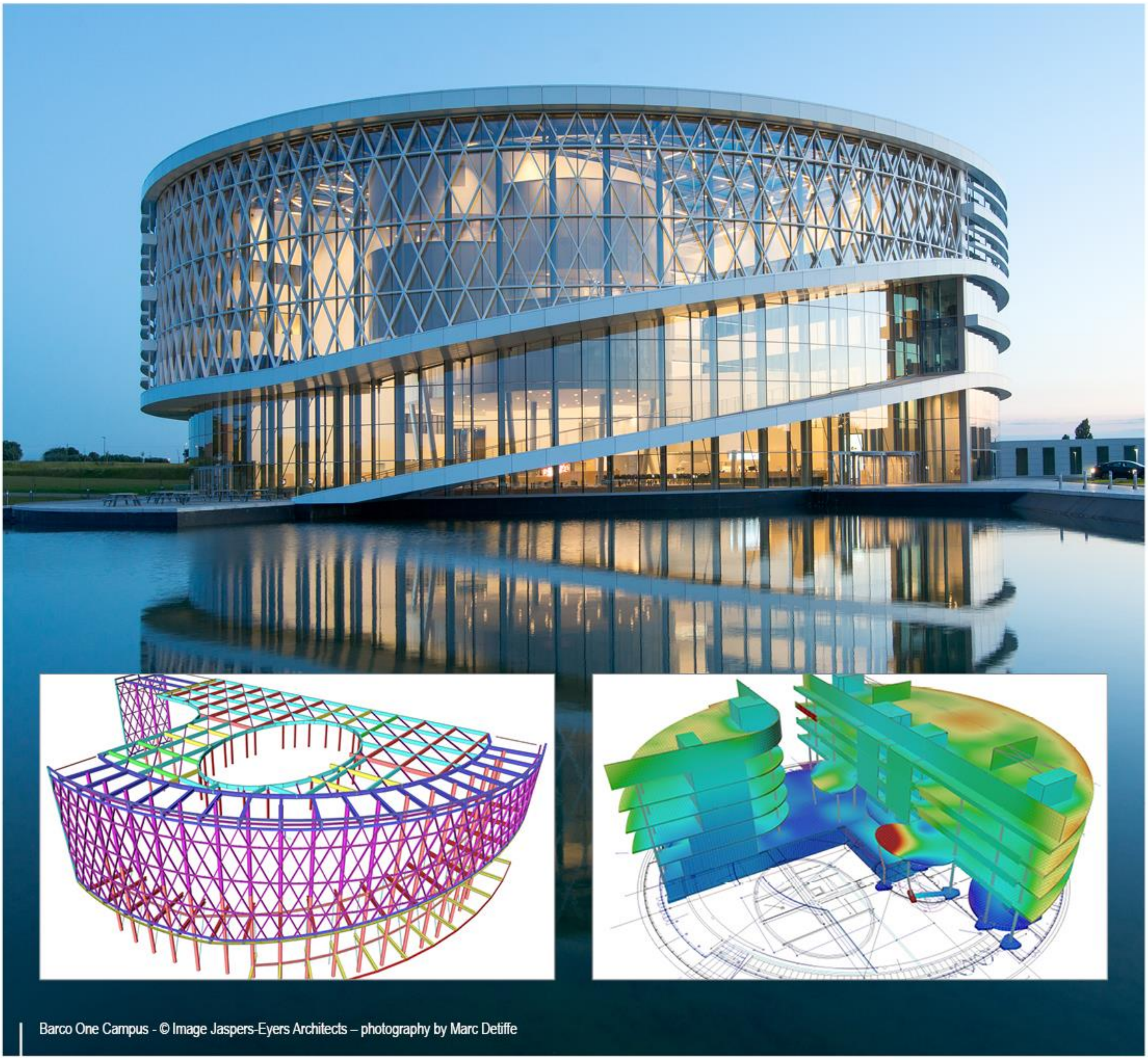
SCIA ENGINEER



Barco One Campus - © Image Jaspers-Eyers Architects – photography by Marc Detiffe

Advanced Training

Cold Formed Steel Check

Table of contents

Introduction

The members are checked according to the regulations given in:

Eurocode 3
Design of steel structures
Part 1 - 3: Supplementary rules for cold-formed members and sheeting

EN 1993-1-3:2006
Corrigendum
EN 1993-1-3:2006/AC:2009

Eurocode 3
Design of steel structures
Part 1 - 5: Plated Structural elements
EN 1993-1-5:2006

Corrigendum
EN 1993-1-5:2006/AC: 2009.

The explained rules are valid for SCIA Engineer 2010.0.

The examples are marked by **Example**

Materials and Combinations

Steel grades

The characteristic values of the material properties are based on EN 1993-1-3 - Table 3.1

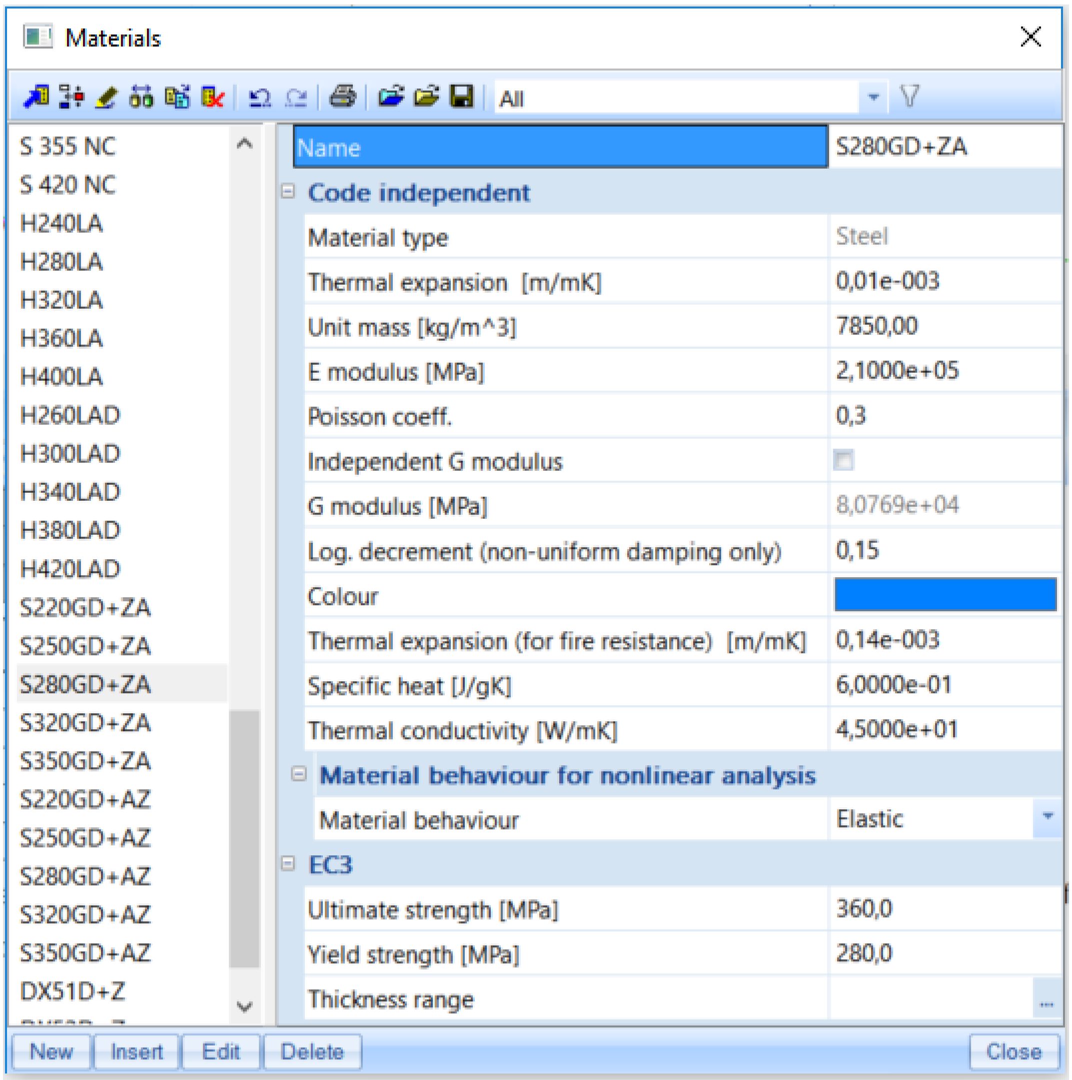
Table 3.1a: Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u

Type of steel	Standard	Grade	f_{yb} N/mm^2	f_u N/mm^2
Hot rolled products of non-alloy structural steels. Part 2: Technical delivery conditions for non alloy structural steels	EN 10025: Part 2	S 235	235	360
		S 275	275	430
		S 355	355	510
Hot-rolled products of structural steels. Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels	EN 10025: Part 3	S 275 N	275	370
		S 355 N	355	470
		S 420 N	420	520
		S 460 N	460	550
		S 275 NL	275	370
		S 355 NL	355	470
		S 420 NL	420	520
		S 460 NL	460	550
Hot-rolled products of structural steels. Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels	EN 10025: Part 4	S 275 M	275	360
		S 355 M	355	450
		S 420 M	420	500
		S 460 M	460	530
		S 275 ML	275	360
		S 355 ML	355	450
		S 420 ML	420	500
		S 460 ML	460	530

Table 3.1b: Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u

Type of steel	Standard	Grade	f_{yb} N/mm^2	f_u N/mm^2
Cold reduced steel sheet of structural quality	ISO 4997	CR 220	220	300
		CR 250	250	330
		CR 320	320	400
Continuous hot dip zinc coated carbon steel sheet of structural quality	EN 10326	S220GD+Z	220	300
		S250GD+Z	250	330
		S280GD+Z	280	360
		S320GD+Z	320	390
		S350GD+Z	350	420
Hot-rolled flat products made of high yield strength steels for cold forming. Part 2: Delivery conditions for thermomechanically rolled steels	EN 10149: Part 2	S 315 MC	315	390
		S 355 MC	355	430
		S 420 MC	420	480
		S 460 MC	460	520
		S 500 MC	500	550
		S 550 MC	550	600
		S 600 MC	600	650
		S 650 MC	650	700
		S 700 MC	700	750
	EN 10149: Part 3	S 260 NC	260	370
		S 315 NC	315	430
		S 355 NC	355	470
		S 420 NC	420	530
Cold-rolled flat products made of high yield strength micro-alloyed steels for cold forming	EN 10268	H240LA	240	340
		H280LA	280	370
		H320LA	320	400
		H360LA	360	430
		H400LA	400	460
Continuously hot-dip coated strip and sheet of steels with higher yield strength for cold forming	EN 10292	H260LAD	240 2)	340 2)
		H300LAD	280 2)	370 2)
		H340LAD	320 2)	400 2)
		H380LAD	360 2)	430 2)
		H420LAD	400 2)	460 2)
Continuously hot-dipped zinc-aluminium (ZA) coated steel strip and sheet	EN 10326	S220GD+ZA	220	300
		S250GD+ZA	250	330
		S280GD+ZA	280	360
		S320GD+ZA	320	390
		S350GD+ZA	350	420
Continuously hot-dipped aluminium-zinc (AZ) coated steel strip and sheet	EN 10326	S220GD+AZ	220	300
		S250GD+AZ	250	330
		S280GD+AZ	280	360
		S320GD+AZ	320	390
		S350GD+AZ	350	420
Continuously hot-dipped zinc coated strip and sheet of mild steel for cold forming	EN 10327	DX51D+Z	140 1)	270 1)
		DX52D+Z	140 1)	270 1)
		DX53D+Z	140 1)	270 1)

In SCIA Engineer, the following materials are provided by default:



Average yield strength

When EC3 is selected and cold formed sections are used, the average yield strength f_{ya} can be used (by setting the proper data flag in the Cross Section input dialog).
The average yield strength is determined as follows (Formula (3.1) of EN 1993-1-3)

$$f_{ya} = f_{yb} + \left(\frac{knt^2}{A_g}\right)\left(f_u - f_{yb}\right) \leq \left(\frac{f_u + f_{yb}}{2}\right)$$

with	f_{yb}	the tensile yield strength = f_y
	f_u	the tensile ultimate strength
	t	the material thickness
	A_g	the gross cross-sectional area
	k	is a coefficient depending on the type of forming :
		k = 7 for cold rolling
		k = 5 for other methods of forming
	n	the number of 90° bends in the section

The average yield strength may not be used if A_{eff} does not equal the gross area A_g (so not for class 4 profiles).

Example - AverageYieldStrength.esa

Manual calculation:

- **CS1 : HE1000X393 – S235**
- A = 50020 mm²
- tf=43.9 mm > 40 mm → fy = 215 N/mm²
- EC3 Check : $N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ (eq. 6.10 of EN 1993-1-1)

  NRd = 50020 x 215 / 1.00 = 10754,3 kN

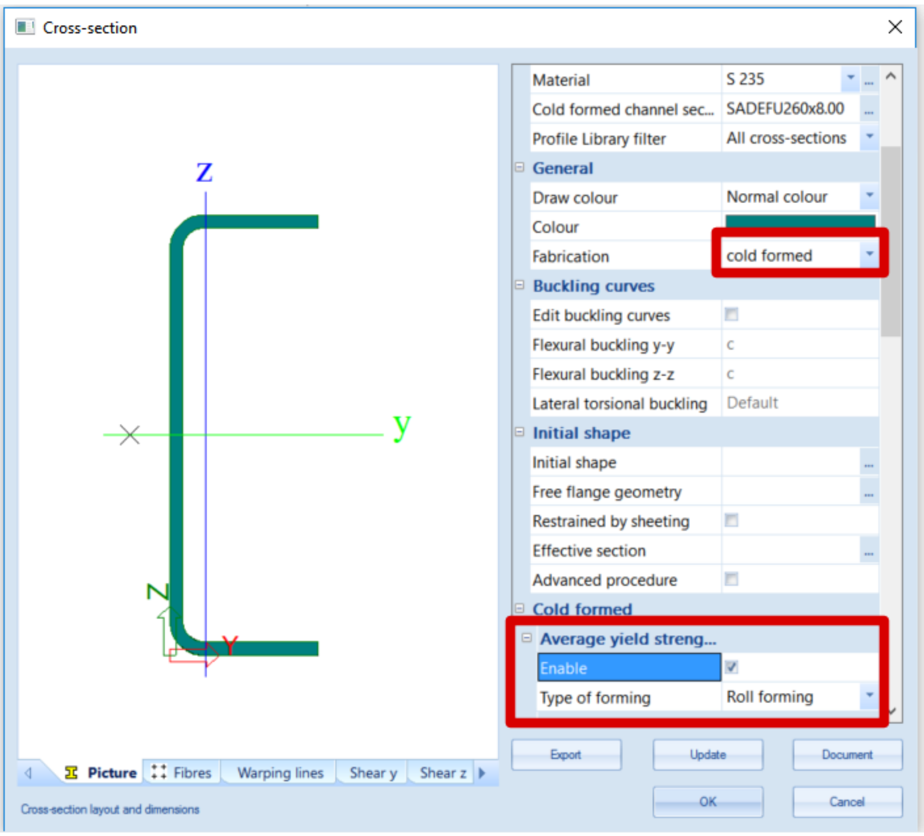
  U.C. = 500/10754,3 = 0,05

- **CS2 : SADEFU260x8.00 – S235 – Cold formed**
- A= 3330mm²
- n=2
- fya = 235 + (7x2x8²) / 3330 x (360-235) = 235 + 33,6 = 268,6 N/mm²
- fya=min(268,77; (360+235)/2.0)=min(268,77;,297,5)=268,77 N/mm²
- EC3 Check : $N_{c,Rd} = A_g \cdot (f_{yb} + (f_{ya} - f_{yb}) \cdot 4 \cdot (1 - \frac{\bar{\lambda}_e}{\bar{\lambda}_{e0}}))/\gamma_{M0}$ (Aeff = Ag →eq. 6.3 of EN1993-1-3)

  NRd = 3330 x (235+(268,6-235) x 4 x (1-0,55/0,67)) / 1,00 = 862,8 kN

  U.C. = 500/862,8 = 0,58

In SCIA Engineer:



Material data		
yield strength fy	235.0	MPa
average yield strength fy,a	268.6	MPa
k	7	
n	2	
tension strength fu	360.0	MPa
fabrication	cold formed	

Axial Compression Check
According to article EN 1993-1-3: 6.1.3 and formula (6.3)

Table of values		
Ag	3330	mm²
Aeff	3330	mm²
Critical Element	3	
Element Type	plane	
Lambda e	0.55	
Lambda e0	0.67	
Nc,Rd	866.34	kN
Unity check	0.58	-

Note
The average yield strength is calculated using the gross section A_g of the initial shape.

In SCIA Engineer the average yield strength is applied in the following resistance calculations:

- Axial Tension
- Axial Compression
- Bending Moment
- Torsional moment
- Flexural buckling
- Torsional (-Flexural) Buckling
- Purlin design – Cross-section resistance

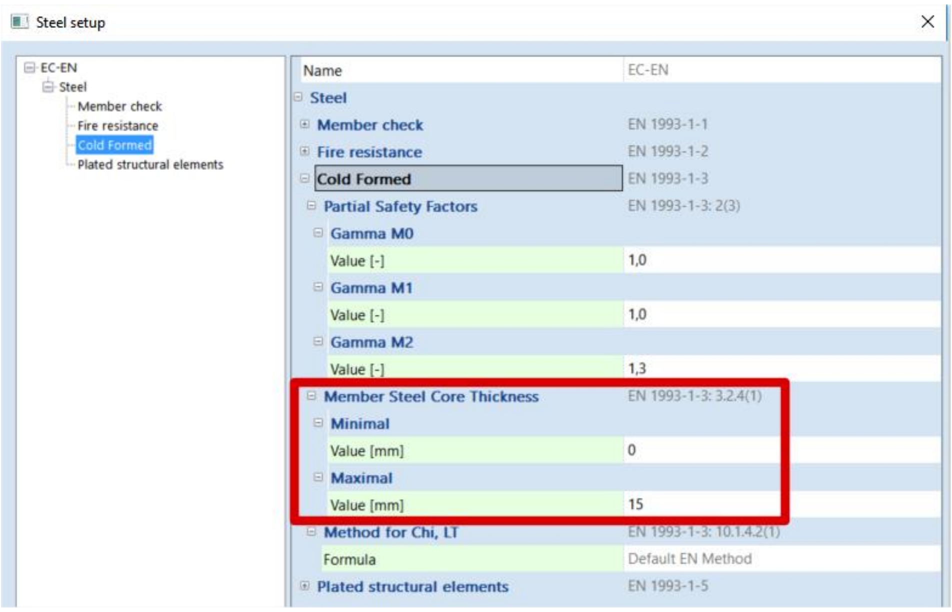
Steel core thickness

Thickness tolerances

The provisions for design by calculation given in the part 1-3 of EN 1993 may be used for steel within a given ranges of core thickness t_{cor}.

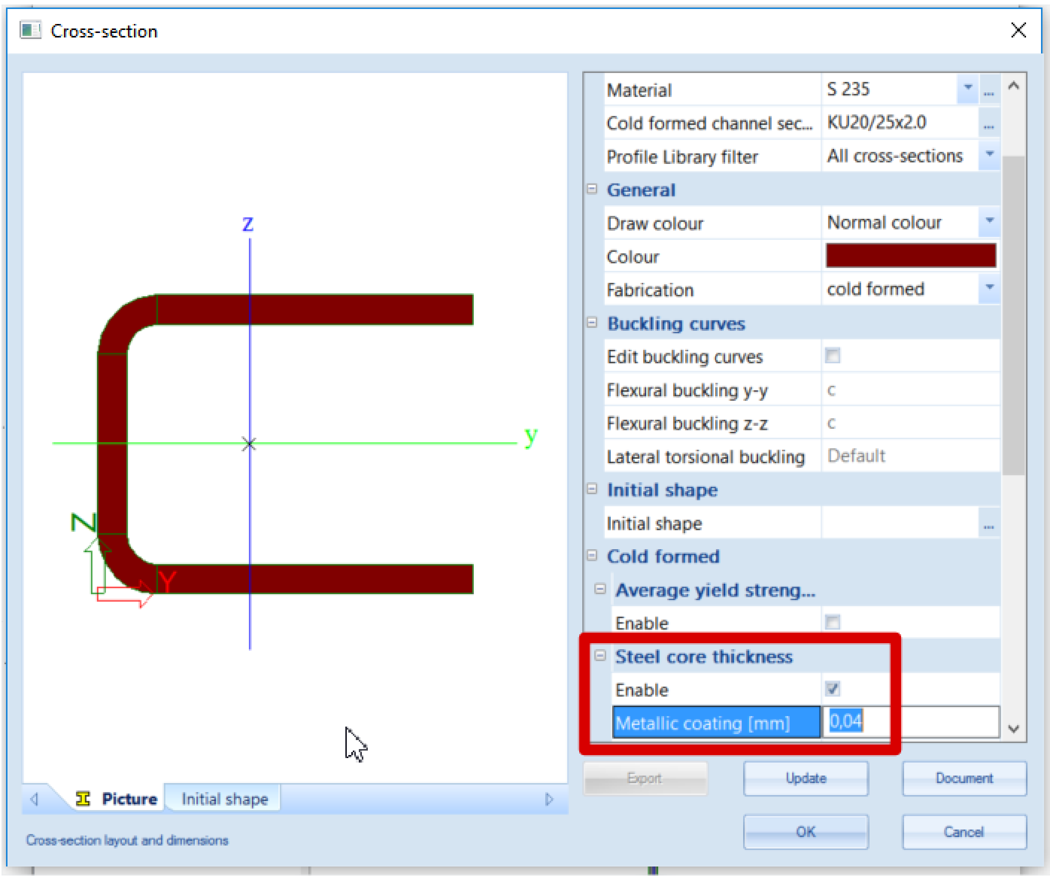
The default value in the EN 1993, article 3.2.4 is:
0,45mm $\leq t_{cor} \leq$ 15mm
But this can be adapted in the National Annex.

In SCIA Engineer this default value is also taken as default but can be adapted in the National Annex parameters:



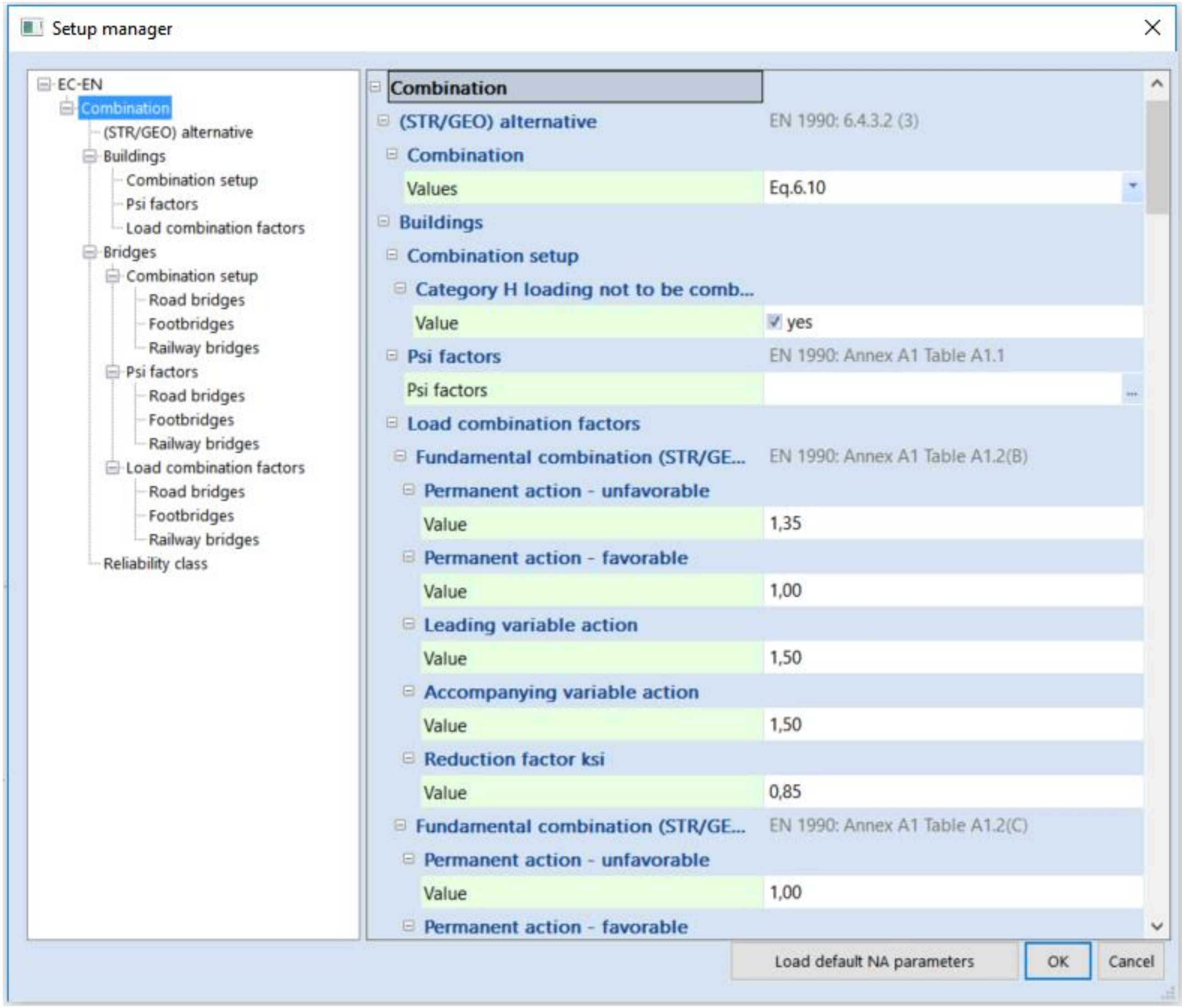
Thickness coating

In SCIA Engineer the user can choose to take the coating-thickness into account in the steel core thickness. This can be activated or deactivated in the cross-section menu.
Default this thickness is taken as 0,04mm but can be adapted for each profile.



