

The relative thermal elongation of steel $\Delta l/l$ should be determined from the following:

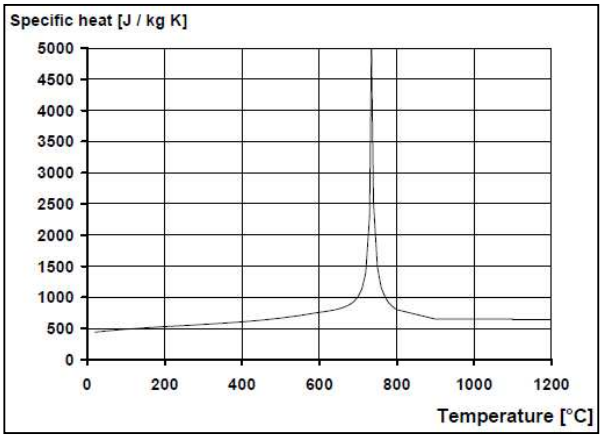
- for $20°C \leq \theta_a < 750°C$: $\frac{\Delta l}{l} = 1{,}2 \text{ x } 10^{-5}\theta_a + 0{,}4 \text{ x } 10^{-8}\theta_a^2 - 2{,}416 \text{ x } 10^{-4}$
- for $750°C \leq \theta_a < 860°C$: $\frac{\Delta l}{l} = 1{,}1 \text{ x } 10^{-2}$
- for $860°C < \theta_a \leq 1200°C$: $\frac{\Delta l}{l} = 2 \text{ x } 10^{-5}\theta_a - 6{,}2 \text{ x } 10^{-3}$



Specific heat

The specific heat c_a [J/kgK] should be determined from the following, with θ_a as the steel temperature:

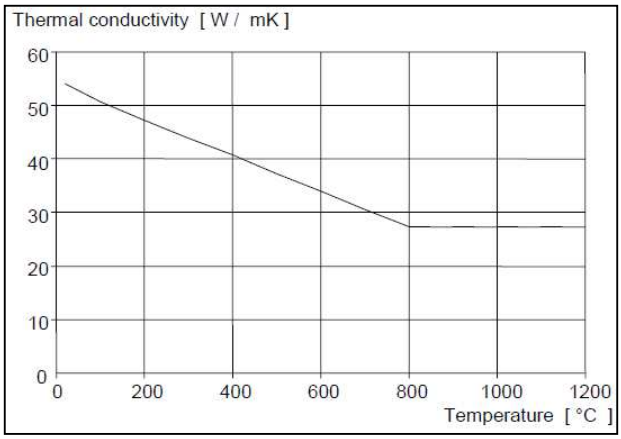
- for $20°C \leq \theta_a < 600°C$: $c_a = 425 + 7{,}73 \text{ x } 10^{-1}\,\theta_a - 1{,}69 \text{ x } 10^{-3}\theta_a^2 + 2{,}22 \text{ x } 10^{-6}\theta_a^3$
- for $600°C \leq \theta_a < 735°C$: $c_a = 666 + \frac{13002}{738-\theta_a}$
- for $735°C \leq \theta_a < 900°C$: $c_a = 545 + \frac{17820}{\theta_{a-731}}$
- for $900°C \leq \theta_a < 1200°C$: $c_a = 650 \text{ J/kgK}$



Thermal conductivity

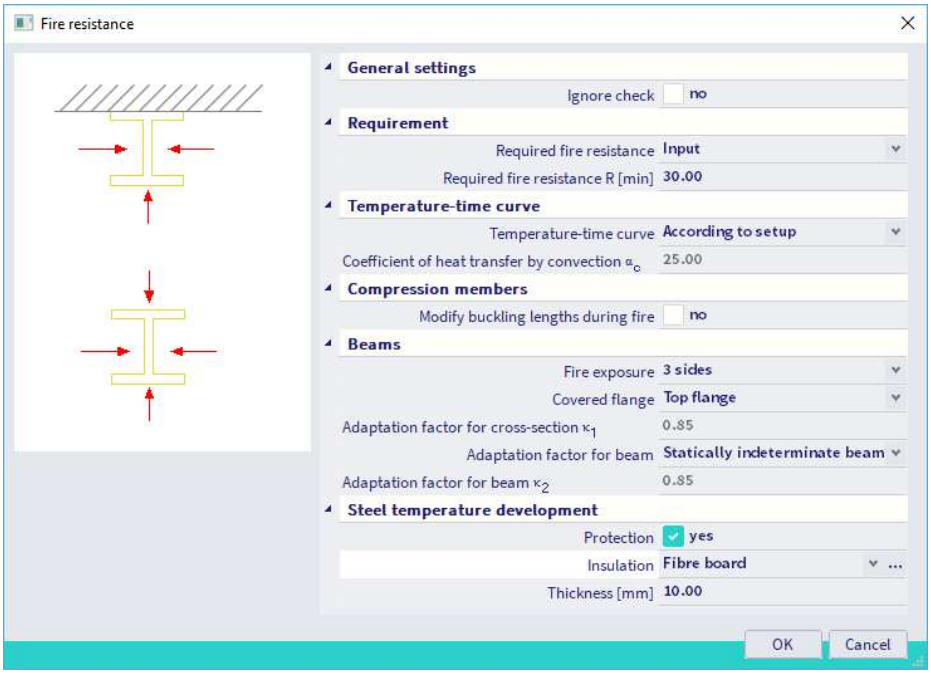
The thermal conductivity λ_a [W/mK] should be determined from the following:

- for $20°C \leq \theta_a < 800°C$: $\lambda_a = 54 - 3{,}33 \text{ x } 10^{-2}\,\theta_a$
- for $800°C \leq \theta_a < 1200°C$: $\lambda_a = 27{,}3 \text{ W/mK}$



11.1.4. Fire resistance properties in SCIA Engineer

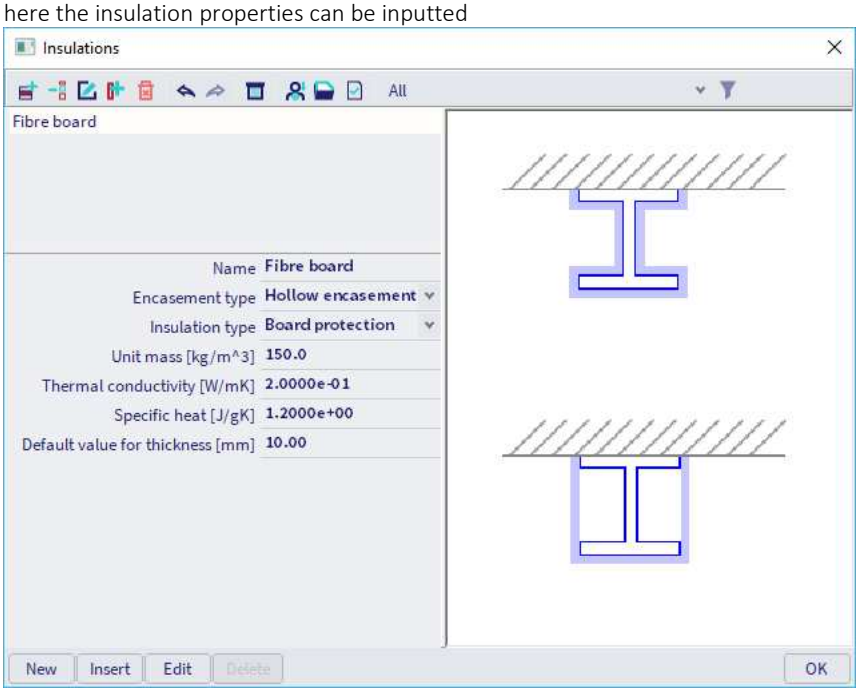
In SCIA Engineer you can input fire resistance properties on a steel member via **Input panel > Steel > Fire resistance check data** or **Process toolbar > Steel > Fire resistance check data**:



Where:

- **Required fire resistance**: inputted by you as 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 parameter 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 $M_{fi,\square,Rd}$
- **Adaption factor for beam**: at the supports of a statically indeterminate beam or all other cases
- **Adaption factor for beam κ_2**: this parameter 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 $M_{fi,\square,Rd}$

- **Protection**: yes or no
- **Insulation**: here the insulation properties can be inputted



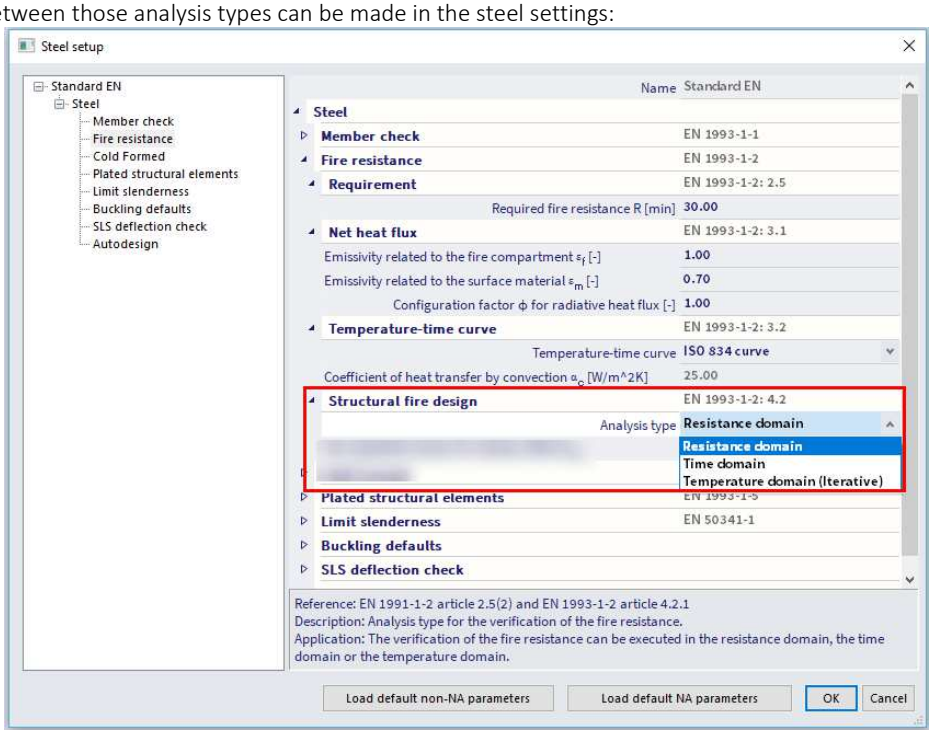
- **Thickness [mm]**: thickness of the insulation

11.2. Calculation methods

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 the steel settings:



11.2.1. Resistance domain

Principle

You 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.

