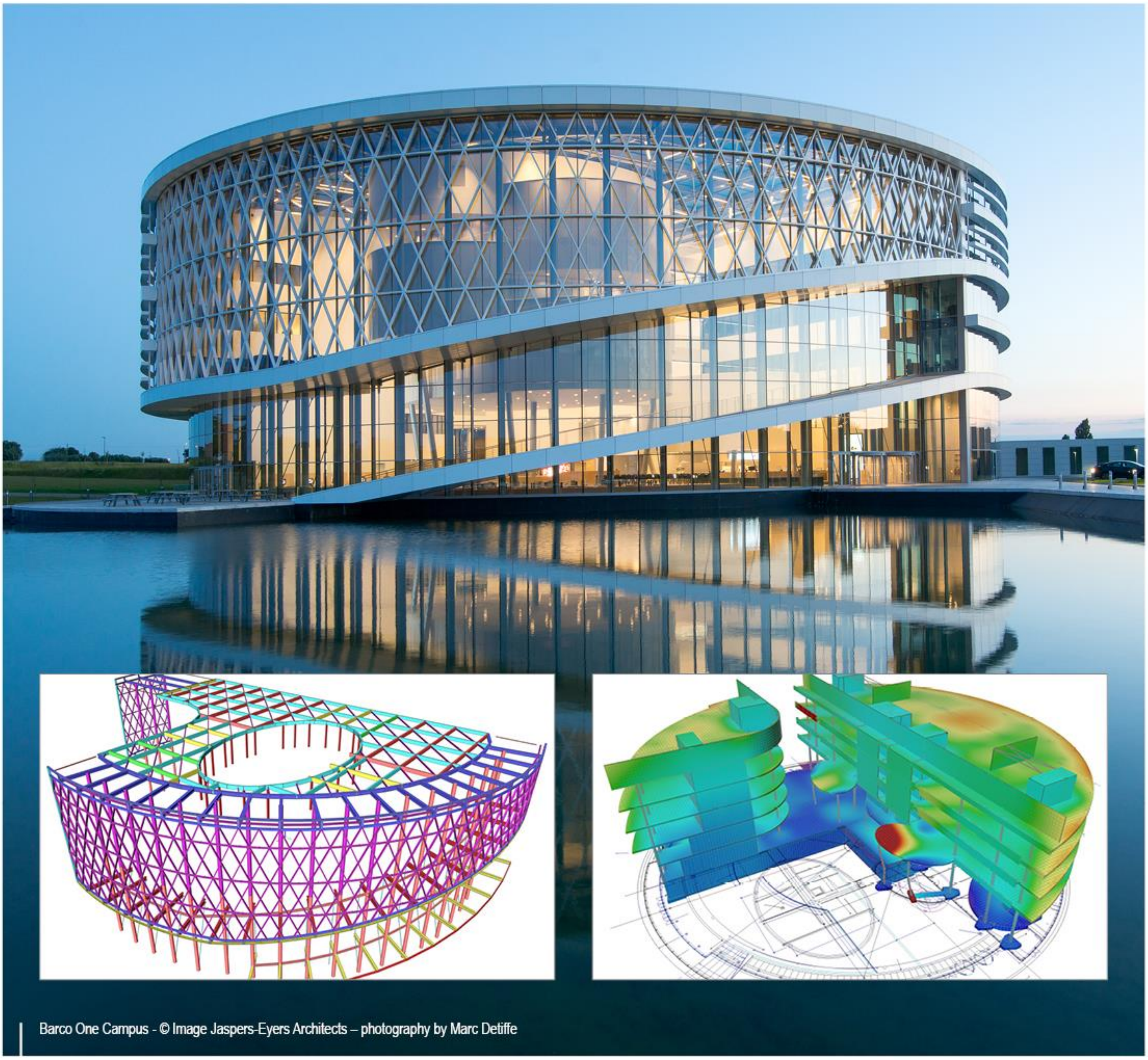
SCIA ENGINEER



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Advanced Training

Aluminium

Table of contents

Introduction

The applied rules for EN 1999-1-1 are explained and illustrated.

EC-EN => EN 1999-1-1:2007

More and detailed references to the applied articles can be found in (Ref.[1])

SCIA Engineer Aluminium Code Check
Theoretical Background
Release : 16.0.1075
Revision : 08/2016

The explained rules are valid for SCIA Engineer 16.0.1075

The examples are marked by '**Example**'
The following examples are available :

Project	Subject
wsa_001.esa	global analysis
wsa_001a.esa	nodal displacement
wsa_001b.esa	relative displacements
wsa_002.esa	classification Z-section
wsa_003.esa	thinwalled cross-section
wsa_004.esa	shear
wsa_005.esa	combined bending - transverse welds
wsa_006.esa	flexural buckling
wsa_008.esa	lateral torsional buckling
wsa_009a.esa	combined stability – xs 1
wsa_009b.esa	combined stability – xs 2
wsa_010.esa	shear buckling - stiffeners

Materials and Combinations

Aluminium grades

The characteristic values of the material properties are based on Table 3.2a for wrought aluminium alloys of type sheet, strip and plate and on Table 3.2b for wrought aluminium alloys of type extruded profile, extruded tube, extruded rod/bar and drawn tube (Ref.[1]).

EN 1999-1-1: 2007 (E)

Table 3.2a - Characteristic values of 0,2% proof strength f_o, ultimate tensile strength f_u (unwelded and for HAZ), min elongation A, reduction factors $\rho_{o,haz}$ and $\rho_{u,haz}$ in HAZ, buckling class and exponent n_p for wrought aluminium alloys - Sheet, strip and plate

Alloy EN-AW	Temper [1)]	Thick-ness [1)] mm	f_o [1)]	f_u	A_{50} [1) 6)]	$f_{o,haz}$ [2)]	$f_{u,haz}$ [2)]	HAZ-factor [2)]		BC 4)	n_p 1), 5)
			N/mm²		%	N/mm²		$\rho_{o,haz}$ [1)]	$\rho_{u,haz}$		
3004	H14 I H24/H34	≤ 6 I 3	180 I 170	220	1 I 3	75	155	0,42 I 0,44	0,70	B	23 I 18
	H16 I H26/H36	≤ 4 I 3	200 I 190	240	1 I 3			0,38 I 0,39	0,65	B	25 I 20
3005	H14 I H24	≤ 6 I 3	150 I 130	170	1 I 4	56	115	0,37 I 0,43	0,68	B	38 I 18
	H16 I H26	≤ 4 I 3	175 I 160	195	1 I 3			0,32 I 0,35	0,59	B	43 I 24
3103	H14 I H24	≤ 25 I 12,5	120 I 110	140	2 I 4	44	90	0,37 I 0,40	0,64	B	31 I 20
	H16 I H26	≤ 4	145 I 135	160	1 I 2			0,30 I 0,33	0,56	B	48 I 28
5005/ 5005A	O/H111	≤ 50	35	100	15	35	100	1	1	B	5
	H12 I H22/H32	≤ 12,5	95 I 80	125	2 I 4	44	100	0,46 I 0,55	0,80	B	18 I 11
	H14 I H24/H34	≤ 12,5	120 I 110	145	2 I 3			0,37 I 0,40	0,69	B	25 I 17
5052	H12 I H22/H32	≤ 40	160 I 130	210	4 I 5	80	170	0,50 I 0,62	0,81	B	17 I 10
	H14 I H24/H34	≤ 25	180 I 150	230	3 I 4			0,44 I 0,53	0,74	B	19 I 11
5049	O / H111	≤ 100	80	190	12	80	190	1	1	B	6
	H14 I H24/H34	≤ 25	190 I 160	240	3 I 6	100	190	0,53 I 0,63	0,79	B	20 I 12
5454	O/H111	≤ 80	85	215	12	85	215	1	1	B	5
	H14IH24/H34	≤ 25	220 I 200	270	2 I 4	105	215	0,48 I 0,53	0,80	B	22 I 15
5754	O/H111	≤ 100	80	190	12	80	190	1	1	B	6
	H14IH24/H34	≤ 25	190 I 160	240	3 I 6	100	190	0,53 I 0,63	0,79	B	20 I 12
5083	O/H111	≤ 50	125	275	11	125	275	1	1	B	6
		50<t≤80	115	270	14 [3)]	115	270			B	
	H12IH22/H32	≤ 40	250 I 215	305	3 I 5	155	275	0,62 I 0,72	0,90	B	22 I 14
	H14IH24/H34	≤ 25	280 I 250	340	2 I 4			0,55 I 0,62	0,81	A	22 I 14
6061	T4 / T451	≤ 12,5	110	205	12	95	150	0,86	0,73	B	8
	T6 / T651	≤ 12,5	240	290	6	115	175	0,48	0,60	A	23
	T651	12,5<t≤80	240	290	6 [3)]						
6082	T4 / T451	≤ 12,5	110	205	12	100	160	0,91	0,78	B	8
	T61/T6151	≤12,5	205	280	10	125	185	0,61	0,66	A	15
	T6151	12,5<t≤100	200	275	12 [3)]			0,63	0,67	A	14
	T6/T651	≤ 6	260	310	6			0,48	0,60	A	25
		6<t≤12,5	255	300	9			0,49	0,62	A	27
	T651	12,5<t≤100	240	295	7 [3)]			0,52	0,63	A	21
7020	T6	≤ 12,5	280	350	7	205	280	0,73	0,80	A	19
	T651	≤ 40			9 [3)]						
8011A	H14 I H24	≤ 12,5	110 I 100	125	2 I 3	37	85	0,34 I 0,37	0,68	B	37 I 22
	H16 I H26	≤ 4	130 I 120	145	1 I 2			0,28 I 0,31	0,59		33 I 33

1) If two (three) tempers are specified in one line, tempers separated by "I" have different technological values but separated by "/" have same values. (The tempers show differences for f_o, A and n_p.).

2) The HAZ-values are valid for MIG welding and thickness up to 15mm. For TIG welding strain hardening alloys (3xxx, 5xxx and 8011A) up to 6 mm the same values apply, but for TIG welding precipitation hardening alloys (6xxx and 7xxx) and thickness up to 6 mm the HAZ values have to be multiplied by a factor 0,8 and so the ρ-factors. For higher thickness – unless other data are available – the HAZ values and ρ-factors have to be further reduced by a factor 0,8 for the precipitation hardening alloys (6xxx and 7xxx) and by a factor 0,9 for the strain hardening alloys (3xxx, 5xxx and 8011A). These reductions do not apply in temper O.

3) Based on A $(= A_{5,65\sqrt{A_o}})$, not A_{50}.

4) BC = buckling class, see 6.1.4.4, 6.1.5 and 6.3.1.

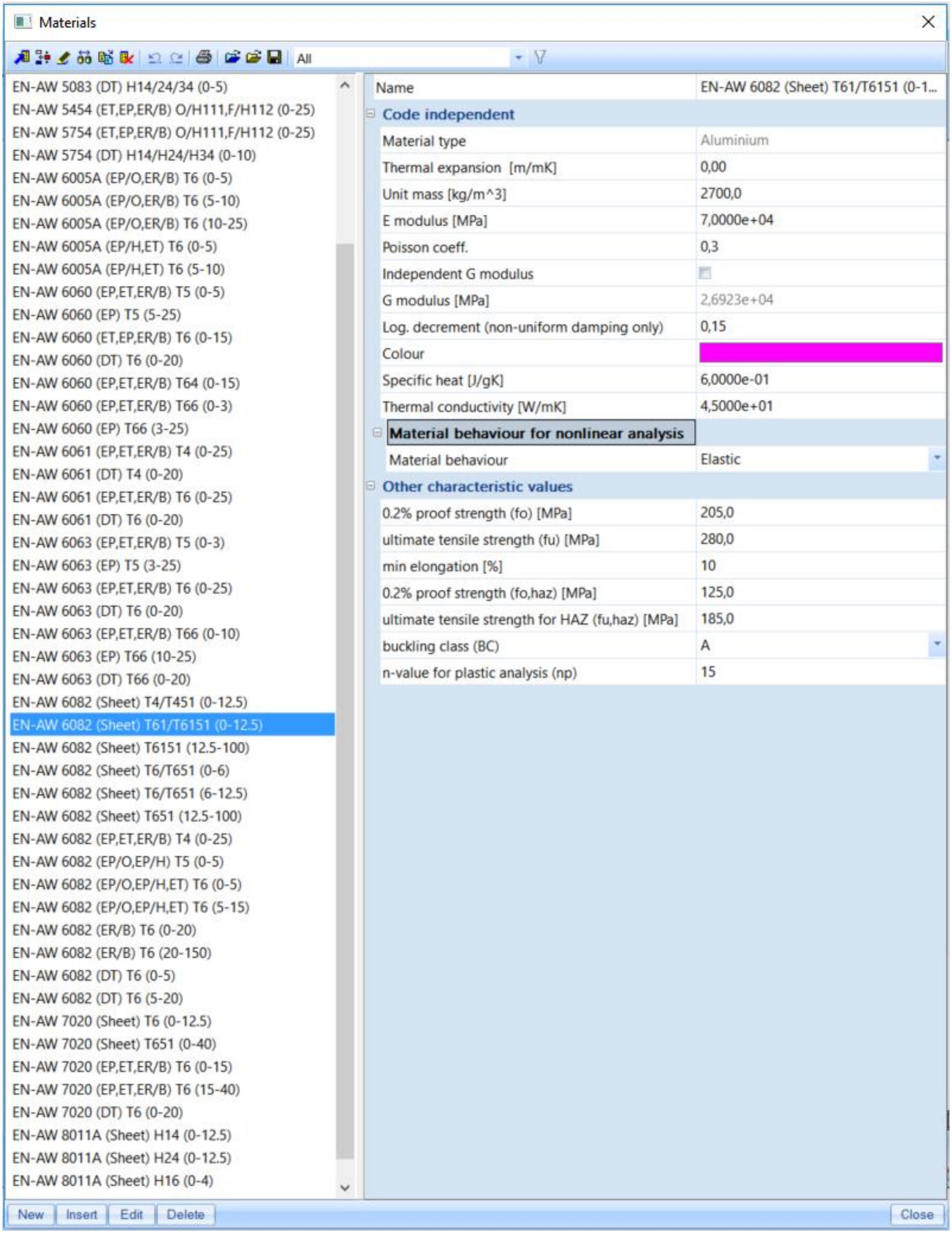
5) n-value in Ramberg-Osgood expression for plastic analysis. It applies only in connection with the listed f_o-value.

6) The minimum elongation values indicated do not apply across the whole range of thickness given, but mostly to the thinner materials. In detail see EN 485-2.

Table 3.2b - Characteristic values of 0,2% proof strength f_o and ultimate tensile strength f_u (unwelded and for HAZ), min elongation A, reduction factors $\rho_{o,haz}$ and $\rho_{u,haz}$ in HAZ, buckling class and exponent n_p for wrought aluminium alloys - Extruded profiles, extruded tube, extruded rod/bar and drawn tube

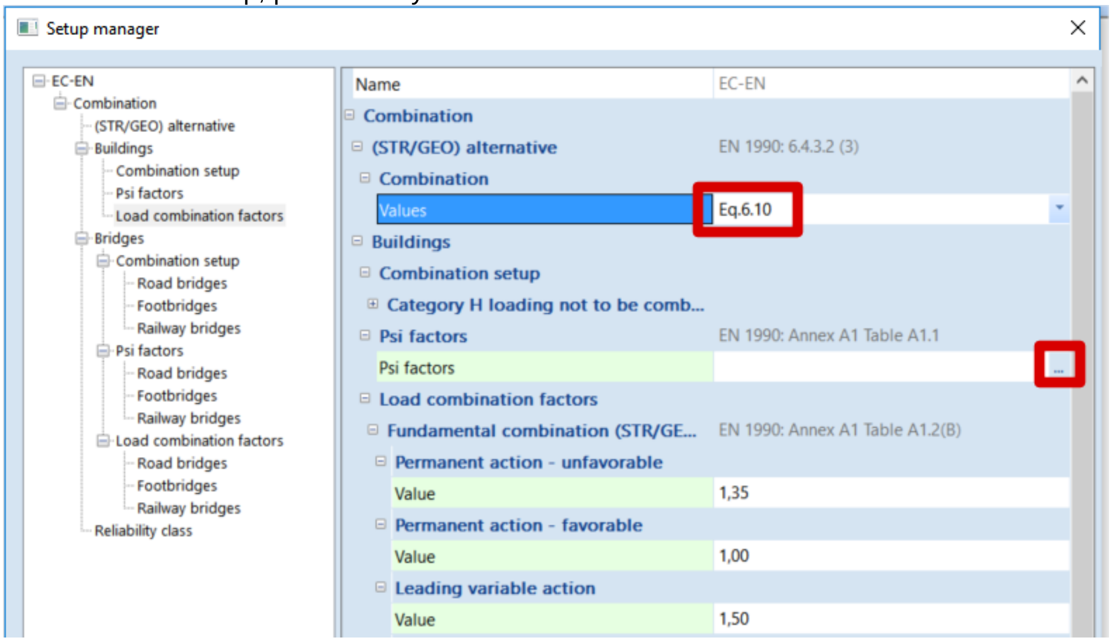
Alloy EN-AW	Product form	Temper	Thick-ness t mm 1) 3)	f_o 1)	f_u 1)	A 5) 2)	$f_{o,haz}$ 4)	$f_{u,haz}$ 4)	HAZ-factor 4)		BC 6)	n_p 7)
				N/mm²		%	N/mm²		$\rho_{o,haz}$	$\rho_{u,haz}$		
5083	ET, EP,ER/B	O / H111, F, H112	$t \le 200$	**110**	**270**	12	110	270	1	1	B	5
	DT	H12/22/32	$t \le 10$	200	280	6	135	270	0,68	0,96	B	14
		H14/24/34	$t \le 5$	235	300	4			0,57	0,90	A	18
6060	EP,ET,ER/B	T5	$t \le$ **5**	120	160	8	50	80	0,42	0,50	B	17
	EP		$5 < t \le 25$	100	140	8			0,50	0,57	B	14
	ET,EP,ER/B	T6	$t \le$ **15**	**140**	**170**	8	60	100	0,43	0,59	A	24
	DT		$t \le 20$	160	215	12			0,38	0,47	A	16
	EP,ET,ER/B	T64	$t \le$ **15**	120	180	12	60	100	0,50	0,56	A	12
	EP,ET,ER/B	T66	$t \le$ **3**	160	215	8	65	110	0,41	0,51	A	16
	EP		$3 < t \le 25$	150	195	8			0,43	0,56	A	18
6061	EP,ET,ER/B,DT	T4	$t<$**25**	110	**180**	**50**	95	150	0,86	0,83	B	8
	EP,ET,ER/B,DT	T6	$t \le$ **20**	240	**260**	**8**	115	175	0,48	0,67	A	55
6063	EP,ET,ER/B	T5	$t \le$ **3**	130	175	8	60	100	0,46	0,57	B	16
	EP		$3 < t \le 25$	110	160	**7**			0,55	0,63	B	13
	EP,ET,ER/B	T6	$t \le$ **25**	**160**	**195**	**8**	65	110	0,41	0,56	A	24
	DT		$t \le 20$	190	220	10			0,34	0,50	A	31
	EP,ET.ER/B	T66	$t \le$ **10**	200	245	**8**	75	130	0,38	0,53	A	22
	EP		$10 < t \le 25$	180	225	8			0,42	0,58	A	21
	DT		$t \le 20$	195	230	10			0,38	0,57	A	28
6005A	EP/O, ER/B	T6	$t \le$ **5**	225	270	8	115	165	0,51	0,61	A	25
			$5 < t \le 10$	215	260	8			0,53	0,63	A	24
			$10 < t \le 25$	200	250	8			0,58	0,66	A	20
	EP/H, ET	T6	$t \le 5$	**215**	**255**	8			0,53	0,65	A	26
			$5 < t \le$ **10**	**200**	**250**	8			0,58	0,66	A	20
6106	EP	T6	$t \le 10$	200	250	8	95	160	0,48	0,64	A	20
6082	EP,ET,ER/B	T4	$t \le$ **25**	110	205	14	100	160	0,91	0,78	B	8
	EP/O, EP/H	T5	$t \le 5$	230	270	8	125	185	0,54	0,69	B	28
	EP/O,EP/H ET	T6	$t \le 5$	250	290	8	125	185	0,50	0,64	A	32
			$5 < t \le$ **15**	260	310	10			0,48	0,60	A	25
	ER/B	T6	$t \le 20$	250	295	8			0,50	0,63	A	27
			$20 < t \le 150$	260	310	8			0,48	0,60	A	25
	DT	T6	$t \le 5$	255	310	8			0,49	0,60	A	22
			$5 < t \le 20$	240	310	10			0,52	0,60	A	17
7020	EP,ET,ER/B	T6	$t \le$ **15**	290	350	10	205	280	0,71	0,80	A	23
	EP,ET,ER/B	T6	$15 < t <$ **40**	**275**	350	10			0,75	0,80	A	19
	DT	T6	$t \le 20$	280	350	10			0,73	0,80	A	18

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



Combinations

In SCIA Engineer, both the SLS and ULS combinations can be set according to the code rules for EC-EN1990. 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.

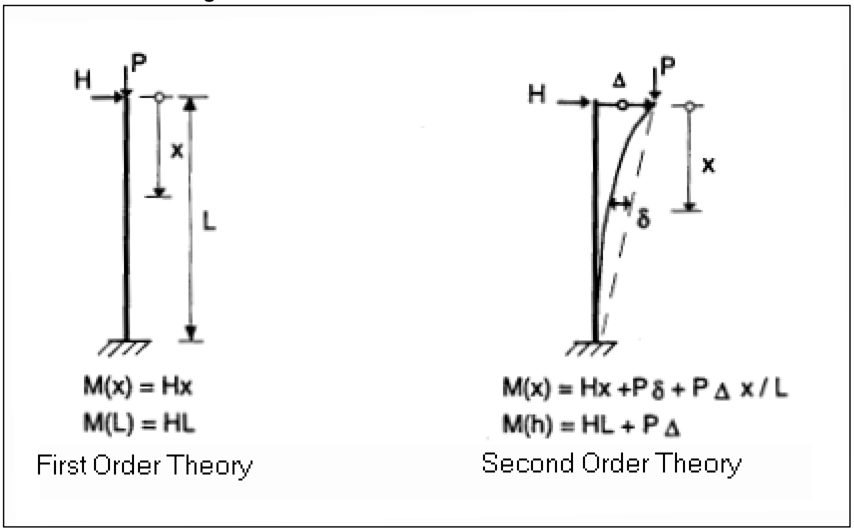
Example

Consider a simple building subjected to an unfavorable permanent load, a Category A Imposed load and a Wind load
for unfavorable permanent actions γ_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 ξ = 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

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.