Combinations

In SCIA Engineer, both the SLS and ULS combinations can be set according to the code rules for EC-EN 1990. In this setup, partial safety factors and Psi factors can be set.



Following EC-EN 1990:2002 the ULS combinations can be expressed in two ways.

- Using Equation 6.10

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}\text{'+'}\gamma_P P\text{'+'}\gamma_{Q,1}Q_{k,1}\text{'+'}\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$$

- Using Equations 6.10a and 6.10b

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}\text{'+'}\gamma_P P\text{'+'}\gamma_{Q,1}\psi_{0,1}Q_{k,1}\text{'+'}\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$$

$$\sum_{j\geq 1}\xi_j\gamma_{G,j}G_{k,j}\text{'+'}\gamma_P P\text{'+'}\gamma_{Q,1}Q_{k,1}\text{'+'}\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$$

Both methods have been implemented in SCIA Engineer. The method which needs to be applied will be specified in the National Annex.

Example

Consider a simple building subjected to an unfavorable permanent load, a Category A Imposed load and a Wind load. This example is calculated using **Set B**.

for unfavorable permanent actions $\gamma_G = 1{,}35$ for the leading variable action $\gamma_{Q,1} = 1{,}50$ for the non-leading variable actions $\gamma_{Q,i} = 1{,}50$
ψ_0 for Wind loads equals 0,6 ψ_0 for an Imposed Load Category A equals 0,7
Reduction factor for unfavourable permanent actions $\xi = 0{,}85$
Using equation 6.10: → Combination 1: 1,35 Permanent + 1,5 Imposed + 0,9 Wind → Combination 2: 1,35 Permanent + 1,05 Imposed + 1,5 Wind
Using equations 6.10a and 6.10b: → Combination 1: 1,35 Permanent + 1,05 Imposed + 0,9 Wind → Combination 2: 1,15 Permanent + 1,5 Imposed + 0,9 Wind → Combination 3: 1,15 Permanent + 1,05 Imposed + 1,5 Wind

Local and distortional buckling

Initial shape

For a cross-section defined as cold formed, the Initial Shape must be defined. This initial shape is supported for the following cross-section types:

- Standard profile library cross-sections
- General thin-walled sections
- General sections with thin-walled representation
- Thin-walled geometric sections
- All other sections which support the centreline and do not have roundings
- Cold formed Pair cross-sections of profile library sections

The inputted types of parts are used further used for determining the classification and reduction factors.
The thin-walled cross-section parts can have for the following types:

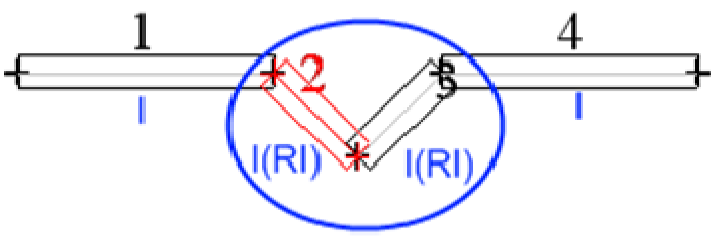
F	Fixed Part – No reduction is needed
I	Internal cross-section part
SO	Symmetrical Outstand
UO	Unsymmetrical Outstand

A part of the cross-section can also be considered as reinforcement:

none	Not considered as reinforcement
RI	Reinforced Internal (intermediate stiffener)
RUO	Reinforced Unsymmetrical Outstand (edge stiffener)
DEF	Double Edge Fold (edge stiffener)

ROU and **DEF** reinforcement types can be set only to elements of type **SO** or **UO**.
RI types can be set only to elements of type **I** or **UO** or **SO**.

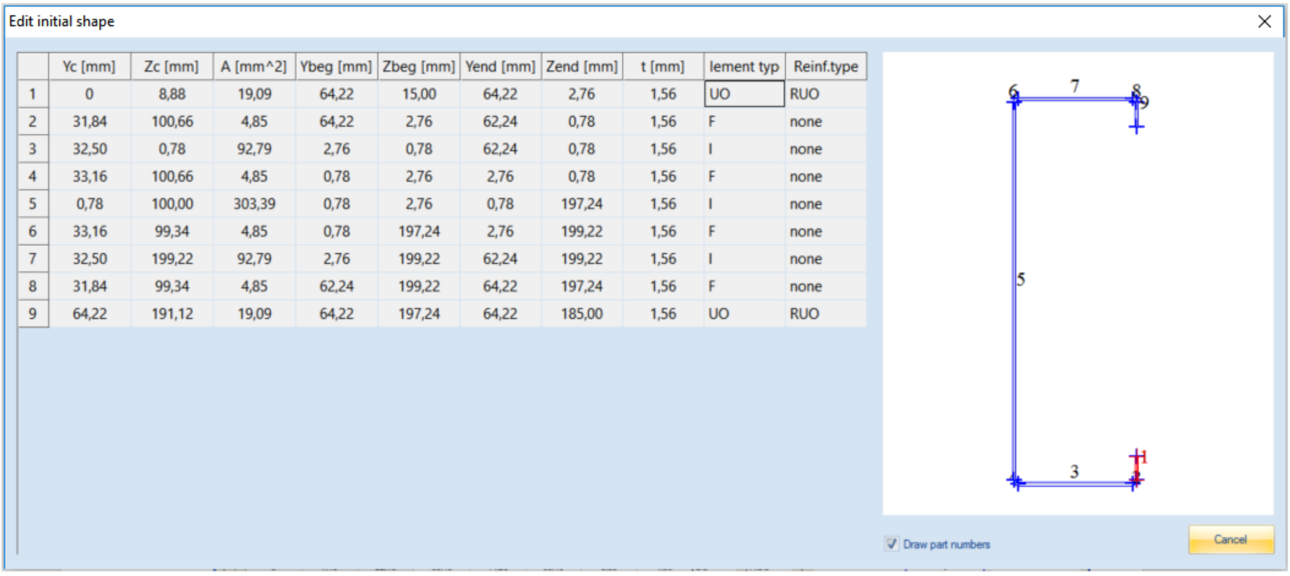
In case a part is specified as reinforcement, a reinforcement ID can be inputted.
For general cross-sections neighbouring elements of type **RI** are seen as one stiffener for the calculation of the stiffener area and inertia:



Note
For standard profile library cross-sections and pair sections, the initial shape is generated automatically.

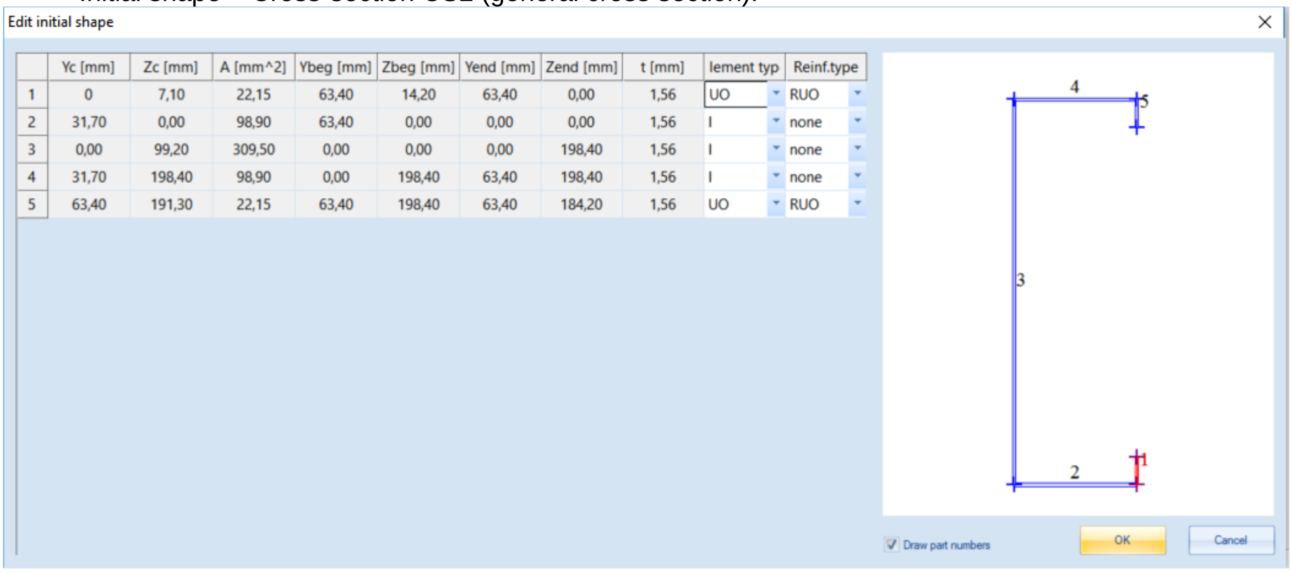
Example – WS CFS 02.esa

Initial shape – Cross-section CS1 (Cold formed C section from Library)



	Yc [mm]	Zc [mm]	A [mm^2]	Ybeg [mm]	Zbeg [mm]	Yend [mm]	Zend [mm]	t [mm]	lement typ	Reinf.type
1	0	8,88	19,09	64,22	15,00	64,22	2,76	1,56	UO	RUO
2	31,84	100,66	4,85	64,22	2,76	62,24	0,78	1,56	F	none
3	32,50	0,78	92,79	2,76	0,78	62,24	0,78	1,56	I	none
4	33,16	100,66	4,85	0,78	2,76	2,76	0,78	1,56	F	none
5	0,78	100,00	303,39	0,78	2,76	0,78	197,24	1,56	I	none
6	33,16	99,34	4,85	0,78	197,24	2,76	199,22	1,56	F	none
7	32,50	199,22	92,79	2,76	199,22	62,24	199,22	1,56	I	none
8	31,84	99,34	4,85	62,24	199,22	64,22	197,24	1,56	F	none
9	64,22	191,12	19,09	64,22	197,24	64,22	185,00	1,56	UO	RUO

Initial shape – Cross-section CS2 (general cross section):



	Yc [mm]	Zc [mm]	A [mm^2]	Ybeg [mm]	Zbeg [mm]	Yend [mm]	Zend [mm]	t [mm]	lement typ	Reinf.type
1	0	7,10	22,15	63,40	14,20	63,40	0,00	1,56	UO	RUO
2	31,70	0,00	98,90	63,40	0,00	0,00	0,00	1,56	I	none
3	0,00	99,20	309,50	0,00	0,00	0,00	198,40	1,56	I	none
4	31,70	198,40	98,90	0,00	198,40	63,40	198,40	1,56	I	none
5	63,40	191,30	22,15	63,40	198,40	63,40	184,20	1,56	UO	RUO

Effective shape

Influence of rounded corners

In cross-sections with rounded corners, the notional flat widths b_p of the plane elements should be measured from the midpoints of the adjacent corner elements as indicated in figure 5.1 EN 1993-1-3 “Notional widths of plane cross sections parts b_p”.

This principle is implemented in SCIA Engineer.

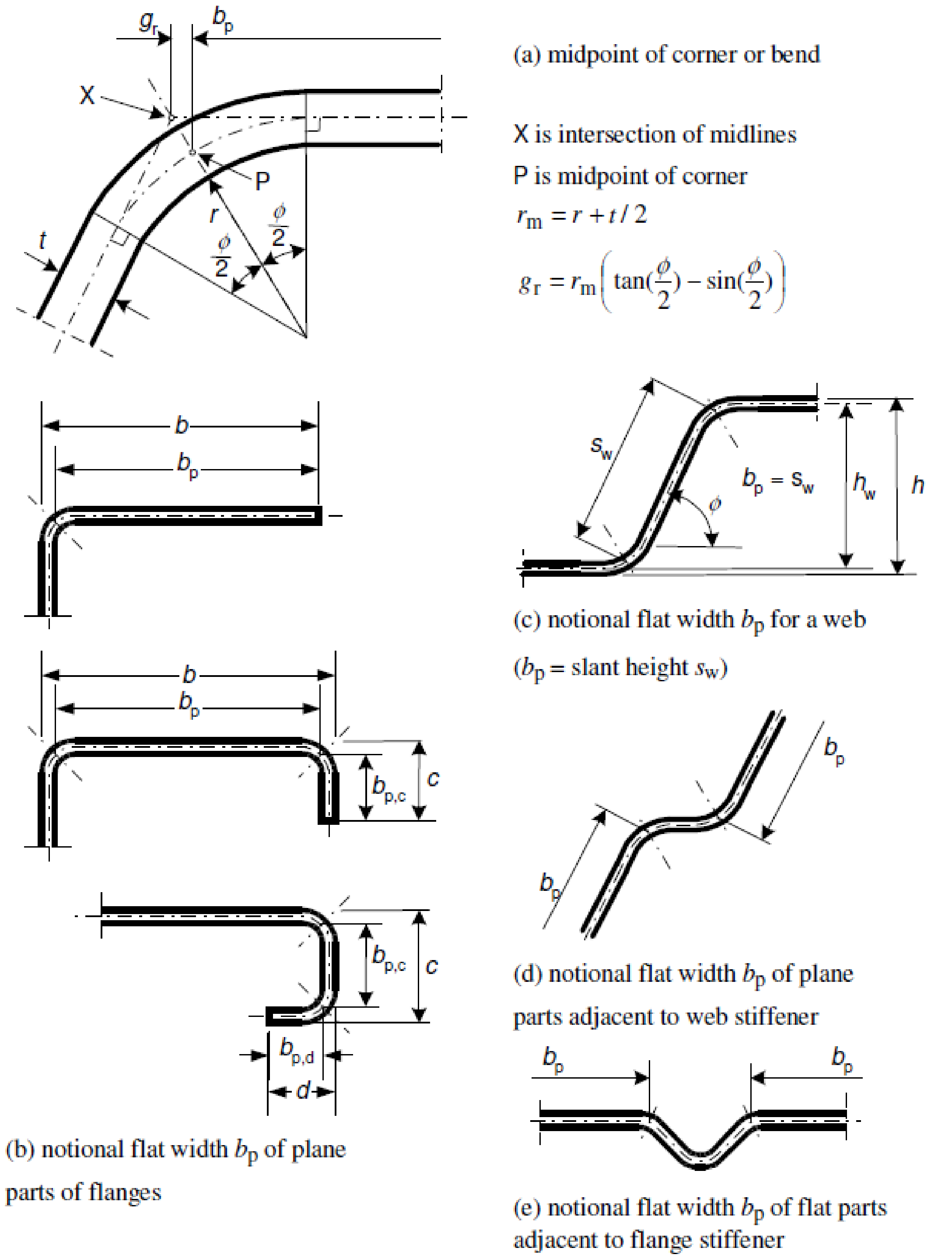


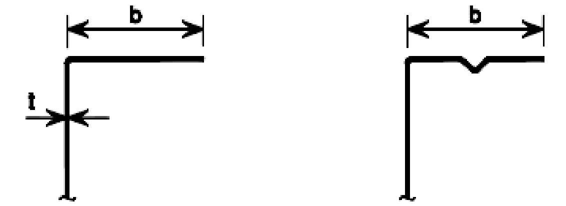
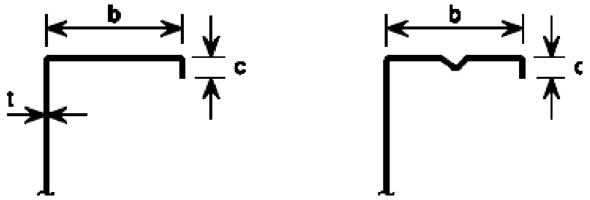
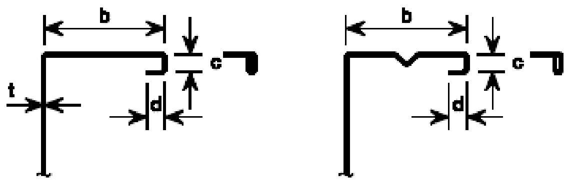
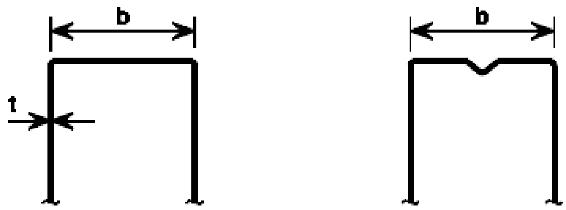
Figure 5.1: Notional widths of plane cross section parts b_p allowing for corner radii

Article 5.1(3) of EN 1993-1-3 gives an alternative procedure to calculate the influence of rounded corners, but this approximate procedure is not supported in SCIA Engineer. SCIA Engineer will always calculate the influence with the exact procedure.

Geometrical proportions

The geometrical proportions are checked according to EN 1993-1-3 article 5.2(1) Table 5.1 “Maximum with-to-thickness ratios”:

Table 5.1: Maximum width-to-thickness ratios

Element of cross-section	Maximum value
	$b/t \leq 50$
	$b/t \leq 60$ $c/t \leq 50$
	$b/t \leq 90$ $c/t \leq 60$ $d/t \leq 50$
	$b/t \leq 500$
	$45° \leq \phi \leq 90°$ $h/t \leq 500 \sin\phi$

The limits for edge stiffeners (c) and double edge folds (d) are checked in case the correct stiffener type (**RUO** or **DEF**) has been set in the initial shape.

Also the limit ratios given in EN 1993-1-3 article 5.2(2) are checked.

$0{,}2 \ \leq c/b \leq 0{,}6$

$0{,}1 \ \leq d/b \leq 0{,}3$

In article 5.2(2) is set that if $c/b < 0{,}2$ or $d/b < 0{,}1$ the lip should be ignored, but in SCIA Engineer lip dimensions c and d are however always accounted for and will not be ignored.

In addition the limit for the internal radius given in EN 1993-1-3 article 5.1(6) is checked:

Where the internal radius r > 0,04 t E / f_y then the resistance of the cross-sections should be determined by tests.

Note

If the maximum value for the width-to-thickness ratios is exceeded, EN 1993-1-3 described that the steel cold formed checks still can be executed if the limit states are verified by an appropriate number of tests.

If this maximum value is exceeded in SCIA Engineer, the program will give a warning message in the preview window, but will perform the check following EN 1993-1-3.

Example – WS CFS 07.esa

Warning message if c/b exceeds the maximum value:

Check of steel

Linear calculation, Extreme : Member
Selection : All
Load cases : LC1

EN 1993-1-3 Cold Formed Code Check
National annex: Standard EN

Member B1	**3,000 m**	**Cold formed C section (200; 80; 2; 6; 80)**	**S280GD+ZA**	**LC1**	**0,53 -**

Basic data EC3 : EN 1993	
partial safety factor Gamma M0 for resistance of cross-sections	1.00
partial safety factor Gamma M1 for resistance to instability	1.00
partial safety factor Gamma M2 for resistance of net sections	1.25

Warning: Cross-Section dimensions ratio is outside the limit: 0,2 <= c/b <= 0,6 (Art. 5.2(2)).

Material data		
yield strength fy	280.0	MPa
tension strength fu	360.0	MPa
fabrication	cold formed	

General procedure for one element

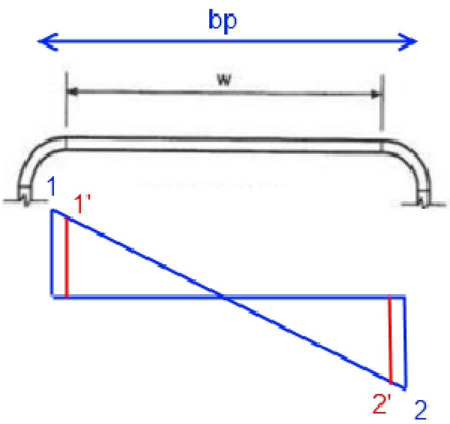
By default EN 1993-1-3 specifies that the stress **f** ($\boldsymbol{\sigma_{com,Ed}}$) to be used for the effective section calculation should be taken as $\boldsymbol{f_y/\gamma_{M0}}$.

The reduction of an element is in general given by:
$b_{eff} = \rho * b$

With
b_{eff} effective width
ρ reduction factor
b full width

<u>Step 1</u>:

For the given stress **f** the normal stress over the rectangular plate element of the initial geometrical shape is calculated. These stresses are calculated based on the nominal width $\mathbf{b_p}$.



1 & 2 - Stresses based on bp

1' & 2' - Stresses based on w

σ_{beg}: normal stress at start point of rectangular shape – compression stress is positive
σ_{end}: normal stress at end point of rectangular shape – compression stress is positive

If the rectangular shape is completely under tension, i.e. beg and end are both tensile stresses, no reduction is needed, p = 1.0

Step 2: Determine f1 and f2:

In case $|\sigma_{beg}| \geq |\sigma_{end}|$
$f_1 = \sigma_{beg}$
$f_2 = \sigma_{end}$

In case $|\sigma_{beg}| < |\sigma_{end}|$
$f_1 = \sigma_{end}$
$f_2 = \sigma_{beg}$

Step 3: Calculate the stress gradient ψ:

$$\psi = f_2/f_1$$

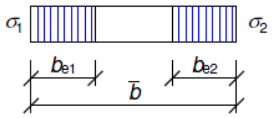
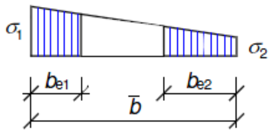
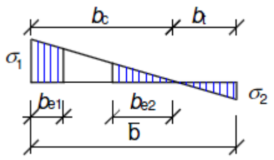
Step 4:
If $\psi = 1$ the element is under uniform compression, else the element is under stress gradient.

Depending on the stress gradient and the element type, the effective width can be calculated as specified in the following paragraphs.

Internal Compression Elements

The effective width of internal compression elements is calculated according to EN 1993-1-5 **article 4.4** and **Table 4.1**.
This applies to elements of type **I**. The notional width $\boldsymbol{b_p}$ is used as $\bar{\boldsymbol{b}}$.

Table 4.1: Internal compression elements

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

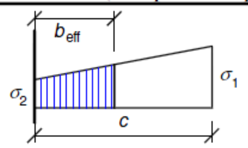
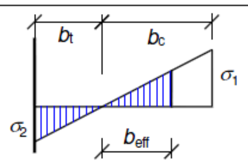
$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi > -3$
Buckling factor k_σ	4,0	$8{,}2\ /\ (1{,}05 + \psi)$	7,81	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	23,9	$5{,}98\ (1 - \psi)^2$

Outstanding Compression Elements

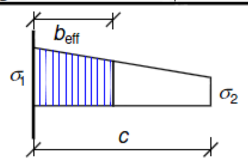
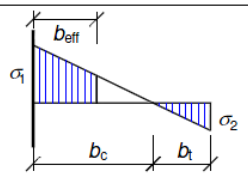
The effective width of internal compression elements is calculated according to EN 1993-1-5 **article 4.4** and **Table 4.2**.

This applies to elements of type **UO** and **SO**. The notional width b_p is used as c.