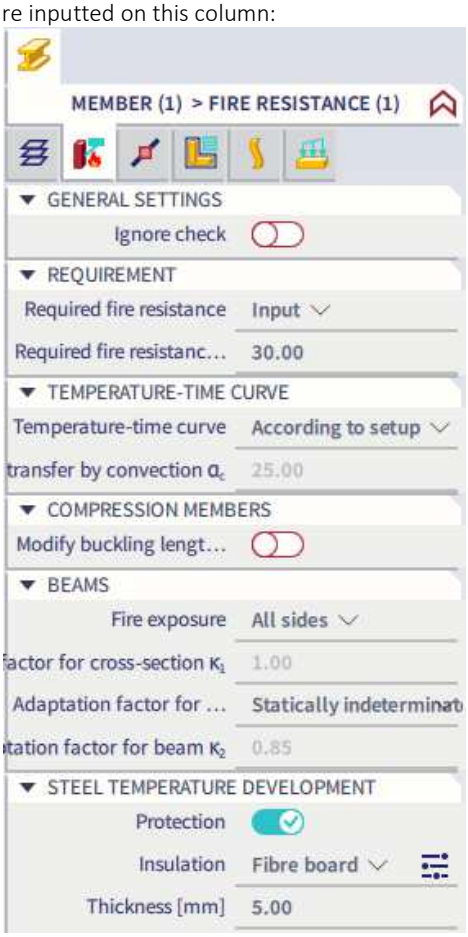
Example in SCIA Engineer

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 **Menu bar > Design > Steel members > Fire check** or **Process toolbar > Steel > Steel ULS fire check** for this column and for result class **Fire** (which contains combination EN-Accidental with ψ_1 coefficients and combination EN-Accidental with ψ_2 coefficients), resulting in a **unity check of 1,18**.

When looking at the detailed output, this calculation is given by SCIA Engineer (it is also possible to turn on the formulas as for the normal ULS checks).

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

Then 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	

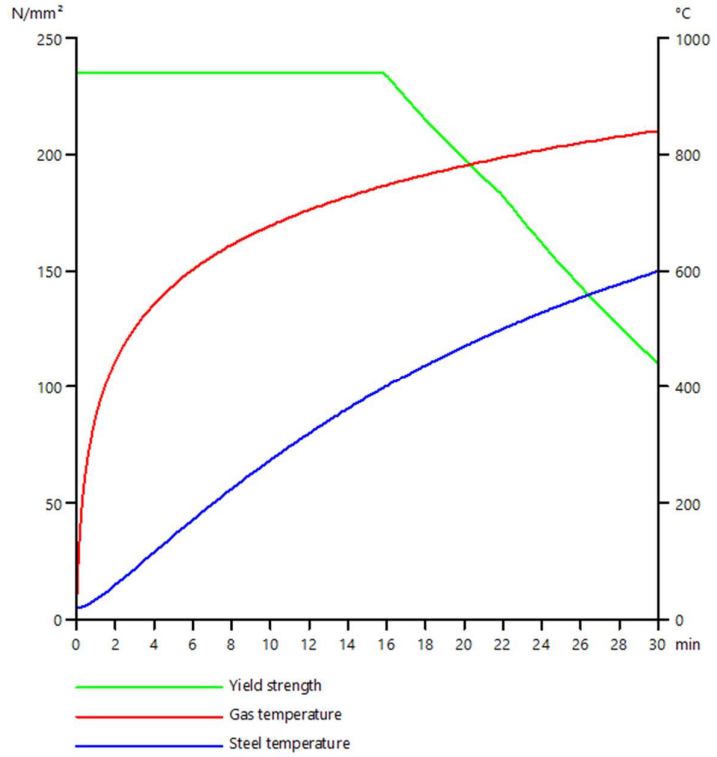
And the fire resistance properties as inputted in SCIA Engineer. Here is also indicated that the fire resistance check has been executed after 30 minutes of fire.

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 yield strength	$k_{y,\theta}$	0.47	
Reduction factor for the 0.2% proof strength	$k_{0.2p,\theta}$	0.30	
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

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 then the unity checks are shown with the reduced properties:

...::SECTION CHECK::...

The critical check is on position 6.900 m

Internal forces		Calculated	Unit
Normal force	$N_{fi,Ed}$	-69.55	kN
Shear force	$V_{y,fi,Ed}$	-0.01	kN
Shear force	$V_{z,fi,Ed}$	-40.21	kN
Torsion	$T_{fi,Ed}$	0.00	kNm
Bending moment	$M_{y,fi,Ed}$	-277.43	kNm
Bending moment	$M_{z,fi,Ed}$	-0.08	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	65221.307	65395.606	1.0	0.4	1.0	6.4	7.6	8.5	11.7	1
3	SO	108.90	17.00	65145.761	64971.462	1.0	0.4	1.0	6.4	7.6	8.5	11.7	1
4	I	685.00	13.20	60923.771	-53505.136	-0.9		0.5	51.9	58.9	67.8	93.9	1
5	SO	108.90	17.00	-57802.671	-57976.970								
7	SO	108.90	17.00	-57727.125	-57552.826								

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)

Cross-section area	A	1.8800e+04	mm^2
Design buckling resistance	$N_{fi,t,Rd}$	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)

Plastic section modulus	$W_{pl,y}$	5.1100e+06	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	1200.85	kNm
Bending resistance	$M_{y,fi,\theta,Rd}$	564.24	kNm
Design bending resistance	$M_{y,fi,t,Rd}$	663.81	kNm
Unity check		0.42	-

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

Plastic section modulus	$W_{pl,z}$	6.3100e+05	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	148.28	kNm
Bending resistance	$M_{z,fi,\theta,Rd}$	69.67	kNm
Design bending resistance	$M_{z,fi,t,Rd}$	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)

Shear correction factor	η	1.20	
Shear area	A_v	9.4086e+03	mm^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	1276.54	kN
Plastic shear resistance for V_y	$V_{y,fi,t,Rd}$	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)

Shear correction factor	η	1.20	
Shear area	A_v	1.1389e+04	mm^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	1545.22	kN
Plastic shear resistance for V_z	$V_{z,fi,t,Rd}$	726.05	kN
Unity check		0.06	-

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)

Plastic bending moment	$M_{y,fi,t,Rd}$	663.81	kNm
Exponent of bending ratio y	α	2.00	
Plastic bending moment	$M_{z,fi,t,Rd}$	81.97	kNm
Exponent of bending ratio z	β	1.00	

Unity check (4.9) = 0.17 + 0.00 = 0.18 -

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: 6.900 m
Decisive utilisation factor η: 0.42
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/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	65221.307	65395.606	1.0	0.4	1.0	6.4	7.6	8.5	11.7	1
3	SO	108.90	17.00	65145.761	64971.462	1.0	0.4	1.0	6.4	7.6	8.5	11.7	1
4	I	685.00	13.20	60923.771	-53505.136	-0.9		0.5	51.9	58.2	67.4	92.7	1
5	SO	108.90	17.00	-57802.671	-57976.970								
7	SO	108.90	17.00	-57727.125	-57552.826								

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1
Note: The decisive position for the stability classification is based on the utilisation factor η according to Semi-Comp+.

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

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	6.900	6.900	m
Buckling factor	k	3.58	0.87	
Buckling length	l_{cr}	24.716	5.978	m
Critical Euler load	N_{cr}	5635.44	3067.01	kN
Slenderness	λ	83.15	112.71	
Relative slenderness	λ_{rel}	0.89	1.20	
Relative slenderness	$\lambda_{rel,\theta}$	1.09	1.48	
Imperfection	α	0.65	0.65	
Reduction factor	χ_{fi}	0.42	0.28	
Buckling resistance	$N_{b,fi,t,Rd}$	864.02	588.88	kN

Flexural Buckling verification			
Cross-section area	A	1.8800e+04	mm^2
Buckling resistance	$N_{b,fi,t,Rd}$	588.88	kN
Unity check		0.12	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-2 article 4.2.3.2 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 4.2.3.3 and formula (4.11)

LTB parameters			
Plastic section modulus	$W_{pl,y}$	5.1100e+06	mm^3
Elastic critical moment	M_{cr}	1785.02	kNm
Relative slenderness	$\lambda_{rel,LT}$	0.82	
Relative slenderness	$\lambda_{rel,LT,\theta}$	1.01	
Imperfection	α_{LT}	0.65	
Reduction factor	$\chi_{LT,fi}$	0.45	
Design buckling resistance	$M_{b,fi,t,Rd}$	254.59	kNm
Unity check		**1.09**	-

Mcr parameters			
LTB length	l_{LT}	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 centre 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 and formula (4.21a),(4.21b)

Bending and axial compression check parameters			
Cross-section area	A	1.8800e+04	mm^2
Plastic section modulus	$W_{pl,y}$	5.1100e+06	mm^3
Plastic section modulus	$W_{pl,z}$	6.3100e+05	mm^3
Design compression force	$N_{fi,Ed}$	69.55	kN
Design bending moment	$M_{y,fi,Ed}$	-277.43	kNm
Design bending moment	$M_{z,fi,Ed}$	-0.08	kNm
Reduction factor	$\chi_{min,fi}$	0.28	
Reduction factor	$\chi_{z,fi}$	0.28	
Reduction factor	$\chi_{LT,fi}$	0.45	
Equivalent moment factor	$\beta_{M,y}$	1.80	
Factor	μ_y	-0.44	
Interaction factor	k_y	1.04	

Bending and axial compression check parameters			
Equivalent moment factor	$\beta_{M,z}$	1.80	
Factor	μ_z	-0.25	
Interaction factor	k_z	1.03	
Equivalent moment factor	$\beta_{M,LT}$	1.80	
Factor	μ_{LT}	0.25	
Interaction factor	k_{LT}	0.97	

Unity check (4.21a) = 0.12 + 0.51 + 0.00 = 0.63 -
Unity check (4.21b) = 0.12 + 1.06 + 0.00 = **1.18** -

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	$M_{f,fi,t,Rd}$	340.52	kNm
Flange factor	c	1.849	m
Contribution of the flange	$V_{bf,fi,t,Rd}$	1.54	kN
Maximum resistance	$V_{b,fi,t,Rd,limit}$	726.05	kN
Resistance	$V_{b,fi,t,Rd}$	648.45	kN
Plastic resistance	$M_{pl,fi,t,Rd}$	564.24	kNm
Shear ratio	$\eta_{3,bar}$	0.06	

Unity check (5.10) = 0.06 -
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**,18**.

To reach the required fire resistance of 30 minutes, you could increase the thickness of the insulation (for example to 6 mm or more).

11.2.2. Time domain

Principle

You 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. 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 = 0 sec, 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! Therefore, this calculation method will rarely be used.

Example in SCIA Engineer

This principle is explained with an example in SCIA Engineer.

Example: Industrial hall.esa

Consider member B28.

Change in the steel settings the analysis type to Time domain.

The fire resistance check is executed via Menu bar > Design > Steel members > Fire check or Process toolbar > Steel > Steel ULS fire check for this column and for result class Fire, resulting in a unity check of 0,46.

When looking at the detailed output, following information is mentioned 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 executed at t = 0 min, so at 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 = 0,46 = μ_0.