According to the EC-EN 1999, 1st Order analysis may be used for a structure, if the increase of the relevant internal forces or moments or any other change of structural behaviour caused by deformations can be neglected. This condition may be assumed to be fulfilled, if the following criterion is satisfied:

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

With:

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

If α_{cr} has a value lower then 10, a 2nd Order calculation needs to be executed. Depending on the type of analysis, both Global and Local imperfections need to be considered.
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 5.3.2(3) Ref.[1]:

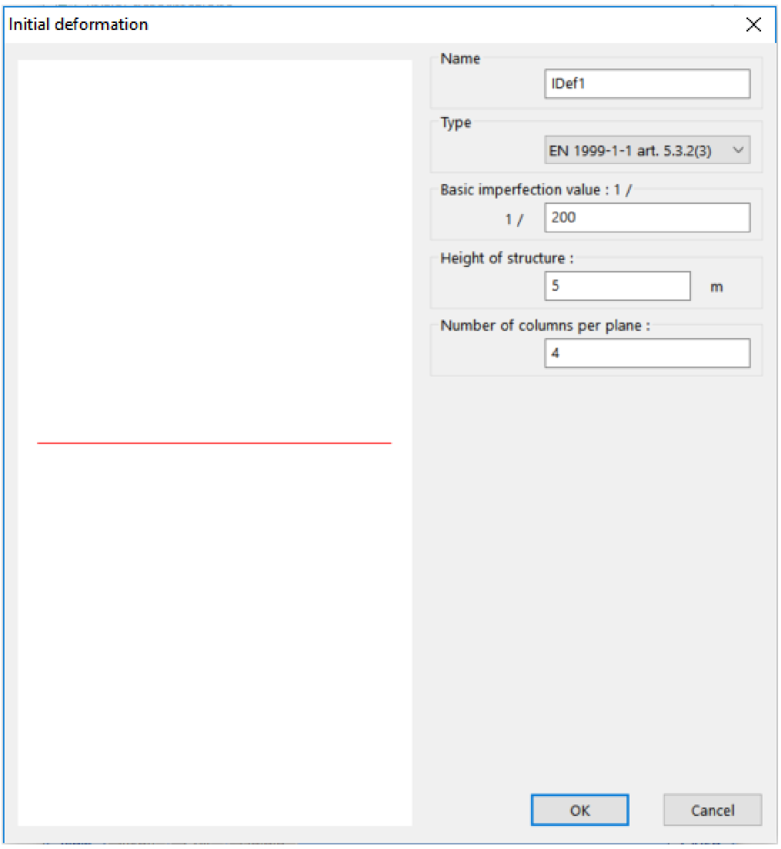
$$\varphi = \frac{1}{200} \cdot \alpha_h \cdot \alpha_m$$

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

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

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 1999-1-1 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

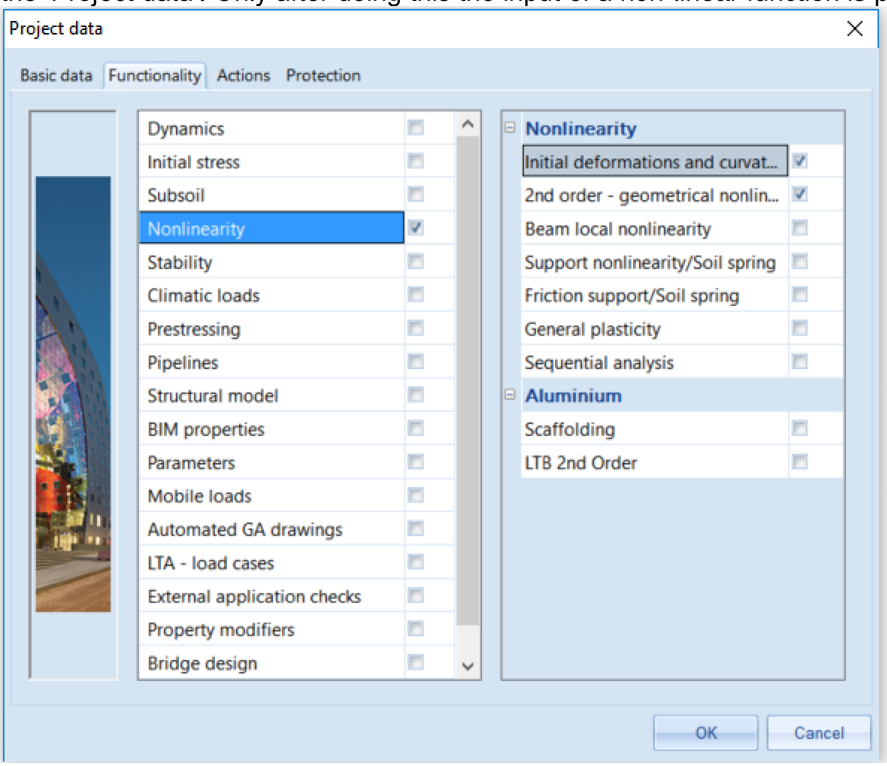
Initial bow imperfection e_0

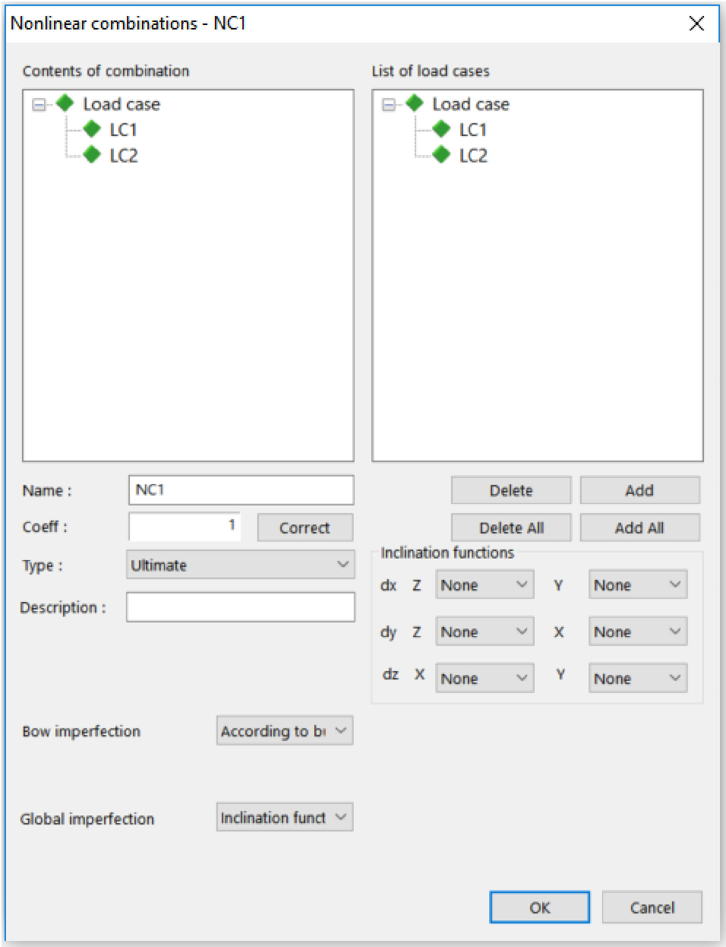
The values of **e0/L** may be chosen in the National Annex. Recommend values are given in the following Table 5.1 Ref.[1]. The bow imperfection has to be applied when the normal force $\mathbf{N_{Ed}}$ in a member is higher than 25% of the member's critical buckling load $\mathbf{N_{cr}}$.

Buckling class acc. to Table 3.2	elastic analysis e_0/L	plastic analysis e_0/L
A	1/300	1/250
B	1/200	1/150

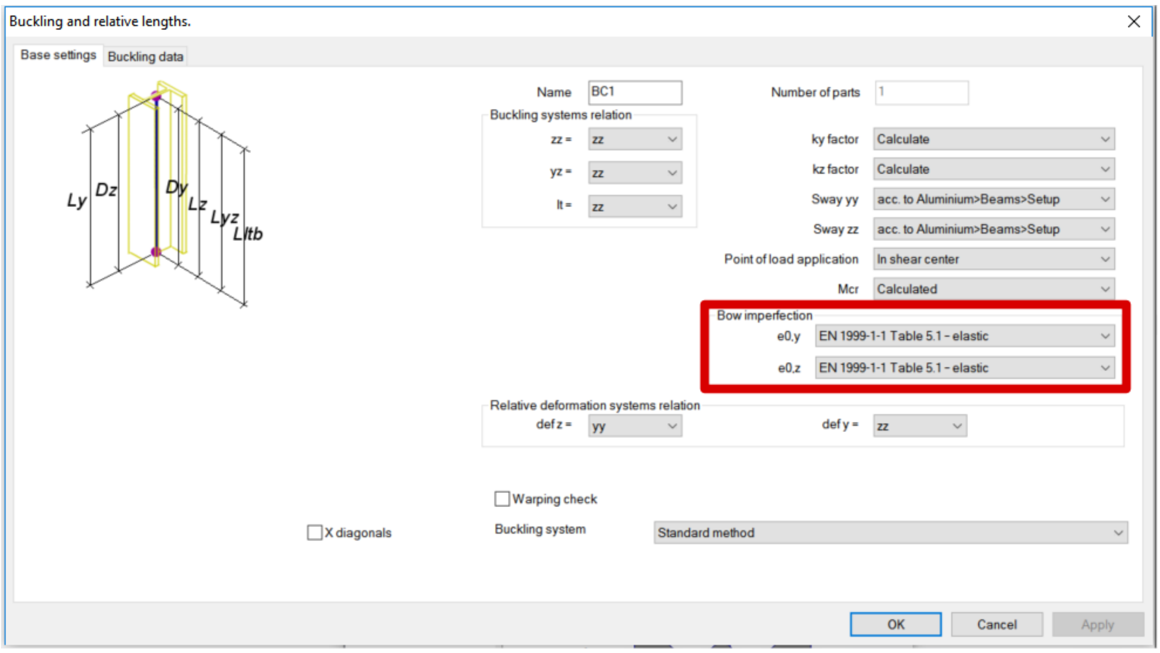
Where L is the member length.
SCIA Engineer can calculate the bow imperfection according to the code automatically for all needed members or the user can input values for $\mathbf{e_0}$. This is done via 'Project data' > 'National Annex' > 'EN 1999: Design of aluminium structures' > 'EN 1999-1-1 (General structural rules)'.

In order to input Global and Bow imperfections in SCIA Engineer, the user has to select the functionality 'Nonlinearity' + 'Initial deformation and curvature' + '2nd Order – geometrical nonlinearity' in the 'Project data'. Only after doing this the input of a non-linear function is possible.



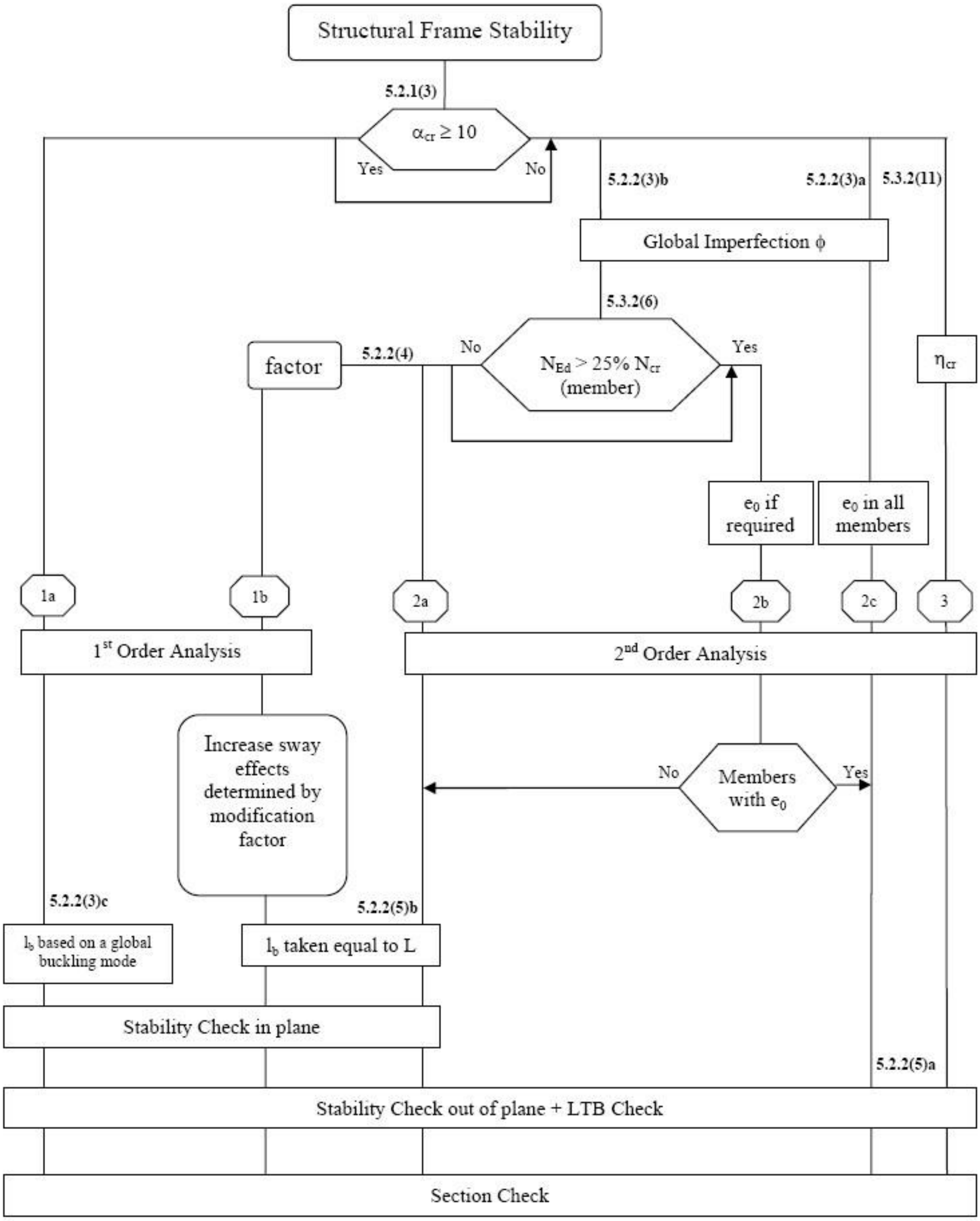


By selecting a specific member, the user can adjust the property “Buckling and relative length” for inputting the Bow imperfection.



The buckling curve used for calculation of the imperfection is the curve indicated in the material properties.

The general procedure for EC-EN1999 is shown in the following diagram.

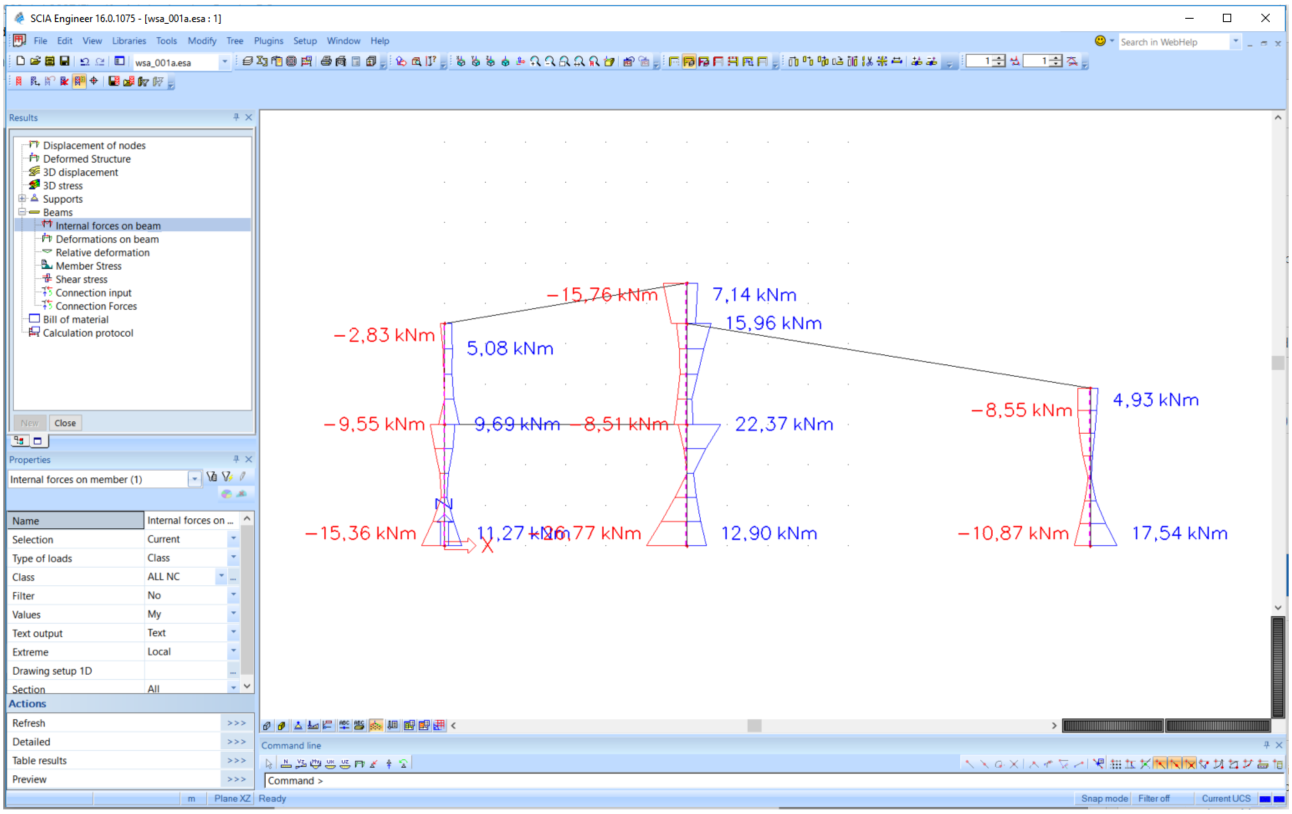


With:	η_{cr}	Elastic critical buckling mode.
	L	Member system length
	l_b	Buckling Length

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.
Path 2 specifies the "equivalent sway method". In further analysis a buckling factor smaller than 1 may be justified.

Example

wsa_001 global analysis (and wsa_001a.esa)
Method 2c according to EC-EN is used - Set ULS combinations - Set non-linear ULS combinations with: global imperfection = according to the code bow imperfection = according to the buckling data - Non-linear calculation using Timoshenko



The bow imperfection can be visualized through 'Aluminium' > 'Beams' > 'Slenderness data'.

Slenderness data

Linear calculation

Member	CS Name	Part	Sway y Sway z	Ly [m] Lz [m]	ky [-] kz [-]	ly [m] lz [m]	Lam y [-] Lam z [-]	e0,y [mm] e0,z [mm]	lyz [m]	l LTB [m]
B1	column A	1	Yes	3,000	1,13	3,387	56,38	10,0	5,500	5,500
			No	5,500	1,00	5,500	196,15	18,3		
B1	column A	2	Yes	2,500	1,45	3,616	60,19	8,3	5,500	5,500
			No	5,500	1,00	5,500	196,15	18,3		
B2	column B	1	Yes	3,000	1,27	3,798	50,49	10,0	6,500	6,500
			No	6,500	1,00	6,500	**213,76**	21,7		
B2	column B	2	Yes	2,500	1,84	4,598	61,12	8,3	6,500	6,500
			No	6,500	1,00	6,500	**213,76**	21,7		
B2	column B	3	Yes	1,000	2,00	2,003	26,64	3,3	6,500	6,500
			No	6,500	1,00	6,500	**213,76**	21,7		
B3	column C	1	Yes	3,900	1,02	3,975	66,17	13,0	3,900	3,900
			No	3,900	1,00	3,900	139,09	13,0		

According to Table 3.2 (Ref.[1]).
Buckling class according to material = EN-AW 6082 (Sheet) T6/T651 (0-6) → A

- Column B1: L_1 = 2500mm → e_0 = 1/300 * 2500 = 8,3mm
- Column B1: L_2 = 3000mm → e_0 = 1/300 * 3000 = 10,0mm

- Column B2: L_1 = 3000mm → e_0 = 1/300 * 3000 = 10,0mm
- Column B2: L_2 = 2500mm → e_0 = 1/300 * 2500 = 8,3mm
- Column B2: L_3 = 1000mm → e_0 = 1/300 * 1000 = 3,3mm

- Column B3: L_1 = 3900mm → e_0 = 1/300 * 3900 = 13,0mm
- Column B3: L_2 = 3900mm → e_0 = 1/300 * 3900 = 13,0mm

Initial shape, classification and reduced shape

Initial shape

For a cross-section with material Aluminium, the Initial Shape can be defined. For a General Cross-section, the 'Thinwalled representation' has to be used to be able to define the Initial Shape.
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

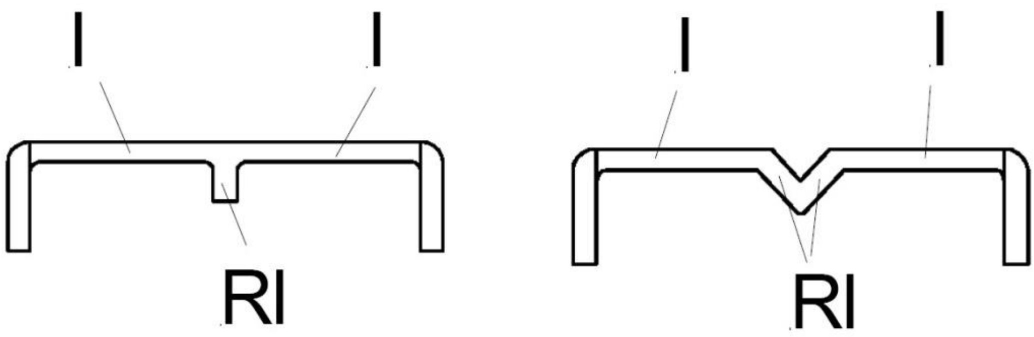
A part of the cross-section can also be considered as reinforcement:

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

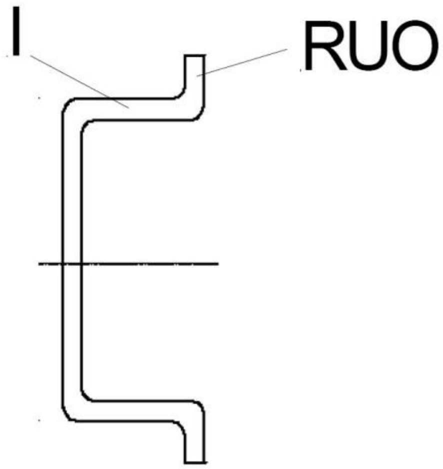
In case a part is specified as reinforcement, a reinforcement ID can be inputted. Parts having the same reinforcement ID are considered as one reinforcement.

The following conditions apply for the use of reinforcement:

- RI: There must be a plate type I on both sides of the RI reinforcement.



- RUO: The reinforcement is connected to only one plate with type I.