Table 4.2: Outstand compression elements

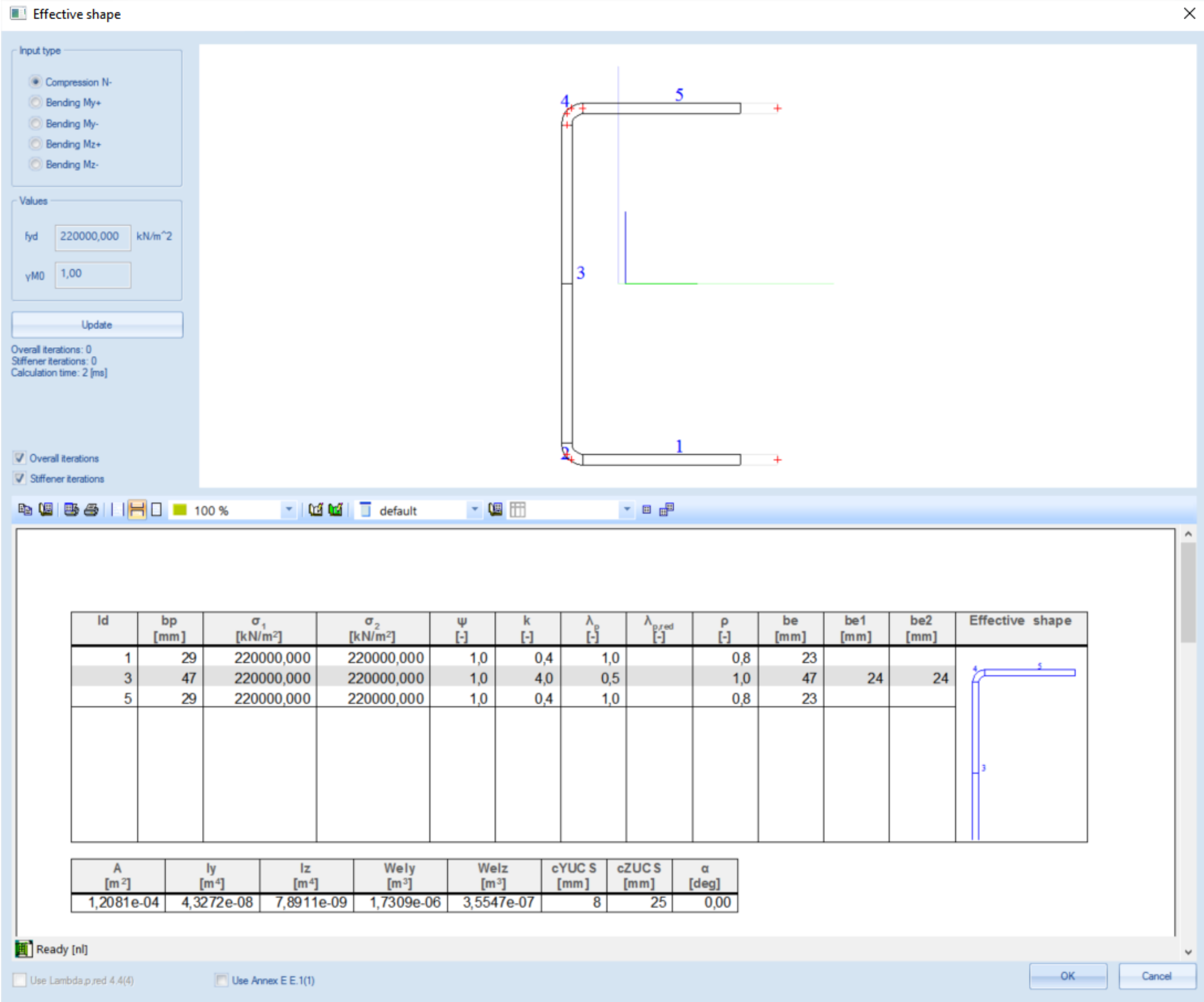
Stress distribution (compression positive)	Effective[p] width b_{eff}
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ c$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ c\ /\ (1\text{-}\psi)$

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \geq \psi \geq -3$
Buckling factor k_σ	0,43	0,57	0,85	$0{,}57 - 0{,}21\psi + 0{,}07\psi^2$

Stress distribution (compression positive)	Effective[p] width b_{eff}
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ c$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ c\ /\ (1\text{-}\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0{,}578\ /\ (\psi + 0{,}34)$	1,70	$1{,}7 - 5\psi + 17{,}1\psi^2$	23,8

➤ **Example WS CFS 06.esa**



Id	bp [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k [-]	λ_p [-]	$\lambda_{p,red}$ [-]	ρ [-]	be [mm]	be1 [mm]	be2 [mm]	Effective shape
1	29	220000,000	220000,000	1,0	0,4	1,0		0,8	23			
3	47	220000,000	220000,000	1,0	4,0	0,5		1,0	47	24	24	
5	29	220000,000	220000,000	1,0	0,4	1,0		0,8	23			

A [m²]	Iy [m⁴]	Iz [m⁴]	Wely [m³]	Welz [m³]	cYUCS [mm]	cZUCS [mm]	α [deg]
1,2081e-04	4,3272e-08	7,8911e-09	1,7309e-06	3,5547e-07	8	25	0,00

General method for Plane elements with Edge or intermediate Stiffeners

Article 5.5.3.1 (En 1993-1-3) gives the general method for plane elements with edge or intermediate stiffeners

The design of compression elements with edge or intermediate stiffeners should be based on the assumption that the stiffener behaves as a compression members with continuous partial restraint, with a spring stiffness that depends on the boundary conditions and the flexural stiffness of the adjacent plane elements.
The spring stiffness per unit length may be determined from:

$$K = u/\delta$$

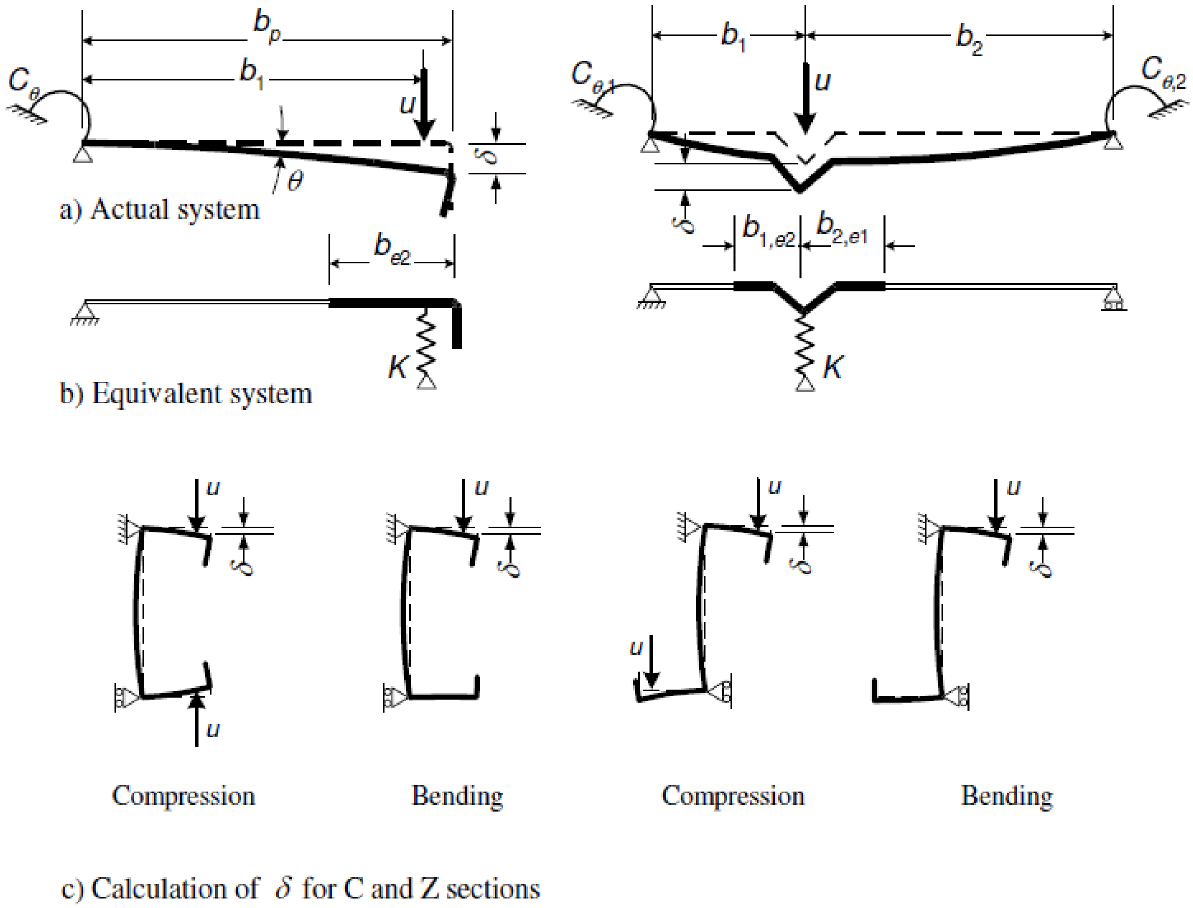


Figure 5.6: Determination of spring stiffness

In case of the edge stiffener of lipped C-sections and lipped Z-sections, C_θ should be determined with the unit load *u* applied as shown in figure 5.6(c). This results in the following expression for the spring stiffness **K** for the flange 1:

$$K = \frac{E\,t^3}{4(1-\nu^2)} \cdot \frac{1}{b_1^2 h_w + b_1^3 + 0{,}5 b_1 b_2 h_w k_f}$$

Where

b_1 — see figure 5.6(a)

b_1 — see figure 5.6(a)

h_w — is the web depth

$k_f = 0$ — if flange 2 is in tension (e.g. for beam in bending about the y-y-axis)

$k_f = A_{s2} / A_{s1}$ — if flange 2 is also in compression (e.g. for beam in axial compression)

$k_f = 1$ — for a symmetric section in compression

A_{s2} and A_{s1} — is the effective area of the edge stiffener (including effective part b_{e2} of the flange, see figure 5.6(b)) of flange 1 and flange 2 respectively.

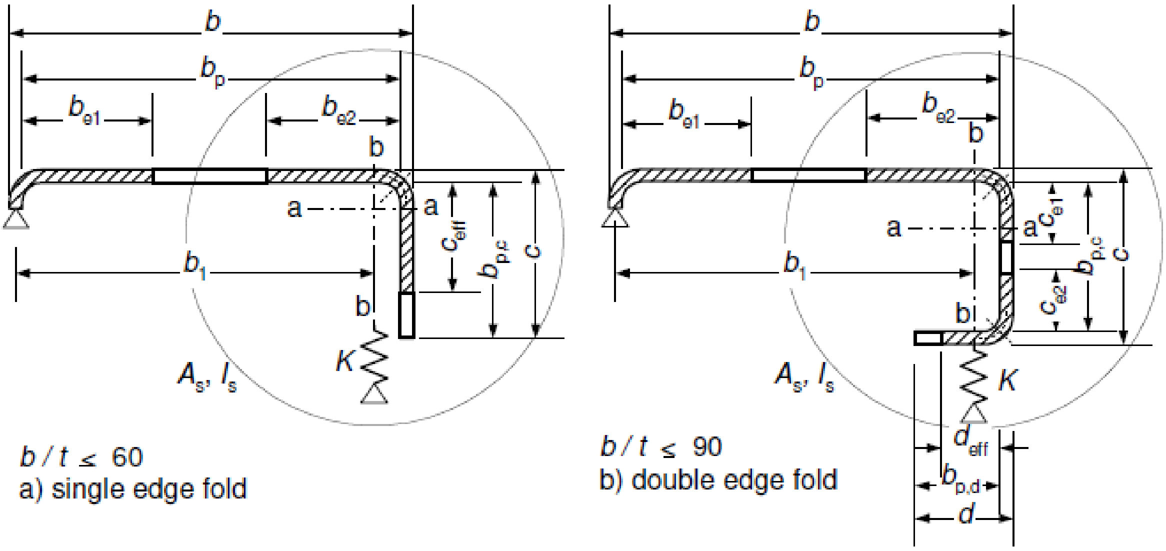
Note

This formula of K (given in the EN 1993-1-3) is based purely on simple sections with two flanges. In case of more complex cross-sections, the only exact procedure is to perform a numerical analysis (finite strip method) to determine the critical stresses for local and distortional buckling. This is referenced as the "general procedure" given in article 5.5.1(7). This method is currently not supported by SCIA Engineer.

Plane elements with Edge Stiffeners

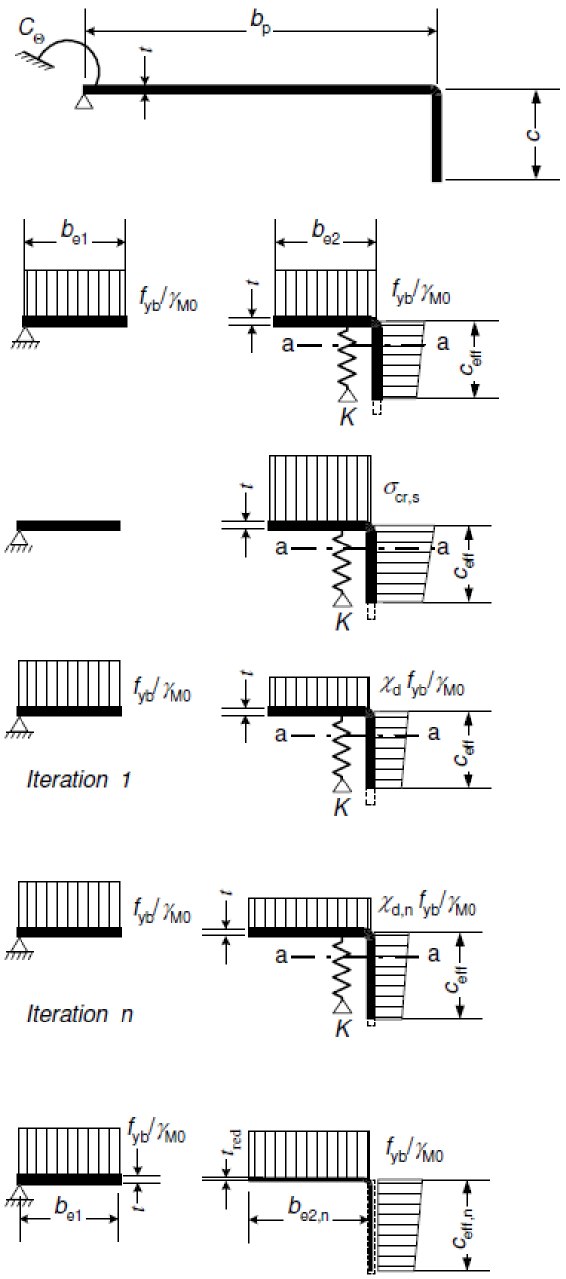
The procedure for determining the effective width/thickness of elements with edge stiffeners is given in EN 1993-1-3 **article 5.5.3.2** and **5.5.3.1**.

This applies to elements of type **RUO** and **DEF**.



Procedure:

- **Step 1** Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb}/\gamma_{M0}$.
-
- **Step 2** Use the initial effective cross-section of the stiffener to determine the reduction factor for distortional buckling, allowing for the effects of the continuous spring restraint.
-
- **Step 3** Optionally iterate to refine the value of the reduction factor for buckling of the stiffener.



a) Gross cross-section and boundary conditions

b) **Step 1**: Effective cross-section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb}/\gamma_{M0}$

c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from step 1

d) Reduced strength $\chi_d f_{yb}/\gamma_{M0}$ for effective area of stiffener A_s, with reduction factor χ_d based on $\sigma_{cr,s}$

e) **Step 3**: Optionally repeat step 1 by calculating the effective width with a reduced compressive stress $\sigma_{com,Ed,i} = \chi_d f_{yb} / \gamma_{M0}$ with χ_d from previous iteration, continuing until $\chi_{d,n} \approx \chi_{d,(n-1)}$ but $\chi_{d,n} \leq \chi_{d,(n-1)}$.

f) Adopt an effective cross-section with b_{e2}, c_{eff} and reduced thickness t_{red} corresponding to $\chi_{d,n}$

Step 1:
Determine the effective width with EN 1993-1-5 **article 4.4** and **Table 4.1**.

(2) The reduction factor ρ may be taken as follows:

– internal compression elements:

$$\rho = 1{,}0 \qquad \text{for } \bar{\lambda}_p \leq 0{,}673$$

$$\rho = \frac{\bar{\lambda}_p - 0{,}055\,(3+\psi)}{\bar{\lambda}_p^2} \leq 1{,}0 \qquad \text{for } \bar{\lambda}_p > 0{,}673 \text{ , where } (3+\psi) \geq 0 \qquad (4.2)$$

– outstand compression elements:

$$\rho = 1{,}0 \qquad \text{for } \bar{\lambda}_p \leq 0{,}748$$

$$\rho = \frac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^2} \leq 1{,}0 \qquad \text{for } \bar{\lambda}_p > 0{,}748 \qquad (4.3)$$

where $\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\,\varepsilon\,\sqrt{k_\sigma}}$

<u>For a single edge fold stiffener:</u>

$c_{eff} = \rho b_{p,c}$

ρ is obtained from EN 1993-1-5, (with the notional width $\boldsymbol{b_p}$ is used as $\overline{\boldsymbol{b}}$), except using a value of the buckling factor k_σ given by the following:

If $b_{p,c} / b_p \leq 0{,}35$ => $k_\sigma = 0{,}35$

If $0{,}35 < b_{p,c} / b_p \leq 0{,}6$ => $k_\sigma = 0{,}5 + 0{,}83 \sqrt[3]{\left(b_{p,c}/b_c - 0{,}35\right)^2}$

<u>For a double edge fold stiffener:</u>

$c_{eff} = \rho b_{p,c}$

ρ and k_σ are obtained from EN 1993-1-5 – Table 4.1, (with the notional width $\boldsymbol{b_p}$ is used as $\overline{\boldsymbol{b}}$)

$d_{eff} = \rho b_{p,d}$

ρ and k_σ are obtained from EN 1993-1-5 – Table 4.2, (with the notional width $\boldsymbol{b_p}$ is used as $\overline{\boldsymbol{b}}$)

If $0{,}35 < b_{p,c} / b_p \leq 0{,}6$ => $k_\sigma = 0{,}5 + 0{,}83 \sqrt[3]{\left(b_{p,c}/b_c - 0{,}35\right)^2}$

Step 2:
The effective cross-sectional area of the edge stiffener A_s is calculated correctly, with the exact value for b_p.

And the elastic critical buckling stress:

$$\sigma_{cr,s} = \frac{2\sqrt{K\ E\ I_s}}{A_s}$$

Step 3 (alternative):
The reduction χ_d for the distortional buckling resistance of an edge stiffener should be obtained from the value of $\sigma_{cr,s}$.

The reduction factor χ_d for distortional buckling resistance (flexural buckling of a stiffener) should be obtained from the relative slenderness $\bar{\lambda}_d$ from:

$\chi_d = 1{,}0$ if $\bar{\lambda}_d \leq 0{,}65$

$\chi_d = 1{,}47 - 0{,}723\ \bar{\lambda}_d$ if $0{,}65 < \bar{\lambda}_d < 1{,}38$

$\chi_d = \frac{0{,}66}{\bar{\lambda}_d}$ if $\bar{\lambda}_d \geq 1{,}38$

Where: $\bar{\lambda}_d = \sqrt{f_y/\sigma_{cr,s}}$

If $\chi_d < 1{,}0$ it may be refined iteratively, starting the iteration with modified values of ρ obtained with $\sigma_{com,Ed,i} = \chi_d f_{yb}/\gamma_{M0}$ so that:

$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi_d}$

The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi_d A_s \frac{f_{yb}/\gamma_{M0}}{\sigma_{com,Ed,i}}$$

<u>Conclusion</u>

In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = t\ A_{s,red} / A_s$ for all the elements include in A_s.

Plane elements with intermediate Stiffeners

The procedure for determining the effective width/thickness of elements with intermediate stiffeners is given in EN 1993-1-3 **article 5.5.3.3** and **5.5.3.1**.
This applies to elements of type **RI**.

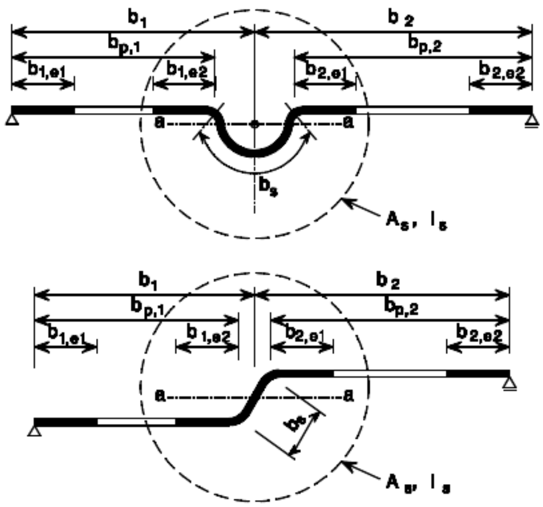
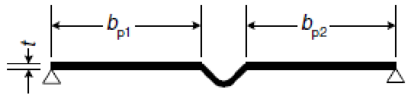
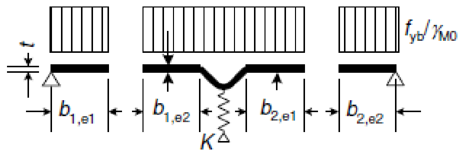


Figure 5.9: Intermediate stiffeners

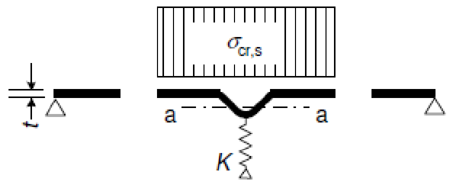
This principle is also shown on the figure below:



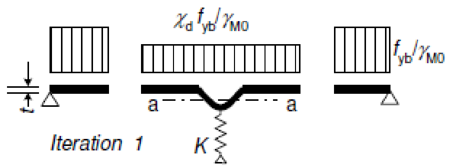
a) Gross cross-section and boundary conditions



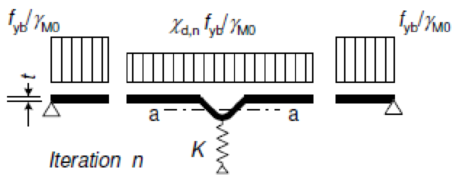
b) **Step 1**: Effective cross-section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb} / \gamma_{M0}$



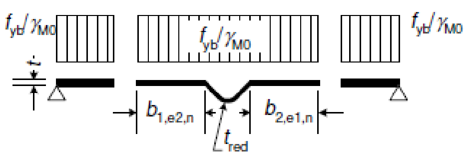
c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from step 1



d) Reduced strength $\chi_d f_{yb} / \gamma_{M0}$ for effective area of stiffener A_s, with reduction factor χ_d based on $\sigma_{cr,s}$



e) **Step 3**: Optionally repeat step 1 by calculating the effective width with a reduced compressive stress $\sigma_{com,Ed,i} = \chi_d f_{yb} / \gamma_{M0}$ with χ_d from previous iteration, continuing until $\chi_{d,n} \approx \chi_{d,(n-1)}$ but $\chi_{d,n} \leq \chi_{d,(n-1)}$.



f) Adopt an effective cross-section with $b_{1,e2}$, $b_{2,e1}$ and reduced thickness t_{red} corresponding to $\chi_{d,n}$

Step 1:
Determine the effective width with EN 1993-1-5 **article 4.4** and **Table 4.1**.

(2) The reduction factor ρ may be taken as follows:

- internal compression elements:

$$\rho = 1{,}0 \qquad \text{for } \bar{\lambda}_p \leq 0{,}673$$

$$\rho = \frac{\bar{\lambda}_p - 0{,}055\,(3+\psi)}{\bar{\lambda}_p^2} \leq 1{,}0 \qquad \text{for } \bar{\lambda}_p > 0{,}673 \text{ , where } (3+\psi) \geq 0 \qquad (4.2)$$

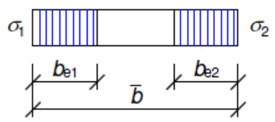
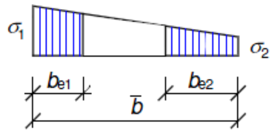
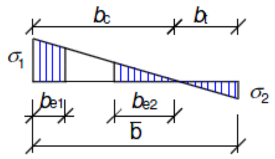
- outstand compression elements:

$$\rho = 1{,}0 \qquad \text{for } \bar{\lambda}_p \leq 0{,}748$$

$$\rho = \frac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^2} \leq 1{,}0 \qquad \text{for } \bar{\lambda}_p > 0{,}748 \qquad (4.3)$$

where $\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\,\varepsilon\sqrt{k_\sigma}}$

Table 4.1: Internal compression elements

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

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

The effective cross-sectional area of the edge stiffener A_s is calculated correctly in SCIA Engineer using the real cross section.

Step 2:

And the elastic critical buckling stress:

$$\sigma_{cr,s} = \frac{2\sqrt{K\,E\,I_s}}{A_s}$$

The reduction χ_d for the distortional buckling resistance of an edge stiffener should be obtained from the value of $\sigma_{cr,s}$.

The reduction factor χ_d for distortional buckling resistance (flexural buckling of a stiffener) should be obtained from the relative slenderness $\bar{\lambda}_d$ from:

$$\chi_d = 1{,}0 \qquad \text{if} \qquad \bar{\lambda}_d \leq 0{,}65$$
$$\chi_d = 1{,}47 - 0{,}723\,\bar{\lambda}_d \qquad \text{if} \qquad 0{,}65 < \bar{\lambda}_d < 1{,}38$$

$$\chi_d = \frac{0,66}{\bar{\lambda}_d} \qquad \text{if} \qquad \bar{\lambda}_d \geq 1,38$$

Where: $\bar{\lambda}_d = \sqrt{f_y/\sigma_{cr,s}}$

Step 3 (alternative):

If $\chi_d < 1,0$ it may be refined iteratively, starting the iteration with modified values of ρ obtained with $\sigma_{com,Ed,i} = \chi_d f_{yb}/\gamma_{M0}$ so that:
$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi_d}$

The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi_d A_s \frac{f_{yb}/\gamma_{M0}}{\sigma_{com,Ed,i}}$$

Conclusion

In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = t\, A_{s,red} / A_s$ for all the elements include in A_s.

General procedure of Effective Shape Calculation

The gross-section properties are used to calculate the internal forces and deformations.

The general procedure which combines the effective calculation of plane elements without and plane elements with stiffeners is given in EN 1993-1-3 article 5.5.2(3) and article 5.5.3.

This procedure can be written out as follows:

- Step 1: The effective width of the flanges and edge/intermediate stiffeners within the flanges are calculated based on gross section properties.

  This includes the optional iterative procedure for the edge/intermediate stiffeners.

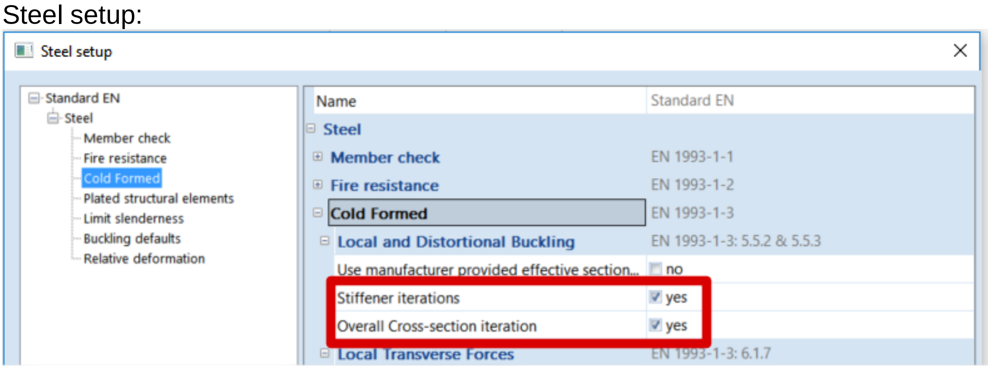
- Step 2: This partially effective shape of the previous step is used to determine the stress gradient and effective width of the web.