Bending and axial compression check
According to EN 1993-1-2 article 4.2.3.5 and formula (4.21a),(4.21b)

Bending and axial compression check parameters			
Cross-section area	A	1.8800e+04	mm²
Plastic section modulus	$W_{pl,y}$	5.1100e+06	mm³
Plastic section modulus	$W_{pl,z}$	6.3100e+05	mm³
Design compression force	$N_{fi,Ed}$	69.55	kN
Design bending moment	$M_{y,fi,Ed}$	-277.43	kNm
Design bending moment	$M_{z,fi,Ed}$	-0.08	kNm
Reduction factor	$\chi_{min,fi}$	0.37	
Reduction factor	$\chi_{z,fi}$	0.37	
Reduction factor	$\chi_{LT,fi}$	0.54	
Equivalent moment factor	$\beta_{M,y}$	1.80	
Factor	μ_y	-0.16	
Interaction factor	k_y	1.00	
Equivalent moment factor	$\beta_{M,z}$	1.80	
Factor	μ_z	-0.02	
Interaction factor	k_z	1.00	
Equivalent moment factor	$\beta_{M,LT}$	1.80	
Factor	μ_{LT}	0.17	
Interaction factor	k_{LT}	0.99	

Unity check (4.21a) = 0.04 + 0.23 + 0.00 = 0.27 -
Unity check (4.21b) = 0.04 + 0.42 + 0.00 = 0.46 -

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

$$\theta_{a,cr} = 39{,}19 \ln\left[\frac{1}{0{,}9674\ \mu_0^{3{,}833}} - 1\right] + 482 = 39{,}19 \ln\left[\frac{1}{0{,}9674\ (0{,}46)^{3{,}833}} - 1\right] + 482 = 596{,}41\ °C$$

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

The unity check is: $\frac{600{,}06°C}{596{,}41°C} = 1{,}01$

All those values are also given in the overview table in the preview of the fire resistance check:

Fire resistance

Verification in Time 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/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
Degree of utilization	μ_0	0.46	
Critical material temperature	$\theta_{a,cr}$	596.41	°C
Fire resistance	t_{cr}	29.65	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 yield strength	$k_{y,\theta}$	1.00	
Reduction factor for the E modulus	$k_{E,\theta}$	1.00	
Unity check		**1.01**	-

Also the **fire resistance time** is given in this table: this member can resist fire for **29,65 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!

11.2.3. Temperature domain (iterative)

Principle

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.

Change in the **steel settings** the **analysis type** to **Temperature domain (iterative)**.

Change in the **steel settings** the **analysis type** to **Time domain**.

The fire resistance check is executed via **Menu bar > Design > Steel members > Fire check** or **Process toolbar > Steel > Steel ULS fire check** for this column and for result class **Fire**, resulting in a **unity check of 0,80**.

When looking at the detailed output, following information is mentioned by SCIA Engineer:

> The check results shown hereafter are given at the critical material temperature $\theta_{a,cr}$ = 752.89 °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 $\theta_{a,cr}$ = 832,40 °C. This temperature is calculated iteratively resulting in a unity check following the EN 1993-1-2 equal to 1,00:

Bending and axial compression check
According to EN 1993-1-2 article 4.2.3.5 and formula (4.21a),(4.21b)

Bending and axial compression check parameters			
Cross-section area	A	1,8800e+04	mm^2
Plastic section modulus	$W_{pl,y}$	5,1100e+06	mm^3
Plastic section modulus	$W_{pl,z}$	6,3100e+05	mm^3
Design compression force	$N_{fi,Ed}$	71,17	kN
Design bending moment	$M_{y,fi,Ed}$	-32,88	kNm
Design bending moment	$M_{z,fi,Ed}$	-0,02	kNm
Reduction factor	$\chi_{min,fi}$	0,34	
Reduction factor	$\chi_{z,fi}$	0,34	
Reduction factor	$\chi_{LT,fi}$	0,53	
Equivalent moment factor	$\beta_{M,y}$	1,78	
Factor	μ_y	-0,28	
Interaction factor	k_y	1,10	
Equivalent moment factor	$\beta_{M,z}$	1,80	
Factor	μ_z	-0,09	
Interaction factor	k_z	1,04	
Equivalent moment factor	$\beta_{M,LT}$	1,78	
Factor	μ_{LT}	0,19	
Interaction factor	k_{LT}	0,90	

And indeed the highest unity check will be equal to 1,00 for this critical temperature.

So for this case the unity check is: $\frac{600,06°C}{832,40°C} = 0,72$

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/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
Critical material temperature	$\theta_{a,cr}$	832,40	°C
Fire resistance	t_{cr}	70,57	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 yield strength	$k_{y,\theta}$	0,09	
Reduction factor for the E modulus	$k_{E,\theta}$	0,08	
Unity check		0,72	-

And also the **fire resistance time** is given in this table: this member can resist fire for **70,57 minutes**.

Annex A: Shear areas

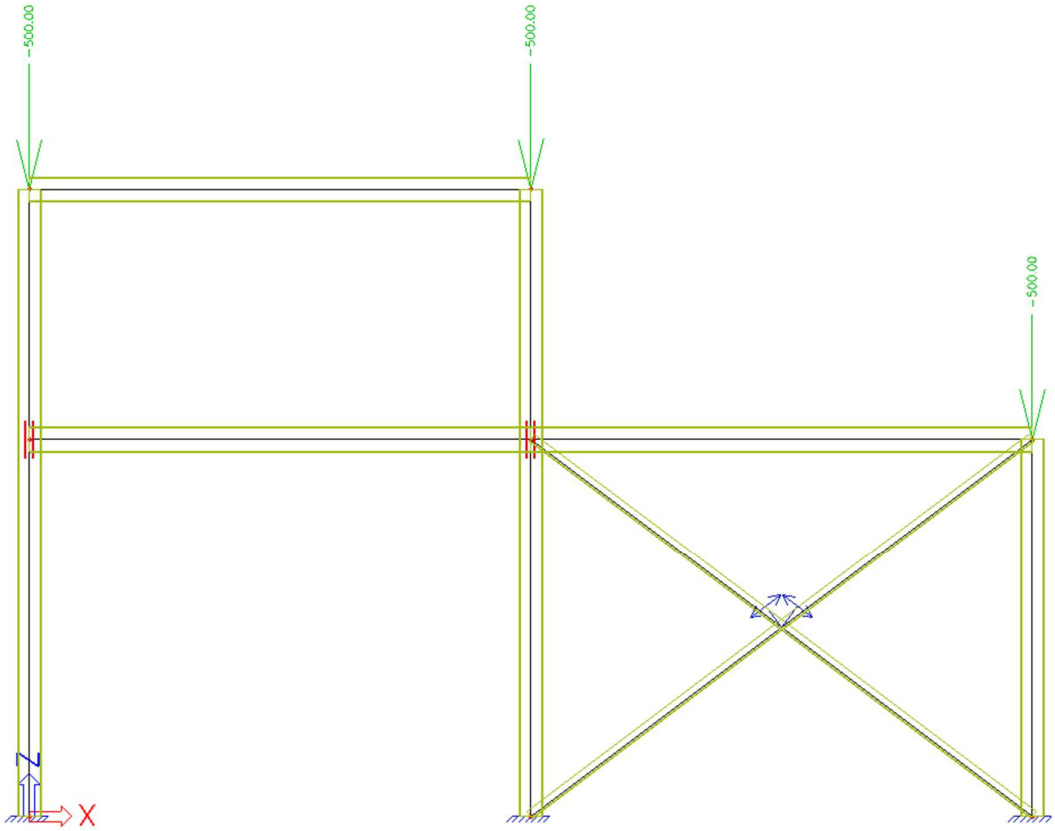
Cross-section type	Shear Area	Source
Rolled I-section (FC 1)	$A_{vz} = A - 2 * b * t_f + (t_w + 2 * r) * t_f \geq \eta * h_w * t_w$ $A_{vy} = 2 * b * t_f + (t_w + r) * t_w$	EN 1993-1-1 ECCS 85
Rolled Asym. I-section (FC 101)	$A_{vz} = A - bt * tt - bb * tb + (t_w + 2 * r) * \left[\frac{tb + tt}{2}\right] \geq \eta * h_w * t_w$ $A_{vy} = bt * tt + bb * tb + (t_w + r) * t_w$	EN 1993-1-1 (mod) ECCS 85 (mod)
Welded I-section (FC 1)	$A_{vz} = \eta * h_w * t_w$ $A_{vy} = A - h_w * t_w$	EN 1993-1-1 EN 1993-1-1
Rolled U-section (FC 5)	$A_{vz} = A - 2 * b * t_f + (t_w + r) * t_f$ $A_{vy} = 2 * b * t_f$	EN 1993-1-1 EN 1993-1-1 (mod)
Welded U-section (FC 5)	$A_{vz} = h_w * t_w$ $A_{vy} = A - h_w * t_w$	EN 1993-1-1 (mod) EN 1993-1-1
Rolled T-section (FC 6)	$A_{vz} = A - b * t_f + (t_w + r) * \frac{t_f}{2}$ $A_{vy} = b * t_f$	EN 1993-1-1 EN 1993-1-1 (mod)
Welded T-section (FC 6)	$A_{vz} = t_w(h - \frac{t_f}{2})$ $A_{vy} = b * t_f$	EN 1993-1-1 EN 1993-1-1 (mod)
Rolled RHS (FC 2)	$A_{vz} = \frac{A * h}{(b + h)}$ $A_{vy} = \frac{A * b}{(b + h)}$	EN 1993-1-1 EN 1993-1-1
Cold-Formed RHS (FC 2)	$A_{vz} = \frac{A * h}{(b + h)}$ $A_{vy} = \frac{A * b}{(b + h)}$	EN 1993-1-1 EN 1993-1-1

Welded RHS (FC 2)	$A_{vz} = \eta * 2 * h_w * t_w$ $A_{vy} = A - 2 * h_w * t_w$	EN 1993-1-1 EN 1993-1-1
CHS (FC 3)	$A_{vz} = \frac{2 * A}{\pi}$ $A_{vy} = \frac{2 * A}{\pi}$	EN 1993-1-1 EN 1993-1-1
Full Rectangular Section (FC 7)	$A_{vz} = A$ $A_{vy} = A$	ECCS 85 ECCS 85
Full Circular Section (FC 11)	$A_{vz} = A$ $A_{vy} = A$	ECCS 85 ECCS 85
IFBA (FC 154)	$A_{vz} = h * t_w + 2 * r * t_o + 2 * r^2 * (1 - \frac{\pi}{4})$ With h the height of the rolled section $A_{vy} = b_o * t_o + b_u * t_u$	ECCS 83 ECCS 83 (mod)
IFBB (FC 155)	$A_{vz} = h * t_w + 2 * r * t_u + 2 * r^2 * (1 - \frac{\pi}{4})$ With h the height of the rolled section $A_{vy} = b_o * t_o + b_u * t_u$	ECCS 83 ECCS 83 (mod)
SFB (FC 153)	$A_{vz} = h * t_w + 4 * r * t_f + 4 * r^2 * (1 - \frac{\pi}{4})$ With h the height of the rolled section $A_{vy} = 2 * b * t_f + b_u * t_u$	ECCS 83 ECCS 83 (mod)
THQ (FC 156)	$A_{vz} = 2 * h * t_w$ With h the web height $A_{vy} = b_o * t_o + b_u * t_u$	ECCS 83 ECCS 83 (mod)
Numerical	$A_{vz} = A_z$ $A_{vz} = A_z$ (Taken from the cross-section) $A_{vy} = A_y$ $A_{vy} = A_y$ (Taken from the cross-section)	

Annex B: Examples on the Eurocode paths

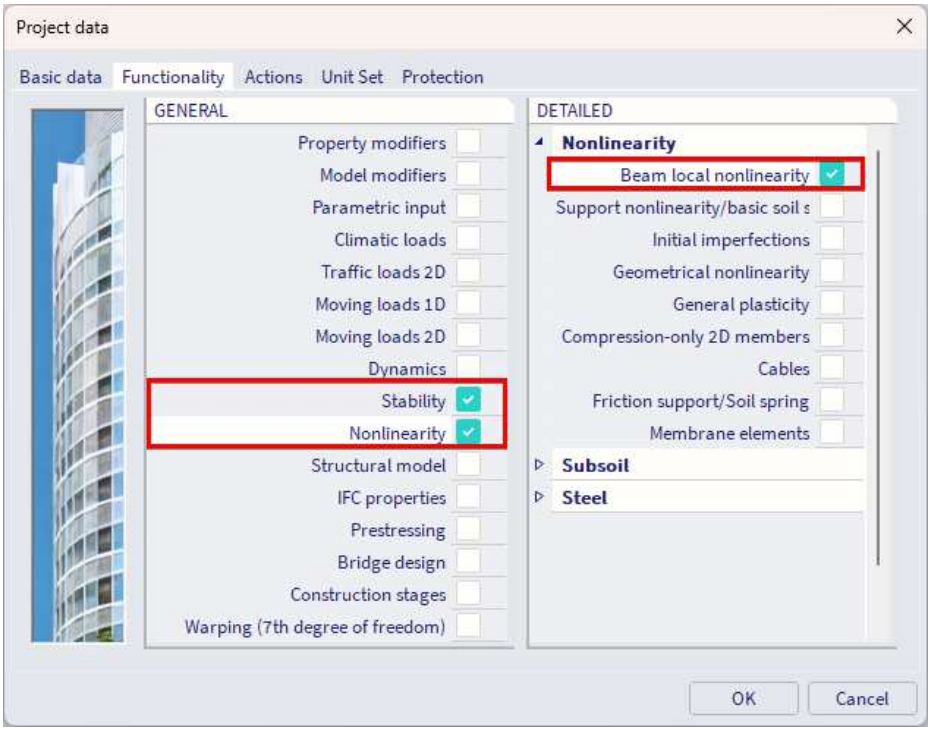
This annex will illustrate the Eurocode paths with some examples. Path 2b is not given since this path cannot be used in practice: or you put a local imperfection on all members as in path 2c or you put no local imperfections as in path 2a. It is possible to put a local imperfection on several members, but the flexural buckling check will always be performed on all members or no member at all.