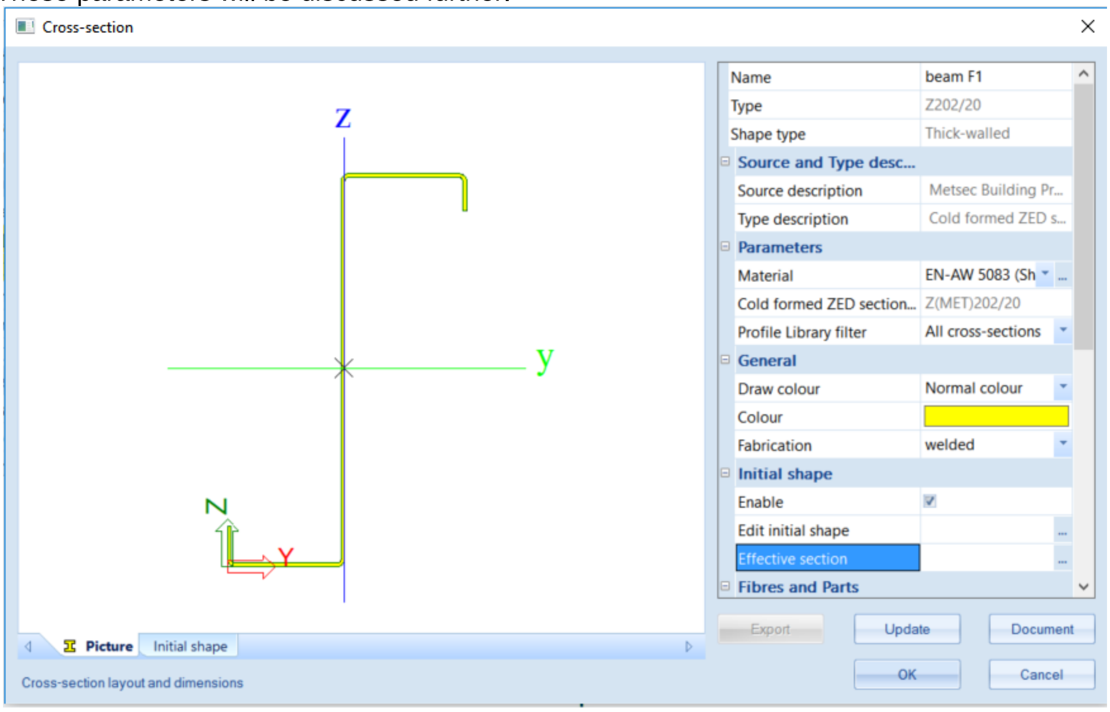
For standard cross-sections, the default type and reinforcement can be found in (Ref.[1]). For non standard section, the user has to evaluate the different parts in the cross-section.

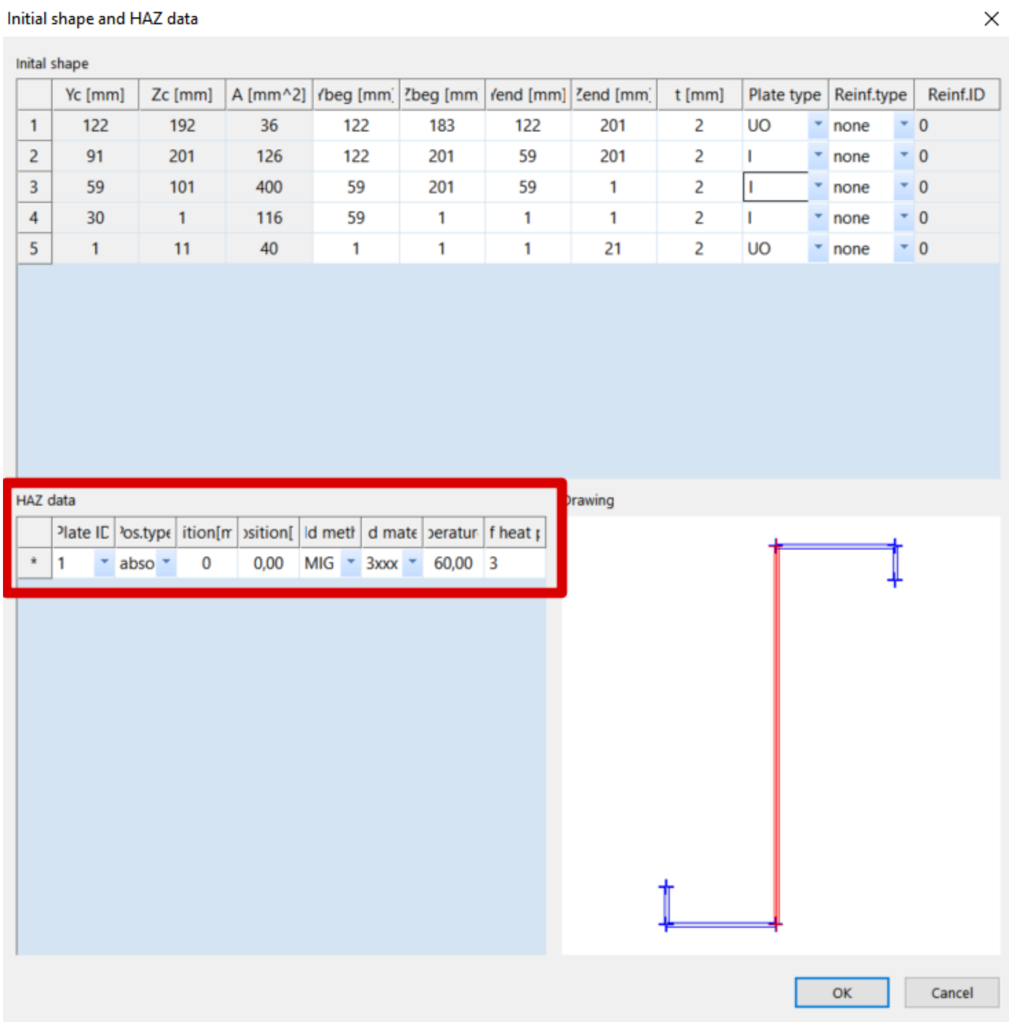
The Initial Shape can be inputted using ‘Cross-sections’ > ‘Edit’ > ‘Initial shape’. When this option is activated, the user can select ‘Edit initial shape’. In this box also welds (HAZ – Heath Affected Zone) can be inputted.

The parameters of the welds (HAZ) are:

- Plate ID
- Position
- Weld Method: MIG or TIG
- Weld Material: 5xxx and 6xxx or 7xxx
- Weld Temperature
- Number of heath paths

These parameters will be discussed further.





Classification

Four classes of cross-sections are defined, as follows (Ref.[1]):

- Class 1 cross-sections are those that can form a plastic hinge with the rotation capacity required for plastic analysis without reduction of the resistance.

- Class 2 cross-sections are those that can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.

- Class 3 cross-sections are those in which the calculated stress in the extreme compression fibre of the aluminium member can reach its proof strength, but local buckling is liable to prevent development of the full plastic moment resistance.

- Class 4 cross-sections are those in which local buckling will occur before the attainment of proof stress in one or more parts of the cross-section.

Classification for members with combined bending and axial forces is made for the loading components separately. No classification is made for the combined state of stress.

Classification is thus done for N, My and Mz separately. Since the classification is independent on the magnitude of the actual forces in the cross-section, the classification is always done for each component/part.

Taking into account the sign of the force components and the HAZ reduction factors, this leads to the following force components for which classification is done:

Compression force	N-
Tension force	N+ with $\rho_{0,HAZ}$
Tension force	N+ with $\rho_{u,HAZ}$
y-y axis bending	My-
y-y axis bending	My+
z-z axis bending	Mz-
z-z axis bending	Mz-

For each of these components, the reduced shape is determined and the effective section properties are calculated.

The following procedure is applied for determining the classification of a part:

- Step 1: calculation of stresses:
  For the given force component (N, My, Mz) the normal stress is calculated over the rectangular plate part for the initial (geometrical) shape.

- Step 2: determination of stress gradient over the plate part.

- Step 3: calculation of slenderness:

• Depending on the stresses and the plate type, the slenderness parameter β is calculated. Used formulas can be found in (Ref.[1]).
  if $\beta \leq \beta_1$: class 1
  if $\beta_1 < \beta \leq \beta_2$: class 2
  if $\beta_2 < \beta \leq \beta_3$: class 3
  if $\beta_3 < \beta$: class 4

  Values for β_1, β_2 and β_3 are according to Table 6.2 of (Ref.[1]):

Material classification according to Table 3.2	Internal part			Outstand part		
	β_1/ε	β_2/ε	β_3/ε	β_1/ε	β_2/ε	β_3/ε
Class A, without welds	11	16	22	3	4,5	6
Class A, with welds	9	13	18	2,5	4	5
Class B, without welds	13	16,5	18	3,5	4,5	5
Class B, with welds	10	13,5	15	3	3,5	4
$\varepsilon = \sqrt{250/f_o}$, f_o in N/mm^2						

Reduced Shape

The gross-section properties are used to calculate the internal forces and deformations.
The reduced shape is used for the Aluminium Code Check and is based on 3 reduction factors:

- **ρ_c** : reduction factor due to 'Local Buckling' of a part of the cross-section. For a cross-section part under tension or with classification different from Class 4, the reduction factor ρ_c is taken as 1,00.
- χ (Kappa): reduction factor due to 'Distortional Buckling'.
- **ρ_{HAZ}**: reduction factor due to HAZ effects.

Reduction factor ρ_c for local buckling

In case a cross-section part is classified as Class 4 (slender), the reduction factor ρ_c for local buckling is calculated according to art. 6.1.5 Ref.[1]:

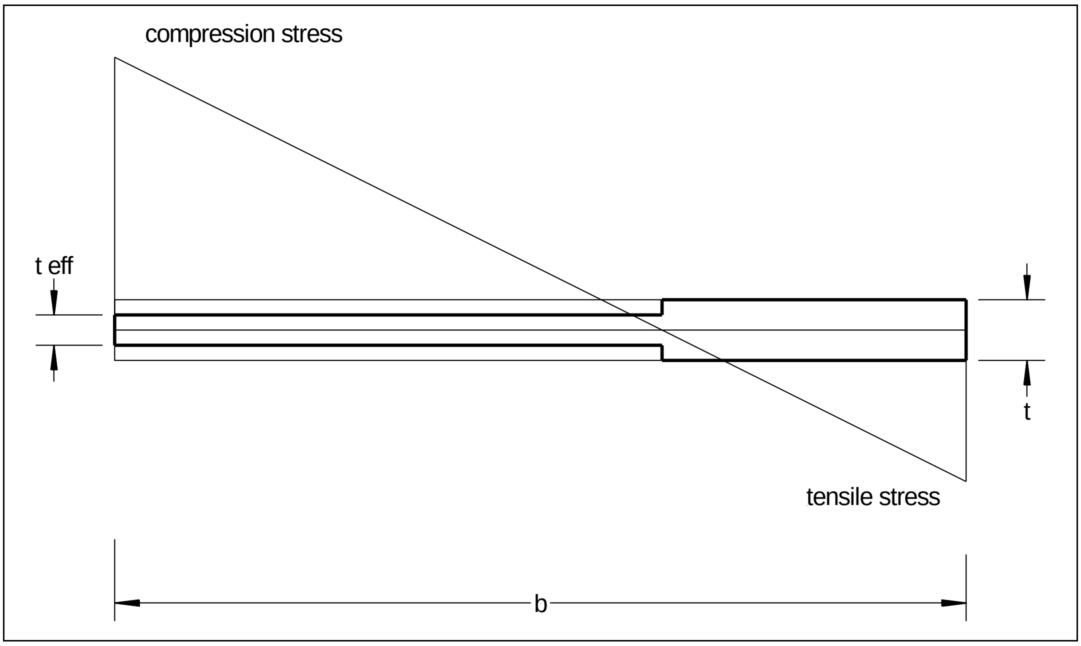
$$\rho_c = \frac{C_1}{(\beta/\varepsilon)} - \frac{C_2}{(\beta/\varepsilon)^2}$$

Table 6.3 - Constants C_1 and C_2 in expressions for ρ_c

Material classification according to Table 3.2	Internal part		Outstand part	
	C_1	C_2	C_1	C_2
Class A, without welds	32	220	10	24
Class A, with welds	29	198	9	20
Class B, without welds	29	198	9	20
Class B, with welds	25	150	8	16

For a cross-section part under tension or with classification different from Class 4 the reduction factor ρ_c is taken as 1,00.

In case a cross-section part is subject to compression and tension stresses, the reduction factor ρ_c is applied only to the compression part as illustrated in the following figure.



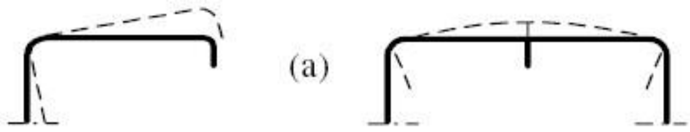
Reduction factor χ (Kappa) for distortional buckling

In SCIA Engineer a general procedure is used according to Ref.[2] p66.
The design of stiffened elements is based on the assumption that the stiffener itself acts as a beam on elastic foundation, where the elastic foundation is represented by a spring stiffness depending on the transverse bending stiffness of adjacent parts of plane elements and on the boundary conditions of these elements.

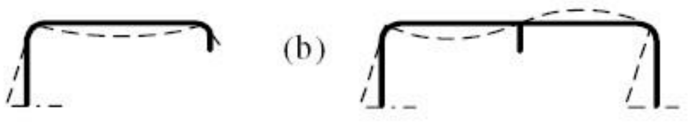
The effect of 'Local and Distortional Buckling' is explained as follows (Ref.[1]):
When considering the susceptibility of a reinforced flat part to local buckling, three possible buckling modes should be considered.

The modes are:

a) Mode 1: the reinforced part buckles as a unit, so that the reinforcement buckles with the same curvature as the part. This mode is often referred to as Distortional Buckling (Figure (a)).



b) Mode 2: the sub-parts and the reinforcement buckle as individual parts with the junction between them remaining straight. This mode is referred as Local Buckling (Figure (b)).



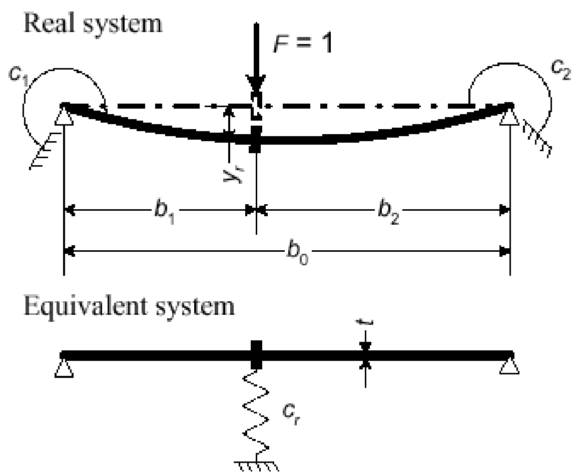
c) Mode 3: this is a combination of Modes 1 and 2 in which sub-part buckles are superimposed on the buckles of the whole part.

The following procedure is applied for calculating the reduction factor for an intermediate stiffener (RI) or edge stiffener (RUO):

Step 1) Calculation of spring stiffness
Step 2) Calculation of Area and Second moment of area
Step 3) Calculation of stiffener buckling load
Step 4) Calculation of reduction factor for distortional buckling

Step 1: Calculation of spring stiffness

Spring stiffness **c = c_r** for RI:



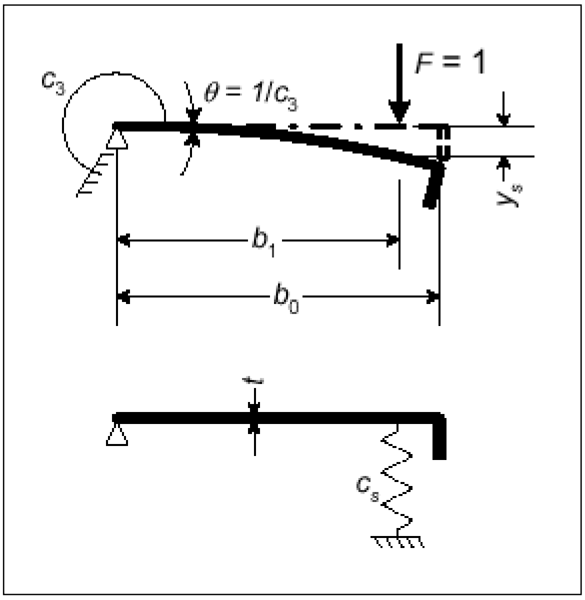
Spring stiffness

$$c_r = \frac{1}{y_r} \quad \text{where}$$

$$y_r = \frac{4(1-\nu^2)\,b_1^2 b_2^2}{E\,t^3(b_1+b_2)}$$

if $c_1 = c_2 = 0$

Spring stiffness **c = c_s** for RUO:



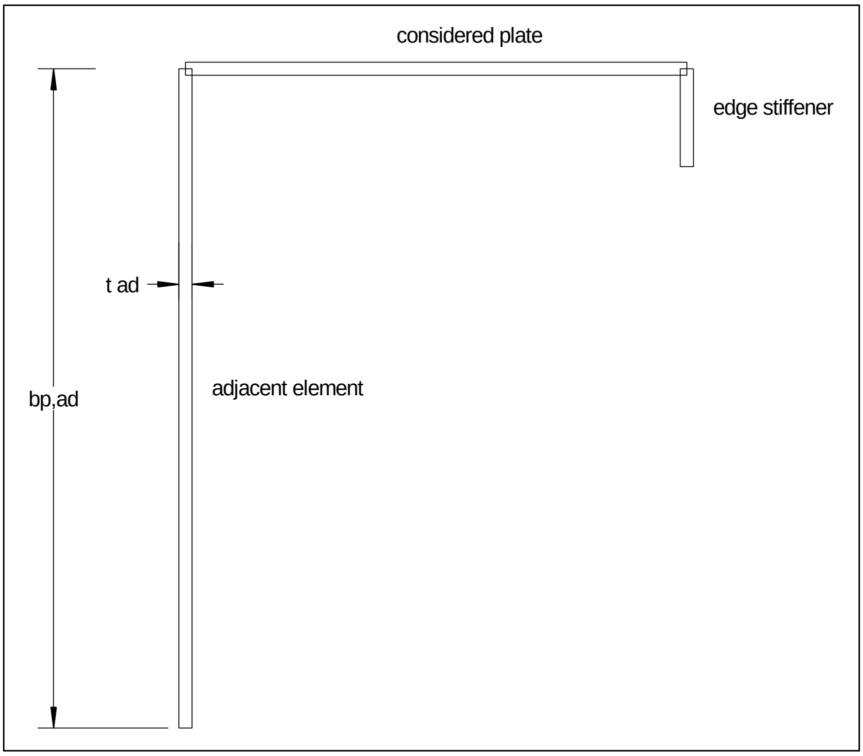
$$c = c_s = \frac{1}{y_s}$$

$$y_s = \frac{4(1-\nu^2)b_1^3}{Et^3} + \frac{b_1^2}{c_3}$$

$$c_3 = \sum \frac{\alpha E t_{ad}^3}{12(1-\nu^2)b_{p,ad}}$$

With:	t_{ad}	Thickness of the adjacent element
	$b_{p,ad}$	Flat width of the adjacent element
	c_3	The sum of the stiffnesses from the adjacent elements
	α	equal to 3 in the case of bending moment load or when the cross section is made of more than 3 elements (counted as plates in initial geometry, without the reinforcement parts) equal to 2 in the case of uniform compression in cross sections made of 3 elements (counted as plates in initial geometry, without the reinforcement parts, e.g. channel or Z sections)

These parameters are illustrated on the following picture:



Step 2: Calculation of Area and Second moment of area

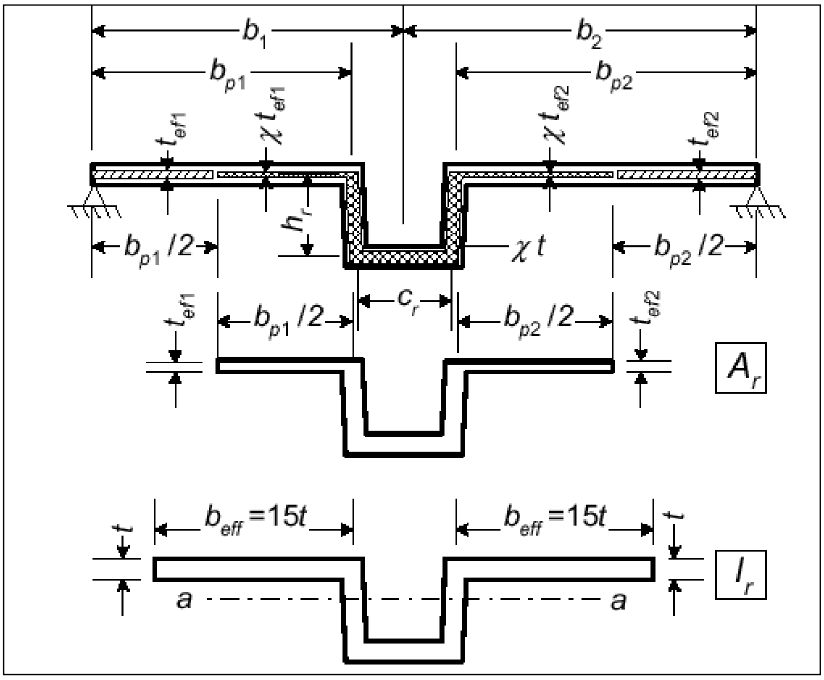
After calculating the spring stiffness the area **Ar** and Second moment of area **Ir** are calculated.