  This includes the optional iterative procedure for the intermediate stiffeners.

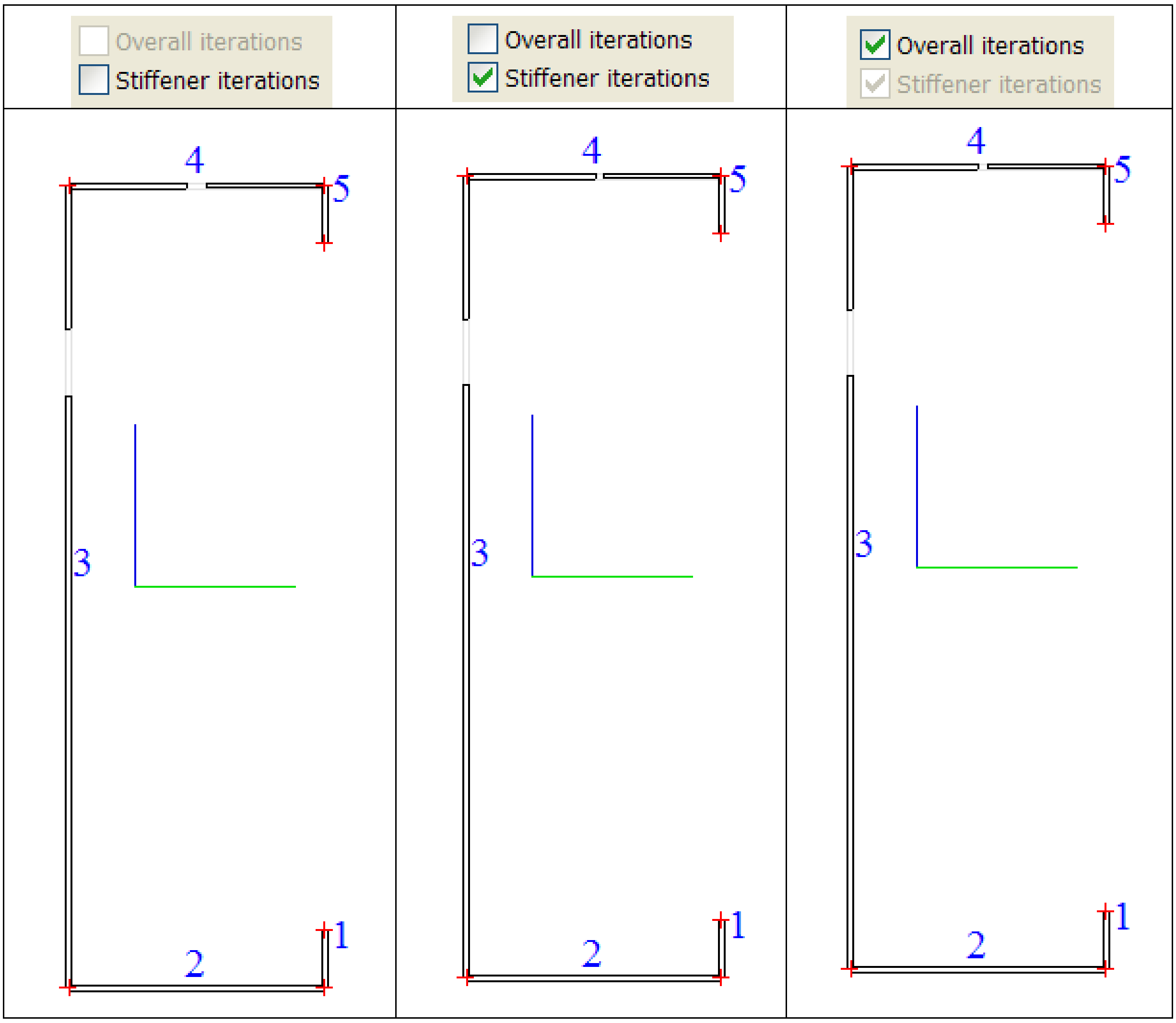
- Step 3: The end result of the previous two steps is the effective cross-section and its properties can be calculated

- Step 4: This process can now be optionally iterated using the stress ratio based on the effective cross-section instead of the gross cross-section.

Both iteration procedures (iteration of stiffeners and iteration of the full cross-section) can be set in the Steel setup:



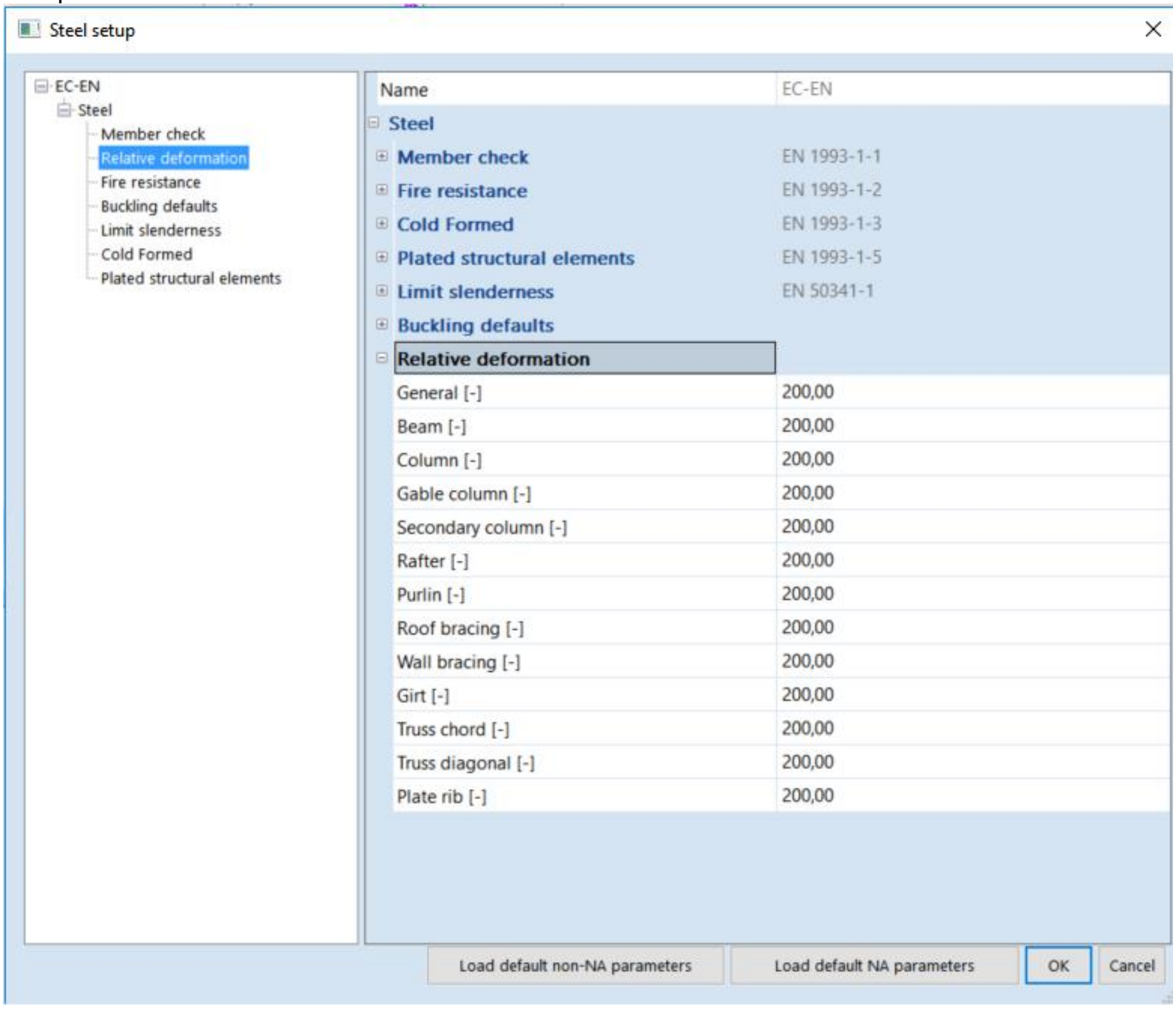
➤ **Example WS CFS 02.esa**
In this example the differences between the options “Stiffener iterations” and “Overall iterations” activated can be seen:
Look at cross-section CS2 – Positive bending around y-y:



SLS check

Relative deformations

For each beam type, limiting values for the relative deflections are set, using the menu 'Steel' > ‘Steel Setup’ > ‘Relative deformations'.



With the option 'Steel’ > ‘Relative deformation' the relative deformations can be checked. The relative deformations are given as absolute value, relative value related to the span, or as unity check related to the limit for the relative value to the span.

Example Relative deformation.esa

- Set beam type for member B196 & B112: Beam and Purlin
- Set system length for relative deformation
- Set limits for relative deformations: Beam 1/500 and Purlin 1/1000
- Relative deformation check on member B196 & B112

Relative deformation	
General [-]	200,0
Beam [-]	500,0
Column [-]	200,0
Gable column [-]	200,0
Secondary column [-]	200,0
Rafter [-]	200,0
Purlin [-]	1000,0
Roof bracing [-]	200,0
Wall bracing [-]	200,0
Girt [-]	200,0
Truss chord [-]	200,0
Truss diagonal [-]	200,0
Plate rib [-]	200,0

Relative deformation

Linear calculation, Extreme : Member, System : Principal
Selection : B112, B196
Combinations : SLS

Member	dx [m]	Case - combination	uy [mm]	Rel uy [1/xx]	uz [mm]	Rel uz [1/xx]	Check uy [-]	Check uz [-]
B112	3,000	SLS/1	**-20,0**	**1/301**	**-50,0**	**1/1081**	**3,33**	**0,93**
B112	0,000	SLS/3	**0,0**	**0**	-22,0	1/2450	**0,00**	0,41
B112	0,000	SLS/2	0,0	0	**-21,8**	**1/2474**	0,00	**0,40**
B196	5,455	SLS/2	**0,0**	**1/10000**	-1,1	1/5295	**0,00**	0,09
B196	0,545	SLS/2	**0,0**	**1/10000**	-1,1	1/5295	**0,00**	0,09
B196	3,000	SLS/3	0,0	1/10000	**-3,9**	**1/1522**	0,00	**0,33**
B196	0,000	SLS/3	0,0	0	**0,0**	**0**	0,00	**0,00**

Manual calculation uy

- B112: L = 6,0 m → limit: 6000/1000 = 6 mm
  uy = 20 mm → Rel uy = 20/6000 = 1/301
  Check = 20mm/6mm = 3,33

- B196: L = 6,0 m → limit: 6000/500 = 12 mm
  uy = 0mm → Rel uy = 1/10000 (= default value for 0 results)
  Check = 0mm/12mm = 0,00

Manual calculation uz

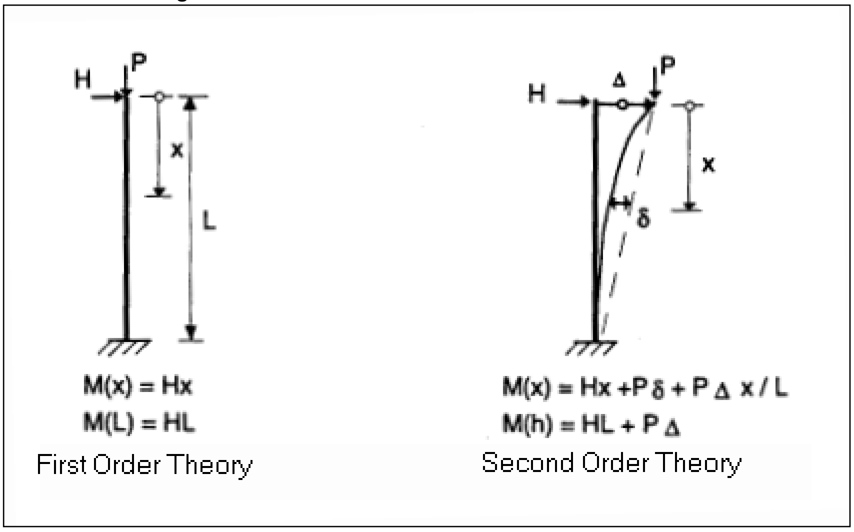
- B112: L = 9 beams x 6,0 m = 54m → limit: 54000/1000 = 54 mm
  uz = 50,0 mm → Rel uz = 50,0/54000 = 1/1081
  Check = 50mm/54mm = 0,93

- B196: L = 6,0m → limit: 6000/500 = 12 mm
  uz = 3,9 mm → Rel uz = 3,9/6000 = 1/1522
  Check: 3,9mm/12mm = 0,33

Structural Analysis

Global analysis

Global analysis aims at determining the distribution of the internal forces and moments and the corresponding displacements in a structure subjected to a specified loading.
The first important distinction that can be made between the methods of analysis is the one that separates elastic and plastic methods. Plastic analysis is subjected to some restrictions. Another important distinction is between the methods, which make allowance for, and those, which neglect the effects of the actual, displaced configuration of the structure. They are referred to respectively as second-order theory and first-order theory based methods. The second-order theory can be adopted in all cases, while first-order theory may be used only when the displacement effects on the structural behavior are negligible.
The second-order effects are made up of a local or member second-order effects, referred to as the P-δ effect, and a global second-order effect, referred to as the P-Δ effect.



For EC3 the global analysis schema for elastic analysis are given in the next chapters.

EC3

<u>Non-sway/sway frame (EC-EN)</u>: a frame may be classified as non-sway if its response to in-plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes.

According to the Eurocode3 EC-EN, 1st Order analysis may be used for a structure, if the increase of the relevant internal forces or moments or any other change of structural behaviour caused by deformations can be neglected. This condition may be assumed to be fulfilled, if the following criterion is satisfied:

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

With:

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

If α_{cr} has a value lower then 10, a 2nd Order calculation needs to be executed. Depending on the type of analysis, both Global and Local imperfections need to be considered.
Eurocode prescribes that 2nd Order effects and imperfections may be accounted for both by the global analysis or partially by the global analysis and partially through individual stability checks of members.

Global frame imperfection φ

The global frame imperfection is given by:

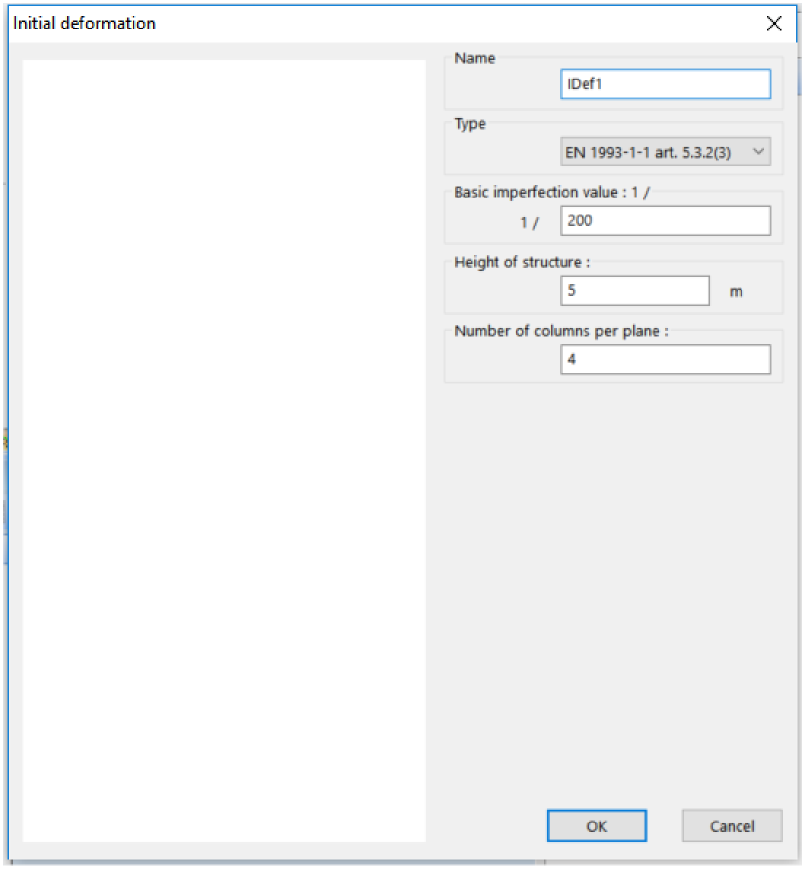
$$\varphi = \frac{1}{200} \cdot \alpha_h \cdot \alpha_m$$

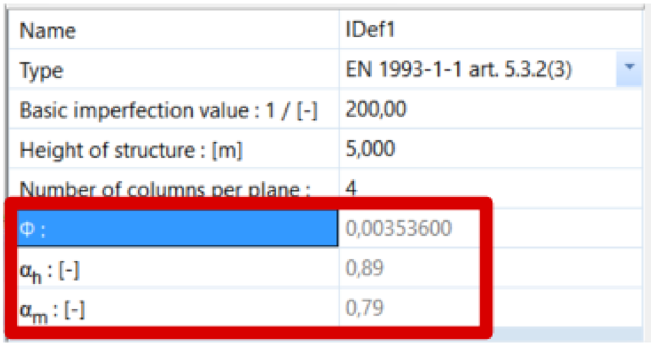
$$\alpha_h = \frac{2}{\sqrt{h}} \qquad \text{but } \frac{2}{3} \leq \alpha_h \leq 1{,}0$$

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

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

This can be calculated automatically by SCIA Engineer





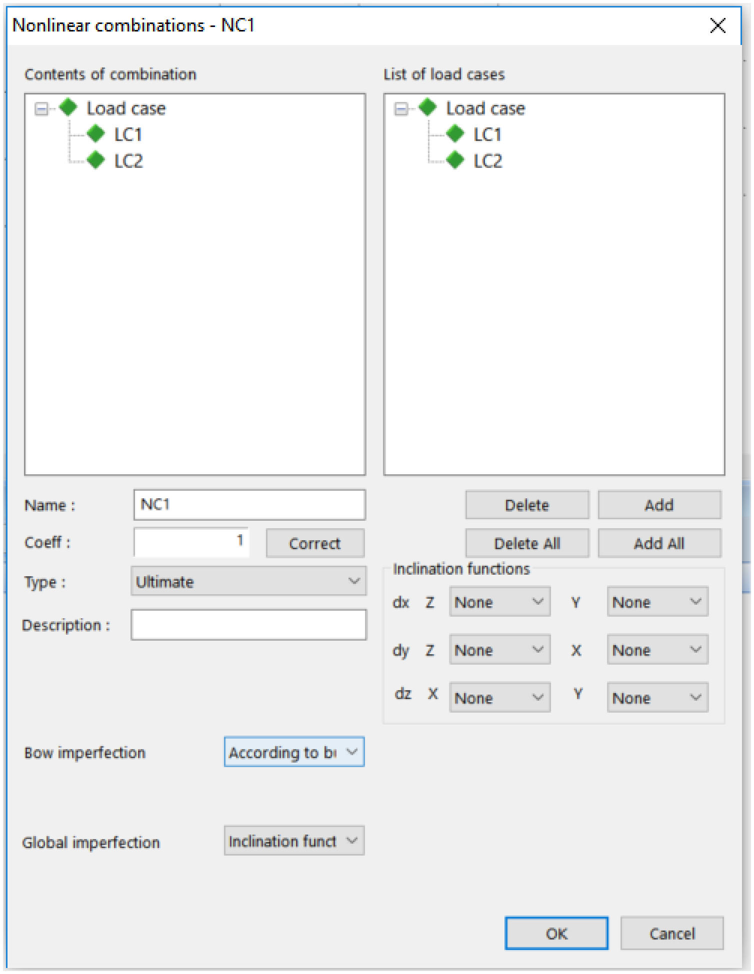
Name	IDef1
Type	EN 1993-1-1 art. 5.3.2(3)
Basic imperfection value : 1 / [-]	200,00
Height of structure : [m]	5,000
Number of columns per plane :	4
Φ :	0,00353600
α_h : [-]	0,89
α_m : [-]	0,79

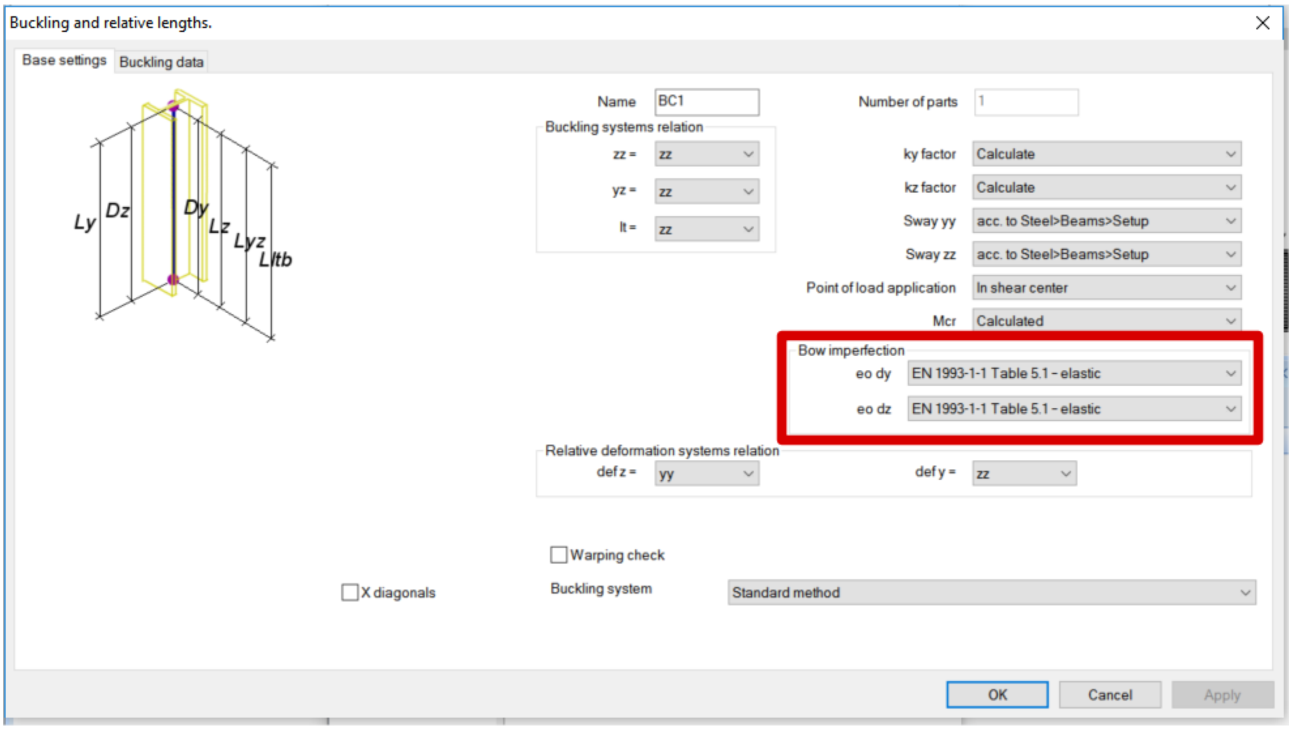
<u>Initial bow imperfection e_0</u>
The initial bow imperfection is given by:

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

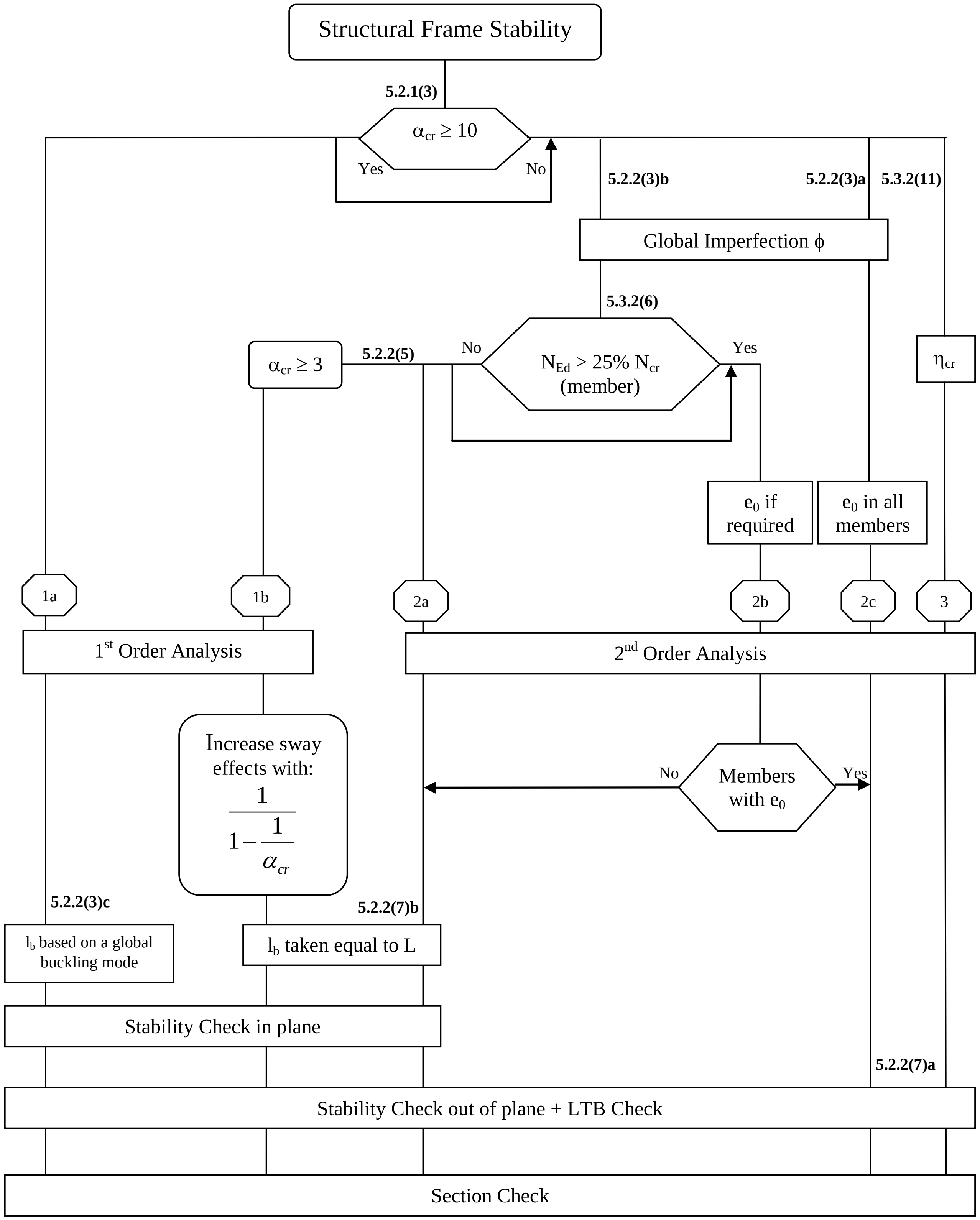
Where L is the member length.
The bow imperfection has to be applied when the normal force N_{Ed} in a member is higher than 25% of the member's critical buckling load N_{cr}.