Example project: 1 load case, containing 3 point loads of 500 kN and a wind load case with load 70 kN/m on the left column. For the examples of paths 1b, 2a, 2c and 3, the loads are increased to 750 kN to have a lower critical alpha coefficient.



Path 1a: 1st order analysis

Functionalities



Stability calculation

We perform a stability analysis to check if the critical alpha coefficient is bigger than 10. If so, it is sufficient to perform a first-order analysis. As mentioned in the chapter 'Buckling shape as imperfection', the **Average number of tiles of 1d element** is set to **10**:



Via Menu bar > Results > Critical load coefficients, a critical alpha coefficient of 14,08 is found, so it is sufficient to perform a first-order analysis.

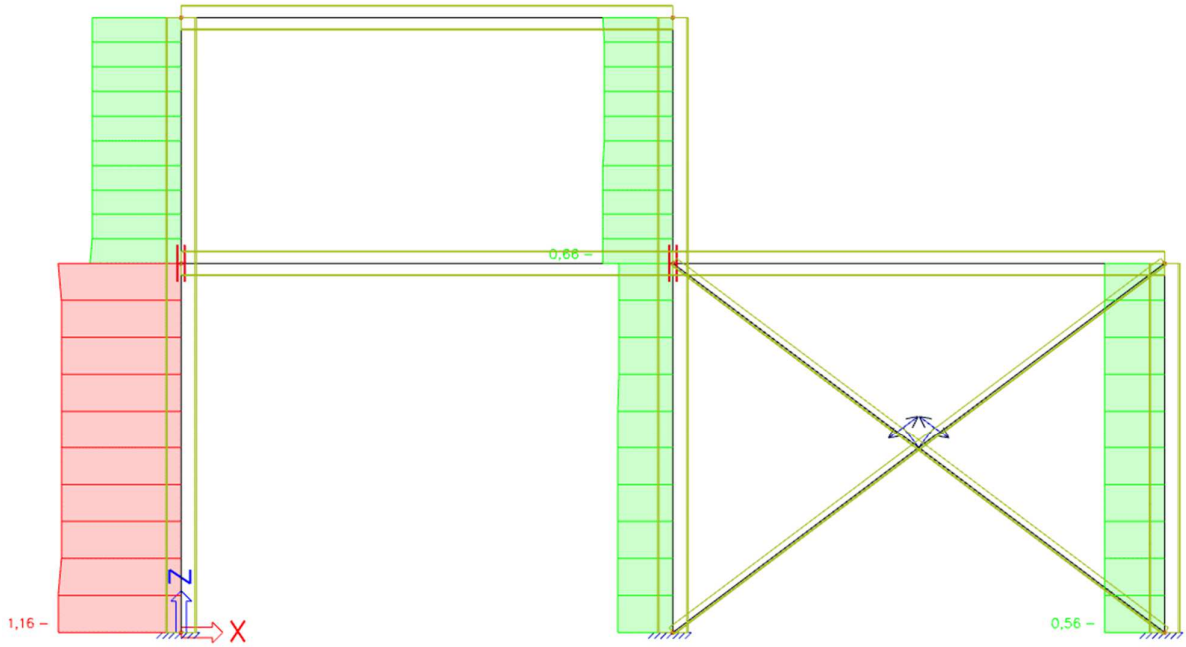
Linear calculation

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 you as user;
- system length of member: proposition of SCIA Engineer to be checked by you as user.

Alternative: manual input of the buckling factor k or the buckling length l

We perform the ULS check on combination CO1: the unity check for flexural buckling is decisive.

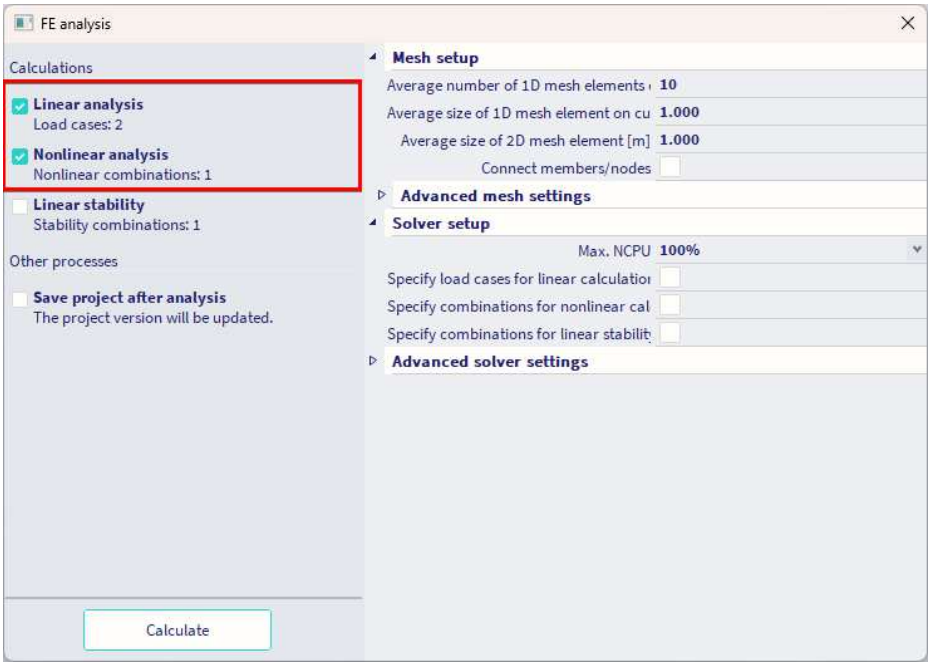


Nonlinear calculation

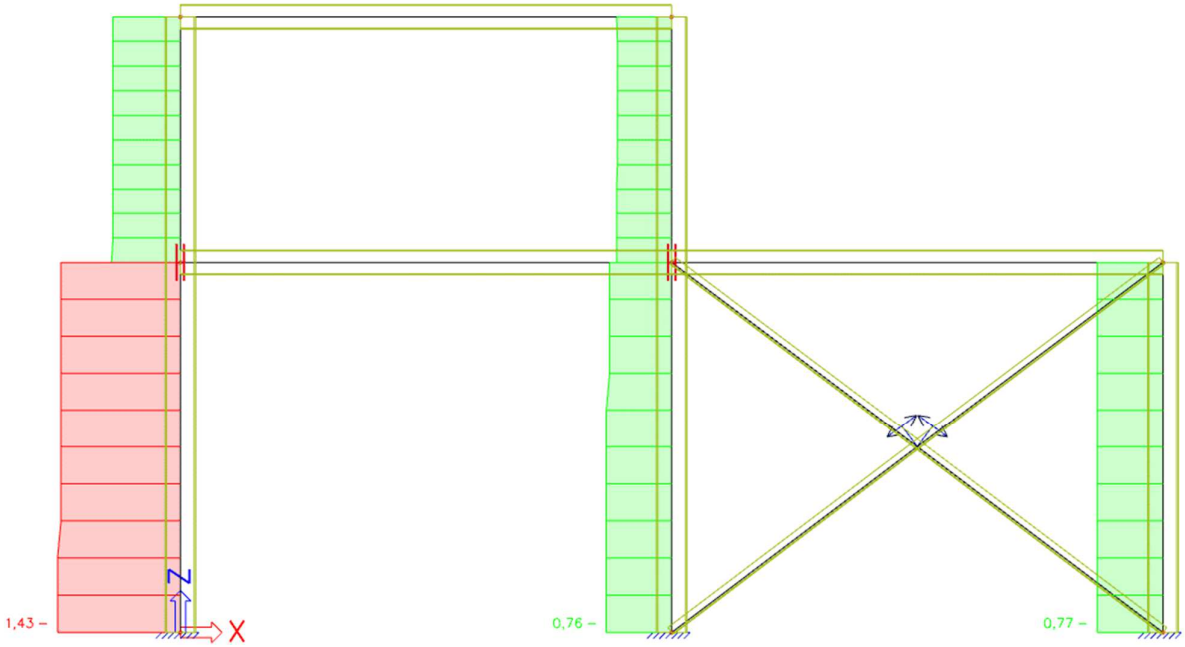
Same setup of buckling properties as for linear calculation.

The only difference is the **Tension only** property (via **Input panel > Structure > Boundary conditions > Nonlinearity 1D**) that has been added to the wind bracings. This means we are still dealing with a 1st order calculation, but a nonlinear calculation must 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!.

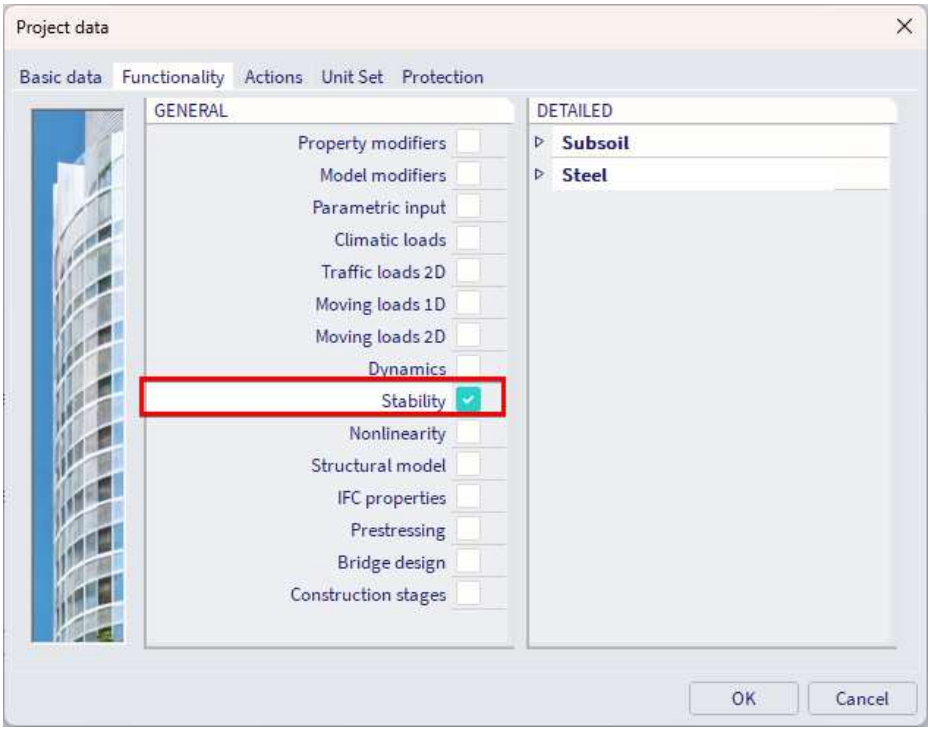


We perform the ULS check on nonlinear combination NC1: the unity check for flexural buckling check is still decisive.