With:

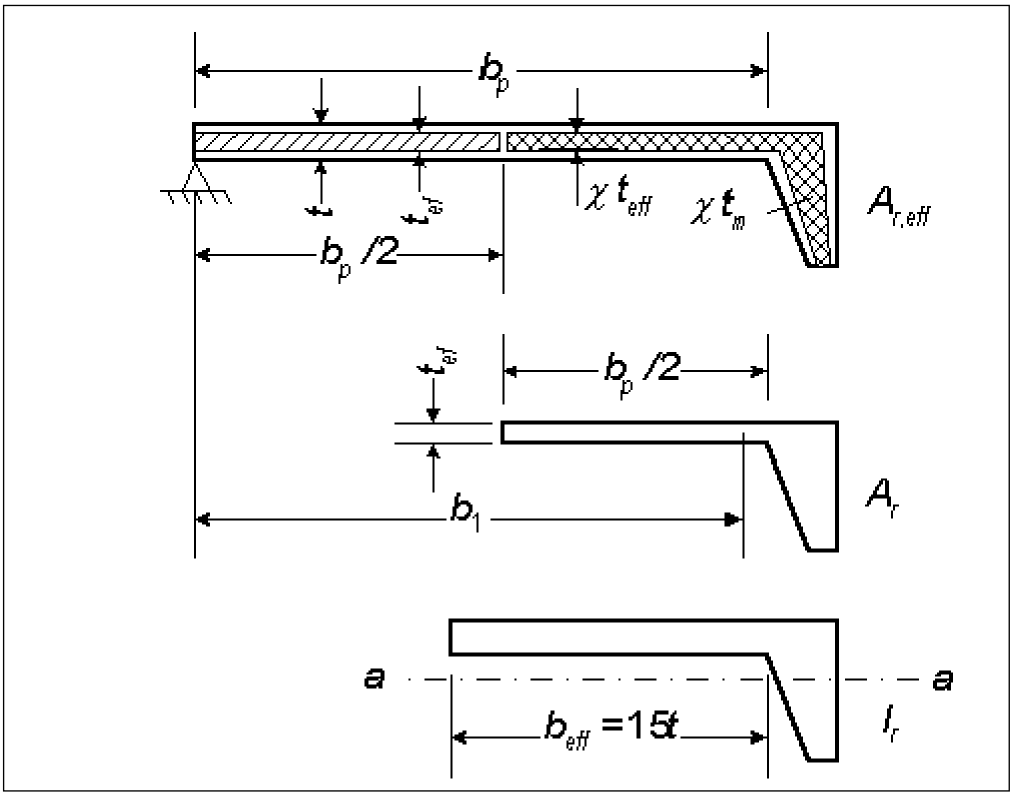
With:	Ar	the area of the effective cross section (based on $t_{eff} = \rho_c\ t$) composed of the stiffener area and half the adjacent plane elements
	Ir	the second moment of area of an effective cross section composed of the (unreduced) stiffener and part of the adjacent plate elements, with thickness t and effective width $\mathbf{b_{eff}}$, referred to the neutral axis a-a
	b_{eff}	For RI reinforcement taken as 15 t For ROU reinforcement taken as 12 t

These parameters are illustrated on the following figures.

Ar and **Ir** for RI:



Ar and **Ir** for RUO:



Step 3: Calculation of stiffener buckling load

The buckling load $\mathbf{N_{r,cr}}$ of the stiffener can now be calculated as follows:

$$N_{r,cr} = 2\sqrt{cEI_r}$$

With:	c	Spring stiffness of Step 1
	E	Module of Young
	Ir	Second moment of area of Step 2

Step 4: Calculation of reduction factor for distortional buckling

Using the buckling load $\mathbf{N_{r,cr}}$ and area **Ar** the relative slenderness λ_c can be determined for calculating the reduction factor χ:

$$\lambda_c = \sqrt{\frac{f_o A_r}{N_{r,cr}}}$$

$$\alpha = 0.20$$

$$\lambda_0 = 0.60$$

$$\phi = 0.50(1.0 + \alpha(\lambda_c - \lambda_0) + \lambda_c^2)$$

$$if \quad \lambda_c < \lambda_0 \quad => \chi = 1.00$$

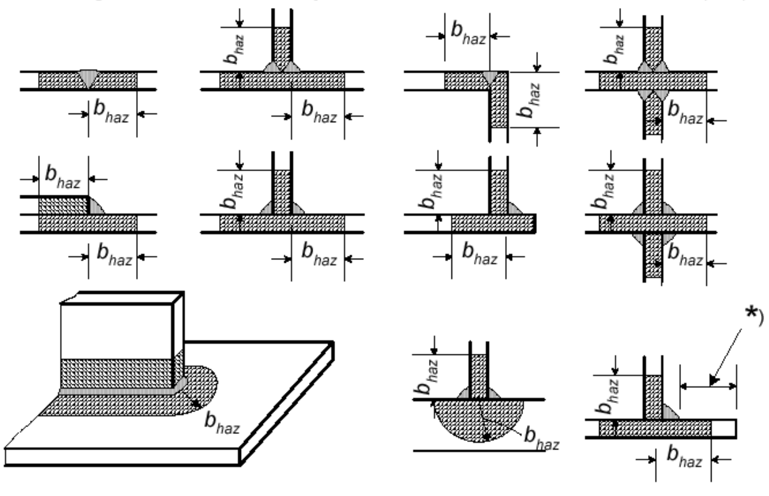
$$if \quad \lambda_c \geq \lambda_0 \quad => \chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}} \leq 1.00$$

With:	f_0	0,2% proof strength
	λ_c	Relative slenderness
	λ_0	Limit slenderness taken as 0,60
	α	Imperfection factor taken as 0,20
	χ	Reduction factor for distortional buckling

The reduction factor is then applied to the thickness of the reinforcement(s) and on half the width of the adjacent part(s).

Reduction factor ρ_{HAZ} for weld effects

The extent of the Heat Affected Zone (HAZ) is determined by the distance b_{haz} according to art.6.1.6.3 of Ref.[1].



*) If this distance is less than $3b_{haz}$ assume that the HAZ extends to the full width of outstand.

The value for $\mathbf{b_{haz}}$ is multiplied by the factors α_2 and 3/n:

For 5xxx & 6xxx alloys: $\alpha_2 = 1 + \frac{(T1 - 60)}{120}$

For 7xxx alloys: $\alpha_2 = 1 + 1.5\frac{(T1 - 60)}{120}$

With:	T1	Interpass temperature
	n	Number of heat paths

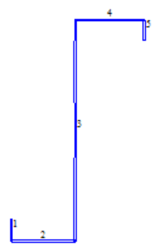
<u>Note:</u>
The variations in numbers of heath paths 3/n is specifically intended for fillet welds. In case of a butt weld the parameter n should be set to 3 (instead of 2) to negate this effect.

The reduction factor for the HAZ is given by:

$$\rho_{u,haz} = \frac{f_{u,haz}}{f_u}$$

$$\rho_{o,haz} = \frac{f_{o,haz}}{f_o}$$

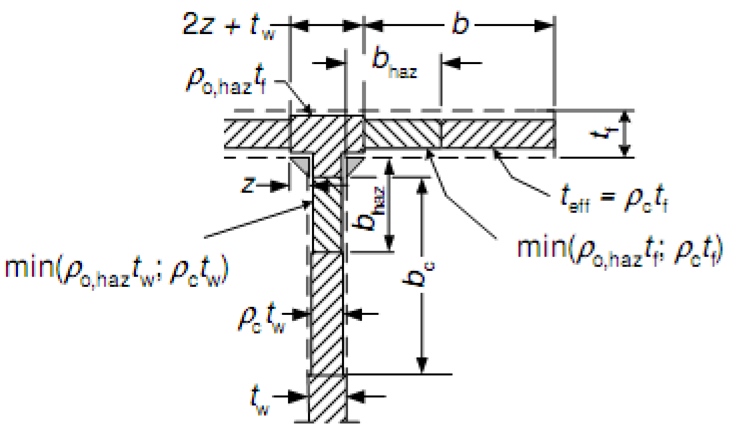
By editing a profile in SCIA Engineer, the user can evaluate for each component (N, My and Mz) the determined classification and reduction factors via the option ‘Run analysis’.



Parts	Id	Psi	Sigma Beg [kN/m²]	Sigma End [kN/m²]	C1	C2	Beta	Beta1	Beta2	Beta3	Class	Beg. x [mm]	End x [mm]	Ro c	Chi	Ro haz	Ro	Reinf. ID	Ar [mm²]	Ir [mm⁴]
	1	0.000	0.000	0.000	9.000	20.000	10.000	2.761	4.417	5.522	4	0.00	20.00	1.000	1.000	1.000	1.000	0	0.00	0.00
	2	0.000	0.000	0.000	29.000	198.000	20.300	9.939	14.356	19.878	4	0.00	58.00	1.000	1.000	1.000	1.000	0	0.00	0.00
	3	0.000	0.000	0.000	29.000	198.000	70.000	9.939	14.356	19.878	4	0.00	75.00	1.000	1.000	1.000	1.000	0	0.00	0.00
												75.00	125.00	1.000	1.000	0.610	0.610			
												125.00	200.00	1.000	1.000	1.000	1.000			
	4	0.000	0.000	0.000	29.000	198.000	22.050	9.939	14.356	19.878	4	0.00	63.00	1.000	1.000	1.000	1.000	0	0.00	0.00
	5	0.000	0.000	0.000	9.000	20.000	6.300	2.761	4.417	5.522	4	0.00	18.00	1.000	1.000	1.000	1.000	0	0.00	0.00

Calculation of the effective properties

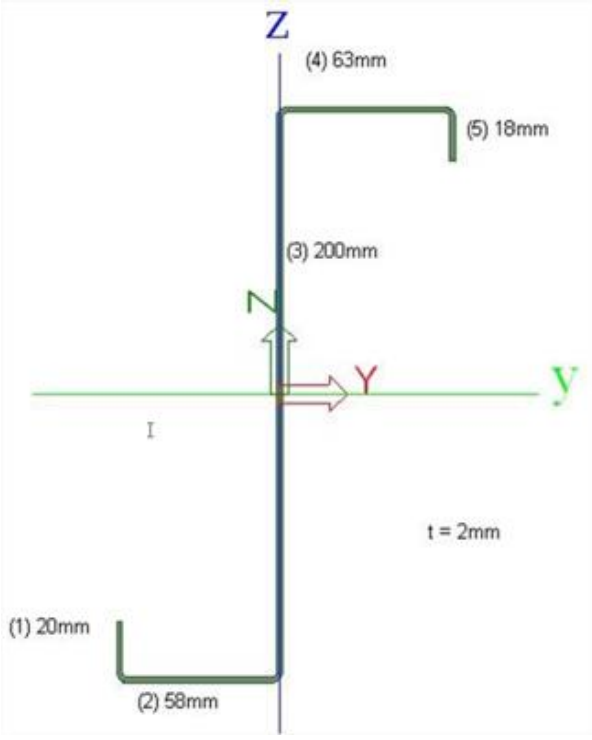
For each part the final thickness reduction ρ is determined as the minimum of $\chi.\rho_c$ and ρ_{haz}.



The section properties are then recalculated based on the reduced thicknesses.

Worked example

Example wsa_002
In this example, a manual check is made for a cold formed ZED section (lipped Z-section).
A simple supported beam with a length of 6m is modelled. The cross-section is taken from the profile library: Z(MET) 202/20 .
The dimensions are indicated:



The material properties are as indicated in EC-EN1999: EN-AW 6082 T61/T6151 (0- 12.5):
f_0 = 205 N/mm², $f_{0,HAZ}$ = 125 N/mm²
f_u = 280 N/mm², $f_{u,HAZ}$ = 280 N/mm²
Buckling Curve: A
Fabrication: welded

A weld is made in the middle of part (3). The parameters of this weld are:

- MIG- weld
- 6xxx alloy
- Interpass temperature = 90°

The 5 parts of the cross-section (type) are as indicated by SCIA Engineer:

Inital shape

	Yc [mm]	Zc [mm]	A [mm^2]	Ybeg [mm]	Zbeg [mm]	Yend [mm]	Zend [mm]	t [mm]	Plate type	Reinf.type	Reinf.ID
1	-58,20	-89,90	40,00	-58,20	-79,90	-58,20	-99,90	2,00	UO	RUO	0
2	-29,20	-99,95	116,00	-58,20	-99,90	-0,20	-100,00	2,00	I	none	0
3	0,00	0,00	400,00	-0,20	-100,00	0,20	100,00	2,00	I	none	0
4	31,70	99,95	126,00	0,20	100,00	63,20	99,90	2,00	I	none	0
5	63,20	90,90	36,00	63,20	99,90	63,20	81,90	2,00	UO	RUO	0

The manual calculation is done for compression (N-).

Classification

According to 6.1.4 Ref.[1]:
ψ = stress gradient = 1 (compression in all parts)

=> $\varepsilon = \sqrt{\dfrac{250}{f_0}} = \sqrt{\dfrac{250}{205}} = 1{,}104$

=> $\eta = 0{,}70 + 0{,}30\psi = 1$

For all parts with no stress gradient (6.1.4.3 Ref.[1]):
$\beta = b/t$

Part	Type	b	t	β
1	RUO	20	2	10
2	I	58	2	29
3	I	200	2	100
4	I	63	2	31,5
5	RUO	18	2	9

Next, the boundaries for class 1, 2 and 3 are calculated according to 6.1.4.4 and Table 6.2 Ref.[1]. Boundaries β_1, β_2 and β_3 are depended on the buckling class (A or B), the presence of longitudinal welds and the type (internal/outstand part).

Part	Type	β_1/ε	β_2/ε	β_3/ε	β_1	β_2	β_3	classification
1	RUO	3	4,5	6	3,31	4,97	6,62	4
2	I	11	16	22	12,14	17,66	24,29	4
3	I	9	13	18	9,94	14,36	19,88	4
4	I	11	16	22	12,14	17,66	24,29	4
5	RUO	3	4,5	6	3,31	4,97	6,62	4

Reduction factor ρ_c for local buckling

ρ_c is calculated according to 6.1.5 and Formulas (6.11) and (6.12) Ref.[1] (all parts class 4):

$$\rho_c = \frac{C_1}{(\beta/\varepsilon)} - \frac{C_2}{(\beta/\varepsilon)^2}$$

Part	β	C_1	C_2	ρ_c
1	10	10	24	0,811
2	29	32	220	0,899
3	100	29	198	0,296
4	31,5	32	220	0,851
5	9	10	24	0,866

Reduction factor χ for distortional buckling

Distortional buckling has to be calculated for Part 1-2 and Part 4-5.

Part 1-2

Step1: calculation of spring stiffness

$$c = c_s = \frac{1}{y_s}$$

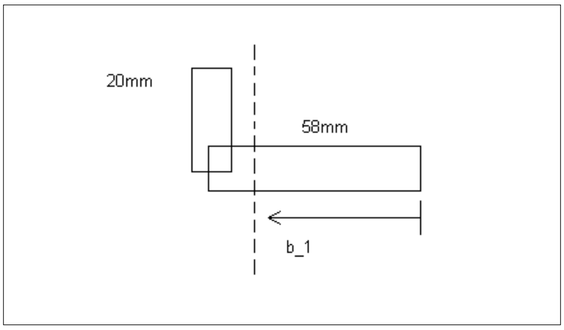
$$y_s = \frac{4(1-\nu^2)b_1^3}{Et^3} + \frac{b_1^2}{c_3}$$

$$c_3 = \sum \frac{\alpha E t_{ad}^3}{12(1-\nu^2)b_{p,ad}}$$

With: $\alpha = 3$ want meer dan drie delen
$E = 70000$ N/mm²
$v = 0,3$
$t_{ad} = 2$ mm
$b_{p,ad} = 200$ mm (lengte van deel 3)

Thus this gives:

$$c_3 = \frac{2 \times 70000 \times 2^3}{12(1-0,3^2) \times 200} = 512,82 Nrad$$



$$b_1 = \frac{(58 \times 2) \times 58/2 + (20 \times 2) \times 58}{(58 \times 2) + (20 \times 2)} = 36,44 mm$$

$$y_s = \frac{4 \times (1-0,3^2) \times 36,44^3}{70000 \times 2^3} + \frac{36,44^2}{512,82} = 2,903 mm^2 / N$$

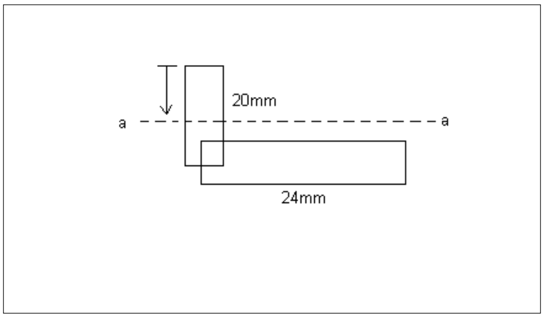
$$c = c_s = \frac{1}{y_s} = \frac{1}{2,903} = 0,344 N / mm^2$$

Step2: calculation of Area and Second moment of area

=> half of the adjacent member = $\frac{58}{2} mm$

ρ_c for Part (2) = 0,899

$$A_r = 20 \times 2 + \frac{58}{2} \times 2 \times 0,899 = 92,142 mm^2$$



b_{eff} = For RUO reinforcement taken as 12xt
t = 2mm

=> b_{eff} = 24mm

$$y = \frac{(20\times 2)\times 20/2 + (24\times 2)\times 20}{(20\times 2)+(24\times 2)} = 15{,}45mm$$

$$I_r = \frac{2\times 20^3}{12} + (20\times 2)\times(15{,}45 - \frac{20}{2})^2 + \frac{24\times 2^3}{12} + (24\times 2)\times(20-15{,}45)^2 = 3531{,}15mm^4$$

Step3: calculation of stiffener buckling load

$$N_{r,cr} = 2\times\sqrt{c\times E\times I_r} = 2\times\sqrt{0{,}344\times 70000\times 3531{,}15} = 18454{,}4N$$

$$\lambda_c = \sqrt{\frac{f_0\times A_r}{N_{r,cr}}} = \sqrt{\frac{205\times 92{,}142}{18454{,}4}} = 1{,}0117$$

$$\alpha = 0{,}2$$

$$\lambda_0 = 0{,}60$$

$$\Rightarrow \lambda_0 > \lambda_0$$

$$\Rightarrow \phi = 0{,}50\times(1+0{,}2\times(1{,}0117-0{,}6)+1{,}0117^2) = 1{,}0529$$

$$\Rightarrow \chi = \frac{1}{\phi+\sqrt{\phi^2-\lambda_c^2}} = 0{,}743$$

Kappa = reduction factor for distortional buckling

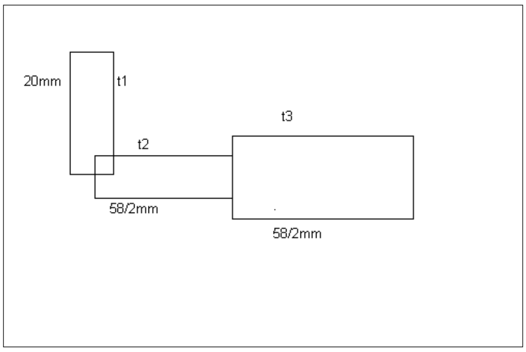
Calculation of effective thickness

t_1, t_2 and t_3 are the thicknesses Part (1) and (2)

$$t_1 = 2\times\rho_c\times\chi = 2\times 0{,}811\times 0{,}743 = 1{,}205mm$$

$$t_2 = 2\times\rho_c\times\chi = 2\times 0{,}899\times 0{,}743 = 1{,}336mm$$

$$t_3 = 2\times\rho_c = 2\times 0{,}899 = 1{,}798mm$$



Part 4-5

Step1: calculation of spring stiffness

$$c = c_s = \frac{1}{y_s}$$

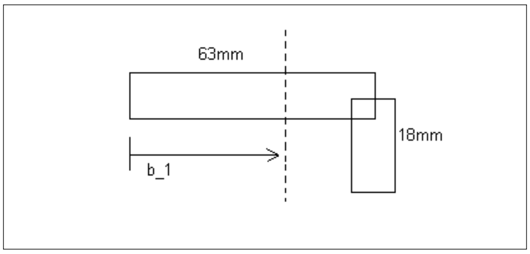
$$y_s = \frac{4(1-\nu^2)b_1^3}{Et^3} + \frac{b_1^2}{c_3}$$

$$c_3 = \sum \frac{\alpha E t_{ad}^3}{12(1-\nu^2)b_{p,ad}}$$

With: α = 3
E = 70000 N/mm²
ν = 0,3
t_{ad} = 2 mm
$b_{p,ad}$ = 200 mm (thickness of Part 3)

Thus this gives:

$$c_3 = \frac{2 \times 70000 \times 2^3}{12(1-0{,}3^2) \times 200} = 512{,}82 Nrad$$



$$b_1 = \frac{(63 \times 2) \times {}^{63}\!/_{2} + (18 \times 2) \times 63}{(63 \times 2) + (18 \times 2)} = 38{,}5mm$$

$$y_s = \frac{4 \times (1-0{,}3^2) \times 368{,}5^3}{70000 \times 2^3} + \frac{38{,}5^2}{512{,}82} = 3{,}2613 mm^2 / N$$

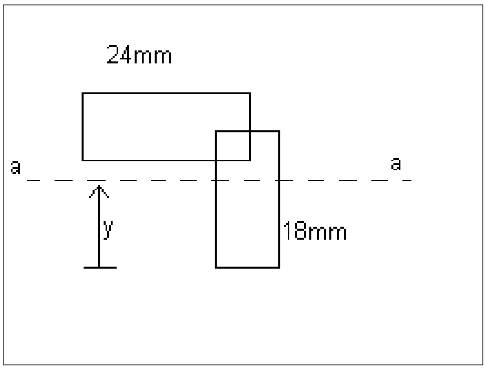
$$c = c_s = \frac{1}{y_s} = \frac{1}{3{,}26} = 0{,}3066 N / mm^2$$

Step2: calculation of Area and Second moment of area

=> half of the adjacent member = $\frac{63}{2} mm$

ρ_c for Part (4) = 0, 851

$$A_r = 18 \times 2 + \frac{63}{2} \times 2 \times 0{,}851 = 89{,}613 mm^2$$



b_{eff} = For RUO reinforcement taken as 12xt
t = 2mm

=> b_{eff} = 24mm

$$y = \frac{(24\times 2)\times 18 + (18\times 2)\times \frac{18}{2}}{(24\times 2)+(18\times 2)} = 14{,}14mm$$

$$I_r = \frac{24\times 2^3}{12} + (24\times 2)\times(18-14{,}14)^2$$
$$+\frac{2\times 18^3}{12} + (18\times 2)\times(14{,}14-\frac{18}{2})^2 = 2654{,}29mm^4$$

Step3: calculation of stiffener buckling load

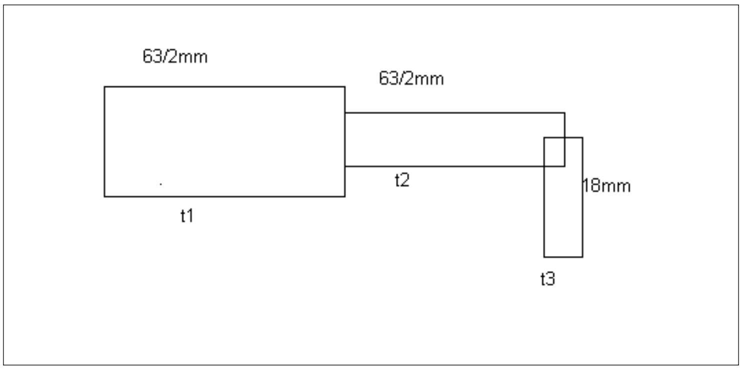
$$N_{r,cr} = 2\times\sqrt{c\times E\times I_r} = 2\times\sqrt{0{,}3066\times 70000\times 2654{,}29} = 15095{,}8N$$
$$\lambda_c = \sqrt{\frac{f_0\times A_r}{N_{r,cr}}} = \sqrt{\frac{205\times 89{,}613}{15095{,}8}} = 1{,}103$$

$$\alpha = 0{,}2$$
$$\lambda_0 = 0{,}60$$
$$\Rightarrow \lambda_0 > \lambda_0$$
$$\Rightarrow \phi = 0{,}50\times(1+0{,}2\times(1{,}103-0{,}6)+1{,}103^2) = 1{,}159$$
$$\Rightarrow \chi = \frac{1}{\phi+\sqrt{\phi^2-\lambda_c^2}} = 0{,}661$$