SCIA Engineer can calculate the bow imperfection according to the code automatically for all needed members:





The buckling curve used for calculation of the imperfection is the curve inputted in the cross-section manager. For standard sections, the curve according to the code is automatically used, for non-standard sections the user needs to input the buckling curve manually.
The general procedure for the new EC-EN is shown in the next diagram.



With: η_{cr} Elastic critical buckling mode.
L Member system length
l_b Buckling Length

Path 1a specifies the so called Equivalent Column Method. In step 1b and 2a "l_b may be taken equal to L". This is according to EC-EN so the user does not have to calculate the buckling factor =1. In further analysis a buckling factor smaller than 1 may be justified.

- **Example WS CFS Hall.esa**

Consider beam B112

Check of steel

Linear calculation, Extreme : Member
Selection : B112
Combinations : ULS

EN 1993-1-3 Cold Formed Code Check
National annex: Standard EN

Member B112	**6,000 m**	**Cold formed Sigma section**	**S 235**	**ULS/1**	**5,16 -**

Basic data EC3 : EN 1993	
partial safety factor Gamma M0 for resistance of cross-sections	1.00
partial safety factor Gamma M1 for resistance to instability	1.00
partial safety factor Gamma M2 for resistance of net sections	1.25

Material data		
yield strength fy	235.0	MPa
tension strength fu	360.0	MPa
fabrication	cold formed	

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

The critical check is on position 3.000 m

Internal forces	Calculated	Unit
N,Ed	3,00	kN
Vy,Ed	0,00	kN
Vz,Ed	-0,18	kN
T,Ed	0,00	kNm
My,Ed	8,20	kNm
Mz,Ed	0,79	kNm

Effective section My+
Effective width calculation
According to EN 1993-1-3 article 5.5.2, 5.5.3 & EN 1993-1-5 article 4.4

Element	bp [mm]	f1 [N/mm ²]	f2 [N/mm ²]	Psi [-]	k,sigma [-]	Lambda,p [-]	Lambda,p,red [-]	Rho [-]	beff [mm]	be1 [mm]	be2 [mm]
1	18	-185.881	-227.096								
3	60	-229.769	-230.496								
5	32	-150.224	-226.329								
7	28	-121.268	-148.555								
9	104	124.215	-119.595	-0.96	22.93	0.39		1.00	53	21	32
11	28	153.804	125.899	0.82	0.00	0.00		1.00	28		
13	32	231.594	155.489	0.67	4.76	0.27		1.00	32	15	17
15	60	235.000	234.274	1.00	4.01	0.54	0.52	1.00	60	30	30
17	18	230.833	189.618	0.82	0.50	0.45	0.43	1.00	18		

Stiffener calculation
According EN 1993-1-3 article 5.5.3

Element	As [m ²]	Is [m ⁴]	b1 [mm]	b2 [mm]	hw [mm]	kf [-]	K [N/mm ²]	Sigma,cr [N/mm ²]	Lambda,d [-]	Chi,d [-]	As,red [m ²]
11	1.2721e-04	1.5383e-08	109	43	0	0.00	3.039	1557.754	0.39	1.00	1.2721e-04
17	9.4895e-05	3.1731e-09	53	41	232	0.00	0.549	402.949	0.76	0.92	8.7101e-05

Effective section Mz+
Effective width calculation
According to EN 1993-1-3 article 5.5.2, 5.5.3 & EN 1993-1-5 article 4.4

Element	bp [mm]	f1 [N/mm ²]	f2 [N/mm ²]	Psi [-]	k,sigma [-]	Lambda,p [-]	Lambda,p,red [-]	Rho [-]	beff [mm]	be1 [mm]	be2 [mm]
1	18	235.000	235.000	1.00	0.50	0.45	0.43	1.00	18		
3	60	226.136	-139.246	-0.62	15.39	0.28	0.26	1.00	37	15	22
5	32	-148.032	-148.032								
7	28	10.150	-145.324	-14.32	0.00	0.00		1.00	28		
9	104	12.937	12.937	1.00	4.00	0.93		0.82	85	43	43
11	28	10.150	-145.324	-14.32	0.00	0.00		1.00	28		
13	32	-148.032	-148.032								
15	60	226.136	-139.246	-0.62	15.39	0.28	0.26	1.00	37	15	22

Element	bp [mm]	f1 [N/mm ²]	f2 [N/mm ²]	Psi [-]	k,sigma [-]	Lambda,p [-]	Lambda,p,red [-]	Rho [-]	beff [mm]	be1 [mm]	be2 [mm]
17	18	235.000	235.000	1.00	0.50	0.45	0.43	1.00	18		

Stiffener calculation
According EN 1993-1-3 article 5.5.3

Element	As [m²]	Is [m⁴]	b1 [mm]	b2 [mm]	hw [mm]	kf [-]	K [N/mm ²]	Sigma,cr [N/mm ²]	Lambda,d [-]	Chi,d [-]	As,red [m²]
1	6.5102e-05	2.5477e-09	59	59	232	1.00	0.307	393.636	0.77	0.91	5.9332e-05
7	1.9638e-04	2.7114e-08	46	105	0	0.00	2.779	1281.125	0.43	1.00	1.9638e-04
11	1.9638e-04	2.7114e-08	105	46	0	0.00	2.779	1281.125	0.43	1.00	1.9638e-04
17	6.5102e-05	2.5477e-09	59	59	232	1.00	0.307	393.636	0.77	0.91	5.9332e-05

Axial tension check
According to article EN 1993-1-3: 6.1.2 and formula (6.1).

Table of values		
Ag	754	mm²
Fn,Rd	217.15	kN
Nt,Rd	177.19	kN
Unity check	0.02	-

Bending Moment Check
Bending Moment My
According to article EN 1993-1-3: 6.1.4.1 and formula (6.4)

Bending about Y axis		
Wel,y	43337	mm³
Weff,y	42159	mm³
Mcy,Rd	9.91	kNm
Unity check	0.83	-

Bending Moment Mz
According to article EN 1993-1-3: 6.1.4.1 and formula (6.4)

Bending about Z axis		
Wel,z	7368	mm³
Weff,z	6896	mm³
Mcz,Rd	1.62	kNm
Unity check	0.48	-

Biaxial Bending
According to article EN 1993-1-3: 6.1.4.1 and formula (6.7)

Bending about Z axis		
Mcy,Rd	9.91	kNm
Mcz,Rd	1.62	kNm
Unity check	**1.31**	-

Combined Tension and Bending Check
According to article EN 1993-1-3: 6.1.8 and formula (6.23), (6.24).

Table of values		
Nt,Rd	177.19	kN
Mcy,Rd,ten	10.12	kNm
Mcz,Rd,ten	2.54	kNm
Mcy,Rd,com	9.92	kNm
Mcz,Rd,com	1.62	kNm

Unity check (6.23) 0.02 + 0.81 + 0.31 = **1.14** -
Unity check (6.24) 0.83 + 0.48 - 0.02 = **1.29** -

The member does NOT satisfy the section check!

...::STABILITY CHECK::...

Lateral Torsional Buckling Check
According to article EN 1993-1-3: 6.2.4
According to article EN 1993-1-1: 6.3.2 and formula (6.55)

LTB Parameters		
Method for LTB Curve	art. 6.3.2.2	
Weff,y	42159	mm^3
Elastic critical moment Mcr	2.05	kNm
Relative slenderness Lambda,LT	2.20	
Limit slenderness Lambda,LT,0	0.40	
LTB curve	b	
Imperfection Alpha,LT	0.34	
Reduction factor Chi,LT	0.18	
Buckling resistance Mb,Rd	1.75	kNm
Unity check	**4.69**	-

Mcr Parameters		
LTB length	6.000	m
k	1.00	
kw	1.00	
C1	2.51	
C2	1.49	
C3	0.41	
Influence of load position	no influence	

Note: C Parameters according to ECCS 119 2006 / Galea 2002

Bending and Axial Tension Check
According to article EN 1993-1-3: 6.3.

Table of values		
Nt,Rd	177.19	kN
Mb,y,Rd	1.75	kNm
Mc,z,Rd,com	1.62	kNm

Unity check: **4.69+0.48-0.02** = **5.16** -

The member does NOT satisfy the stability check!

Remarks about this checks are given below:

Axial Tension

The axial tension check is executed according to EN 1993-1-3, **article 6.1.2**.

Axial Compression

The axial compression check is executed according to EN 1993-1-3, **article 6.1.3**.

The choice between formula **(6.2)** and **(6.3)** is made by comparing the gross area $\mathbf{A_g}$ from the initial shape with the effective area $\mathbf{A_{eff}}$ of the effective shape for **Compression**:

- Profile Library sections can have different gross properties compared to the initial shape since the gross properties come from certain sources and are mostly rounded off.
- For general cross-sections the gross shape can differ from the initial shape since the initial shape concerns a thin walled representation.

Bending moment

The bending moment check is executed according to EN 1993-1-3, **article 6.1.4.1**.

The choice between formula **(6.4)** and **(6.5)** is made by comparing the elastic section modulus $\mathbf{W_{el}}$ from the initial shape with the effective section modulus $\mathbf{W_{eff}}$ of the effective shape for bending:

- Profile Library sections can have different gross properties compared to the initial shape since the gross properties come from certain sources and are mostly rounded off.
- For general cross-sections the gross shape can differ from the initial shape since the initial shape concerns a thin walled representation.

This check (formula **(6.5)**) is only applied in the following cases (EN 1993-1-3 **article 6.1.4.1(2)**

- There is only single bending My or Mz

- There is no torsion, no Torsional (-Flexural) Buckling), no Lateral Torsional Buckling and no distortional buckling
- The angle between the web and flange exceeds 60°.

Otherwise this formula has to be replaced by formula **(6.6)**.

Articles **6.1.4.2** and **6.1.4.3** from EN 1993-1-3 concerning the plastic reserve of the tension flange and the effects of shear lag are not supported.

Shear force

The shear force check is executed according to EN 1993-1-3, **article 6.1.5**.
The shear resistance is calculated for each 'web' element separately and the cross-section resistance is taken as the sum of these resistances.

Formula **(6.8)** is rewritten as follows for both directions:

$$V_{b,Rd,y} = \sum_i V_{b,Rd,yi} = \sum_i \frac{l_{c,i} * t_i * f_{bv,i}}{\gamma_{M0}} cos^2(\alpha_i)$$

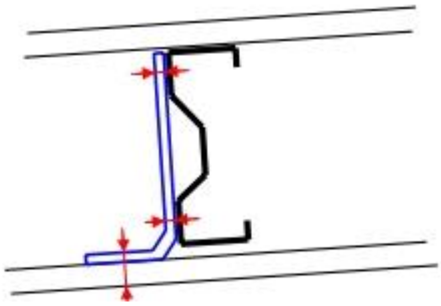
$$V_{b,Rd,z} = \sum_i V_{b,Rd,zi} = \sum_i \frac{l_{c,i} * t_i * f_{bv,i}}{\gamma_{M0}} sin^2(\alpha_i)$$

With:

α_i Angle of element **i** related to the principal axis y-y axis

$l_{c,i}$ Centreline length of element **i**

Note
Different formulas are given for web with and without longitudinal stiffeners (see EN 1993-1-3 formula (6.10a) and (6.10b)).
By default the shear check is executed "without stiffening at the support".
In case Local transverse forces data are inputted which have the checkbox "Local Transverse Forces" not activated, the Shear check in those sections is executed "with stiffening at the support".



Torsional moment

The combined stress Check including torsion and warping is executed according to EN 1993-1-3, **article 6.1.6**.
The average yield strength is $\mathbf{f_{ya}}$ in all three formulas **(6.11a)**, **(6.11b)**, **(6.11c)** will only be used in case for all three force components separately (N, M_y, M_z) the average yield strength may be used ($A_{eff} = A_g$; $W_{eff,y} = W_{el,y}$; $W_{eff,z} = W_{el,z}$).

Local transverse forces

General procedure

The local transverse forces check is executed according to EN 1993-1-3 **art 6.1.7** and following.

The check is executed on the positions where there is a jump in the **Vz** shear force diagram.

Remarks:

- The shear force diagram of both the actual member as well as adjacent members is evaluated. Adjacent members are defined as members which are in the same buckling system.
- The Flange Condition depends on the definition of the initial shape. In case there is an element with reinforcement type **ROU** or **DEF** the setting is taken as “Stiffened”.
- The distances for One-flange/Two-flange and End/Interior are evaluated taking into account adjacent members. Adjacent members are defined as members which are in the same buckling system.
- In case the cross-section has multiple webs, for determining the load condition the maximal web height is used.
- As opposed to EN 1993-1-3 **art.6.1.7.2(4)**, the exact inputted bearing length **ss** will be used at all times i.e. the simplification of using the minimal length for both opposing loads is not supported.

Cross-sections with a single unstiffened web

As indicated on EN 1993-1-3 **Figure 6.6**, the local transverse force resistance is taken relative to the support, not according to the principal z-axis. Therefore **FEd**, is determined according to the LCS axis system and not according to the principal axis system!

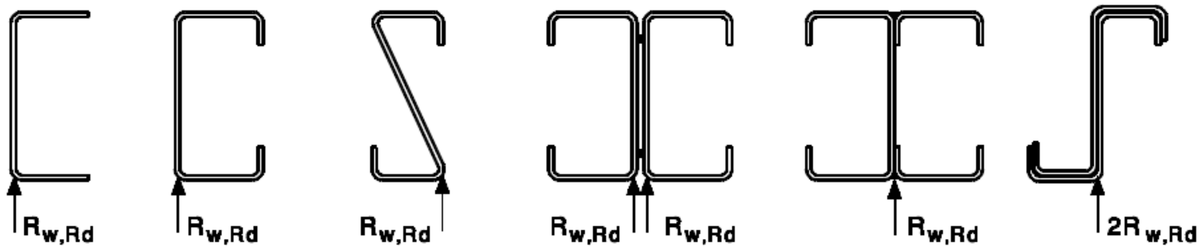


Figure 6.6: Examples of cross-sections with a single web

This paragraph specifies the general procedure to determine the local transverse web resistance which is applied for any type of cross-section except for **FC 115** (Cold formed Omega).

<u>Step 1</u>

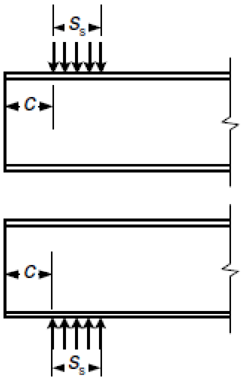
In a first step the web height $\mathbf{h_w}$ is determined for each “web” element:

- Only elements of type **I** are accounted for. In addition elements with stiffener types **RUO** and **DEF** are not accounted for.
- For each of those elements **i** the centreline length $\mathbf{l_{c,i}}$ is read from the Initial shape
- For each of those elements **i** the angle $\boldsymbol{\phi_i}$ is determined as the angle of the element relative to the horizontal axis (based on **Figure 6.6**). In addition, only elements with an angle $\boldsymbol{\phi_i} \geq$ **45°** are accounted for.
- The web height for each element **i** is calculated as: $h_{w,i} = l_{c,i} * sin\phi_i$

In case none of the cross-section elements fulfill the above conditions, the local transverse forces check is not supported for the cross-section.

<u>Step 2</u>

When $\mathbf{h_{w,i}}$ is determined, the local transverse resistance $\mathbf{R_{w,Rd,i}}$ for each of those elements is determined based on EN 1993-1-3 **art.6.1.7.2 – Table 6.7** with coefficients k_1 to k_5 determined in EN 1993-1-3 **article 6.1.7.2(3)**.



a) For a single local load or support reaction

i) $c \leq 1{,}5\, h_w$ clear from a free end:

- for a cross-section with stiffened flanges:

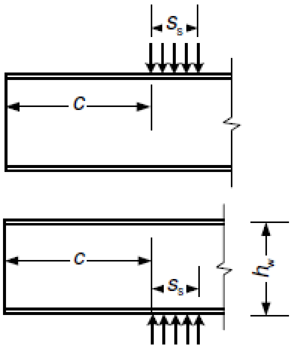
$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[9{,}04 - \frac{h_w/t}{60}\right]\left[1 + 0{,}01\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15a)$$

- for a cross-section with unstiffened flanges:
- if $s_s/t \leq 60$:

$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[5{,}92 - \frac{h_w/t}{132}\right]\left[1 + 0{,}01\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15b)$$

- if $s_s/t > 60$:

$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[5{,}92 - \frac{h_w/t}{132}\right]\left[0{,}71 + 0{,}015\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15c)$$



ii) $c > 1{,}5\, h_w$ clear from a free end:

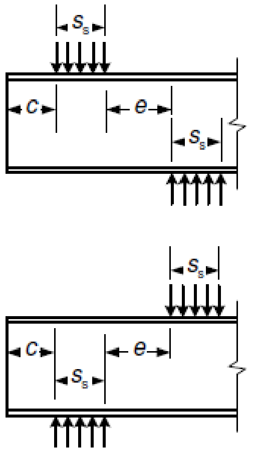
- if $s_s/t \leq 60$:

$$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[14{,}7 - \frac{h_w/t}{49{,}5}\right]\left[1 + 0{,}007\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15d)$$

- if $s_s/t > 60$:

$$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[14{,}7 - \frac{h_w/t}{49{,}5}\right]\left[0{,}75 + 0{,}011\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15e)$$

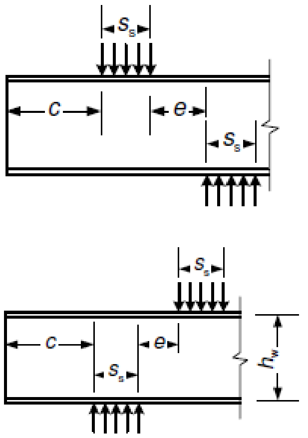
Figure 6.7a) : Local loads and supports — cross-sections with a single web



b) For two opposing local transverse forces closer together than $1{,}5\ h_w$:

i) $c \leq 1{,}5\ h_w$ clear from a free end:

$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[6{,}66 - \frac{h_w/t}{64}\right]\left[1 + 0{,}01\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15f)$$



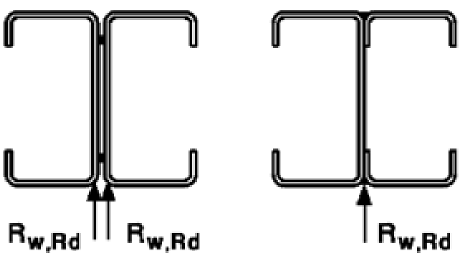
ii) $c > 1{,}5\ h_w$ clear from a free end:

$$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[21{,}0 - \frac{h_w/t}{16{,}3}\right]\left[1 + 0{,}0013\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \qquad (6.15g)$$

Figure 6.7b): Local loads and supports — cross-sections with a single web

The final cross-section resistance is taken as the sum of the individual element resistances.

In case **Web rotation prevented** was set using **Local Transverse Forces data** instead of EN 1993-1-3 **Figure 6.7a & 6.7b** the formulas given in EN 1993-1-3 **art. 6.1.7.2(4)** are used.
Example of a prevented web rotation:



Omega sections

Specifically for **FC 115** (Cold formed Omega) cross-sections the special procedure for sections with two or more unstiffened webs is applied. The local transverse resistance $\mathbf{R_{w,Rd,i}}$ for each of those webs is determined according to EN 1993-1-3 **art. 6.1.7.3**.

Other cross-sections with two or more unstiffened webs will always be calculated according to the General Procedure, not this special procedure.

Stiffened webs

This paragraph outlines the special procedure in case of stiffened webs according to EN 1993-1-3 **art. 6.1.7.4**.

This method is used only in case there are one or more elements with stiffener type **RI**.

The procedure consists of four steps.

Step 1: Creating "composed" webs

In a first step, "composed" webs are created using the same procedure as outlined in Sections with Internal stiffeners.
This includes the determination of the centreline length **lc,i** of those "composed" webs.

Step 2: Evaluation of „composed" webs

The special procedure outlined in EN 1993-1-3 **art. 6.1.7.4** is only valid under certain conditions.

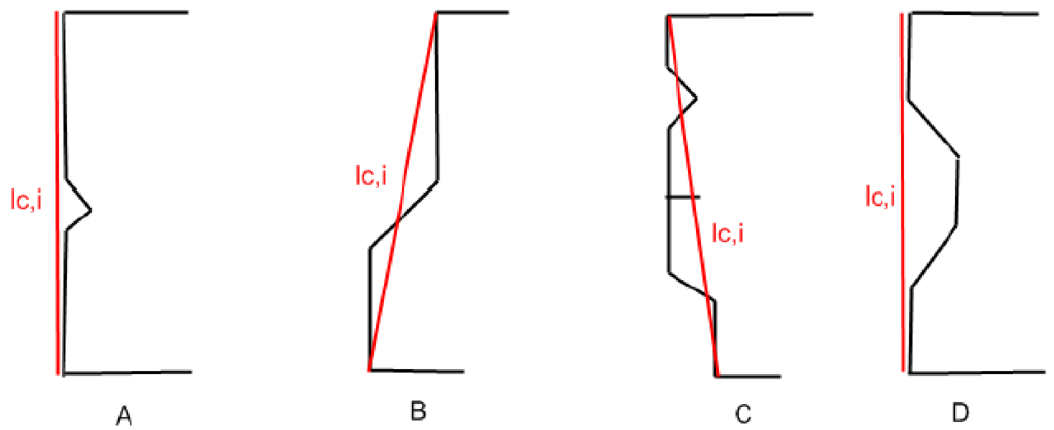
Therefore, each "composed" web is evaluated to see if it meets the following requirements:

- There is one or more elements with stiffener type **RI**
- Each **RI** element should have element type **I** (i.e. it is at both sides connected to other elements signifying it's a fold instead of a stiffener).
- Elements connected to this **RI** element should not have stiffener type **RI**. This implies that the procedure is not applied in case of neighbouring stiffener elements i.e. elements forming "one" big stiffener.

Composed webs which do **NOT meet these requirements** are further evaluated in **step 3**.

Composed webs which **meet all requirements** are further evaluated in **step 4**.

Examples of cross sections with composed webs



- Section **A** contains two **RI** stiffeners which are connected. The web thus does not meet the requirements (calculated as described in **step 3**).

- Section **B** contains a single **RI** stiffener which meets all the requirements. This stiffener is thus a „true" two fold stiffener so the special article applies (calculated as described in **step 4**).

- Section **C** contains several **RI** stiffeners however not all match the requirements (one is an outstand stiffener, others are connected etc). The web thus does not meet the requirements (calculated as described in **step 3**).

- Section **D** has a composed web which contains two **RI** stiffeners. Both meet all the requirements and are thus „true‟ two fold stiffeners (calculated as described in **step 4**).

Step 3: Composed webs witch do NOT meet the requirements

For composed webs which do not meet the requirements, the special article is not valid.
The local transverse force resistance of these webs will be determined according to the procedure for cross-sections with a single unstiffened web.

In this case, the centre line length $\mathbf{l_{c,i}}$ of the composed web is used in the determination of $\mathbf{h_w}$.
The angle ϕ_i is determined as the angle of the centre line length relative to the horizontal axis.

Step 4: Composed webs which meet all requirements

For composed webs which meet all requirements, the special procedure outlined in EN 1993-1-3 **article 6.1.7.4** is applied.

The "system line" of this web is taken as the centre line length $\mathbf{l_{c,i}}$.

The eccentricity **e** is determined at each end of an **RI** within the „composed‟ web. Eccentricity $\mathbf{e_{min}}$ and $\mathbf{e_{max}}$ are then taken as the min and max value for the considered composed web.