Path 1b: 1st order analysis

Functionalities



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 must be higher than 3 to be able to use path 1b of the structural frame stability scheme. In this example, the critical load coefficient is equal to 9,55 (which is in between 3 and 10).

Critical load coefficients

N -	f []
Linear stability combination : SL1	
1	9,55
2	27,20
3	32,44
4	38,33

Linear combination

First you can check if condition EN1993-1-1 5.3.2 (4) is fulfilled or not.

(4)B For building frames sway imperfections may be disregarded where

$$H_{Ed} \geq 0{,}15\ V_{Ed} \qquad (5.7)$$

If the wind load would be small than 15% of the vertical load, you need to take into account the effect of a global imperfection, e.g. via equivalent horizontal forces. In our example this isn't necessary.

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:

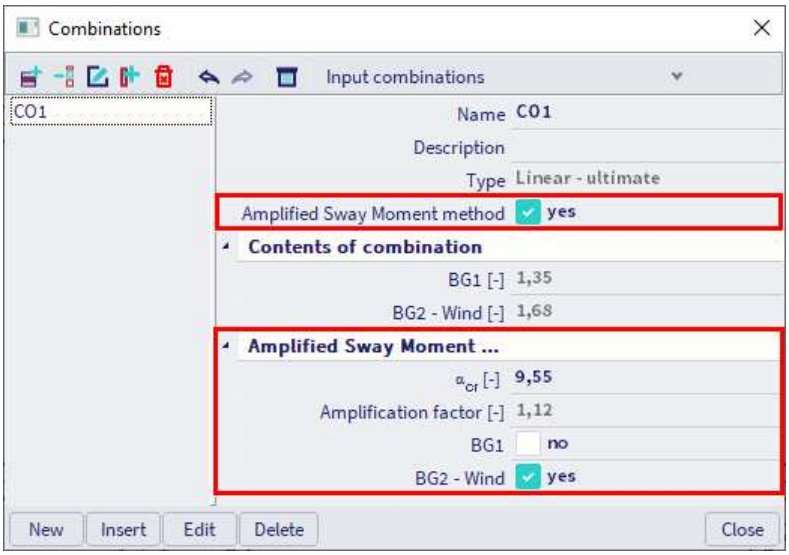
The critical check is on position 0,000 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-984,03	kN
Shear force	$V_{y,Ed}$	0,00	kN
Shear force	$V_{z,Ed}$	204,43	kN
Torsion	T_{Ed}	0,00	kNm
Bending moment	$M_{y,Ed}$	-161,85	kNm
Bending moment	$M_{z,Ed}$	0,00	kNm

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

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	3,000	5,000	m
Buckling factor	k	1,00	1,00	
Buckling length	l_{cr}	3,000	5,000	m
Critical Euler load	N_{cr}	24065,39	3040,94	kN
Slenderness	λ	27,34	76,92	
Relative slenderness	λ_{rel}	0,29	0,82	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		b	c	
Imperfection	α	0,34	0,49	
Reduction factor	χ	0,97	0,65	
Buckling resistance	$N_{b,Rd}$	1973,20	1326,32	kN

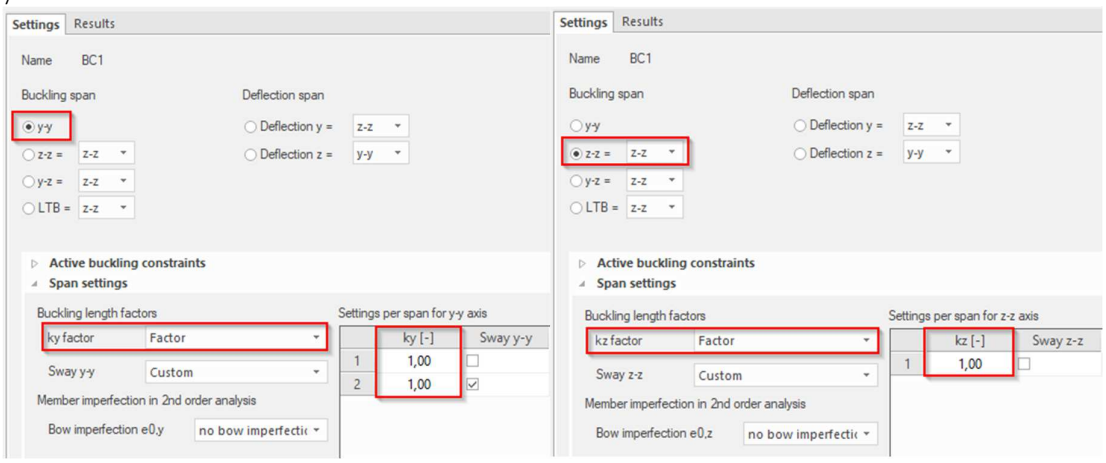
Next the **Amplified Sway Moment Method** can be activated in the linear combinations and the critical alpha value has to be inserted:



We only consider the amplification factor on the wind load case. The coefficient of initially 1,5 increases to 1,68 due to the amplification factor of 1,12.

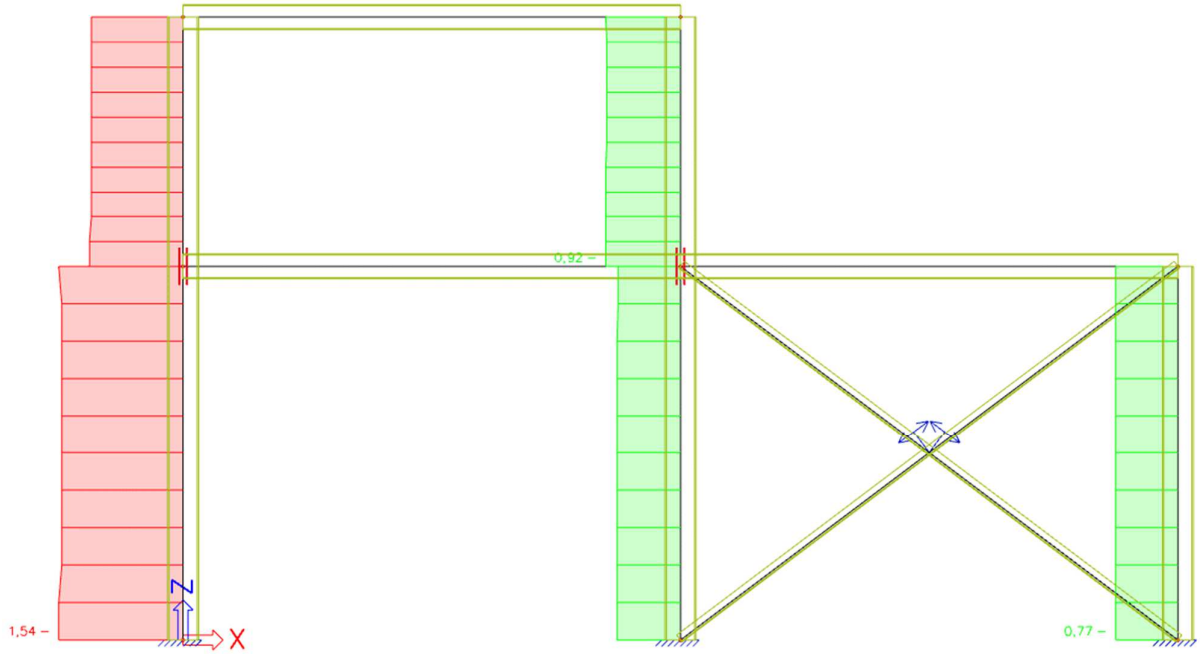
Linear calculation

The buckling lengths are taken equal to the system lengths. This can be done via 'System lengths and buckling settings' for y-y and z-z:



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

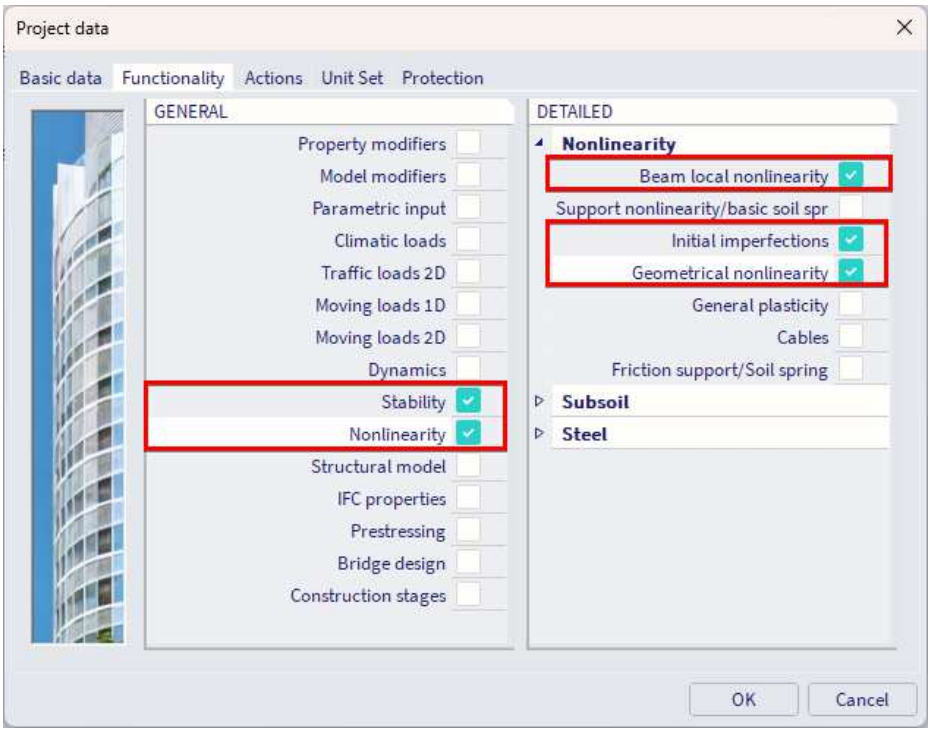
We perform the ULS check for combination CO1: the unity check for combined compression and bending is decisive.