Kappa = reduction factor for distortional buckling

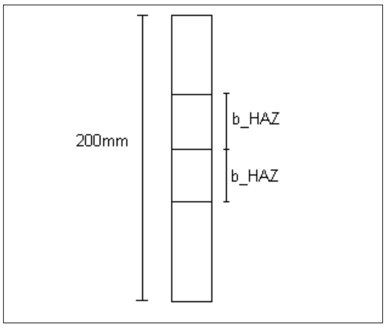
Calculation of effective thickness

t_1, t_2 and t_3 are the thicknesses Part (4) and (5)
$$t_1 = 2\times\rho_c = 2\times 0{,}851 = 1{,}702mm$$
$$t_2 = 2\times\rho_c\times\chi = 2\times 0{,}851\times 0{,}661 = 1{,}125mm$$
$$t_3 = 2\times\rho_c\times\chi = 2\times 0{,}866\times 0{,}661 = 1{,}145mm$$



Reduction factor ρ_{HAZ} for weld effects

The weld is situated in the middle of Part (3)



Data:
t = 2mm
MIG-weld:
Following Ref [1] 6.1.6.3:

(3) For a MIG weld laid on unheated material, and with interpass cooling to 60°C or less when multi-pass welds are laid, values of b_{haz} are as follows:

$0 < t \leq 6$ mm: $b_{haz} = 20$ mm
$6 < t \leq 12$ mm: $b_{haz} = 30$ mm
$12 < t \leq 25$ mm: $b_{haz} = 35$ mm
$t > 25$ mm: $b_{haz} = 40$ mm

$$0 < t \leq 6mm \Rightarrow b_{HAZ} = 20mm$$

Temperature (6xxx alloy):

$$\alpha_2 = 1 + \frac{90-60}{120} = 1{,}25$$

Thus this gives:

$$b_{HAZ} = 1{,}25 \times 20 = 25mm \Rightarrow HAZ - zone = 2 \times b_{HAZ} = 50mm$$

$$\rho_{0,HAZ} = \frac{f_{0,HAZ}}{f_0} = \frac{125}{205} = 0{,}610$$

ρ_c in Part (3) = 0,296.
This means that Local Buckling is limiting and not the HAZ-effect (ρ_{HAZ} = 0,61)

Thickness of Part (3):

$$t_1 = 2 \times \rho_c \times \chi = 2 \times 0{,}296 = 0{,}592$$

Calculation of effective Area for uniform compression (N-)

Part (1): $20 \times 1{,}205 = 24{,}1 mm^2$

Part (2): $58/2 \times 1{,}336 = 38{,}7 mm^2$

$58/2 \times 1{,}798 = 52{,}1 mm^2$

$75 \times 0{,}592 = 44{,}4 mm^2$

Part (3): $50 \times 0{,}592 = 29{,}6 mm^2$

$75 \times 0{,}592 = 44{,}4 mm^2$

Part (4): $\frac{63}{2} \times 1{,}702 = 53{,}6 mm^2$

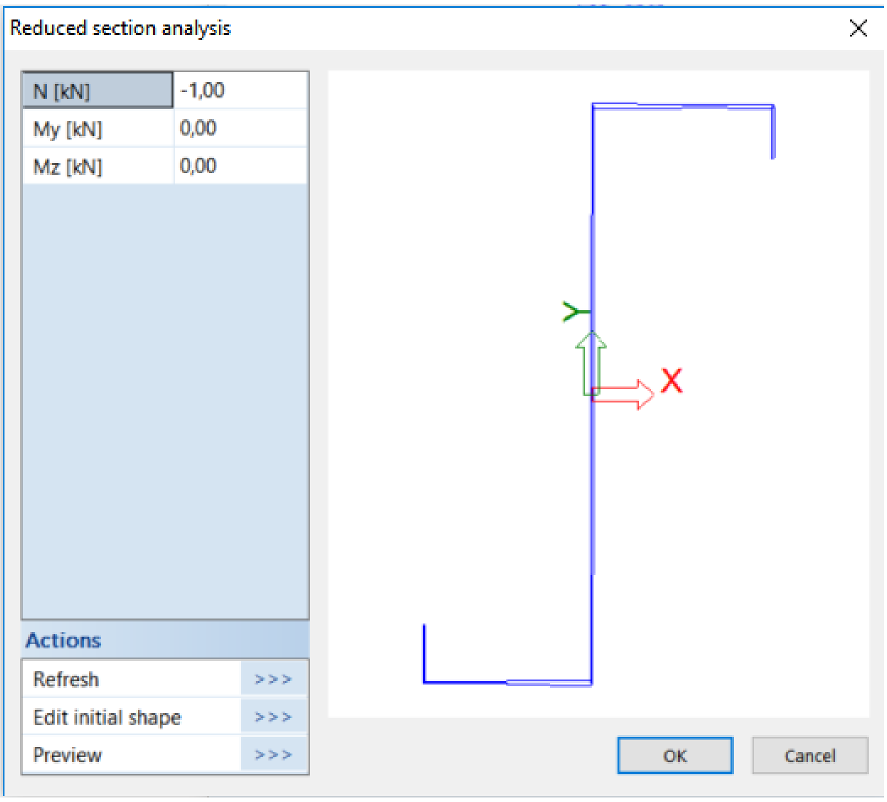
$\frac{63}{2} \times 1{,}064 = 35{,}4 mm^2$

Part (5): $18 \times 1{,}145 = 20{,}6 mm^2$

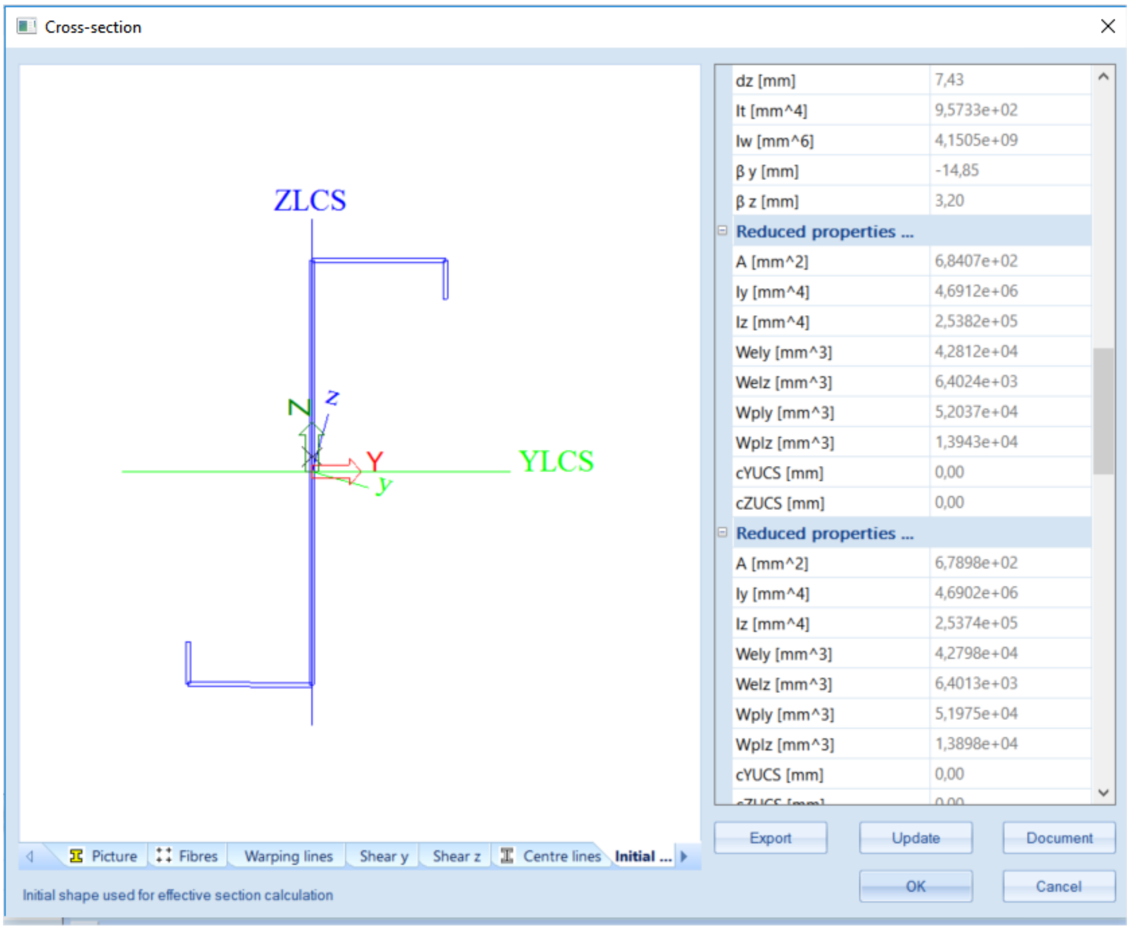
The total effective Area is the sum of the above values = 343 mm²

Comparison with SCIA Engineer

Via ‘Profile’ > ‘Edit’ > ‘Effective section’, the user can manually check the calculated classification, reduction factors and intermediate results.



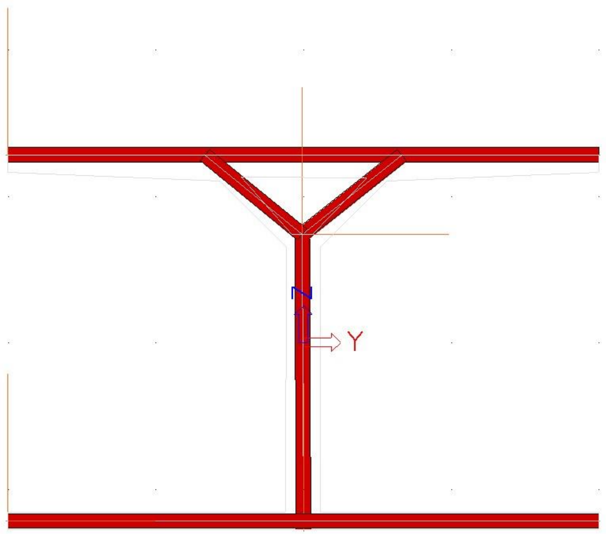
Parts	Id	Psi	Sigma_Beg [kN/m^2]	Sigma_End [kN/m^2]	C1	C2	Beta	Beta1	Beta2	Beta3	Class	Beg. x [mm]	End. x [mm]	Ro_c	Chi	Ro_haz	Ro	Reinf. ID	Ar [mm^2]	Ir [mm^4]
	1	1,000	-1392,755	-1392,755	10,000	24,000	10,000	3,313	4,969	6,626	4	0,00	20,00	0,812	0,429	1,000	0,348	0	92,17	887,51
	2	1,000	-1392,755	-1392,755	32,000	220,000	29,000	12,147	17,669	24,295	4	0,00	29,00	0,900	0,429	1,000	0,386	0	0,00	0,00
												29,00	58,00	0,900	1,000	1,000	0,900			
	3	1,000	-1392,755	-1392,755	29,000	198,000	100,000	9,939	14,356	19,878	4	0,00	75,00	0,296	1,000	1,000	0,296	0	0,00	0,00
												75,00	125,00	0,296	1,000	0,610	0,296			
												125,00	200,00	0,296	1,000	1,000	0,296			
	4	1,000	-1392,755	-1392,755	32,000	220,000	31,500	12,147	17,669	24,295	4	0,00	31,50	0,851	1,000	1,000	0,851	0	0,00	0,00
												31,50	63,00	0,851	0,579	1,000	0,493			
	5	1,000	-1392,755	-1392,755	10,000	24,000	9,000	3,313	4,969	6,626	4	0,00	18,00	0,866	0,579	1,000	0,501	0	89,64	1893,82



General Cross-section

Example

wsa_003 thinwalled cross-section
- read profile from DWG-file (dwg profile.dwg) - convert into thinwalled representation to be used in Aluminium Check. - set scale, select polylines, select opening, import, convert to thinwalled representation - only after this, reduced shape can be used



SLS check

Nodal displacement

Example

wsa_001a nodal displacement
- SLS combinations - Limit for horizontal deflection δ for Beam B1 is h/150 → 5500/150 = 36,7 mm - Maximum horizontal deformation = 21 mm < 36,7 mm

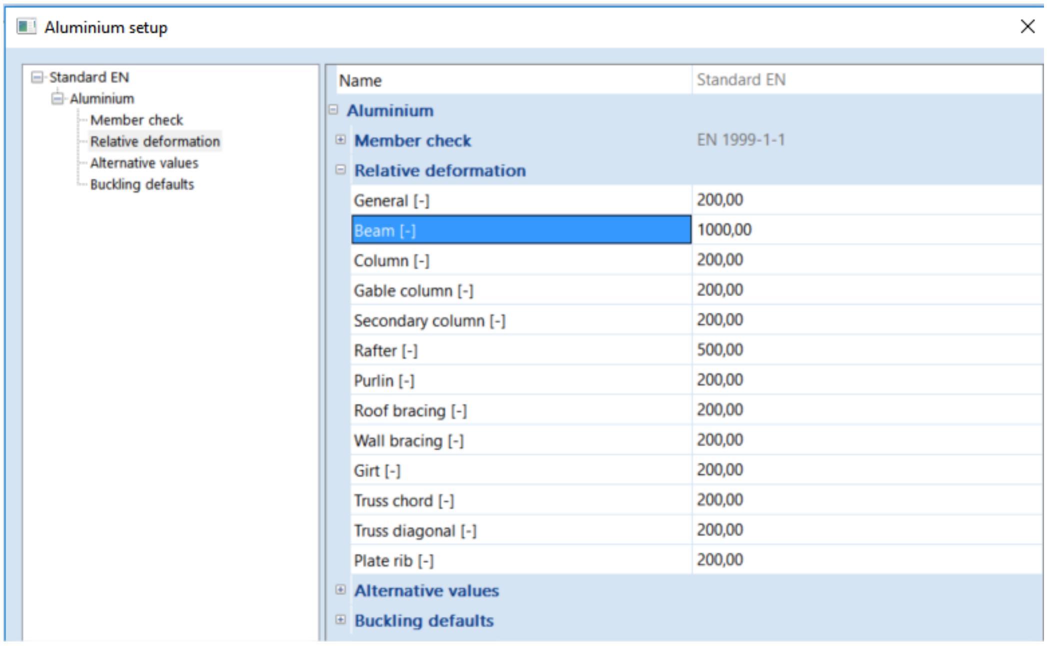
Displacement of nodes

Linear calculation, Extreme : Global
Selection : All
Combinations : SLS

Node	Case	Ux [mm]	Uz [mm]
N4	SLS/1	**-20,5**	0,3
N6	SLS/2	**21,1**	0,2
N4	SLS/3	0,3	**-0,3**
N4	SLS/4	-19,7	**0,3**

Relative deformations

For each beam type, limiting values for the relative deflections are set, using the menu 'Aluminium' > 'Setup' > 'Member check' > 'Relative deformations'.
With the option 'Aluminium' > 'Beams' > 'Member check' > '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

wsa_001b relative deformations
- Set beam type for member B5 & B6: Beam and Rafter - Set limits for relative deformations: Beam 1/1000 and Rafter 1/500 - Relative deformation check on member B5 & B6

Relative deformation

Linear calculation, Extreme : Global, System : LCS
Selection : B5,B6
Combinations : SLS

Case - combination	Member	dx [m]	uz [mm]	Rel uz [1/xx]	Check uz [-]
SLS/1	B6	5,064	**-6,2**	**1/1629**	0,31
SLS/2	B6	5,064	**8,9**	1/1136	0,44
SLS/3	B5	2,765	8,5	**1/713**	**1,40**
SLS/4	B5	2,765	-3,3	1/1849	**0,54**

- B5: L = 6.1m → limit: 6083/1000 = 6,1mm

  Uz = 8.5mm → 8.5/6083 = 1/715
  Check: (1/713)/(1/1000) = **1,40**

- B6: L = 10.127m → limit: 10127/500 = 20,3mm

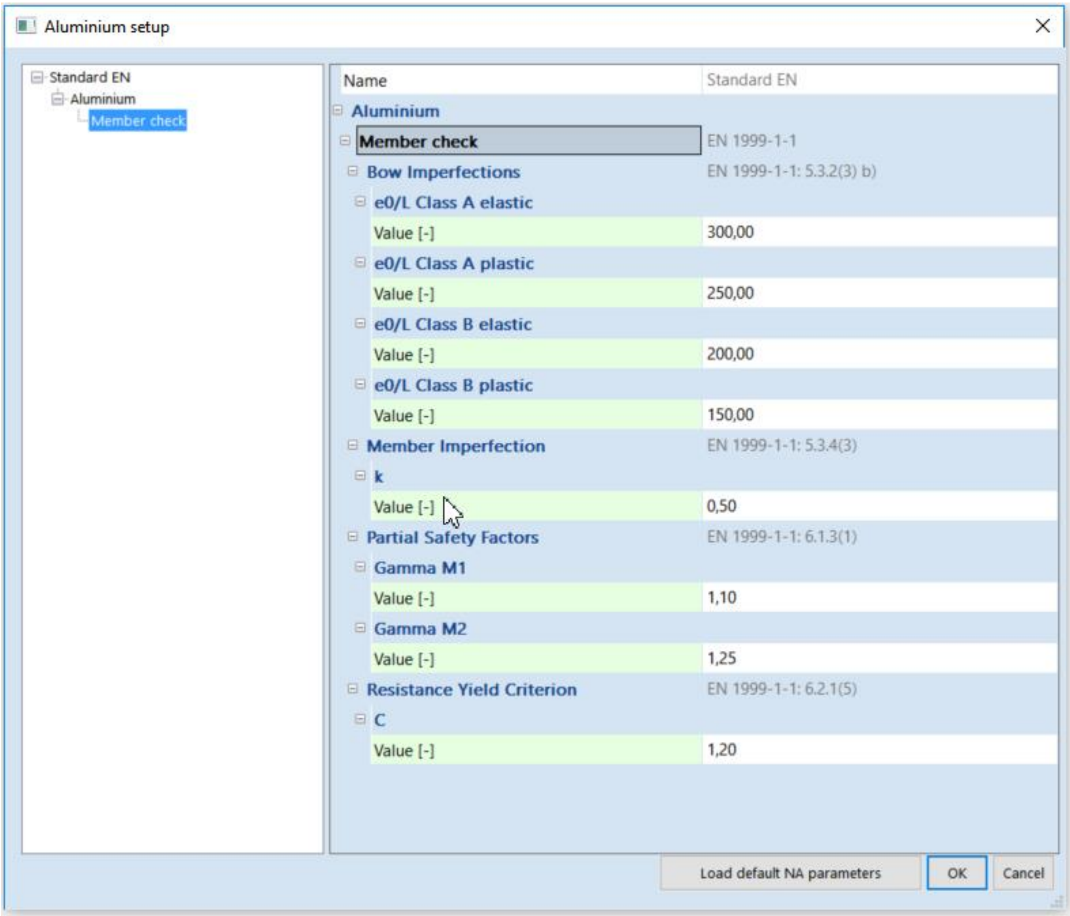
  Uz = 8.9mm → 8.9/10127 = 1/1137
  Check: (1/1136)/(1/500) = **0,44**

Additional Data

Setup

The national annexes of the Aluminium Code Check can be adapted under 'Project data' > 'National annex' > 'EN 1999: Design of aluminium structures' > 'EN 1999-1-1 (General structural rules)'. In this window the following options can be adapted:

- Bow imperfections for each class
- Member imperfections
- Partial Safety Factors
- Resistance Yield Criterion



Using 'Aluminium' > 'Setup', the user can change the basic setup-parameters for the Aluminium Code Check. A change of these values will affect all members.

In 'Member check', the following parameters can be adapted:

- Sway type
- Buckling length ratios
- Calculation of xs for unknown buckling shape
- Calculation of xs for known buckling shape

Next to these parameters, the user can input:

Elastic check only
All sections will be classified as class 3.

Section check only
Only section check is performed. No stability check is performed.

Only LTB stability check in 2nd Order calculation

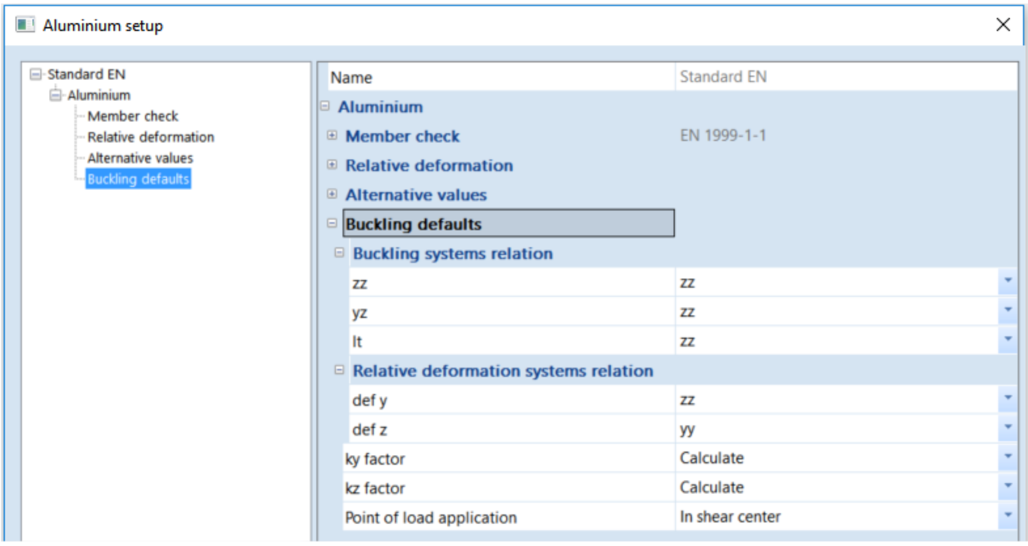
After performing a non-linear calculation with global and local (bow) imperfections and second order effects, only LTB needs to be checked.

In ‘Member check’ > ‘Relative deformations’, the user can input admissible deformations for different type of beams.

In ‘Member check’ > ‘Alternative values’, the user can choose between alternative values for different parameters according to EC-EN 1999-1-1.

In ‘Member check’ > ‘National Annex’, the user can choose between alternative values for different parameters according to the National Annex

In ‘Buckling defaults’, the user can input the default buckling system applied on all members. Via the property window of a separate beam, the buckling parameters can be changed locally.