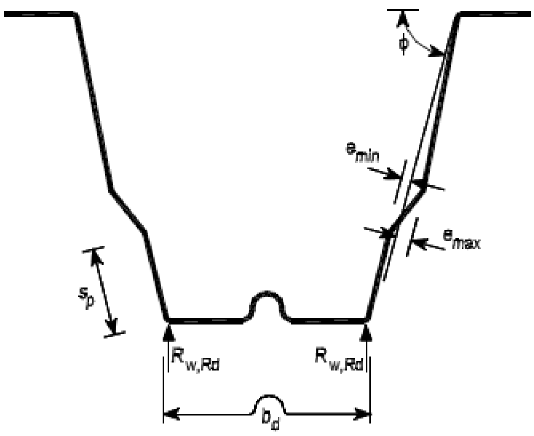


Figure 6.10: Stiffened webs

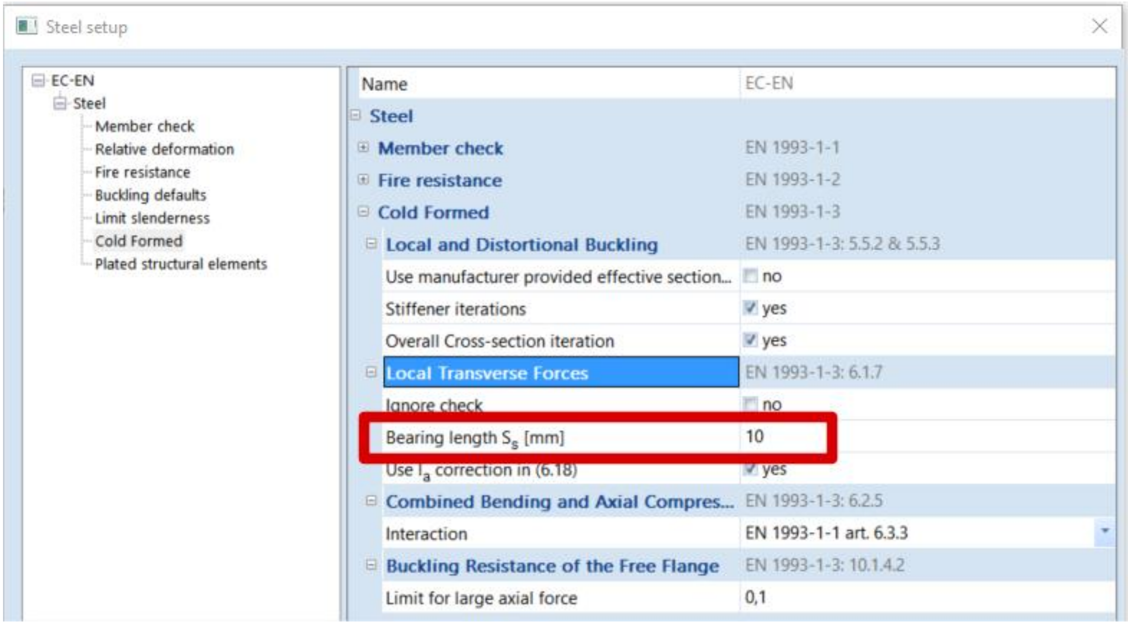
The article is applied in case the following limit is fulfilled:

$$2 < \frac{e_{max}}{t} < 12$$

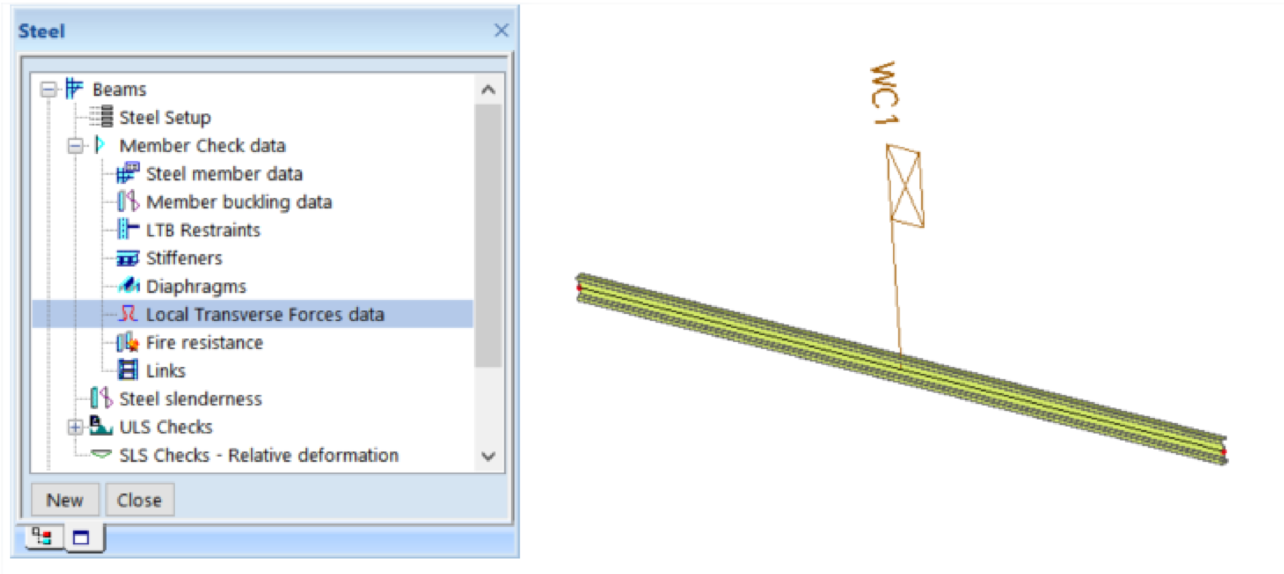
In case this limit is not fulfilled, the special article is not applied and the composed web is considered as a web which does not meet all requirements. For such a web the procedure outlined in **step 3** is applied.

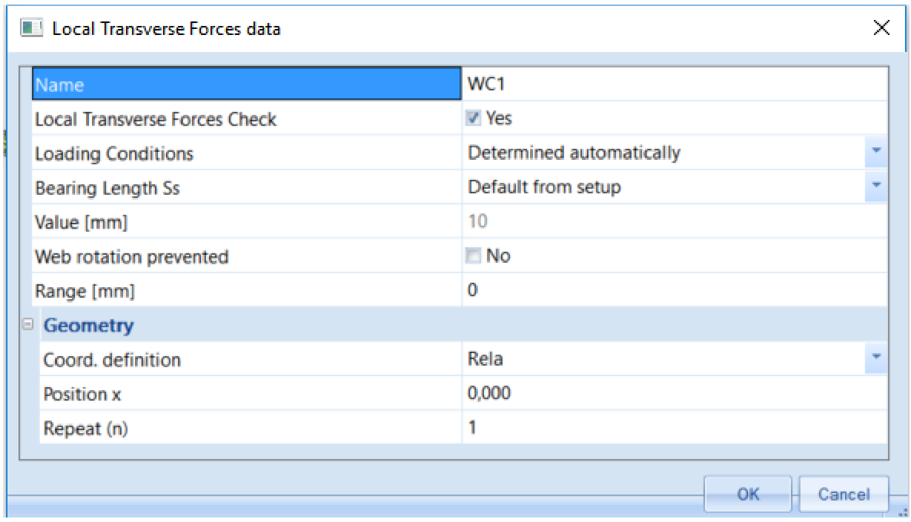
Local Transverse Force data in SCIA Engineer

In SCIA Engineer a point force is inputted as a point, but in the calculation of the check for the Local Transverse Force, a bearing length Ss will be used. Default this value is inputted as 10mm in SCIA Engineer. The default value can be adapted in "Steel > Beams > Steel Setup > Cold Formed":



It is also possible to change this bearing length for one beam only or change the default properties for this beam manually with the option “Steel > Beams > Member Check data > Local Transverse Force data”





➤ **Example WS CFS Hall.esa**

Consider beam B122 and look at the detailed output:

Local Transverse Forces Check
According to article EN 1993-1-3: 6.1.7.2, 6.1.7.4 and formula (6.15d)

Table of values		
Flange condition	Stiffened	
Loading condition	Interior one-flange (IOF)	
Web rotation	Not prevented	
Inside bend radius r	4	mm
Bearing length Ss	10	mm
k	1.03	
k1	0.99	
k4	0.99	

Element	lc[mm]	Phi [deg]	hw [mm]	t [mm]	k2	k3	k5	Rw,Rd,i [kN]
4-5-6-7-8-9-10-11-12-13-14	198	90.00	198	2	0.84	1.00	0.94	11.02

Element	emin[mm]	emax [mm]	bd [mm]	sp [mm]	Kappa,a,s
4-5-6-7-8-9-10-11-12-13-14	0	27	63	32	0.77

Note: The stiffened web consisting of elements 4-5-6-7-8-9-10-11-12-13-14 does not satisfy the condition of formula (6.21). Therefore article 6.1.7.4 is not applied.

Table of values		
Load/Reaction FEd	-35.00	kN
Rw,Rd	11.02	kN
Unity check	**3.18**	-

Combined tension and Bending

The Combined Tension and Bending check is executed according to EN 1993-1-3, **article 6.1.8**.

Combined Compression and Bending

The Combined Compression and Bending check is executed according to EN 1993-1-3, **article 6.1.9**.

Additional moments due to the shift in neutral axis are calculated at the beginning of the check and added to the internal forces. This ensures specific bending checks are executed also in case there is no initial moment but only an additional moment.

Combined shear force, axial force and bending moment

The Combined Compression and Bending check is executed according to EN 1993-1-3, **article 6.1.10**.

Formula **(6.27)** is rewritten as follows for both directions:

Shear V_y

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} + \left(1 - \frac{M_{f,Ed}}{M_{z,pl,Rd}}\right)\left(\frac{2V_{y,Ed}}{V_{y,b,Ed}} - 1\right)^2 \leq 1$$

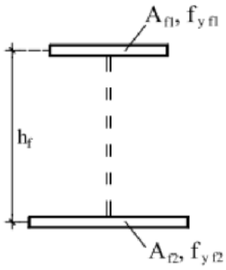
Shear V_z

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \left(1 - \frac{M_{f,Ed}}{M_{y,pl,Rd}}\right)\left(\frac{2V_{z,Ed}}{V_{z,b,Ed}} - 1\right)^2 \leq 1$$

$M_{f,Ed}$ is the design moment resistance of the cross-section consisting of the effective area of flanges only (see EN 1993-1-5)
$\boldsymbol{M_{f,Ed}}$ is taken as **zero** in case of $\mathbf{V_y}$. In case of weak axis bending the “web” becomes a “flange”. Since there is only a single “flange” in that case, the moment resistance of this flange is negligible. In addition, in case of more webs like in a box section EN 1993-1-5 **art. 7.1 (5)** specifies $\boldsymbol{M_{f,Ed}}$ = 0. Therefore, as a general conservative approach for $\mathbf{V_y}$ the value of $\boldsymbol{M_{f,Ed}}$ is taken as 0

Remarks:

According to [Ref.2] pp70 $\boldsymbol{M_{f,Ed}}$ is calculated as follows:



$$M_{f,Ed} = \min\left(\mathrm{h_f} \cdot \mathrm{A_{f1}} \frac{\mathrm{f_{y,f1}}}{\gamma_{\mathrm{M0}}}, \mathrm{h_f} \cdot \mathrm{A_{f2}} \frac{\mathrm{f_{y,f2}}}{\gamma_{\mathrm{M0}}}\right)$$

This is generalized in the following way:

- Only elements with element types **I**, **UO** and **SO** are accounted for
- Only elements which have an angle with the principal y-y axis which is **45°** are considered. In case there is only one or none of such element, $\boldsymbol{M_{f,Ed}}$ **= 0**.
- Of these elements, the one with the lowest $\mathbf{b_{eff}}$ is considered. The width $\mathbf{b_{eff}}$ concerns the effective with of this element, read from the effective shape for bending.
- $\mathbf{A_f = b_{eff} * t}$ with t the thickness of the considered element.
- Next only elements which have an angle with the principal y-y axis which is > **45°**are considered. In case there are no such elements, set $\boldsymbol{M_{f,Ed} = 0}$.
- Of these elements, the one with the highest value of $\mathbf{l_c * \sin(\alpha)}$ is considered, with $\mathbf{l_c}$ the centreline length of the element.
- $\mathbf{h_f = l_c * \sin(\alpha)}$
- $\boldsymbol{M_{f,Ed}}$ is now be calculated as: $M_{f,Ed} = \mathrm{h_f} \cdot \mathrm{A_f} \frac{\mathrm{f_y}}{\gamma_{\mathrm{M0}}}$

Combined bending moment and local Load or Support reaction

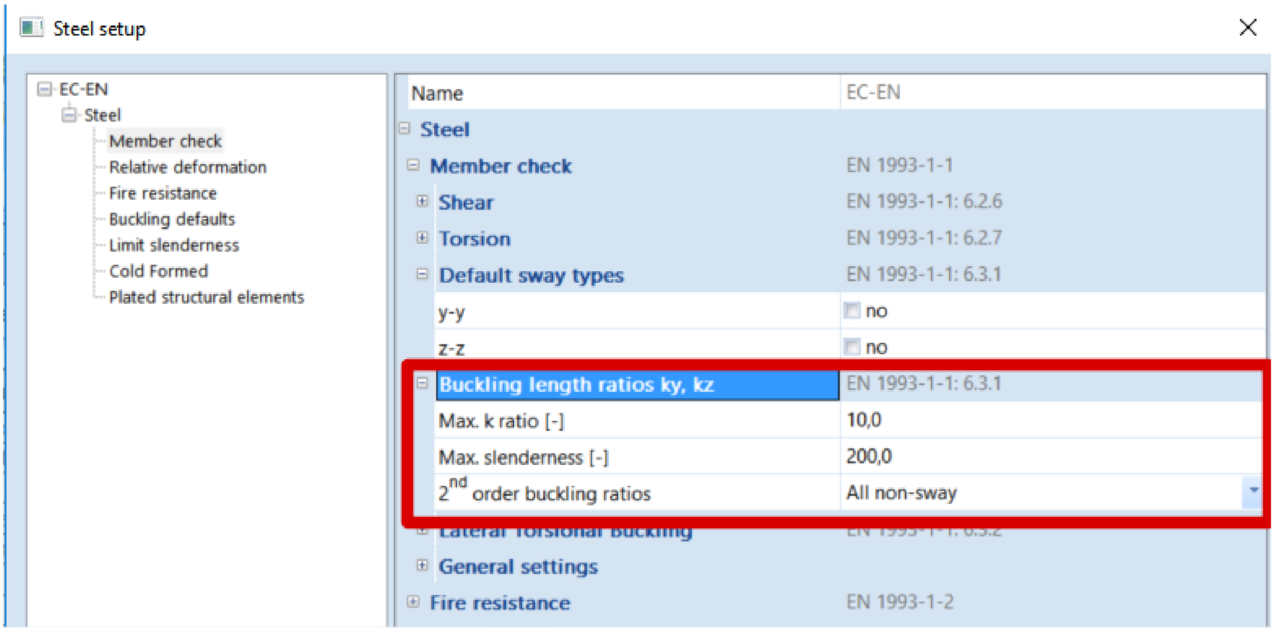
The Combined Bending moment and local Load or Support reaction is executed according to EN 1993-1-3, **article 6.1.11**.

Stability checks

General remarks

The different system lengths and sway type have to be introduced. The defaults can be overruled by the user.

During the non-linear analysis, the sway type can be set by user input, or by 'non-sway'.
'Steel > Beams > Steel Setup':



Buckling Ratio

General method

For the calculation of the buckling ratios, some approximate formulas are used. These formulas are treated in the Theoretical Background (Ref.[32]).
The following formulas are used for the buckling ratios :

- for a non sway structure :

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

- for a sway structure :

$$l/L = x\sqrt{\frac{\pi^2}{\rho_1 x} + 4}$$

with	L	the system length
	E	the modulus of Young
	I	the moment of inertia
	Ci	the stiffness in node I
	Mi	the moment in node I
	Fi	the rotation in node I

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

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

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

The values for M_i and ϕ_i are approximately determined by the internal forces and the deformations, calculated by load cases which generate deformation forms, having an affinity with the buckling form.

The following load cases are considered :

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

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

Example WS CFS 03.esa

consider B1

- L = 4000 mm
- set as sway
- E = 210000 N/mm²
- Iy = 22340000 mm⁴

- in node N1 :
  - fiy = 42138,4 mrad
  - My = 64768,2 kN
  - Ci = 1537,3 kNm/rad = 1,537 x 10^9 Nmm/rad

- in node N2 for LC1:
  - fiy = 32348,4 mrad
  - My = 15469,39 kN
  - Ci = 478,2 kNm/rad = 4,78 x 10^8 Nmm/rad

- ρ_1 = 1,31
- ρ_2 = 0,41
- x = 0,71

⇨ **buckling ratio = 2,71**

Ncr = $\pi^2 EI / L^2$ = 210000 x 162700000 / $(2.71 \times 4000)^2$ = **393,14 kN**

Steel slenderness

Linear calculation

Member	CS Name	Part	Sway y Sway z	Ly [m] Lz [m]	ky [-] kz [-]	ly [m] lz [m]	Lam y [-] Lam z [-]	lyz [m]	l LTB [m]
B1	CS1	1	Yes	4,000	2,64	10,577	149,78	4,000	4,000
			No	4,000	1,00	4,000	98,49		

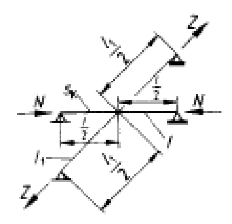
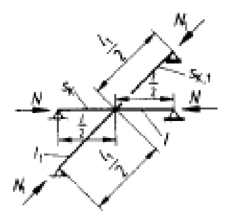
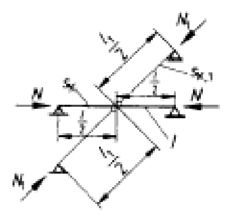
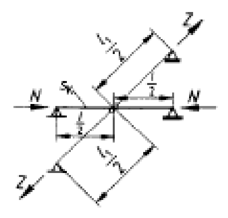
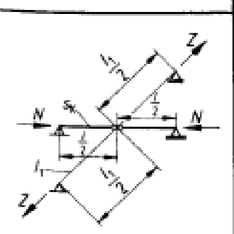
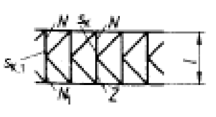
Stability calculation for LC3: a load of 1 kN on the column

Critical load coefficients

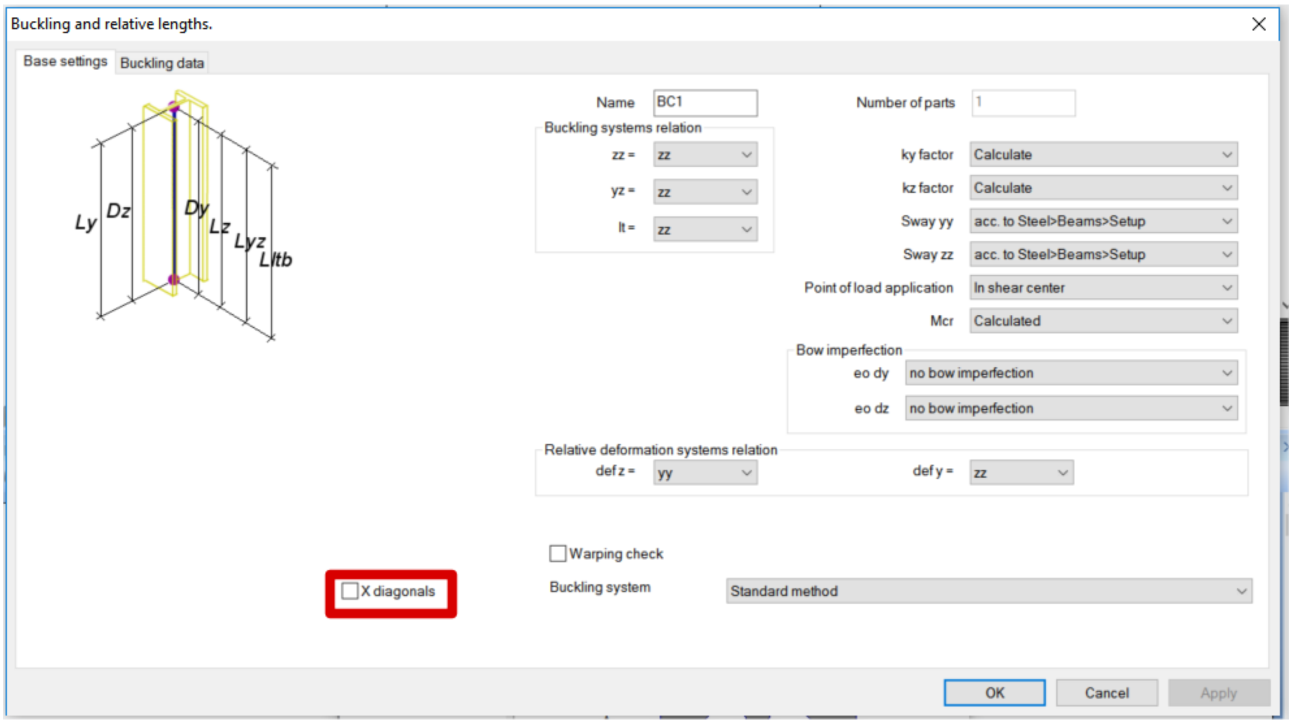
N -	f []
Stability combination : S1	
1	425,84

Crossing diagonals

When the option ‘crossing diagonal’ is selected, the buckling length perpendicular to the diagonal plane, is calculated according to DIN18800 Teil 2, table 15. This means that the buckling length s_K is dependent on the load distribution in the element, and it is not a purely geometrical data.

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

with	s_K	buckling length
	l	member length
	l_1	length of supporting diagonal
	I	moment of inertia (in the buckling plane) of the member
	I_1	moment of inertia (in the buckling plane) of the supporting diagonal
	N	compression force in member
	N_1	compression force in supporting diagonal
	Z	tension force in supporting diagonal
	E	elastic modulus

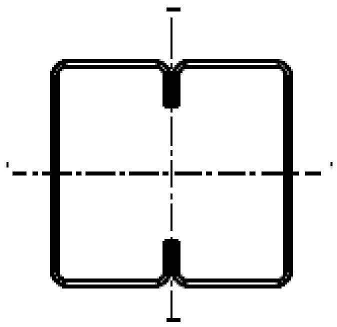
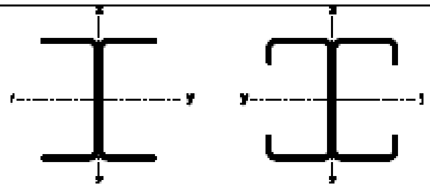
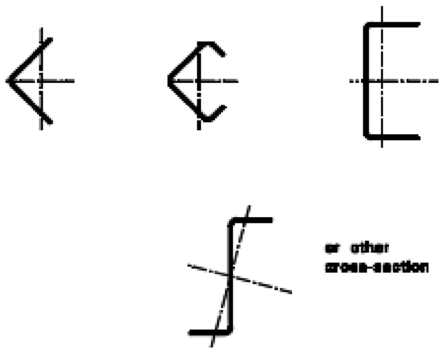


When using cross-links, this option is automatically activated. The user must verify if this is wanted or not. For example, when modelling purlins and rafters using cross-links, the option crossing diagonals may not be activated.

Flexural Buckling

The Flexural Buckling Check is executed according to EN 1993-1-3, **article 6.2.2** and EN 1993-1-1 **article 6.3.1**.

Table 6.3: Appropriate buckling curve for various types of cross-section

Type of cross-section		Buckling about axis	Buckling curve
	if f_{yb} is used	Any	b
	if f_{ya} is used *)	Any	c
		y - y	a
		z - z	b
		Any	b
		Any	c
*) The average yield strength f_{ya} should not be used unless $A_{eff} = A_g$			

This has been implemented in SCIA Engineer as follows:

Form code	Description	about axis	Curve
1	I section	y-y z-z	a b
101	Asymmetric I section	y-y z-z	a b
114	Cold formed C section	any	b
116	Cold formed C-Section eaves beam	any	b
117	Cold formed C-Plus section	any	b
118	Cold formed ZED section	any	b
119	Cold formed ZED section asymmetric lips	any	b
120	Cold formed ZED section inclined lip	any	b
121	Cold formed Sigma section	any	b
122	Cold formed Sigma section stiffened	any	b
123	Cold formed Sigma-Plus section	any	b
124	Cold formed Sigma section eaves beam	any	b
125	Cold formed Sigma-Plus section eaves beam	any	b
126	Cold formed ZED section both lips inclined	any	b
	2CFCo with a = 0	y-y z-z	a b
	2CFCc with a = 0	Closed section rule 6.2.2(3)	
	2CFUo with a = 0	y-y z-z	a b
	2CFUc with a = 0	Closed section rule 6.2.2(3)	
	2CFLT with a = 0	any	c
	Any other section	any	c

All other sections fall in the “other cross-section” case of curve **c** for any axis.

Torsional (-Flexural) Buckling

The Flexural Buckling Check is executed according to EN 1993-1-3, **article 6.2.3** and EN 1993-1-1 **article 6.3.1.4**.

The buckling curve for torsional (-flexural) buckling is taken as the z-z buckling curve according to the table given in Flexural Buckling.