Note: you could also calculate nonlinear with a global imperfection, but then you should adapt the coefficients in the nonlinear combination to take into account the amplification factor. But path 2a te gaan

Path 2a: 2nd order analysis – global imperfection

Functionalities



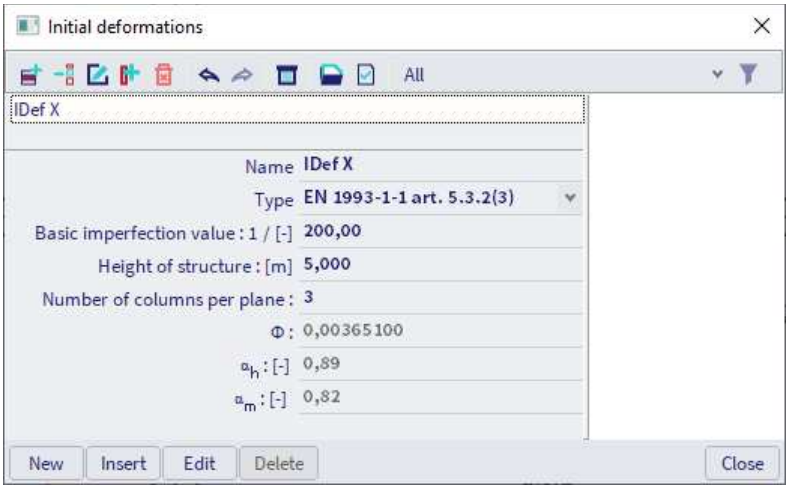
Stability calculation

The stability calculation will be performed to determine the critical alpha coefficient. Since the critical alpha value is smaller than 10, we cannot follow path 1a of the Eurocode overview so we could follow path 2a.

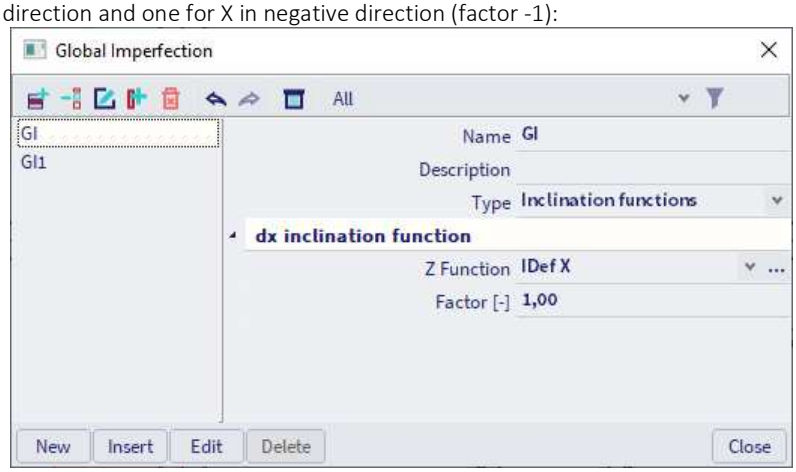
Global imperfection:

The next step is to insert the global imperfection. This will be done using 'Main menu > Libraries > Structure and 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).



You can create two global imperfections (Main menu > Libraries > Load cases, combinations > Global Imperfections), one for X in positive direction and one for X in negative direction (factor -1):

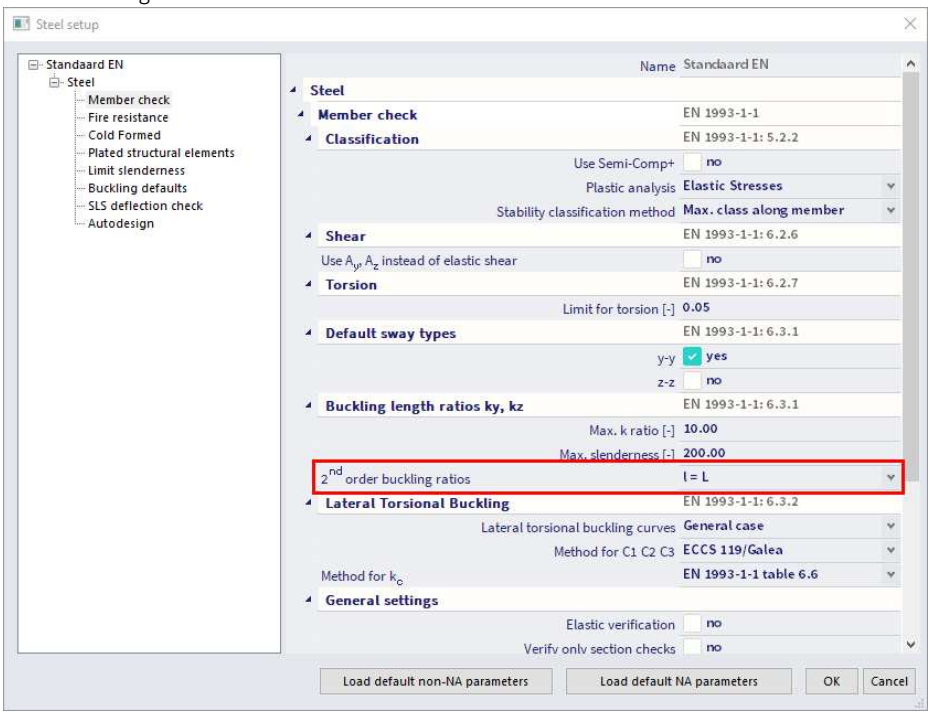


These global imperfections can now be chosen into the nonlinear combinations.

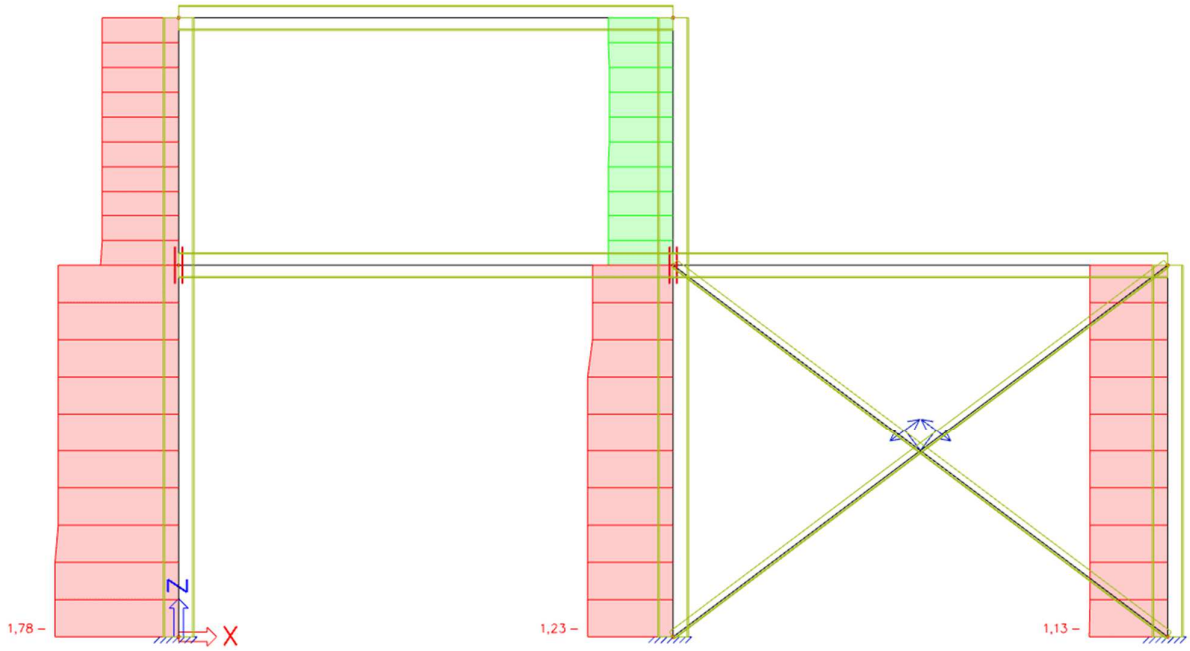


Steel setup

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.

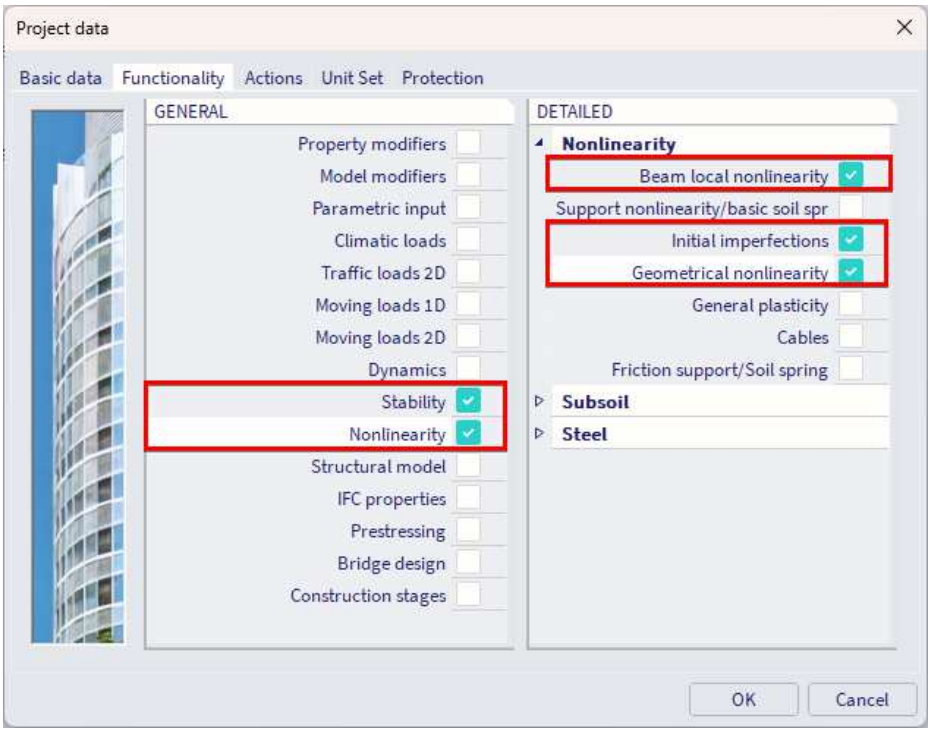


We perform the ULS check for the result class 'RC NL': the unity check for combined compression and bending is decisive.



Path 2c: 2nd order analysis – global + local imperfection

Functionalities



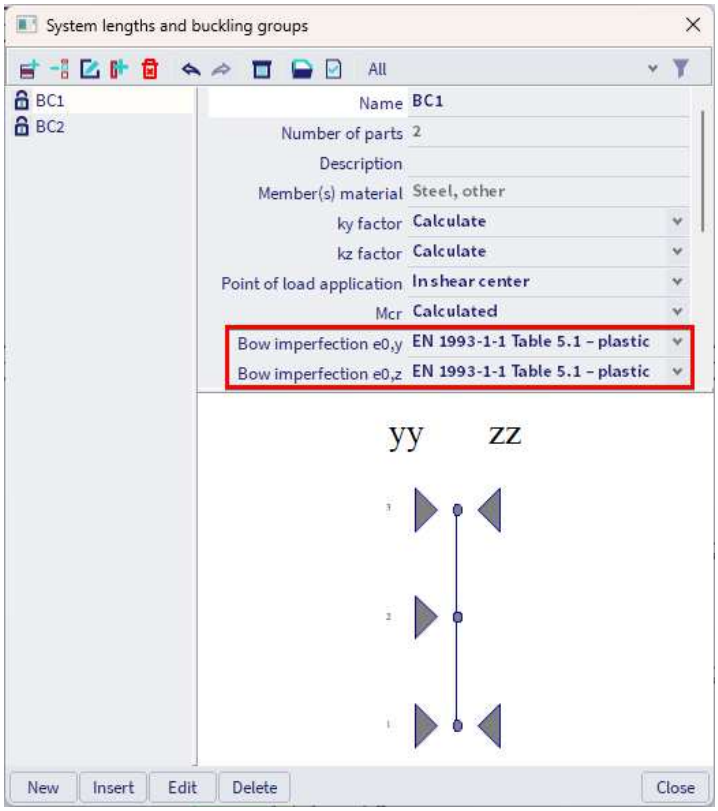
Stability calculation

The stability calculation will be performed to determine the critical alpha coefficient. Since the critical alpha value is smaller than 10, we cannot follow path 1a of the Eurocode overview so we could follow path 2c.