Aluminium member data

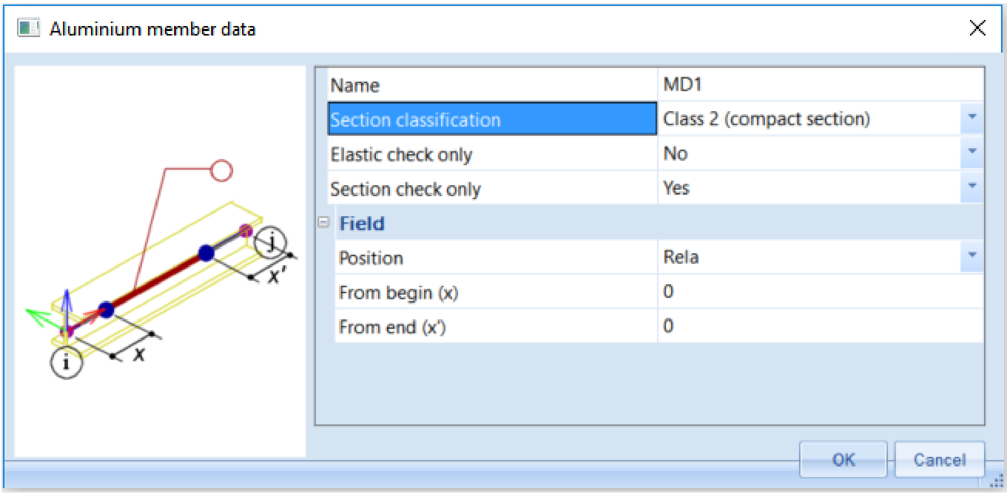
The default values used in the Setup menu can be overruled for a specific member using Member Data.

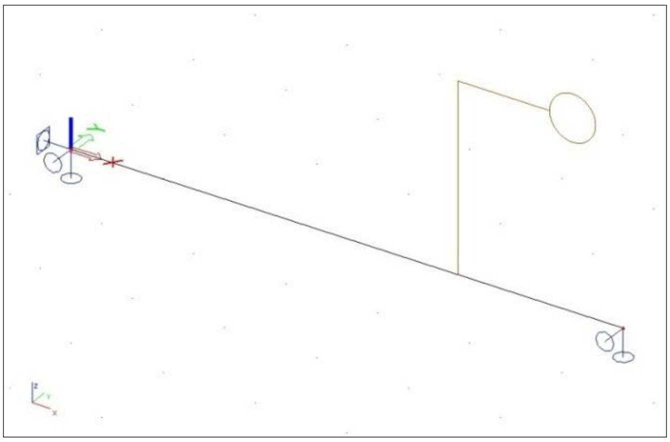
Section classification
For the selected members, the section classification generated by the program, will be overruled by this user settings

Elastic check only
The selected members will be classified as class 3.

Section check only
For the selected members, only section check is performed. No stability check is performed.

Field
Only the internal forces inside the field are considered during the steel code check



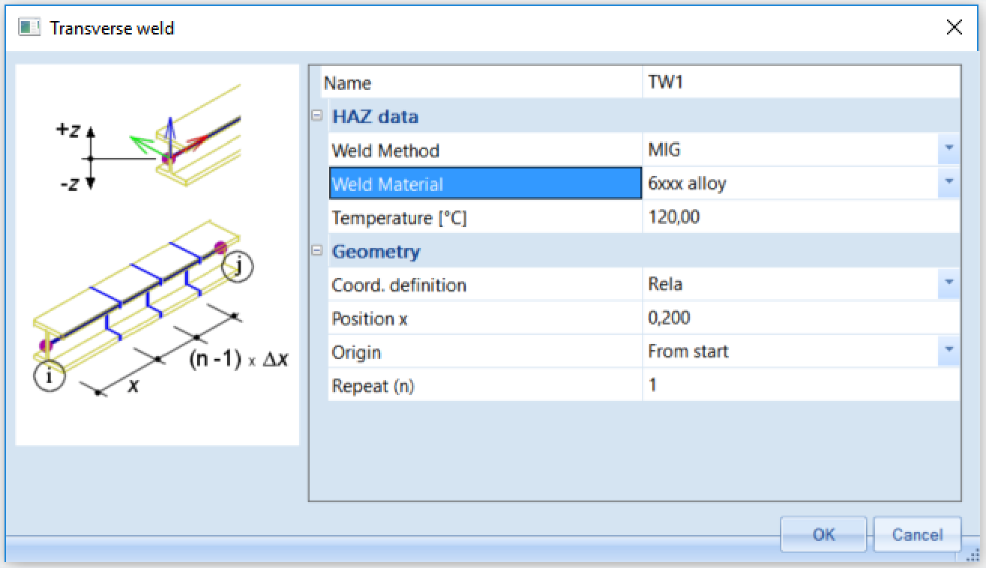


Stability check data

Transverse welds

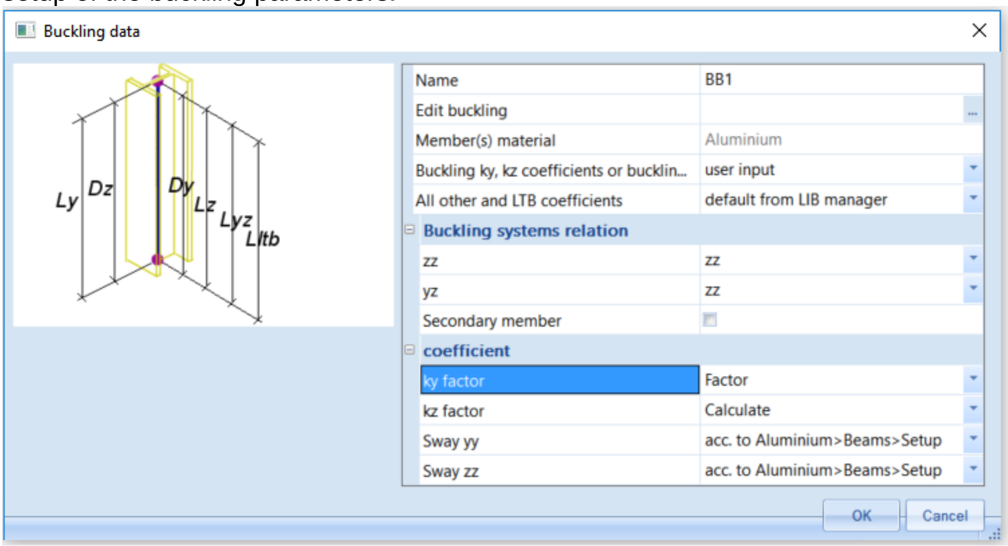
Via ‘Transverse welds’, the user can input different welds in certain sections of the member. Data needed for calculation of these welds are:

- Weld method: MIG or TIG
- Weld material: type of alloy
- Temperature of welding
- Geometry: position of weld in member



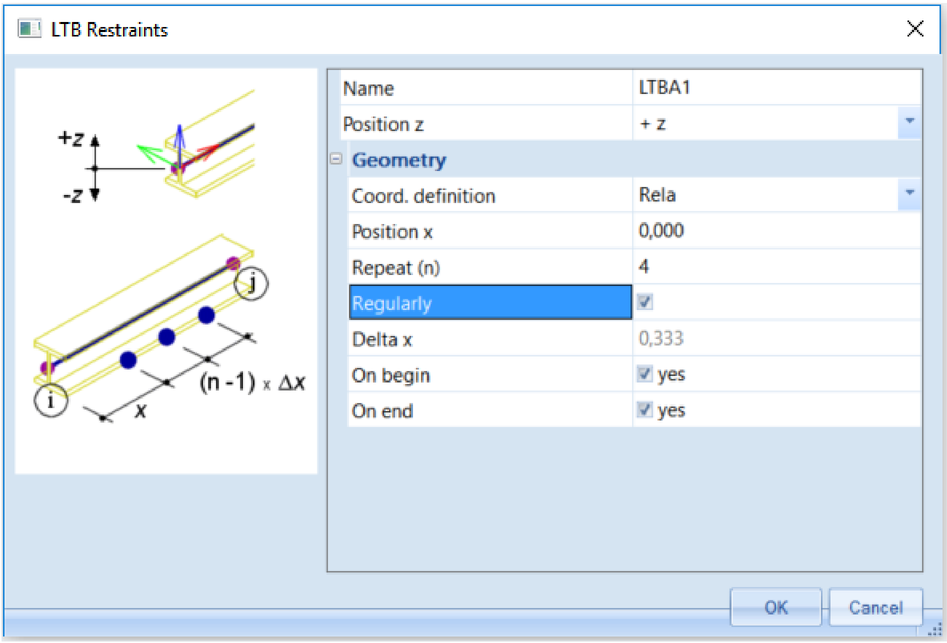
Member buckling data

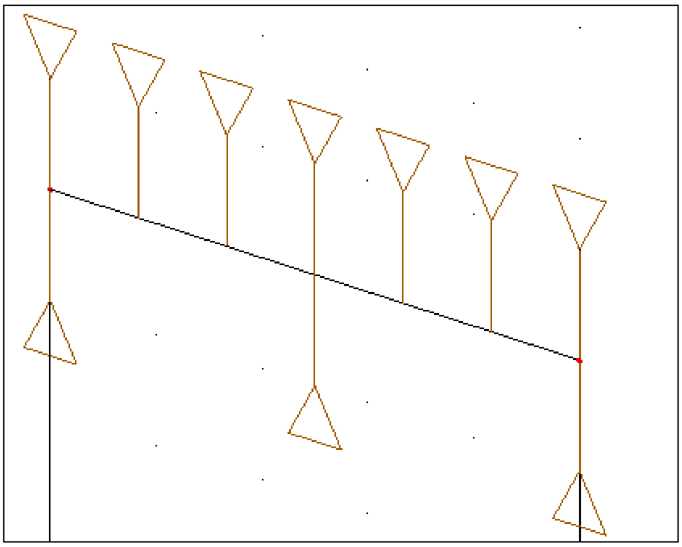
This group of parameters specifies where the member data relating to buckling are taken from. This can be taken from the Buckling Data Library. This data is displayed in the property window when a beam is selected: ‘Property’ > ‘Buckling and relative lengths’.
Using Member Buckling Data, the user can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



LTB restraints

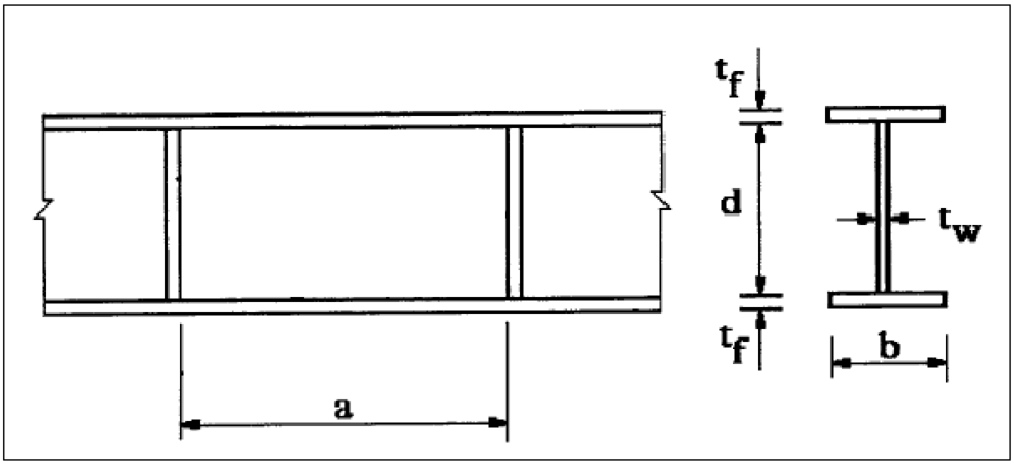
The default LTB data, defined in the buckling data dialog, are overruled by the LTB restraints. Fixed LTB restraints are defined on top flange or on bottom flange. The LTB lengths for the compressed flange are taken as distance between these restraints. The LTB moments factors are calculated between these restraints.



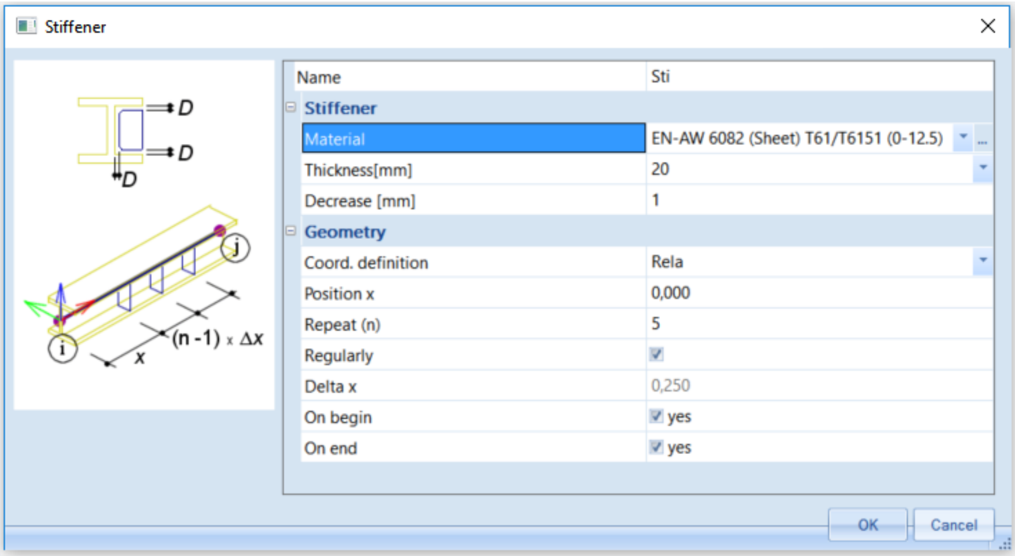


Stiffeners

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

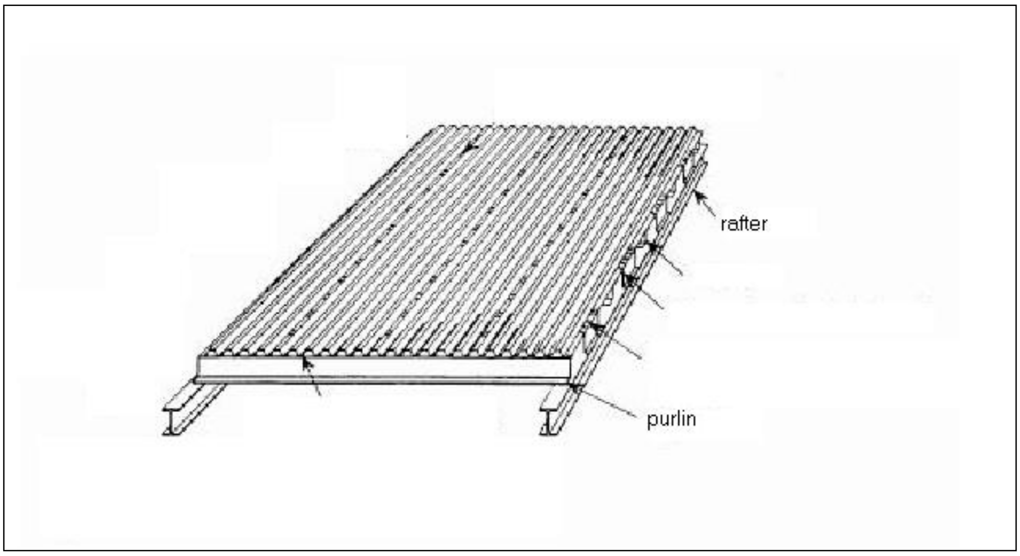


t_f
d
t_w
t_f
b
a

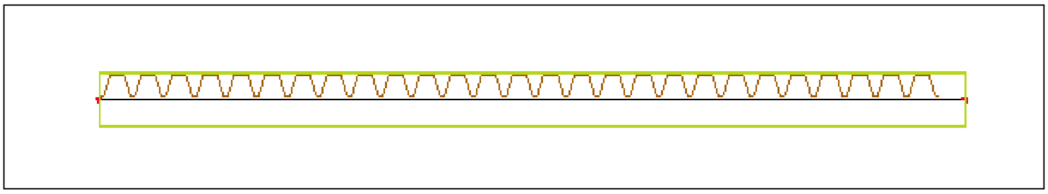


Stiffener
D
D
D
j
i
(n -1) x Δx
x
Name
Sti
Stiffener
Material
EN-AW 6082 (Sheet) T61/T6151 (0-12.5)
Thickness[mm]
20
Decrease [mm]
1
Geometry
Coord. definition
Rela
Position x
0,000
Repeat (n)
5
Regularly
Delta x
0,250
On begin
yes
On end
yes
OK
Cancel

Diaphragms



rafter
purlin



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.
Bold pitch	Bolts may be either: in every rib (i.e. "br"), in each second rib (i.e. "2 br").
Frame distance	The distance of frames
Length	The length of the diaphragm (shear field.)

ULS Check

Aluminium Slenderness

Via ‘Aluminium’ > ‘Slenderness data’, the user can ask for the system length, buckling ratio, buckling length, relative slenderness and bow imperfection according to the 2 local axis. In addition, also the Lateral Torsional Buckling length and the torsion buckling length can be displayed.

Slenderness data

Linear calculation

Member	CS Name	Part	Sway y Sway z	Ly [m] Lz [m]	ky [-] kz [-]	ly [m] lz [m]	Lam y [-] Lam z [-]	e0,y [mm] e0,z [mm]	lyz [m]	l LTB [m]
B1	column A	1	Yes	3,000	1,13	3,387	56,38	10,0	5,500	5,500
			No	5,500	1,00	5,500	196,15	18,3		
B1	column A	2	Yes	2,500	1,45	3,616	60,19	8,3	5,500	5,500
			No	5,500	1,00	5,500	196,15	18,3		

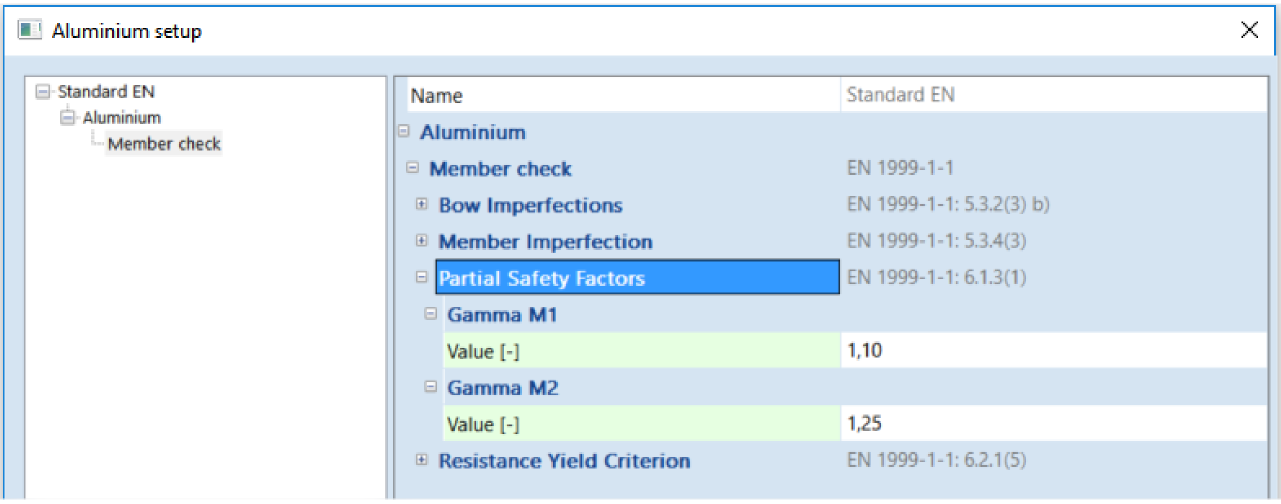
Section check

Partial safety factors

The partial safety factors may be chosen in the National Annex. Recommend values are given in Table 6.1 (Ref.[1]).

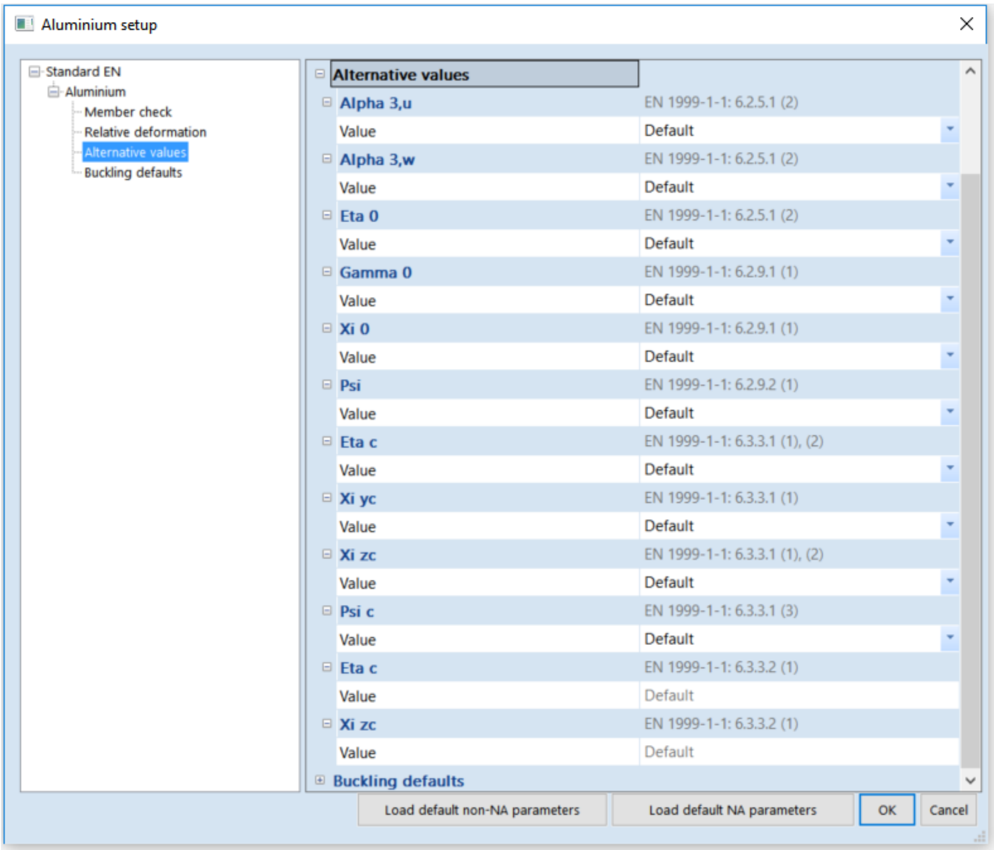
Resistance of cross-sections whatever the class is	$\gamma_{M1} = 1{,}10$
Resistance of member to instability assessed by member checks	$\gamma_{M1} = 1{,}10$
Resistance of cross-sections in tension to fracture	$\gamma_{M2} = 1{,}25$

Using the menu 'Project data’ > ‘National annex’ > ‘EN 1999: Design of aluminium structures’ > ‘EN 1999-1-1 (general structural rules)’, the user can input values for γ_{M1} and γ_{M2}.



Bending moments

According to section 6.2.5.1 Ref.[1], alternative values for $\alpha_{3,u}$ and $\alpha_{3,w}$ can be chosen. In SCIA Engineer, the user can input these alternative values using ‘Aluminium’ > ‘Setup’ > ‘Member check’ > ‘Alternative values’.