The value of the elastic critical load $\mathbf{N_{cr}}$ is taken as the smallest of $\mathbf{N_{cr,T}}$ (Torsional buckling) and $\mathbf{N_{cr,T}}$ (Torsional-flexural Buckling).

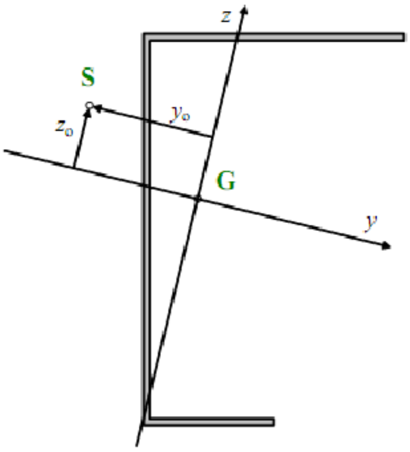
Calculation of $\mathbf{N_{cr,T}}$

The design buckling resistance $N_{b,Rd}$ for torsional or torsional-flexural buckling (according to EC3) shall be obtained using buckling curve b, and with relative slenderness given by :

$$N_{cr,T} = \frac{1}{i_0^2}\left(G\, I_t + \frac{\pi^2\, E\, I_w}{l_T^2}\right)$$

With

E	Modulus of Young
G	Shear Modulus
I_t	Torsion constant
I_w	Warping constant
l_T	Buckling length for the torsional buckling mode
y_0 and z_0	Coordinates of the shear centre with respect to the centroid
i_y	radius of gyration about the strong axis
i_z	radius of gyration about the weak axis



Calculation of $N_{cr,TF}$

The elastic critical load $N_{cr,TF}$ for torsional buckling is calculated according to Ref.[3].
$N_{cr,TF}$ is taken as the smallest root of the following cubic equation in N:

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

With

$N_{cr,y}$	Critical axial load for flexural buckling around the y-y axis
$N_{cr,z}$	Critical axial load for flexural buckling around the z-z axis
$N_{cr,T}$	Critical axial load for torsional buckling

Lateral Torsional Buckling

The Lateral Torsional Buckling Check is executed according to EN 1993-1-3, **article 6.2.4** and EN 1993-1-1 **article 6.3.2.2**.

For standard cases, the elastic critical moment for LTB Mcr is given by the respective codes. For determining the moment factors (EC3/NEN6771 : C1, C2 - DIN18800 : ζ) for lateral torsional buckling (LTB), we use the standard tables which are defined in NEN6771 table 9.1.,10 end 11. In this code the factors are defined for standard cases. 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 is maximum at the start or at the end of the beam.
The standard moment distributions which is closest to the current moment distribution, is taken for the calculation of the factors C1 and C2.
For the other supported sections, the elastic critical moment for LTB Mcr is given by

$$Mcr = \frac{\pi^2 EI_z}{L^2}\sqrt{\frac{Iw}{I_z}+\frac{L^2GI_t}{\pi^2 EI_z}}$$

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

Haunched sections (I+Ivar, Iw+PIvar, 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.

Bending and axial compression

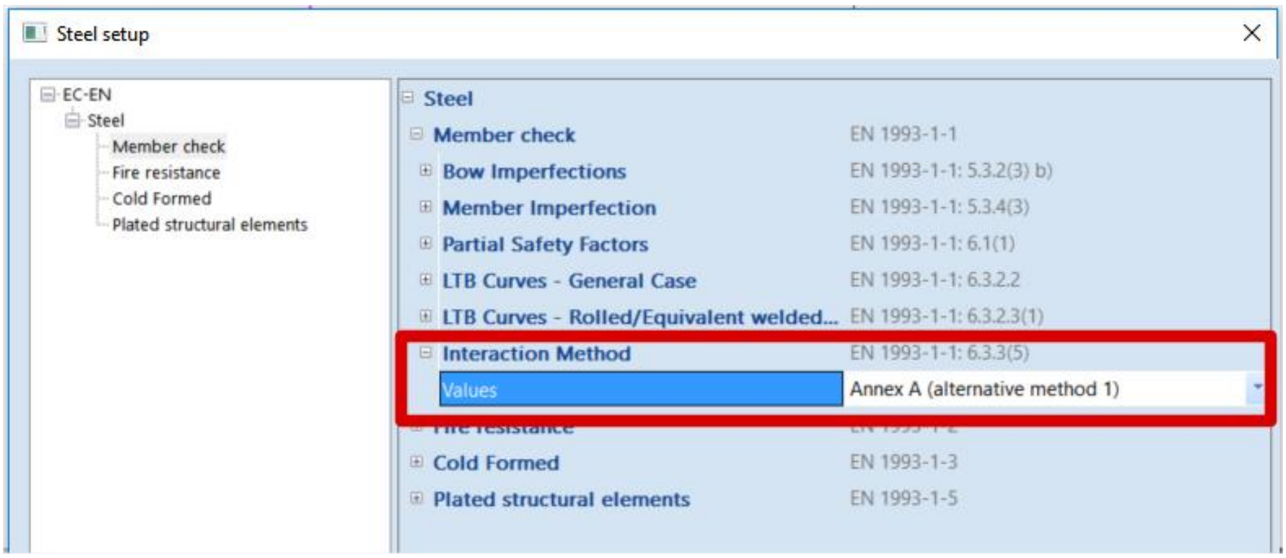
For determining the Combined Bending and Axial Compression Check is executed according to EN 1993-1-3, **article 6.2.5** EN 1993-1-3 allows two possibilities:

- Use the EN 1993-1-1 interaction according to article 6.3.3.
- Use the alternative according to EN 1993-1-3 article 6.2.5(2).

The choice between these two methods is set in “Steel > Beams > Steel Setup”:

EN 1993-1-3 formula (**6.36**) includes the strong axis bending resistance $M_{b,Rd}$. There is however no indication for a weak axis bending moment. Therefore, in case a weak axis bending moment is present, this interaction cannot be applied and the general interaction according to EN 1993-1-1 is applied.

For interaction described in EN 1993-1-1 article 6.3, two methods can be chosen following Annex A or Annex B of the EN 1993-1-1. In the National annex is described for each country which one should be used. This can also be defined in SCIA Engineer:



Bending and axial tension

The Combined Bending and Tension Check is executed according to EN 1993-1-3, **article 6.3**.

The code specifies that the same equations as for compression should be used. These interaction equations are however not fully valid in case of tension.

The purpose of the interaction check for bending and tension is to check the stresses at the compression fiber. In the AISI NAS 2007 Ref [4] code the formula given in **article C5** can be rewritten using EC-EN notations as follows:

$$\frac{M_{y,Ed}}{M_{b,y,Rd}} + \frac{M_{z,Ed}}{M_{c,z,Rd,com}} - \frac{N_{Ed}}{N_{t,Rd}} \leq 1$$

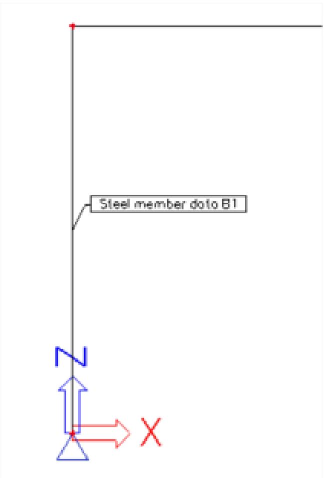
With

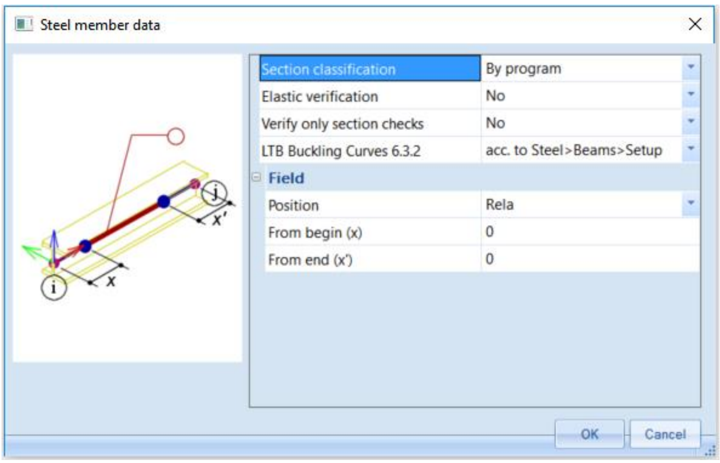
$M_{b,y,Rd}$	The Lateral Torsional Buckling resistance
$M_{c,z,Rd,com}$	The moment resistance for the compression fiber in case of M_z.
$N_{t,Rd}$	The Tension Resistance

Additional data

Steel Member data

The default values used in the Setup menu can be overruled for a specific member using Member Data.





<u>Section classification</u>
For the selected members, the section classification generated by the program, will be overruled by this user settings. This has only effect when the introduced classification is supported.

<u>Elastic check only</u>
The selected members will be classified as class 3 (EL-EL). It means no class 1, class 2 and slender section support.

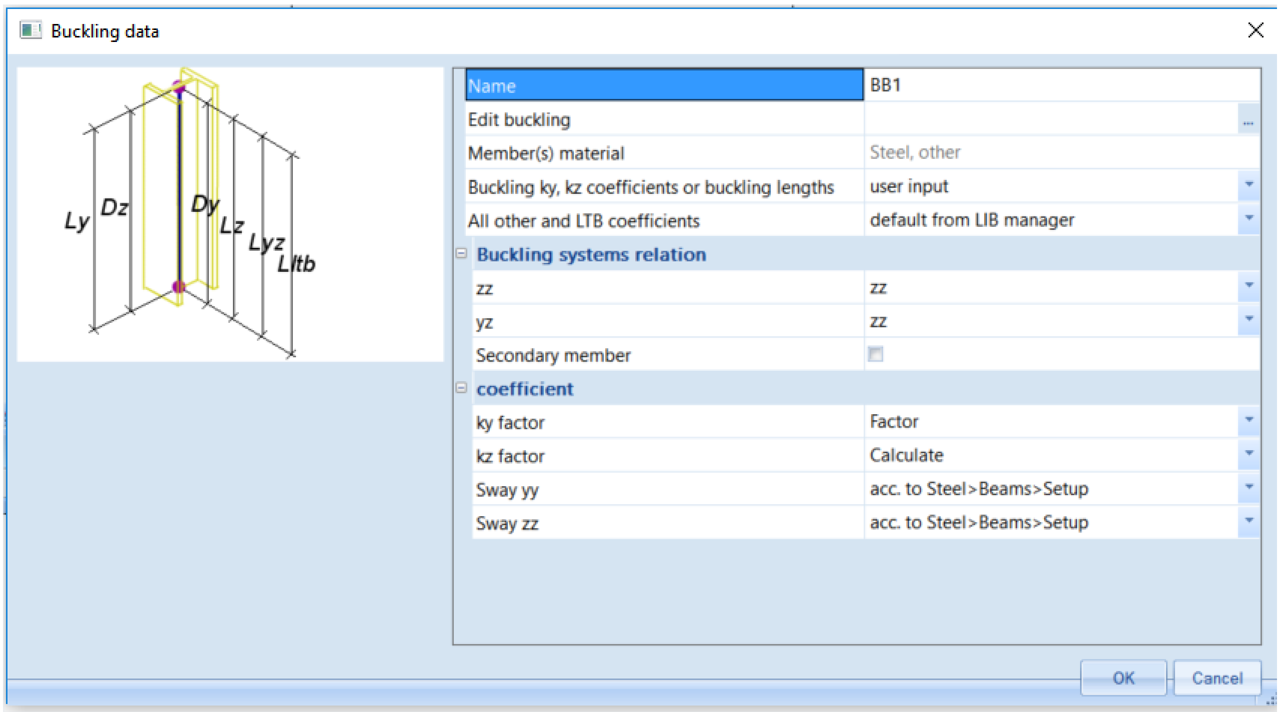
<u>Section check only</u>
For the selected members, only section check is performed. Cfr. the 'exact method' for DIN18800.

<u>LTB Buckling Curves 6.3.2</u>
For the selected members, the "general case" or the "rolled section/equivalent welded" is used for the LTB buckling curves.

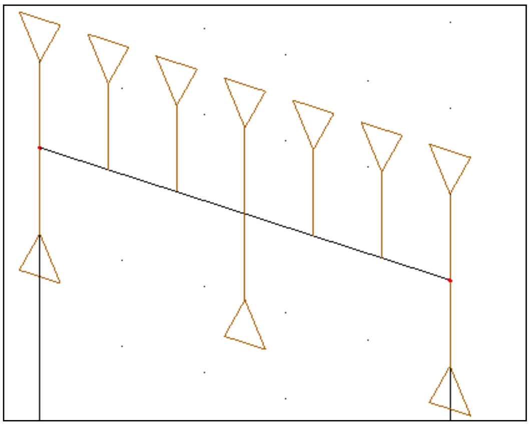
<u>Field</u>
Only the internal forces inside the field are considered during the steel code check.

Member Buckling Data

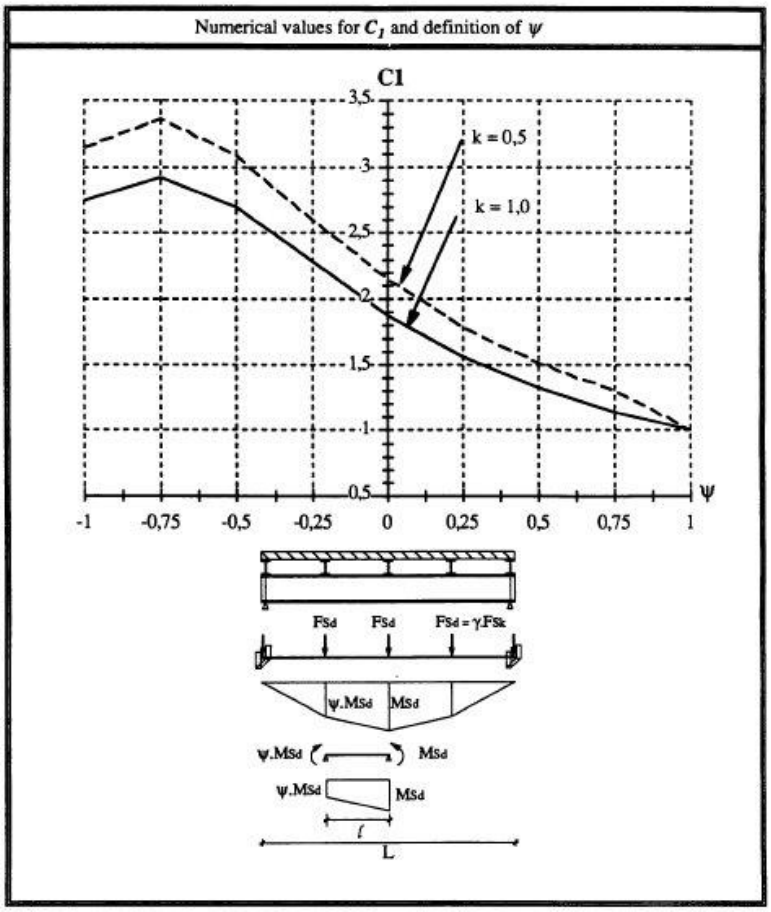
This group of parameters specifies where the member data relating to buckling are taken from. This can be taken from the Buckling Data Library. This data is displayed in the property window when a beam is selected: ‘Property’ > ‘Buckling and relative lengths’.
Using Member Buckling Data, the user can input for every beam of a buckling system a different setup of the buckling parameters.

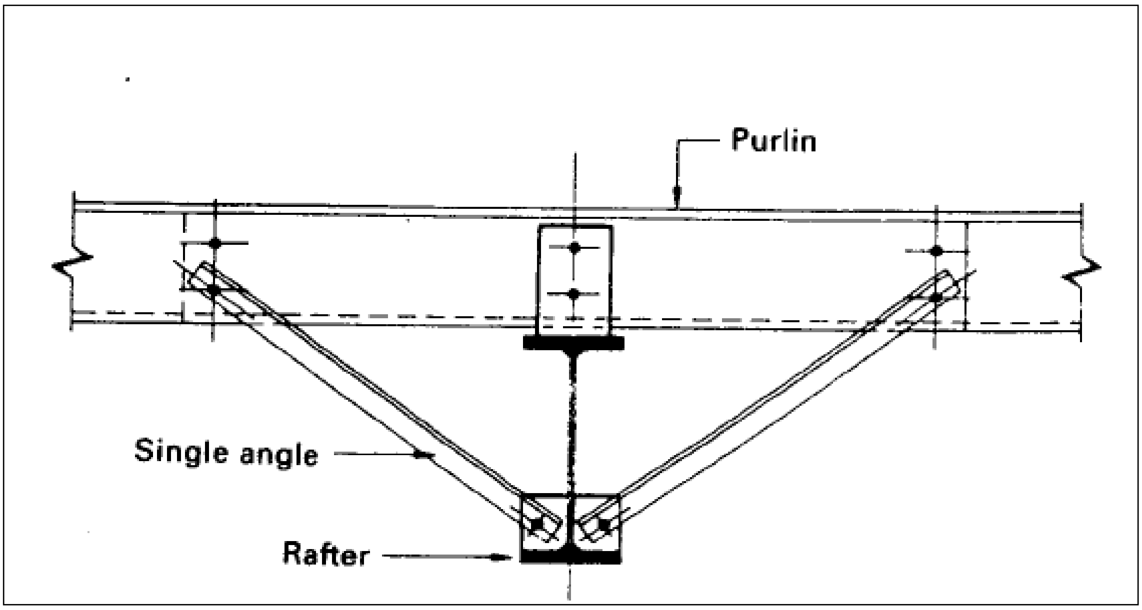


LTB Restraints



The default LTB data, defined in the buckling data dialog, are overruled by the 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 distance between these restraints. The LTB moments factors are calculated between these restraints.





Example WS CFS 04.esa

<table>
<tr>
<td>

Consider beam B1

Inputted section:

LTB length = 4,0 m

C1 = 1.13

C2= 0.45

C3 = 0.53

Mcr = 1,57 kNm

</td>
<td>

Lateral Torsional Buckling Check
According to article EN 1993-1-3: 6.2.4
According to article EN 1993-1-1: 6.3.2 and formula (6.55)

LTB Parameters		
Method for LTB Curve	art. 6.3.2.2	
Weff,y	19067	mm^3
Elastic critical moment Mcr	1.57	kNm
Relative slenderness Lambda,LT	1.69	
Limit slenderness Lambda,LT,0	0.40	
LTB curve	b	
Imperfection Alpha,LT	0.34	
Reduction factor Chi,LT	0.28	
Buckling resistance Mb,Rd	1.26	kNm
Unity check	**1.59**	-

Mcr Parameters		
LTB length	4.000	m
k	1.00	
kw	1.00	
C1	1.13	
C2	0.45	
C3	0.53	
Influence of load position	no influence	

Note: C Parameters according to ECCS 119 2006 / Galea 2002

The member does NOT satisfy the stability check!

</td>
</tr>
<tr>
<td>

Consider beam B2

Inputted section:

LTB length = 1,333 m

C1 = 1.02

C2= 0.05

C3 = 1.00

Mcr = 10,84 kNm

</td>
<td>

Lateral Torsional Buckling Check
According to article EN 1993-1-3: 6.2.4
According to article EN 1993-1-1: 6.3.2 and formula (6.55)

LTB Parameters		
Method for LTB Curve	art. 6.3.2.2	
Weff,y	19067	mm^3
Elastic critical moment Mcr	10.84	kNm
Relative slenderness Lambda,LT	0.64	
Limit slenderness Lambda,LT,0	0.40	
LTB curve	b	
Imperfection Alpha,LT	0.34	
Reduction factor Chi,LT	0.81	
Buckling resistance Mb,Rd	3.65	kNm
Unity check	0.55	-

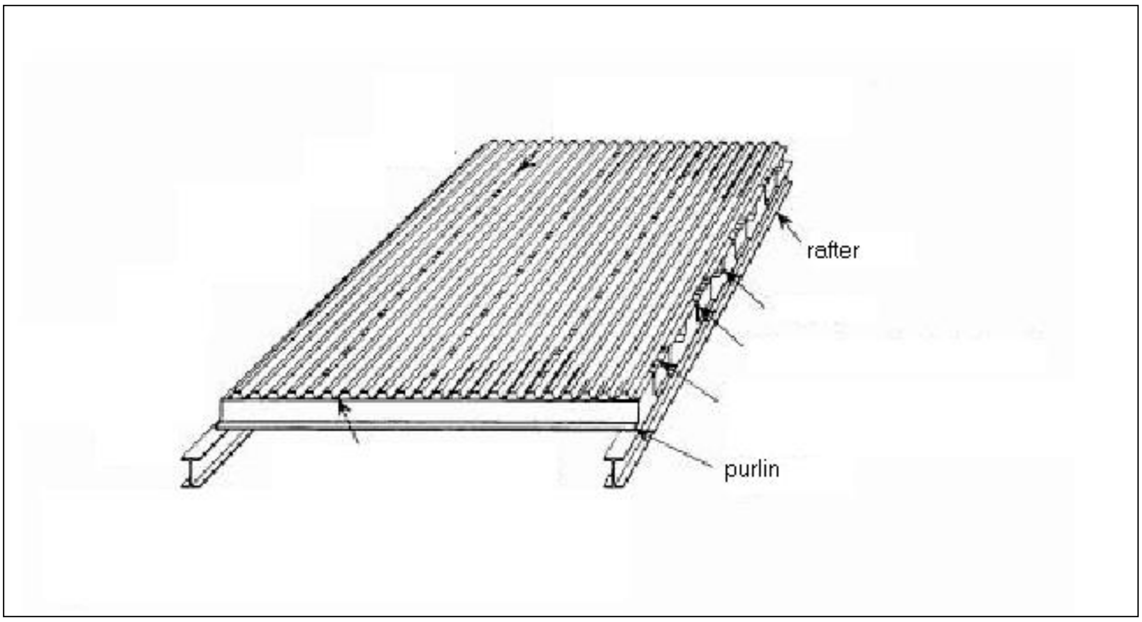
Mcr Parameters		
LTB length	1.333	m
k	1.00	
kw	1.00	
C1	1.02	
C2	0.05	
C3	1.00	
Influence of load position	no influence	

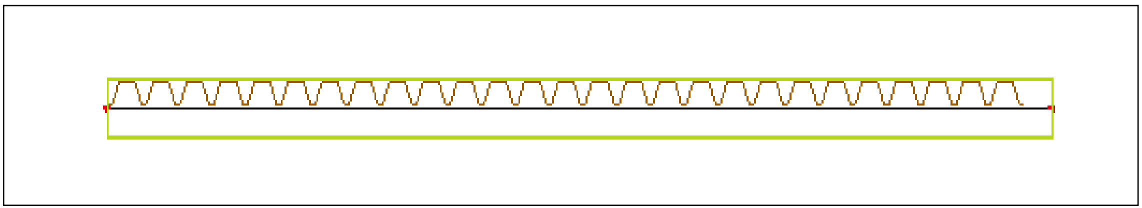
Note: C Parameters according to ECCS 119 2006 / Galea 2002

The member satisfies the stability check.

</td>
</tr>
</table>

Purlin design

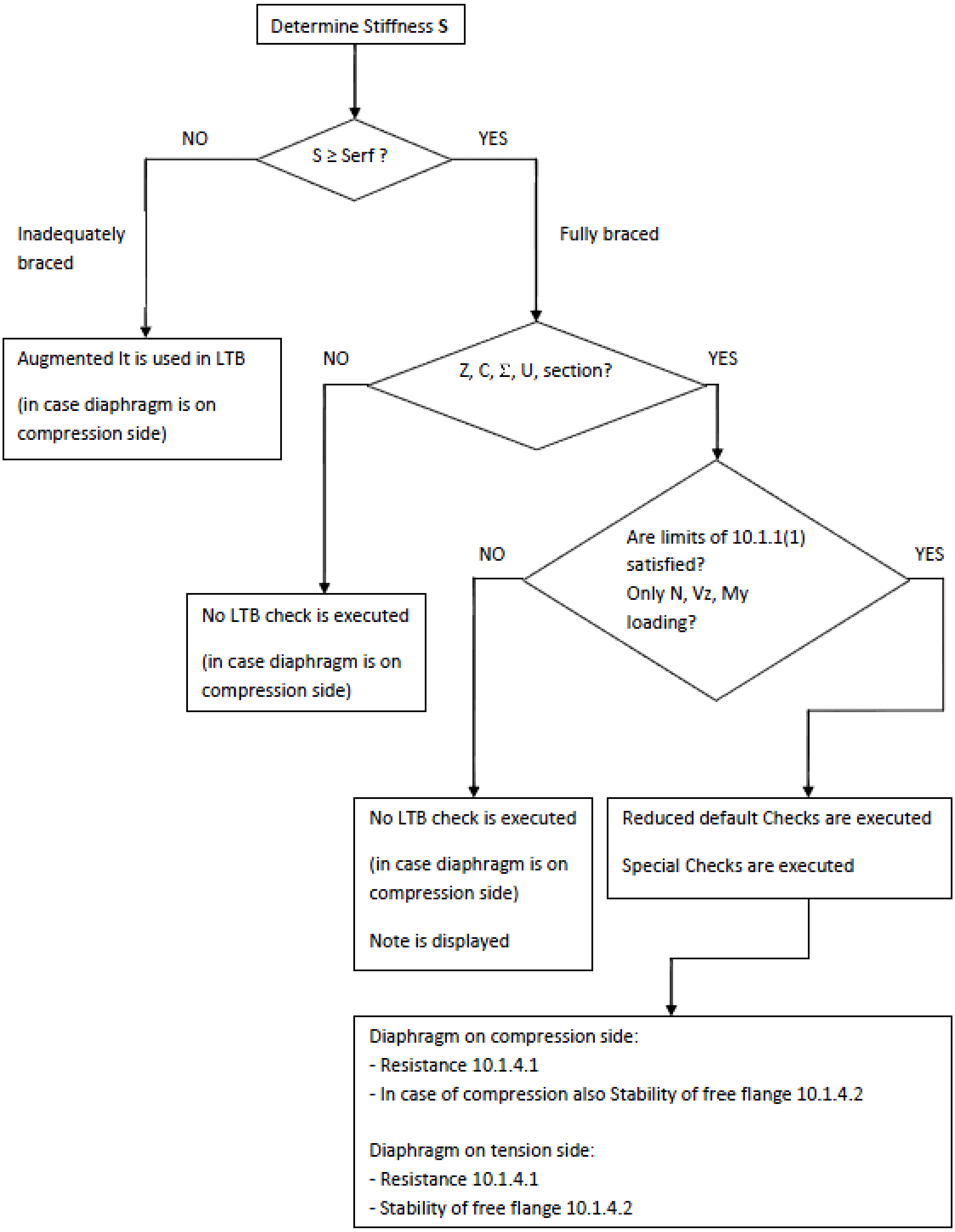




The settings for the diaphragm are:

k	The value of coefficient k depends on the number of spans of the diaphragm: k = 2 for 1 or 2 spans, k = 4 for 3 or more spans.
Diaphragm position	The position of the diaphragm may be either positive or negative. Positive means that the diaphragm is assembled in a way so that the width is greater at the top side. Negative means that the diaphragm is assembled in a way so that the width is greater at the bottom side.
Bolt position	Bolts may be located either at the top or bottom side of the diaphragm.
Bolt pitch	Bolts may be either: in every rib (i.e. "br"), in each second rib (i.e. "2 br").
Frame distance	The distance of frames
Length	The length of the diaphragm (shear field.)