Global imperfection: see previous chapter.

Bow imperfection

The bow imperfection e0 is inserted in the **System lengths and buckling groups** (or you can set it for the entire project via the **steel settings** and put the **diagonals** manually to **No bow imperfection** via the **System lengths and buckling settings**):



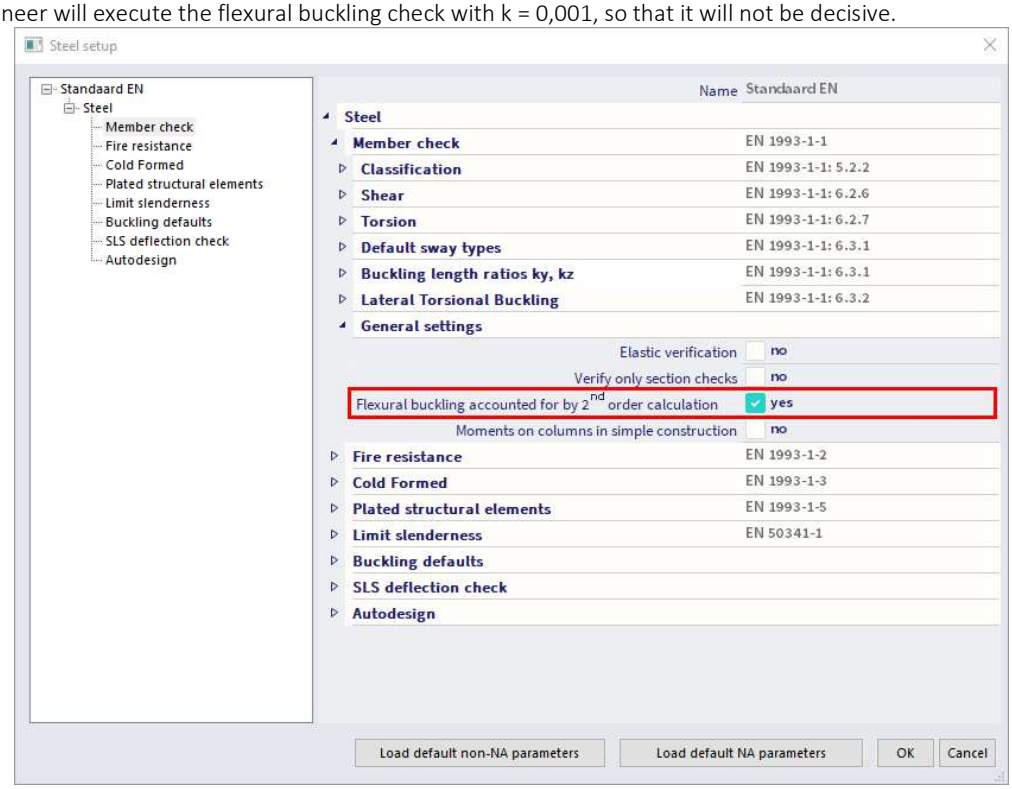
And in the nonlinear combination, a reference is made to this inserted buckling data:



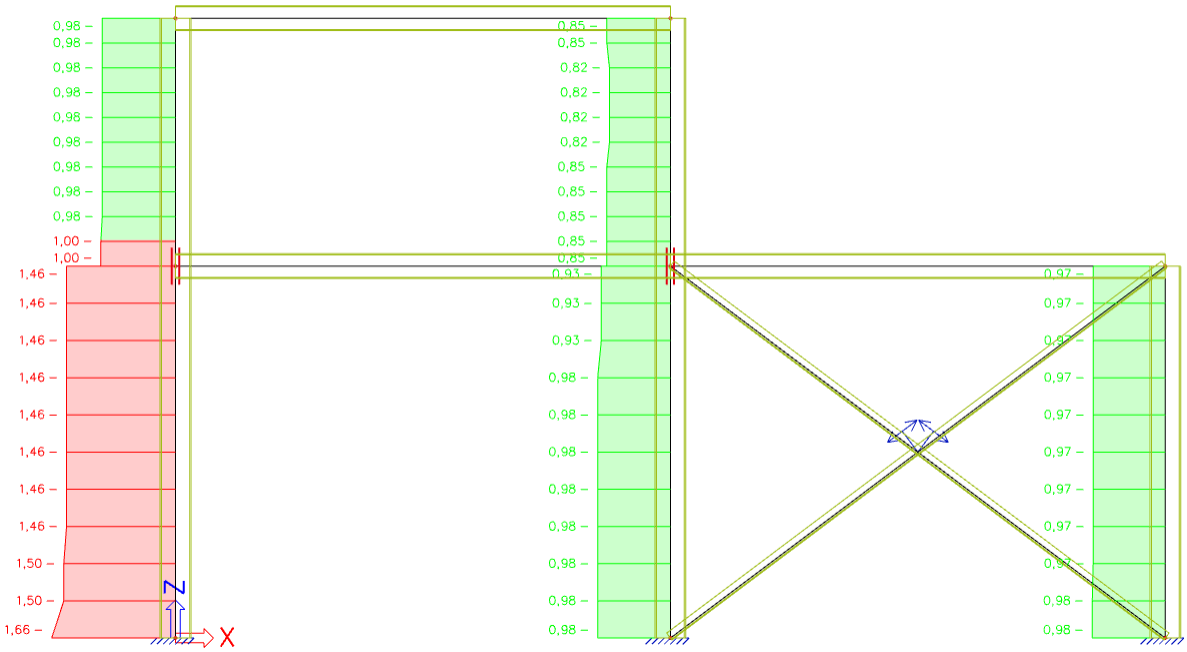
Steel setup

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.

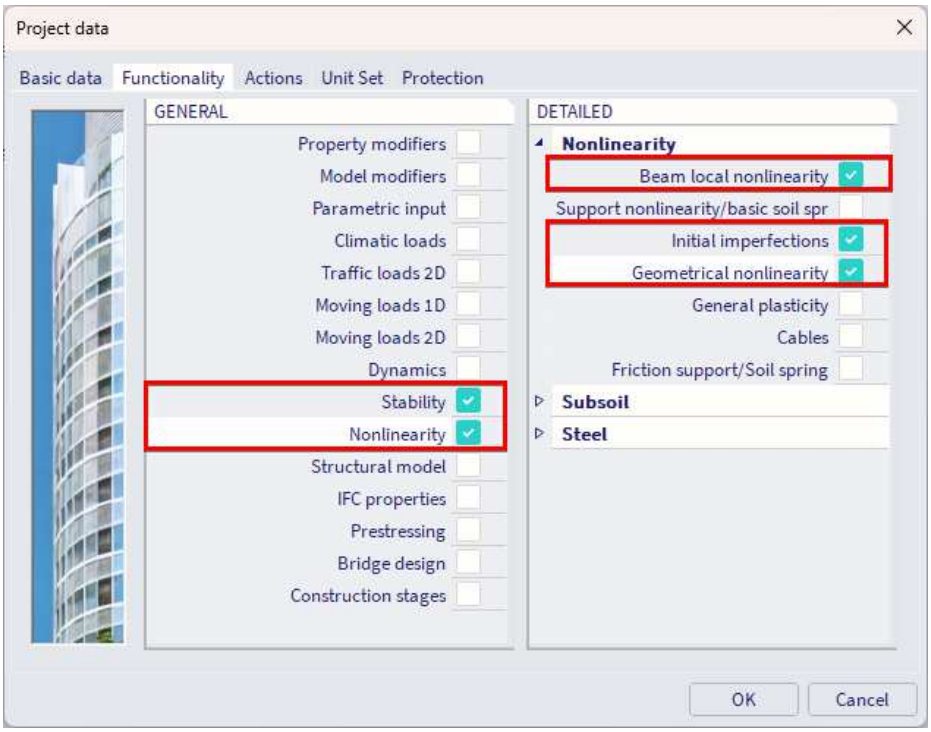


We perform the ULS check for the result class 'RC NL': the section check for combined bending, axial force and shear force is decisive (unity check for combined compression and bending is 1,66 for the left column).



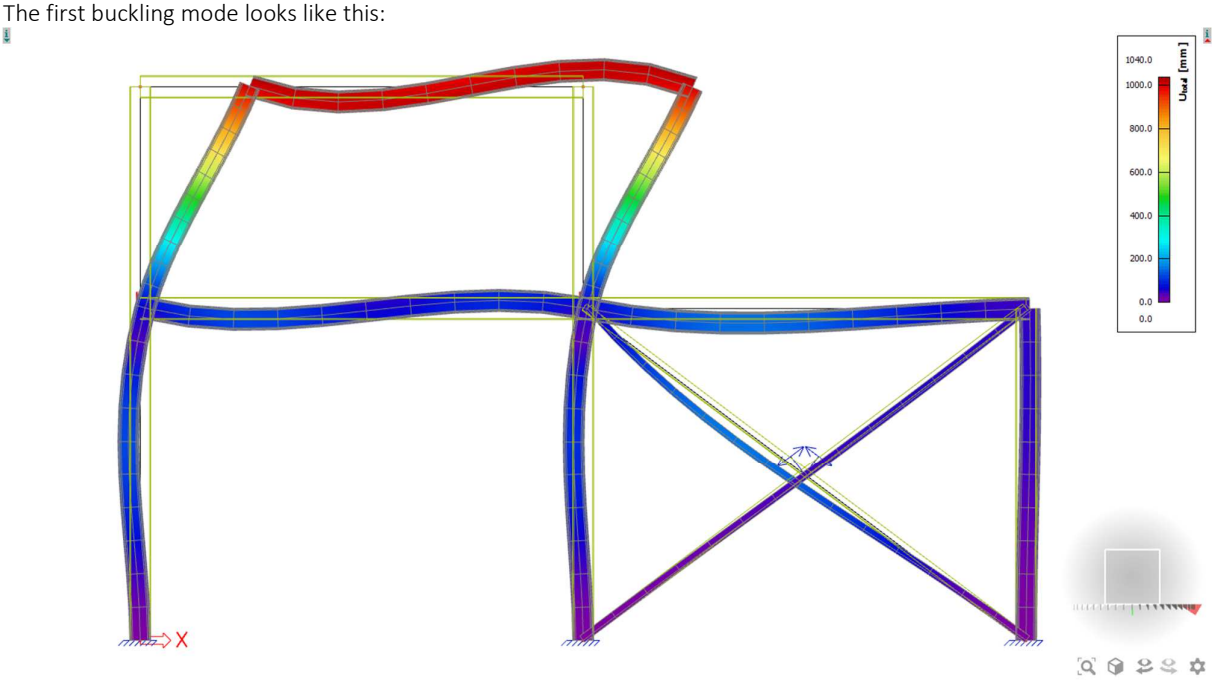
Path 3: 2nd order analysis – buckling shape as imperfection

Functionalities

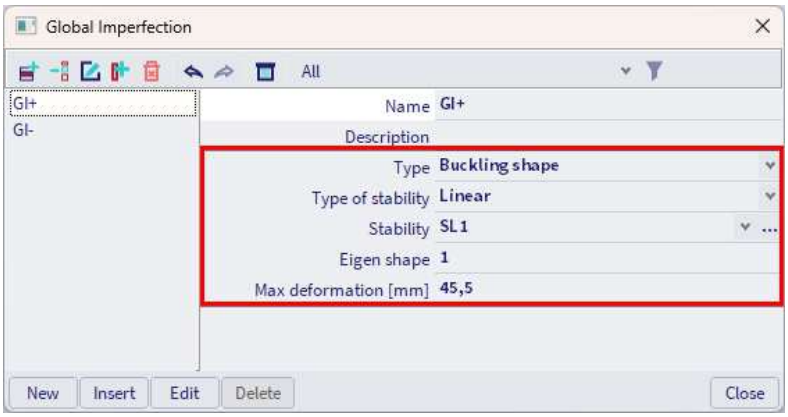


Stability calculation: see previous chapters.