Shear

The design value of the shear force V_{Ed} at each cross-section shall satisfy (Ref.[1]):

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

Where $\mathbf{V_{Rd}}$ is the design shear resistance of the cross-section.

Slender and non-slender sections

The formulas to be used in the shear check are dependent on the slenderness of the cross-section parts.

For each part **i** the slenderness β is calculated as follows:

$$\beta_i = \left(\frac{h_w}{t_w}\right)_i = \left(\frac{x_{end} - x_{beg}}{t}\right)_i$$

With:	x_{end}	End position of plate i .
	x_{beg}	Begin position of plate i.
	t	Thickness of plate i.

For each part **i** the slenderness β is then compared to the limit **39ε**

With $\varepsilon = \sqrt{250/f_0}$ and f_0 in N/mm²

$\beta_i \le 39\varepsilon$ => Non-slender plate
$\beta_i > 39\varepsilon$ => Slender plate

I) All parts are classified as non-slender

$\beta_i \le 39\varepsilon$

The Shear check shall be verified using art. 6.2.6. Ref.[1]

II)One or more parts are classified as slender

$\beta_i > 39\varepsilon$

The Shear check shall be verified using art. 6.5.5. Ref.[1].
For each part **i** the shear resistance $\mathbf{V_{Rd,i}}$ is calculated.

Non-slender part:

Formula (6.88) Ref.[1] is used with properties calculated from the reduced shape for N+($\rho_{u,HAZ}$)

For Vy: $A_{net,y,i} = (x_{end} - x_{beg})_i \cdot \rho_{u,HAZ} \cdot t_i \cdot \cos^2 \alpha_i$

For Vz: $A_{net,z,i} = (x_{end} - x_{beg})_i \cdot \rho_{u,HAZ} \cdot t_i \cdot \sin^2 \alpha_i$

With:

i	The number (ID) of the plate
x_{end}	End position of plate i
x_{beg}	Begin position of plate i
t	Thickness of plate i
$\rho_{u,HAZ}$	Haz reduction factor of plate i
α	Angle of plate i to the Principal y-y axis

Slender part:

Formula (6.88) Ref.[1] is used with properties calculated from the reduced shape for N+($\rho_{u,HAZ}$) in the same way as for a non-slender part. => $\mathbf{V_{Rd,i,yield}}$

Formula (6.89) is used with **a** the member length or the distance between stiffeners (for I or U-sections)
=> $\mathbf{V_{Rd,i,buckling}}$

=> For this slender part, the resulting $\mathbf{V_{Rd,i}}$ is taken as the minimum of $\mathbf{V_{Rd,i,yield}}$ and $\mathbf{V_{Rd,i,buckling}}$

For each part $\mathbf{V_{Rd,i}}$ is then determined.

=> The $\mathbf{V_{Rd}}$ of the cross-section is then taken as the sum of the resistances $\mathbf{V_{Rd,i}}$ of all parts.

$$V_{Rd} = \sum_i V_{Rd_i}$$

<u>Note:</u>
For a solid bar, round tube and hollow tube, all parts are taken as non-slender by default and formula (6.31) is applied.

Example

wsa_004 shear check
- calculate project - aluminium check, detailed output

Part	Type	β	39 ε	Slender?	Avy,i	Avz,i	VRD,y,yield,i	VRD,z,yield,i
1	RUO	10	43,07	no	2,9	37,1	0,31	4
2	I	29	43,07	no	53,9	4,1	5,8	0,45
					53,9	4,1	5,8	0,45
3	I	100	43,07	yes	10,5	139,5	1,13	15
					4,6	61,5	0,5	6,61
					10,5	139,5	1,13	15
4	I	31,5	43,07	no	58,5	4,5	6,3	0,48
					58,5	4,5	6,3	0,48
5	RUO	9	43,07	no	2,6	33,4	0,28	3,6

- In addition: for the slender part 3
- a/b = 6000/200 = 30 with a = 6m and b = 200mm and v_1 = 0,280

- Sum (VRD,y,yield,i) = 27,44 kN
- Sum (VRD,z,yield,i) = 46,08 kN
- VRD,y = 0,31+11,60+**0,85**+12,60+0,28 = 25,63 kN
- VRD,z = 4,00+0,88+**11,21**+0,96+3,60 = 20,64 kN

Shear check

According to EN 1999-1-1 article 6.5.5 and formula (6.87).

Shear force Vy

Part ID	Beta	VRd,Yielding [kN]	VRd,Buckling [kN]
1	10,00	0,31	
2	29,00	11,60	
3	100,00	2,73	0,85
4	31,50	12,60	
5	9,00	0,28	

Table of values		
Vy,Rd	25,63	kN
Unity check	0,22	-

Shear force Vz

Part ID	Beta	VRd,Yielding [kN]	VRd,Buckling [kN]
1	10,00	4,00	
2	29,00	0,88	
3	100,00	36,11	11,21
4	31,50	0,96	
5	9,00	3,60	

Table of values		
Vz,Rd	20,64	kN
Unity check	0,08	-

Calculation of Shear Area

The calculation of the shear area is dependent on the cross-section type.
The calculation is done using the reduced shape for N+($\rho_{0,HAZ}$)

a) Solid bar and round tube

The shear area is calculated using art. 6.2.6 and formula (6.31) Ref.[1]:

$$A_v = \eta_v \cdot A_e$$

With: η_v 0,8 for solid section
0,6 for circular section (hollow and solid)
A_e Taken as area **A** calculated using the reduced shape for N+($\rho_{0,HAZ}$)

b) All other Supported sections

For all other sections, the shear area is calculated using art. 6.2.6 and formula (6.30) Ref.[1].

The following adaptation is used to make this formula usable for any initial cross-section shape:

$$A_{vy} = \sum_{i=1}^{n} (x_{end} - x_{beg}) \cdot \rho_{0,HAZ} \cdot t \cdot \cos^2 \alpha$$

$$A_{vz} = \sum_{i=1}^{n} (x_{end} - x_{beg}) \cdot \rho_{0,HAZ} \cdot t \cdot \sin^2 \alpha$$

With:

i	The number (ID) of the plate
x_{end}	End position of plate i
x_{beg}	Begin position of plate i
t	Thickness of plate i
$\rho_{0,HAZ}$	HAZ reduction factor of plate i
α	Angle of plate i to the Principal y-y axis

Should a cross-section be defined in such a way that the shear area $\mathbf{A_v}$ (A_{vy} or A_{vz}) is zero, then $\mathbf{A_v}$ is taken as **A** calculated using the reduced shape for N+($\rho_{0,HAZ}$).

Note:
For sections without initial shape or numerical sections, none of the above mentioned methods can be applied. In this case, formula (6.29) is used with Av taken as Ay or Az of the gross-section properties.

Torsion with warping

In case warping is taken into account, the combined section check is replaced by an elastic stress check including warping stresses.

$$\sigma_{tot,Ed} \leq \frac{f_0}{\gamma_{M1}}$$

$$\tau_{tot,Ed} \leq \frac{f_0}{\sqrt{3}\gamma_{M1}}$$

$$\sqrt{\sigma_{tot,Ed}^2 + 3\tau_{tot,Ed}^2} \leq \sqrt{C}\,\frac{f_0}{\gamma_{M1}}$$

$$\sigma_{tot,Ed} = \sigma_{N,Ed} + \sigma_{My,Ed} + \sigma_{Mz,Ed} + \sigma_{w,Ed}$$

$$\tau_{tot,Ed} = \tau_{Vy,Ed} + \tau_{Vz,Ed} + \tau_{t,Ed} + \tau_{w,Ed}$$

With: f_0 0,2% proof strength

$\sigma_{tot,Ed}$	Total direct stress
$\tau_{tot,Ed}$	Total shear stress
γ_{M1}	Partial safety factor for resistance of cross-sections
C	Constant (by default 1,2)
$\sigma_{N,Ed}$	Direct stress due to the axial force on the relevant effective cross-section
$\sigma_{My,Ed}$	Direct stress due to the bending moment around y axis on the relevant effective cross-section
$\sigma_{Mz,Ed}$	Direct stress due to the bending moment around z axis on the relevant effective cross-section
$\sigma_{w,Ed}$	Direct stress due to warping on the gross cross-section
$\tau_{Vy,Ed}$	Shear stress due to shear force in y direction on the gross cross-section
$\tau_{Vz,Ed}$	Shear stress due to shear force in z direction on the gross cross-section
$\tau_{t,Ed}$	Shear stress due to uniform (St. Venant) torsion on the gross cross-section
$\tau_{w,Ed}$	Shear stress due to warping on the gross cross-section

The direct stress due to warping is given by Ref.[3] 7.4.3.2.3, Ref.[4]. A more detailed explanation can be found in Ref.[20].

Bending, shear and axial force

According to section 6.2.9.1.(1) and 6.2.9.2 (1) Ref.[1], alternative values for γ_0, η_0, ε_0 and ψ can be chosen. In SCIA Engineer, the user can input these alternative values using ‘Aluminium’ > ‘Setup’ > ‘Member check’ > ‘Alternative values’.

Localised welds

In case transverse welds are inputted, the extend of the HAZ is calculated as specified in paragraph “Calculation of Reduction factor ρ_{HAZ} effects” of the Aluminium Code Check Theoretical Background and compared to the least width of the cross-section.

The reduction factor ω_0 is then calculated according to art. 6.2.9.3 Ref.[1].
When the width of a member cannot be determined (Numerical section, tube …) formula (6.44) is applied.

Note:
Since the extend of the HAZ is defined along the member axis, it is important to specify enough sections on average member in the Solver Setup when transverse welds are used.
Note:
Formula (6.44) is limited to a maximum of **1,00** in the same way as formula (6.64).

Shear reduction

Where V_{Ed} exceeds 50% of V_{Rd} the design resistances for bending and axial force are reduced using a reduced yield strength as specified in art. 6.2.8 & 6.2.10. Ref.[1].

For Vy the reduction factor ρ_y is calculated
For Vz the reduction factor ρ_z is calculated

The bending resistance $M_{y,Rd}$ is reduced using ρ_z
The bending resistance $M_{z,Rd}$ is reduced using ρ_y

The axial force resistance N_{Rd} is reduced by using the maximum of ρ_y and ρ_z

Example

wsa_005 bending - transverse welds
- calculate project - aluminium check combination UGT, detailed output of Beam B6 - classification for My- = 4 - check ends of Beam B6 - Combined Bending, Axial force and Shear force Check

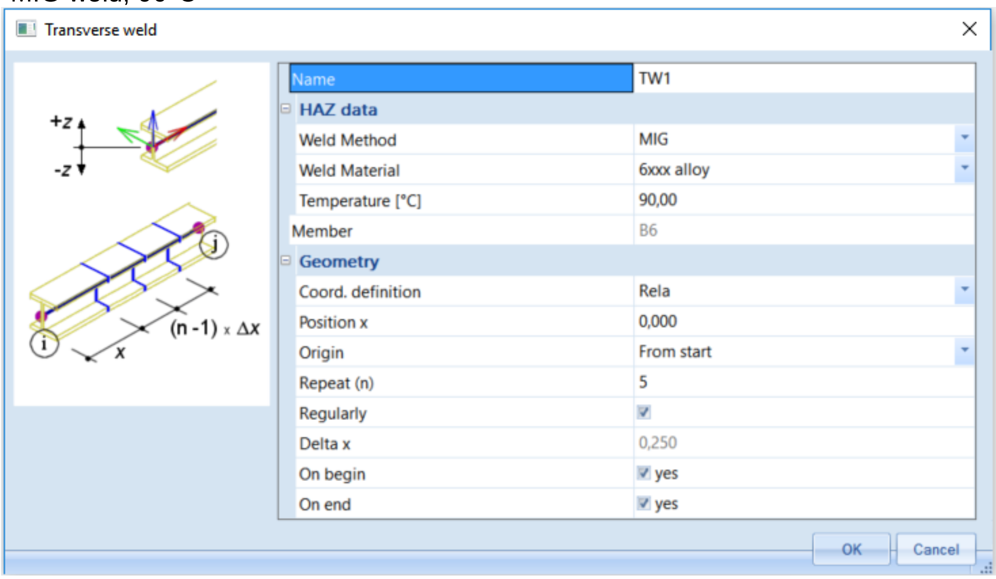
Combined Bending, Axial force and Shear force Check.
According to EN 1999-1-1 article 6.2.9.1 & 6.2.10 and formula (6.40),(6.41).

Table of values		
Eta0 (6.42a)	1,00	
Gamma0 (6.42b)	1,00	
Xi 0 (6.42c)	1,00	
w0	1,00	
NRd	1659,85	kN
My,Rd	342,68	kNm
Mz,Rd	47,36	kNm

Unity check (6.40) = 0,00 + 0,08 = 0,08 -
Unity check (6.41) = 0,00 + 0,08 + 0,00 = 0,08 -
The member satisfies the section check.

wsa_005 bending - transverse welds

- input 5 transverse welds regulary distance, begin and end of the beam B6
- MIG weld, 90°C



- check ends of Beam B6

Combined Bending, Axial force and Shear force Check.
According to EN 1999-1-1 article 6.2.9.1 & 6.2.10 and formula (6.40),(6.41).

Table of values		
Eta0 (6.42a)	1,00	
Gamma0 (6.42b)	1,00	
Xi 0 (6.42c)	1,00	
w0	0,63	
NRd	1659,85	kN
My,Rd	342,68	kNm
Mz,Rd	47,36	kNm

Unity check (6.40) = 0,00 + 0,12 = 0,13 -
Unity check (6.41) = 0,00 + 0,12 + 0,00 = 0,13 -
The member satisfies the section check.

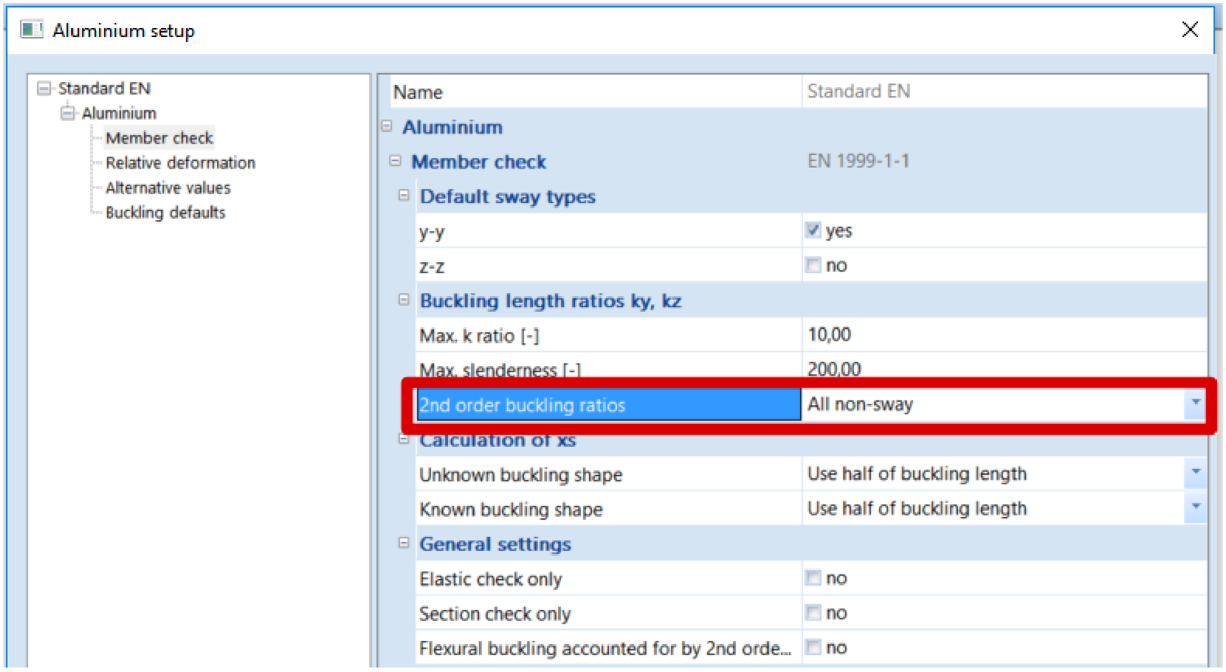
Stability check

Flexural Buckling

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’. See ‘Aluminium’ > ‘Beams’ > ‘Aluminium Setup’:



Buckling Ratio

General formula

For the calculation of the buckling ratios, some approximate formulas are used. These formulas are treated in reference [5], [6] and [7].

The following formulas are used for the buckling ratios (Ref[7],pp.21):

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	System length
	E	Modulus of Young
	I	Moment of inertia
	C_i	Stiffness in node i
	M_i	Moment in node i
	φ_i	Rotation in node i

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

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

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

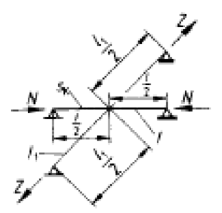
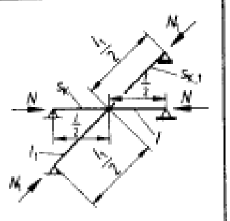
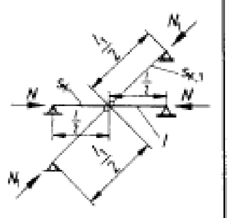
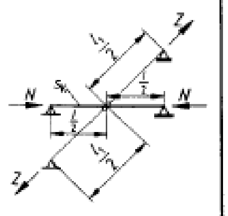
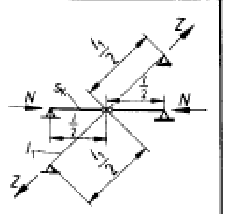
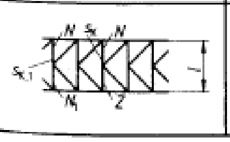
The values for M_i and ϕ_i are approximately determined by the internal forces and the deformations, calculated by load cases which generate deformation forms, having an affinity with the buckling shape. (See also Ref.[8], pp.113 and Ref.[9],pp.112).

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.

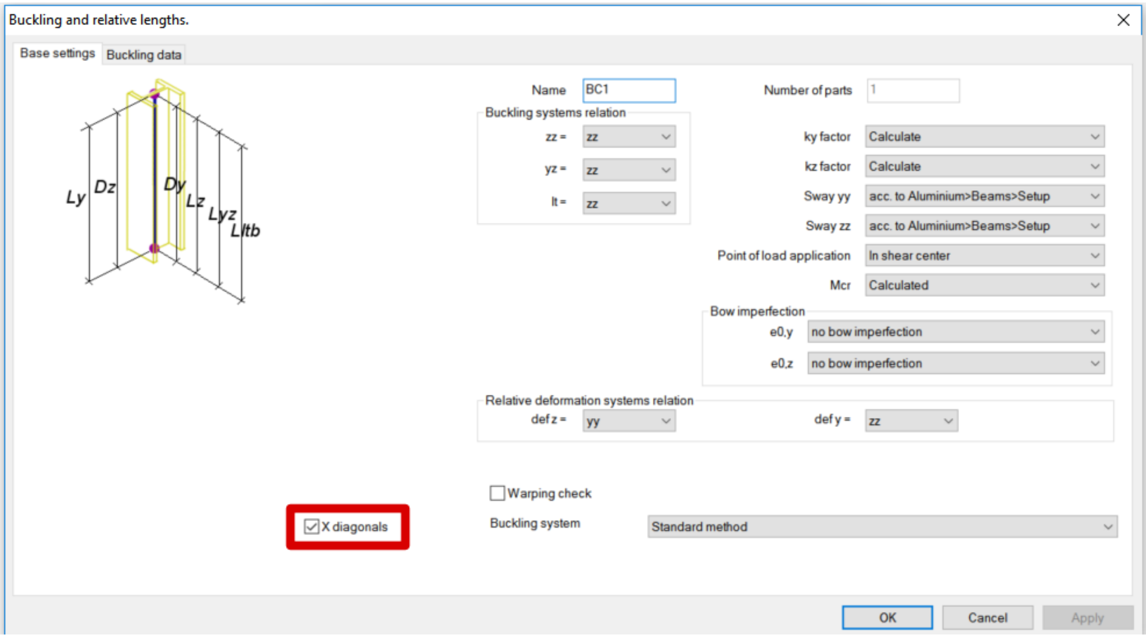
The used approach gives good results for frame structures with perpendicular rigid or semi-rigid beam connections. For other cases, the user has to evaluate the presented bucking ratios. In such cases a more refined approach (from stability analysis) can be applied.

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 Ref.[10]. 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 $s_K \ge 0{,}5\, l$	
2		$s_K = l\sqrt{\dfrac{1+\frac{N_1\cdot l}{N\cdot l_1}}{1+\frac{I_1\cdot l^3}{I\cdot l_1^3}}}$ jedoch $s_K \ge 0{,}5\, l$	$s_{K,1} = l_1\sqrt{\dfrac{1+\frac{N\cdot l_1}{N_1\cdot l}}{1+\frac{I\cdot l_1^3}{I_1\cdot l^3}}}$ jedoch $s_{K,1} \ge 0{,}5\, l_1$
3		durchlaufender Druckstab $s_K = l\sqrt{1+\frac{\pi^2}{12}\cdot\frac{N_1\cdot l}{N\cdot l_1}}$	gelenkig angeschlossener Druckstab $s_{K,1} = 0{,}5\, l_1$ wenn $(E\cdot I)_d \ge \frac{N_1\cdot l^3}{\pi^2\cdot l_1}\left(\frac{\pi^2}{12}+\frac{N\cdot l_1}{N_1\cdot l}\right)$
4		$s_K = l\sqrt{1-0{,}75\,\frac{Z\cdot l}{N\cdot l_1}}$ jedoch $s_K \ge 0{,}5\, l$	
5		$s_K = 0{,}5\, l$ wenn $\frac{N\cdot l_1}{Z\cdot l} \le 1$ oder wenn gilt $(E\cdot I_1)_d \ge \frac{3\,Z\cdot l_1^2}{4\,\pi^2}\left(\frac{N\cdot l_1}{Z\cdot l}-1\right)$	
6		$s_K = l\left(0{,}75-0{,}25\left\lvert\frac{Z}{N}\right\rvert\right)$ jedoch $s_K \ge 0{,}5\, l$	$s_{K,1} = l\left(0{,}75+0{,}25\,\frac{N_1}{N}\right)$ $N_1 < N$

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



When using cross-links, this option is automatically activated. The user must verify if this is wanted or not.