Overview


Determine Stiffness S
S ≥ Serf ?
NO
YES
Inadequately braced
Fully braced
Augmented It is used in LTB
(in case diaphragm is on compression side)
Z, C, Σ, U, section?
NO
YES
No LTB check is executed
(in case diaphragm is on compression side)
Are limits of 10.1.1(1) satisfied?
Only N, Vz, My loading?
NO
YES
No LTB check is executed
(in case diaphragm is on compression side)
Note is displayed
Reduced default Checks are executed
Special Checks are executed
Diaphragm on compression side:
- Resistance 10.1.4.1
- In case of compression also Stability of free flange 10.1.4.2
Diaphragm on tension side:
- Resistance 10.1.4.1
- Stability of free flange 10.1.4.2

$$S = \frac{a \cdot 10^4}{K_1 + \frac{K_2}{L_s}} S_{erf} = \left(E\, I_w \frac{\pi^2}{L^2} + G\, I_t + E\, I_z \frac{\pi^2}{L^2} 0{,}25\, h^2\right) \frac{70}{h^2} I_{t,id} = I_t + vorC_\theta \frac{l^2}{\pi^2\, G}$$

$$S_{erf} = \left(E\, I_w \frac{\pi^2}{L^2} + G\, I_t + E\, I_z \frac{\pi^2}{L^2} 0{,}25\, h^2\right) \frac{70}{h^2} I_{t,id} = I_t + vorC_\theta \frac{l^2}{\pi^2\, G}$$

$$I_{t,id} = I_t + vorC_\theta \frac{l^2}{\pi^2\, G}$$

First of all the lateral stiffness **S** of the diaphragm is determined and compared to the required stiffness **S_{erf}.**

The lateral stiffness **S** is calculated according to Ref. [5],**3.5** and Ref. [6] ,**3.3.4**.

$$S = \frac{a \cdot 10^4}{K_1 + \frac{K_2}{L_s}}$$

As specified in EN 1993-1-3 article 10.1.1 the shear stiffness S is replaced by 0,2S in case the diaphragm is connected every second rib only.

And the required stiffness S_{erf} is determined according to EN 1993-1-3 **article 10.1.1**.

$$S_{erf} = \left(E\, I_w \frac{\pi^2}{L^2} + G\, I_t + E\, I_z \frac{\pi^2}{L^2} 0{,}25\, h^2\right) \frac{70}{h^2}$$

In case S < S_{erf} the member is seen as **inadequately braced**.
In this case, when the diaphragm is located on the compression side, the Lateral Torsional Buckling check is executed using the augmented torsional stiffness **I_t**.

$$I_{t,id} = I_t + vorC_\theta \frac{l^2}{\pi^2\, G}$$

With:

l	The LTB length
G	The shear modulus
vorhC_θ	The actual rotational stiffness of diaphragm

In case S $\geq$ S_{erf} the member is seen as **fully braced**.
In this case, a first test is executed to evaluate if the special purlin checks according to EN 1993-1-3 **Chapter 10** can be applied: this chapter is applied only in case the cross-section concerns a **Z**, **C**, **Σ** or **U** section.

Note
The code specifies that the chapter is also valid for hat (Omega) sections however in all further paragraphs no specific formulas are given for Omega sections. For example the free flange geometry is described only for Z, C and Σ sections, not for Omega sections. Therefore Omega sections are not supported for this special chapter in SCIA Engineer.

In case the cross-section **does not match** any of the above, the default checks are executed. Since the member is seen as fully braced, **no Lateral Torsional Buckling check** needs to be executed in case the diaphragm is located on the compression side.

In case the cross-section **does match** the list of set form codes, a second test is executed. More specifically, the special purlin checks according to EN 1993-1-3 **Chapter 10** can be applied only in case:

- The dimensional limits of **article 10.1.1(1)** are satisfied
- The section is only loaded by **N, V_z, M_y** (chapter 10 specifies only checks related to in plane effect N, V_z an M_y).

For a section which meet all requirements, the following is done:

- Reduced default Checks are executed i.e. not all default checks will be executed
- Special purlin checks according to Chapter 10

Section Check	Article
Axial tension	6.1.2
Axial compression	6.1.3
Bending moment	6.1.4
Shear force	6.1.5
Torsional moment	**NOT**
Local Transverse Forces	6.1.7
Combined tension and bending	**NOT**
Combined compression and bending	**NOT**
Combined shear, axial force and bending moment	6.1.10
Combined Bending and Local Transverse Force	6.1.11
Stability Check	**Article**
Flexural buckling only for y-y	6.2.2
Torsional and Torsional-Flexural buckling	**NOT**
Lateral-Torsional buckling	**NOT**
Bending and axial compression	**NOT**
Bending and axial tension	**NOT**

⇨ The Torsional moment check will never occur in this case since the prerequisite is to have only N, V_z, My.

⇨ The combined axial and bending checks are not executed since they are replaced by the special purlin checks.

⇨ The flexural buckling check is executed for y-y buckling in accordance with EN 1993-1-3 **art. 10.1.4.2(2)**.

⇨ Torsional buckling and Lateral-torsional buckling are prohibited by the fully braced diaphragm. The compression in the free flange is included in the special purlin checks.

⇨ The combined stability checks are not executed since they are replaced by the special purlin checks.

Note
In contrast to article 10.1.3.3(2) the Local Transverse Load Check and its interaction with the bending moment is executed even if the support reaction is a tensile force.

Example WS CFS Hall 2.esa
Consider member B112

Without the diaphragm the following check will be displayed:

....::STABILITY CHECK::...

Lateral Torsional Buckling Check
According to article EN 1993-1-3: 6.2.4
According to article EN 1993-1-1: 6.3.2 and formula (6.55)

LTB Parameters		
Method for LTB Curve	art. 6.3.2.2	
Weff,y	42159	mm^3
Elastic critical moment Mcr	2.05	kNm
Relative slenderness Lambda,LT	2.20	
Limit slenderness Lambda,LT,0	0.40	
LTB curve	b	
Imperfection Alpha,LT	0.34	
Reduction factor Chi,LT	0.18	
Buckling resistance Mb,Rd	1.75	kNm
Unity check	**4.69**	-

Mcr Parameters		
LTB length	6.000	m
k	1.00	
kw	1.00	
C1	2.51	
C2	1.49	
C3	0.41	
Influence of load position	no influence	

Note: C Parameters according to ECCS 119 2006 / Galea 2002

Bending and Axial Tension Check
According to article EN 1993-1-3: 6.3.

Table of values		
Nt,Rd	177.19	kN
Mb,y,Rd	1.75	kNm
Mc,z,Rd,com	1.62	kNm

Unity check: **4.69+0.48-0.02 = 5.16** -

The member does NOT satisfy the stability check!

With the diaphragm, the properties of the diaphragm are given

Diaphragm data

Table of values		
Actual stiffness S	8603.38	kN
Required stiffness Serf	785.63	kN
S >= Serf	Fully Braced	
c0Mk	5.63	kNm/m
c0Pk	2.00	kNm/m
c0Ak	1.10	kNm/m
c100	2.60	kNm/m
vorh ck	0.63	kNm/m

It,id = 9.2720e-10 + 2.8421e-08 = 2.9348e-08 m^4

But the limits for the internal forces (only N, V_z and M_y are not fulfilled), so no purlin check will be executed, but also no Lateral Torsional Buckling check is performed:

....::STABILITY CHECK::...

Bending and Axial Tension Check
According to article EN 1993-1-3: 6.3.

Table of values		
Nt,Rd	177.19	kN
Mb,y,Rd	9.92	kNm
Mc,z,Rd,com	1.62	kNm

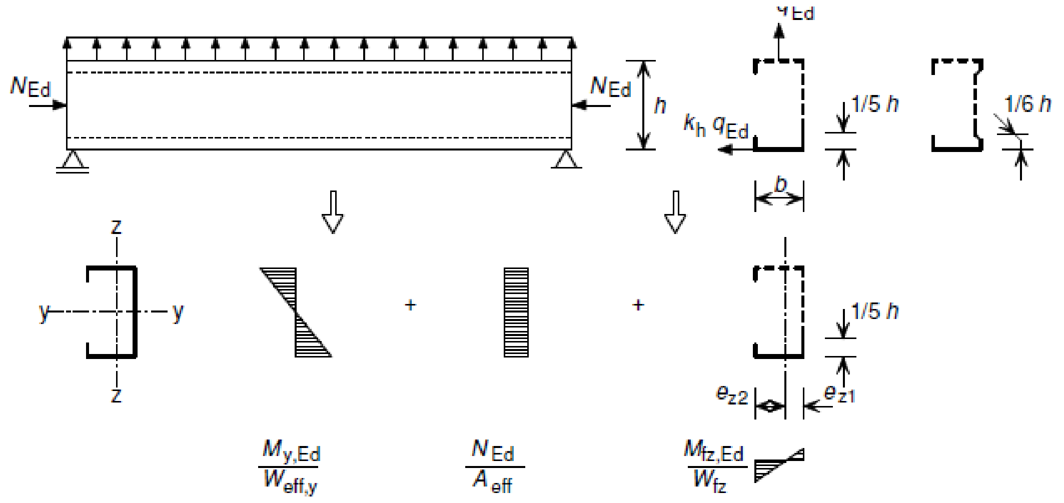
Unity check: **0.83+0.48-0.02 = 1.29** -

The member does NOT satisfy the stability check!

Design resistance

Resistance of cross-sections

The cross section should be verified as indicated below:



So superpose the following forces:

- In-plane bending moment $M_{y,Ed}$
- The axial force N_{Ed}
- An equivalent lateral load $q_{h,Ed}$ acting on the free flange, due to torsion an lateral bending

The maximum stresses in the cross-section should satisfy the following:

- Restrained flange

$$\sigma_{max,Ed} = \frac{M_{y,Ed}}{W_{eff,y}} + \frac{N_{Ed}}{A_{eff}} \leq f_y/\gamma_M$$

- Free flange

$$\sigma_{max,Ed} = \frac{M_{y,Ed}}{W_{eff,y}} + \frac{N_{Ed}}{A_{eff}} + \frac{M_{fz,Ed}}{W_{fz}} \leq f_y/\gamma_M$$

Where:

W_{fz} is the gross elastic section modulus of the free flange plus the contributing part of the web for bending about the z-z-axis.

Unless a more sophisticated analysis is carried out the contributing part of the web may be taken equal to 1/5 of the web height from the point of web-flange intersection in case of C-and Z-section and 1/6 if the web height in case of Σ-section.

$M_{fz,Ed}$ is the bending moment in the free flange due to the horizontal load $q_{h,Ed}$:

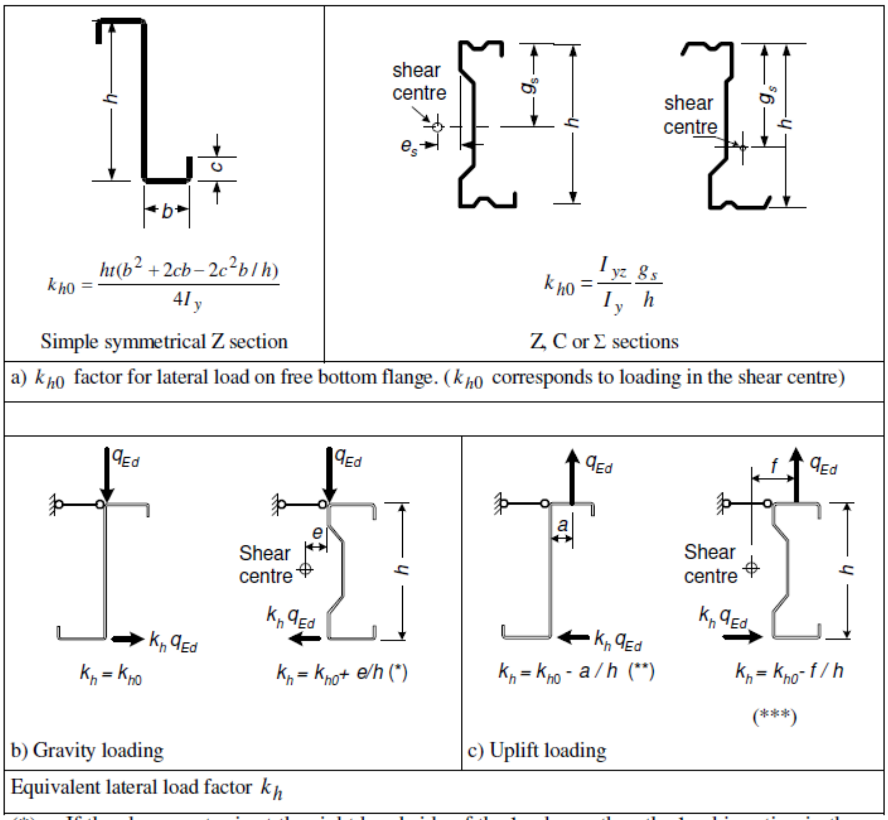
$q_{h,Ed} = k_h\ q_{Ed}$ (see also figure below)

And

$M_{fz,Ed} = \kappa_R\ M_{0,fz,Ed}$

$M_{0,fz,Ed}$ is the initial lateral bending moment in the free flange without any spring support

κ_R is a correction factor for the effective spring support and may be determined for the relevant location and boundary condition, using the theory of beams on the elastic Winkler foundation.
Table 10.1 from EN 1993-1-3 provides the formulas to determine $M_{0,fz,Ed}$ for specific positions within the beams.



(*) If the shear centre is at the right hand side of the load q_{Ed} then the load is acting in the opposite direction.

(**) If $a/h > k_{h0}$ then the load is acting in the opposite direction.

(***) The value of f is limited to the position of the load q_{Ed} between the edges of the top flange.

Note
In case the free flange is in tension, $M_{fz,Ed}$ is taken equal to zero (see also EN 1993-1-3 article 10.1.4.1(5)).

The **lateral spring stiffness K** is determined according to En 1993-1-3, article 10.1.5(4).

$$\frac{1}{K} = \frac{4\,(1-\nu^2)\,h^2\,(h_d + b_{mod})}{E\,t^3} + \frac{h^2}{C_D}$$

Where:

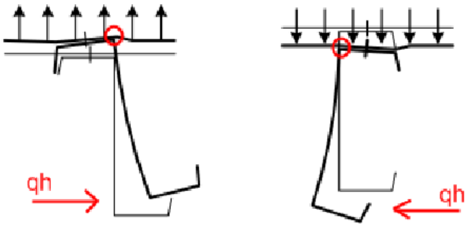
t is the thickness of the purlin

h, a, b, b_{mod}, h_d see figures below

C_D is the total rotational spring stiffness and will be taken as **vorhC**. The calculation of this value is also given below.

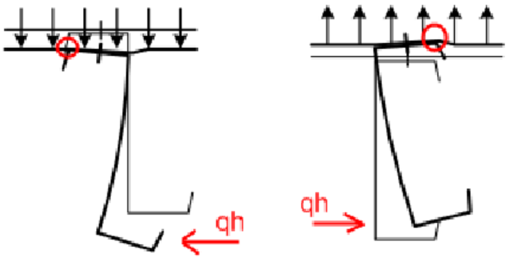
DETERMINATION OF PROPERTIES h, a, b, b_{mod} and h_d

If q_h brings the purlin into contact with the sheeting at the purlin web



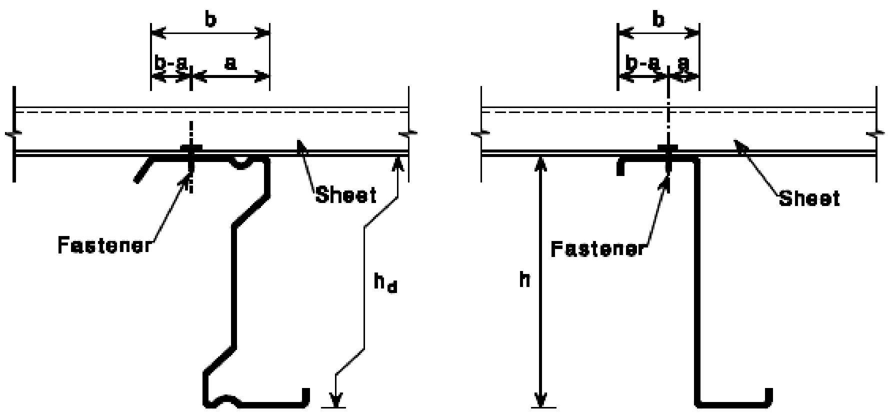
$b_{mod} = a$

If q_h brings the purlin into contact with the sheeting at the tip of the purlin flange



$b_{mod} = 2a + b$

Determination of **a** and **b**



DETERMINATION OF vorhC:

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

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

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

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

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

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

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

With:

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

In below some values for the rotation coefficient:

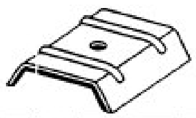
Positioning of sheeting		Sheet fastened through		Pitch of fasteners		Washer diameter	C_{100}	$b_{T,max}$
Positive	Negative	Trough	Crest	$e = b_R$	$e = 2b_R$	[mm]	[kNm/m]	[mm]
For gravity loading:								
×		×		×		22	5,2	40
×		×			×	22	3,1	40
	×		×	×		K_a	10,0	40
	×		×		×	K_a	5,2	40
	×	×		×		22	3,1	120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



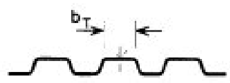
The values in this table are valid for:

- 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.

Sheet fastened:

- through the trough:

- through the crest:



Buckling resistance of the Free Flange

If the free flange is in compression, its buckling resistance should be verified, using:

$$\frac{1}{\chi_{LT}}\left(\frac{M_{y,Ed}}{W_{eff,y}}+\frac{N_{Ed}}{W_{fz}}\right)+\frac{M_{fz,Ed}}{W_{fz}} \leq f_{yb}/\gamma_{M1}$$

And the buckling length will be calculated by:

$$l_{fz} = \eta_1 L_a (1 + \eta_2 R^{\eta_3})^{\eta_4}$$

And η_1 to η_4 are given in the tables below:

Table 10.2a : Coefficients η_i for down load with 0, 1, 2, 3, 4 anti-sag bars

Situation	Anti sag-bar Number	η_1	η_2	η_3	η_4
End span	0	0.414	1.72	1.11	-0.178
Intermediate span		0.657	8.17	2.22	-0.107
End span	1	0.515	1.26	0.868	-0.242
Intermediate span		0.596	2.33	1.15	-0.192
End and intermediate span	2	0.596	2.33	1.15	-0.192
End and intermediate span	3 and 4	0.694	5.45	1.27	-0.168

Table 10.2b : Coefficients η_i for uplift load with 0, 1, 2, 3, 4 anti-sag bars

Situation	Anti sag-bar Number	η_1	η_2	η_3	η_4
Simple span	0	0.694	5.45	1.27	-0.168
End span		0.515	1.26	0.868	-0.242
Intermediate span		0.306	0.232	0.742	-0.279
Simple and end spans	1	0.800	6.75	1.49	-0.155
Intermediate span		0.515	1.26	0.868	-0.242
Simple span	2	0.902	8.55	2.18	-0.111
End and intermediate spans		0.800	6.75	1.49	-0.155
Simple and end spans	3 and 4	0.902	8.55	2.18	-0.111
Intermediate span		0.800	6.75	1.49	-0.155

If the compression over the length L is almost constant, due to the application of **relatively large axial force**, the buckling length should be determined using the values of η_i for the case shown as “more than three anti-sag bars per span”, but the actual spacing L_a.

“The relatively large axial force” is specified in SCIA Engineer as follows:

$$if\ \frac{N_{Ed}}{A_{eff} * f_{yb}/\gamma_{M1}} \geq Limit => Large\ axial\ force$$

$$if\ \frac{N_{Ed}}{A_{eff} * f_{yb}/\gamma_{M1}} < Limit => small\ axial\ force$$

Default this limit value is set on 1 in SCIA Engineer, but this can be changed in the Steel Setup:

Example WS CFS 05.esa

Consider member B2 - section

First the properties of the diaphragm are given.

Diaphragm data

Table of values		
Actual stiffness S	8603.38	kN
Required stiffness Serf	649.96	kN
S >= Serf	Fully Braced	
c0Mk	5.63	kNm/m
c0Pk	5.56	kNm/m
c0Ak	2.50	kNm/m
c100	10.00	kNm/m
vorh ck	1.32	kNm/m

It,id = 1.3008e-09 + 5.9618e-08 = 6.0919e-08 m^4

The condition to perform a purlin check are fulfilled (only N, V_z and M_y)

The critical check is on position 3.000 m

Internal forces	Calculated	Additional moments	Total	Unit
N,Ed	-20,00		-20,00	kN
Vy,Ed	0,00		0,00	kN
Vz,Ed	0,75		0,75	kN
T,Ed	0,00		0,00	kNm
My,Ed	2,25	0,00	2,25	kNm
Mz,Ed	0,00	-0,03	-0,03	kNm

Below the default section check, the check on the **beam restrained by sheeting – resistance of cross-section** is given:

Beam restrained by sheeting - Resistance of cross-section

According to article EN 1993-1-3: 10.1.4.1 and formula (10.3a), (10.3b).

Equivalent Lateral Load		
Vertical load qEd	-1.00	kN/m
kh0	-0.00	
e	19	mm
h	150	mm
kh	0.13	
Lateral load qh,Ed	0.13	kN/m

Lateral Bending Moment		
Boundary Conditions	Hinged-Hinged	
Mfz,Ed	0.00	kNm

Lateral spring stiffness		
a	25	mm
b	50	mm
hd	150	mm
bmod	25	mm
CD	1.32	kNm/m
Lateral stiffness K	46.70	kN/m^2
Ifz	5.8294e-08	m^4
La	6.000	m
R	50.76	

Table of values		
Aeff	5.4180e-04	m^2
Weff,y restrained flange	2.6834e-05	m^3
Weff,y free flange	2.6834e-05	m^3
Wfz	2.8462e-06	m^3
Gamma M	1.00	

Unity check (10.3a) 0.36 + 0.16 = 0.51 -
Unity check (10.3b) (-0.36) + 0.16 + 0.00 = 0.20 -

The check on the **Buckling resistance of the free flange** is not performed for beam B2. $\frac{N_{Ed}}{A_{eff}*f_{yb}/\gamma_{M1}} < Limit = 0{,}1$ so the check does not have to be executed:

For beam B1, the normal force has been increased and $\frac{N_{Ed}}{A_{eff}*\frac{f_{yb}}{\gamma_{M1}}} \geq Limit = 0{,}1$

⇨ **check on the purlin for the buckling resistance of the free flange will be displayed:**

Beam restrained by sheeting - Buckling Resistance of free flange
According to article EN 1993-1-3: 10.1.4.2 Table 10.2a and formula (10.7).
According to article EN 1993-1-1: 6.3.2.3.

Buckling length		
La	6.000	m
R	50.76	
Eta 1	0.69	
Eta 2	5.45	
Eta 3	1.27	
Eta 4	-0.17	
Buckling length lfz	1.355	m
ifz	1.7467e-02	m
Reduced slenderness Lambda,fz	0.83	

Reduction factor		
Limit slenderness Lambda,LT,0	0.40	
LTB curve	b	
Imperfection Alpha,LT	0.34	
Reduction factor Chi,LT	0.80	
Unity check	0.05	-

Note: The buckling length of the free flange is determined according to article 10.1.4.2(5) due to a "relatively large axial force".

The member satisfies the stability check.

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