The first buckling mode looks like this:



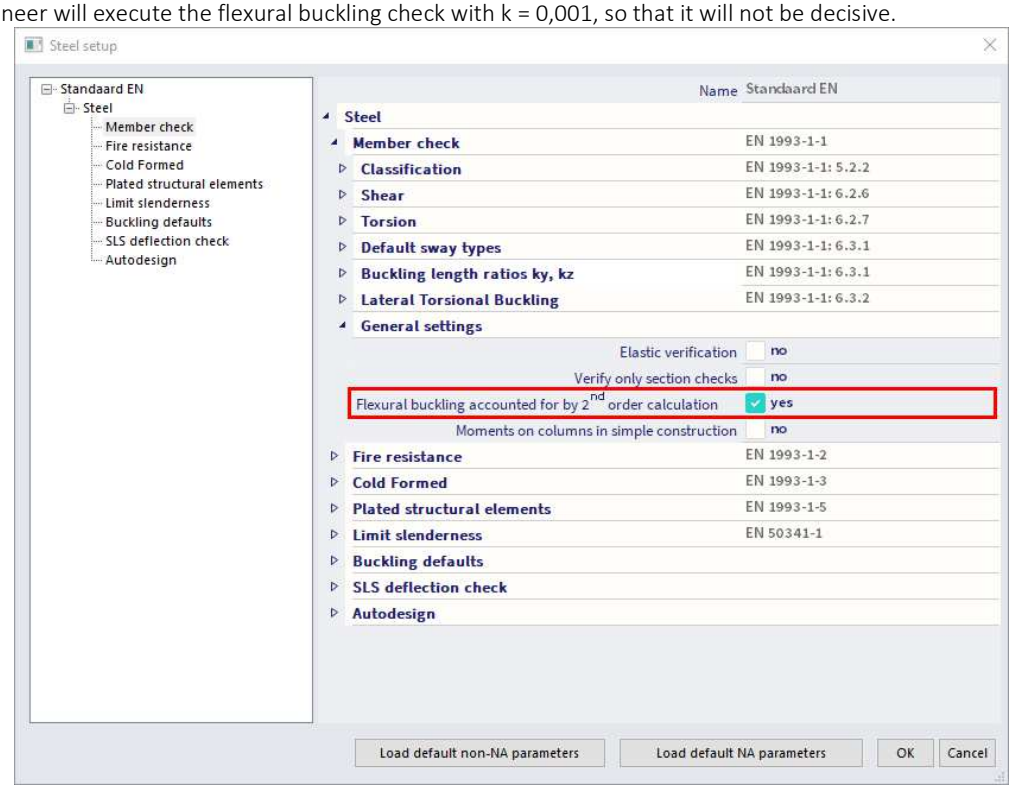
Calculation of η_{init} → to be filled in as 'max. deformation' (see 'Global Imperfection' dialog)



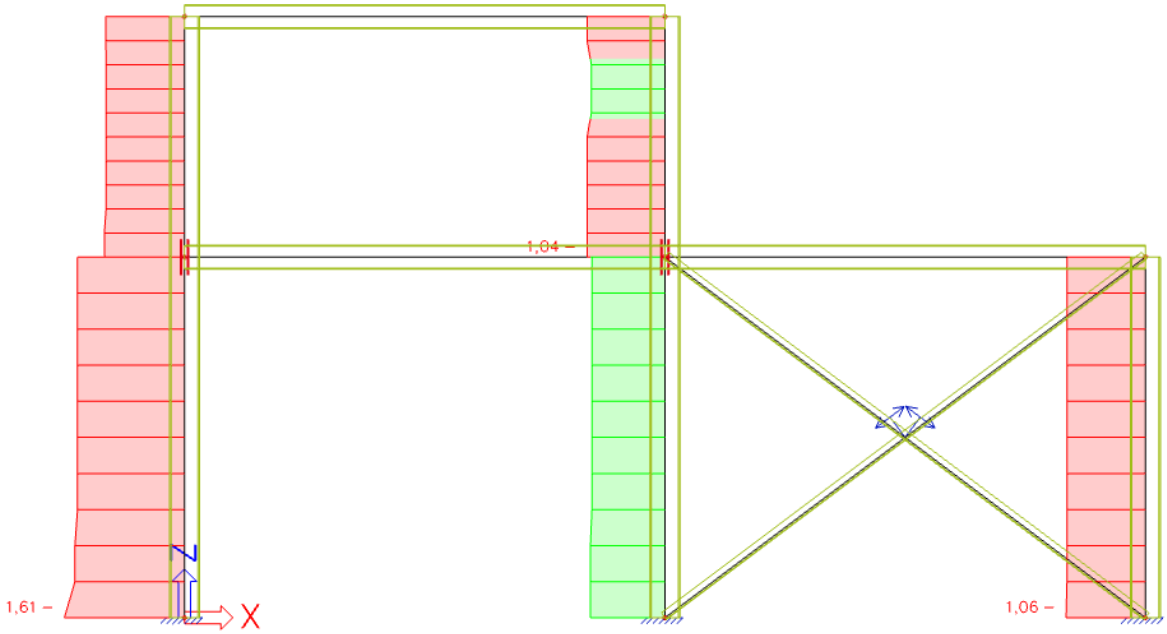
Steel setup

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.



We perform the ULS check for the result class ‘RC NL’: the section check for combined bending, axial force and shear force is decisive (unity check for combined compression and bending is 1,61 for the left column).



References and literature

[1] Eurocode 3:
Design of steel structures
Part 1 – 1 : General rules and rules for buildings
ENV 1993-1-1:1992, 1992

[2] Essentials of Eurocode 3
Design Manual for Steel Structures in Building
ECCS – N° 65, 1991

[3] R. Maquoi
Elements de constructions metallique
Ulg. Faculté des Sciences Appliquées
1988

[4] ENV 1993-1-3:1996
Eurocode 3 : Design of steel structures
Part 1-3: General rules
Supplementary rules for cold-formed thin gauge members and sheeting
CEN 1996

[5] Eurocode 3
Design of steel structures
Part 1 – 1/A1: General rules and rules for buildings
ENV 1993-1-1:1992/A1, 1994

[6] Eurocode 3
Design of steel structures
Part 1 – 2: General rules – Structural fire design
EN 1993-1-2:2005, 2005

[7] Model Code on Fire Engineering
ECCS – N° 111
May 2001

[8] Eurocode 1
Basis of design and actions on structures
Part 2-2: Actions on structures – Actions on structures exposed to fire
EN 1991-1-2:2002

[9] DIN 18800 Teil 1
Stahlbauten
Bemessung und Konstruktion
DK 693.814.014.2, November 1990

[10] DIN 18800 Teil 2
Stahlbauten
Stabilitätsfälle, Knicken von Stäben and Stabwerken
DK 693.814.074.5, November 1990

[11] DIN 18800 Teil 3
Stahlbauten
Stabilitätsfälle, Plattenbeulen
DK 693.814.073.1, November 1990

[12] G. Hünersen, E. Fritzsche
Stahlbau in Beispielen
Berechnungspraxis nach DIN 18800 Teil 1 bis Teil 3 (11.90)
Werner Verlag, Düsseldorf 1991

[13] E. Kahlmeyer
Stahlbau nach DIN 18800 (11.90)
Werner Verlag, Düsseldorf

[14] Beuth-Kommentare
Stahlbauten
Erläuterungen zu DIN 18 800 Teil 1 bis Teil 4, 1.Auflage
Beuth Verlag, Berlin-Köln 1993

[15] Stahlbau Kalender 1999
DSTV
Ernst & Sohn, 1999

[16] Roik, Carl, Lindner
Biegetorsionsprobleme gerader dünnwandiger Stäbe
Verlag von Wilhelm Ernst & Sohn
1972

[17] DASt-Richtlinie 016
Bemessung und konstruktive Gestaltung von Tragwerken aus dünnwandigen kaltgeformted
Bauteilen
Stahlbau-Verlagsgesellschaft - 1992

[18] H. Rubin
Interaktionsbeziehungen für doppeltsymmetrische I- und Kasten-Querschnitte bei zweiachsiger Biegung und Normalkraft
Der Stahlbau 5/1978, 6/1978

[19] Stahl im Hochbau
14. Auflage, Band I / Teil 2
1986, Verlag Stahleisen mbH, Düsseldorf

[20] Staalconstructies TGB 1990
Basiseisen en basisrekenregels voor overwegend statisch belaste constructies
NEN 6770, december 1991

[21] Staalconstructies TGB 1990
Stabiliteit
NEN 6771, december 1991

[22] Staalconstructies TGB 1990
Stabiliteit
NEN 6771, januari 2000

[23] NEN 6072
Rekenkundige bepaling van de brandwerendheid van bouwdelen
Staalconstructies
December 1991

[24] NEN 6072/A2 – Wijzigingsblad
Rekenkundige bepaling van de brandwerendheid van bouwdelen
Staalconstructies
December 2001

[25] NEN 6702
Belastingen en vervormingen TGB 1990
December 1991

[26] Handleiding moduull STACO VGI
Staalbouwkundig Genootschap
Staalcentrum Nederland
5684/82

[27] Newmark N.M. A simple approximate formula for effective end-fixity of columns
J.Aero.Sc. Vol.16 Feb.1949 pp.116

[28] Stabiliteit voor de staalconstructeur
uitgave Staalbouwkundig Genootschap

[29] Rapportnr. BI-87-20/63.4.3360
Controleregels voor lijnvormige constructie-elementen
IBBC Maart 1987

[30] Y. Galéa
Flambement des poteaux à inertie variable
Construction Métallique 1-1981

[31] I. Vayas
Interaktion der plastischen Grenzschnittgrössen doppelsymmetrischer I-Querschnitte
Stahlbau 69 (2000), Heft 9

[32] SCIA Engineer Steel Code Check
Theoretical Background
04/2011

[33] Eurocode 3
Design of steel structures
Part 1 – 1: General rules and rules for buildings
EN 1993-1-1:2005

[34] Rules for Member Stability in EN 1993-1-1
Background documentation and design guidelines
ECCS - N° 119
2006

[35] Lateral-Torsional buckling of steel beams:
A general expression for the moment gradient factor.
A. López, D. J. Yong, M. A. Serna
Stability and Ductility of Steel Structures, 2006

[36] Valorization Project Semi-Comp+
N° RFS2-CT-2010-00023
Background Information
22 March 2012

[37] SN001a-EN-EU
NCCI: Critical axial load for torsional and flexural torsional buckling modes
Access Steel, 2006
www.access-steel.com