Stability analysis

When member buckling data from stability are defined, the critical buckling load $\mathbf{N_{cr}}$ for a prismatic member is calculated as follows:

$N_{cr} = \lambda \cdot N_{Ed}$

Using Euler's formula, the buckling ratio **k** can then be determined:

$$N_{cr} = \frac{\pi^2 . E . I}{(k . s)^2} \Rightarrow k = \frac{1}{s} . \sqrt{\frac{\pi^2 . E . I}{N_{cr}}}$$

With:	λ	Critical load factor for the selected stability combination
	N_{Ed}	Design loading in the member
	E	Modulus of Young
	I	Moment of inertia
	s	Member length

Example

wsa_006 flexural buckling
- calculate project - aluminium check combination UGT, detailed output of Beam B1 - critical check on 3,00m - classification for N- = 4 and My- = 4 - Flexural buckling check

Flexural Buckling check

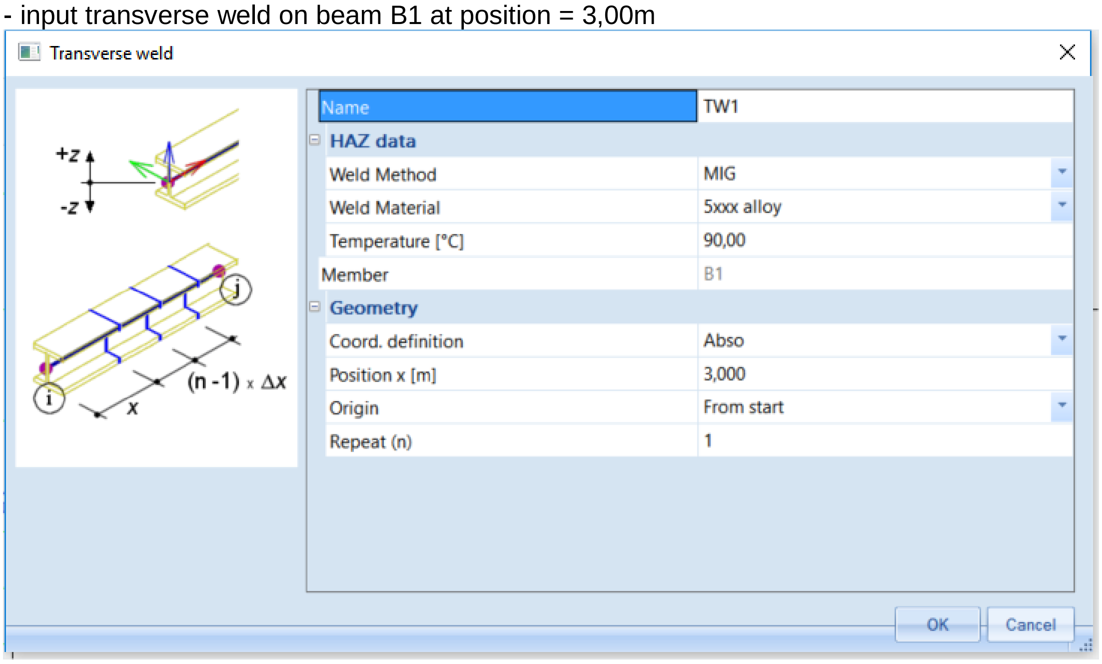
According to EN 1999-1-1 article 6.3.1.1 and formula (6.48).

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System Length L	3,000	5,500	m
Buckling factor k	1,13	1,00	
Buckling length Lcr	3,387	5,500	m
Critical Euler Load Ncr	787,52	65,05	kN
Relative slenderness Lambda	1,01	3,52	
Limit slenderness Lambda,0	0,10	0,10	
Imperfection Alpha	0,20	0,20	
Reduction factor Chi	0,65	0,08	
Welding factor Kappa	1,00	1,00	
Buckling resistance Nb,Rd	474,55	55,80	kN

Table of values		
Aeff	3092	mm^2
Nb,Rd	55,80	kN
Unity check	0,29	-

wsa_006 flexural buckling

- input transverse weld on beam B1 at position = 3,00m



- aluminium check combination UGT, detailed output of Beam B1
- critical check on 3,00m
- classification for N- = 4 and My- = 4
- Flexural buckling check

Flexural Buckling check
According to EN 1999-1-1 article 6.3.1.1 and formula (6.48).

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System Length L	3,000	5,500	m
Buckling factor k	1,13	1,00	
Buckling length Lcr	3,387	5,500	m
Critical Euler Load Ncr	787,52	65,05	kN
Relative slenderness Lambda	1,01	3,52	
Limit slenderness Lambda,0	0,10	0,10	
Imperfection Alpha	0,20	0,20	
Reduction factor Chi	0,65	0,08	
Welding factor Kappa	0,63	0,63	
Buckling resistance Nb,Rd	297,14	34,94	kN

Table of values		
Aeff	3092	mm²
Nb,Rd	34,94	kN
Unity check	0,46	-

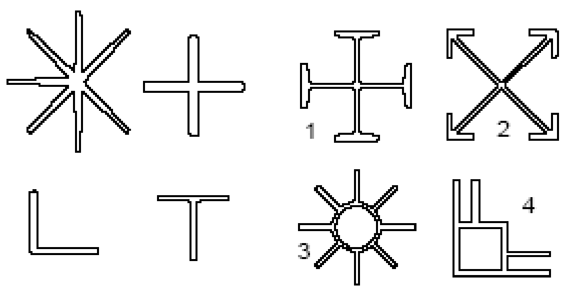
The difference between the two examples can be found in the value for $N_{b,Rd}$.
Around the y-axis:

$$N_{b,Rd,without\ weld} = 474{,}55\ kN$$
$$N_{b,Rd,with\ weld} = 474{,}55\ kN \cdot \kappa = 474{,}55\ kN \cdot 0{,}63 = 298{,}97 kN$$

Torsional (-Flexural) Buckling

If the section contains only Plate Types F, SO, UO it is regarded as '**Composed entirely of radiating outstands**'. In this case $\mathbf{A_{eff}}$ is taken as **A** calculated from the reduced shape for N+($\rho_{0,HAZ}$) according to Table 6.7 Ref.[1].

In all other cases, the section is regarded as '**General**'.
In this case $\mathbf{A_{eff}}$ is taken as **A** calculated from the reduced shape for N-



a) "Outstand" sections b) "General" cross sections

<u>Note:</u>
The Torsional (-Flexural) buckling check is ignored for sections complying with the rules given in art. 6.3.1.4 (1) Ref.[1].
The value of the elastic critical load $\mathbf{N_{cr}}$ is taken as the smallest of $\mathbf{N_{cr,T}}$ (Torsional buckling) and $\mathbf{N_{cr,TF}}$ (Torsional-Flexural buckling).

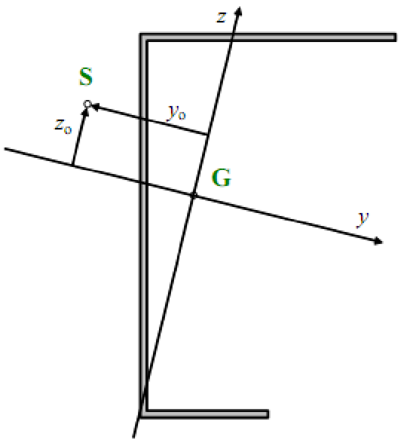
Calculation of $N_{cr,T}$

The elastic critical load $\mathbf{N_{cr,T}}$ for torsional buckling is calculated according to Ref.[11].

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

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

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 center 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 $\mathbf{N_{cr,TF}}$ for torsional flexural buckling is calculated according to Ref.[11].

$\mathbf{N_{cr,TF}}$ is taken as the smallest root of the following cubic equation in N:

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

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

Example

wsa_007 torsional - flexural buckling
- calculate project - aluminium check for Loadcase “LC1” - critical check on 3,00m - classification for N- = 4 , My+ = 4 and My- = 4 - Torsional - Flexural buckling check

Torsional (-Flexural) Buckling check
According to EN 1999-1-1 article 6.3.1.1 & 6.3.1.4 and formula (6.48).

Table of values		
Cross-section Type	General	
Torsional Buckling length	6,000	m
Ncr,T	14,99	kN
Ncr,TF	4,86	kN
Relative slenderness Lambda,T	3,71	
Limit slenderness Lambda,0	0,40	
Imperfection Alpha	0,35	
Aeff	327,39	mm^2
Reduction factor Chi	0,07	
Buckling resistance Nb,Rd	4,06	kN
Unity check	**2,47**	-

Lateral Torsional Buckling

The Lateral Torsional buckling check is verified using art. 6.3.2.1 Ref.[1].

For the calculation of the elastic critical moment $\mathbf{M_{cr}}$ the following methods are available:

- General formula (standard method)
- LTBII Eigenvalue solution
- Manual input

Note:
The Lateral Torsional Buckling check is ignored for circular hollow sections according to art. 6.3.3 (1) Ref.[1].

Calculation of M_{cr} – General Formula

For I sections (symmetric and asymmetric) and RHS (Rectangular Hollow Section) sections the elastic critical moment for LTB $\mathbf{M_{cr}}$ is given by the general formula F.2. Annex F Ref. [12]. For the calculation of the moment factors C1, C2 and C3 reference is made to the paragraph “Calculation of Moment factors for LTB” of the Aluminium Code Check Theoretical Background.
For the other supported sections, the elastic critical moment for LTB $\mathbf{M_{cr}}$ is given by:

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

With:	E	Modulus of elasticity
	G	Shear modulus
	L	Length of the beam between points which have lateral restraint (= l_{LTB})
	I_w	Warping constant
	I_t	Torsional constant
	I_z	Moment of inertia about the minor axis

See also Ref. [13], part 7 and in particular part 7.7 for channel sections.
Composed rail sections are considered as equivalent asymmetric I sections.

Diaphragms

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

See Ref.[14], Chapter 10.1.5., Ref.[15],3.5 and Ref.[16],3.3.4.

The torsional constant I_t is adapted with the stiffness of the diaphragms:

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

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

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

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

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

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

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

With:	l	LTB length
	G	Shear modulus
	vorh C_θ	Actual rotational stiffness of diaphragm
	$C_{\theta M,k}$	Rotational stiffness of the diaphragm
	$C_{\theta A,k}$	Rotational stiffness of the connection between the diaphragm and the beam
	$C_{\theta P,k}$	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 per unit width of the diaphragm
	s	Spacing of the beam
	b_a	Width of the beam flange (in mm)
	C_{100}	Rotation coefficient - see table
	h	Height of the beam
	t	Thickness of the beam flange
	s	Thickness of the beam web

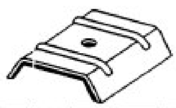
Positioning of sheeting		Sheet fastened through		Pitch of fasteners		Washer diameter	C_{100}	$b_{T,max}$
Positive	Negative	Trough	Crest	$e = b_R$	$e = 2b_R$	[mm]	[kNm/m]	[mm]
For gravity loading:								
×		×		×		22	5,2	40
×		×			×	22	3,1	40
	×		×	×		K_a	10,0	40
	×		×		×	K_a	5,2	40
	×	×		×		22	3,1	120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

Key:

b_R is the corrugation width [185 mm maximum];

b_T is the width of the sheeting flange through which it is fastened to the purlin.

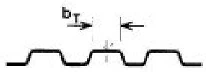
K_a indicates a steel saddle washer as shown below with $t \geq 0,75$ mm



Sheet fastened:

- through the trough:

- through the crest:



The values in this table are valid for:

- sheet fastener screws of diameter: $\phi = 6,3$ mm;
- steel washers of thickness: $t_w \geq 1,0$ mm;
- sheeting of nominal core thickness: $t \geq 0,66$ mm.

LTBII Eigenvalue solution

For calculation of Mcr using LTBII reference is made to chapter "LTBII: Lateral Torsional Buckling 2nd Order Analysis" of the Aluminium Code Check Theoretical Background.

Example

wsa_008 lateral torsional buckling
- calculate project - aluminium check, LC1 - critical check on 3,00m - classification for N- = 4 , My+ = 4 and My- = 4 - Lateral Torsional buckling check - LTB-length = 6,00m

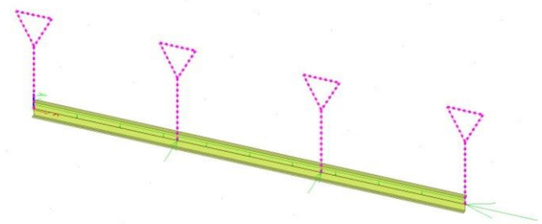
Lateral Torsional Buckling Check
According to EN 1999-1-1 article 6.3.2.1 and formula (6.54).

LTB Parameters		
Alpha	0,688	
Wel,y	41599,23	mm^3
Elastic critical moment Mcr	0,72	kNm
Relative slenderness Lambda,LT	2,860	
Limit slenderness Lambda,0,LT	0,400	
Imperfection Alpha,LT	0,200	
Reduction factor Chi, LT	0,114	
Buckling resistance Mb,Rd	0,61	kNm
Unity check	**2,73**	-

Mcr Parameters		
LTB length	6,000	m
k	1,00	
kw	1,00	
C1	1,13	
C2	0,45	
C3	0,53	
Influence of load position	No influence	

wsa_008 lateral torsional buckling

- input 4 LTB-restraints regulary on topflange of beam
- aluminium check, LC1
- critical check on 3,00m
- classification for N- = 4 , My+ = 4 and My- = 4
- Lateral Torsional buckling check
- LTB-length = 2,00m



Lateral Torsional Buckling Check
According to EN 1999-1-1 article 6.3.2.1 and formula (6.54).

LTB Parameters		
Alpha	0,688	
Wel,y	41599,23	mm^3
Elastic critical moment Mcr	5,71	kNm
Relative slenderness Lambda,LT	1,014	
Limit slenderness Lambda,0,LT	0,400	
Imperfection Alpha,LT	0,200	
Reduction factor Chi, LT	0,698	
Buckling resistance Mb,Rd	3,72	kNm
Unity check	0,45	-

Mcr Parameters		
LTB length	2,000	m
k	1,00	
kw	1,00	
C1	1,10	
C2	0,16	
C3	1,00	
Influence of load position	No influence	

Bending and Axial compression

Flexural Buckling

According to section 6.3.3.1.(1), (2), (3) Ref.[1], alternative values for η_c, ε_{yc}, ε_{zc}, ψ_c can be chosen. In SCIA Engineer, the user can input these alternative values using 'Aluminium' > 'Setup' > 'Member check' > 'Alternative values'.

Lateral Torsional Buckling

Members containing localized welds

In case transverse welds are inputted, the extend of the HAZ is calculated as specified in chapter "Calculation of Reduction factor ρ_{HAZ}" and compared to the least width of the cross-section.
The reduction factors, HAZ softening factors ω_0, ω_x and ω_{xLT} are calculated according to art. 6.3.3.3 Ref.[1].

Unequal end moments and/or transverse loads

If the section under consideration is not located in a HAZ zone, the reduction factors ω_x and ω_{xLT} are then calculated according to art. 6.3.3.5. Ref.[1].

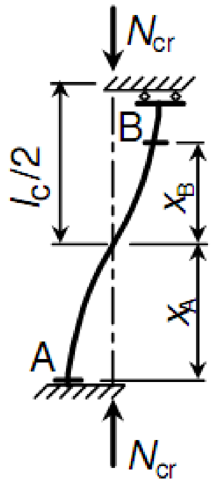
In this case ω_0 is taken equal to **1,00**.

Calculation of x_s

The distance $\mathbf{x_s}$ is defined as the distance from the studied section to a simple support or point of contra flexure of the deflection curve for elastic buckling of axial force only.
By default $\mathbf{x_s}$ is taken as half of the buckling length for each section. This leads to a denominator of **1,00** in the formulas of the reduction factors following Ref.[18] and [19].
Depending on how the buckling shape is defined, a more refined approach can be used for the calculation of $\mathbf{x_s}$.

Known buckling shape
The buckling shape is assumed to be known in case the buckling ratio is calculated according to the General Formula specified in chapter "Calculation of Buckling ratio – General Formula". The basic assumption is that the deformations for the buckling load case have an affinity with the buckling shape.

Since the buckling shape (deformed structure) is known, the distance from each section to a simple support or point of contra flexure can be calculated. As such $\mathbf{x_s}$ will be different in each section. A simple support or point of contra flexure are in this case taken as the positions where the bending moment diagram for the buckling load case reaches zero.



Note:
Since for a known buckling shape $\mathbf{x_s}$ can be different in each section, accurate results can be obtained by increasing the numbers of sections on average member in the 'Solver Setup' of SCIA Engineer.

Unknown buckling shape
In case the buckling ratio is not calculated according to the General Formula specified in chapter "Calculation of Buckling ratio – General Formula", the buckling shape is taken as unknown. This is thus the case for manual input or if the buckling ratio is calculated from stability.

When the buckling shape is unknown, $\mathbf{x_s}$ can be calculated according to formula (6.71) Ref.[1]:

$$cos\left(\frac{x_s\pi}{l_c}\right) = \frac{(M_{Ed,1}-M_{Ed,2})}{\pi.M_{Rd}}.\frac{N_{Rd}}{N_{Ed}}.\frac{1}{1/\chi-1} \quad \text{but } \mathbf{x_s} \geq 0$$

With:		
	l_c	Buckling length
	$M_{Ed,1}$ and $M_{Ed,2}$	Design values of the end moments at the system length of the member
	N_{Ed}	Design value of the axial compression force
	M_{Rd}	Bending moment capacity
	N_{Rd}	Axial compression force capacity
	χ	Reduction factor for flexural buckling

Since the formula returns only one value for x_s, this value will be used in each section of the member.

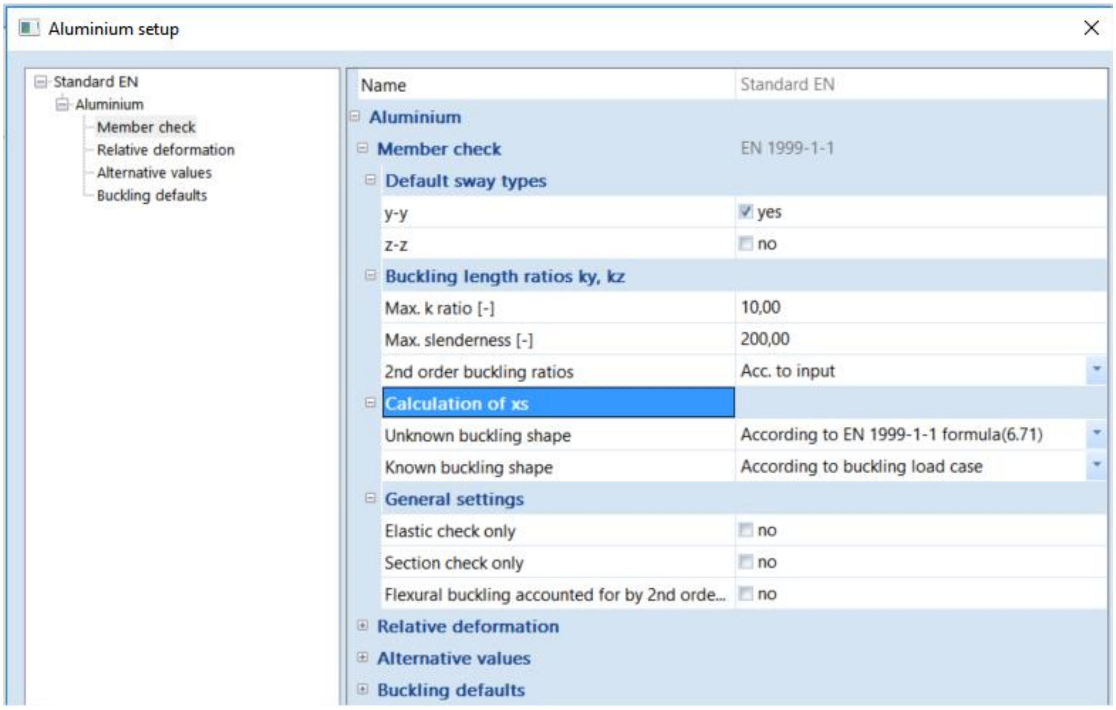
The application of the formula is however limited:

- The formula is only valid in case the member has a linear moment diagram.
- Since the left side of the equation concerns a cosine, the right side has to return a value between -1,00 and +1,00

If one of the two above stated limitations occur, the formula is not applied and instead x_s is taken as half of the buckling length for each section.

Note:
The above specified formula contains the factor π in the denominator of the right side of the equation. This factor was erroneously omitted in formula (6.71) of EN 1999-1-1:2007.

The user can change the calculation protocol for x_s. This input can be changed in the menu 'Aluminium' > 'Setup' > 'Member check'. Here the user can choose between the formulas discussed above or to use half of the buckling length for x_s.



Example

wsa_009a xs1
- B1: default calculation of buckling factors ky and kz according to General Formula → buckling shape is "known" for both directions - B2: default calculation of buckling factors ky, manual input of buckling factor kz → buckling shape is known for yy, but unknown for zz -B3: default calculation of buckling factors ky, manual input of buckling factor kz → buckling shape is known for yy, but unknown for zz - calculation of xs for unknown buckling shape: according to formula (6.71) Ref.[1]. - calculation of xs for known buckling shape: according to buckling load case - check is done at the ends of the beams

Results for Beam B1
- check moments My and Mz - B1: xs_y = xs_z = 6,00m

Table of values		
Method for xs,y	Buckling Loadcase	
Method for xs,z	Buckling Loadcase	
xs,y	6,000	m
xs,z	6,000	m
w0	1,000	
wx,y	1,000	
wx,z	2,692	
wxLT	1,353	

Results for Beam B2
- check moments My and Mz - B1: xs_y = 6,00m and xs_z = 1,50m - for xs_z, the buckling shape is unknown. Thus formula (6.71) will be used, but the limitations of this formula are not respected. As such, Half of the buckling length will be used. The buckling length = kz * L = 0,5 * 6,00m = **1,5**m

Table of values		
Method for xs,y	Buckling Loadcase	
Method for xs,z	Half of Buckling Length	
xs,y	6,000	m
xs,z	1,500	m
w0	1,000	
wx,y	1,000	
wx,z	1,000	
wxLT	1,000	

Note: Formula (6.71) cannot be applied due to non-linear moment diagram or imaginary arc cosine.

Results for Beam B3
- check moments My and Mz -B1: xs_y = 6,00m and xs_z = 1,134m - for xs_z, the buckling shape is unknown. Thus formula (6.71) will be used.

Table of values		
Method for xs,y	Buckling Loadcase	
Method for xs,z	Formula (6.71)	
xs,y	6,000	m
xs,z	1,157	m
w0	1,000	
wx,y	1,000	
wx,z	1,010	
wxLT	1,008	
MEd,1,z	10,00	kNm
MEd,2,z	0,00	kNm

Example

wsa_009b xs2
- B1 and B2: default calculation of buckling factors ky and kz according to General Formula → buckling shape is “known” for both directions

- Length of beams = 4,00m - calculation of xs for unknown buckling shape: use half of buckling length - calculation of xs for known buckling shape: according to buckling load case

Results for Beam B1
- check moments for LC1 = buckling load case = load case as in General formula → inflextion point for My is to be found at dx = +-3,00m Thus the distance left the support in yy-direction is +-1,00m → xs_y = 4,00m – 3,00m = 0,994m - xs_z = 4,00m

Combined Bending and Axial Compression Check
According to EN 1999-1-1 article 6.3.3.1, 6.3.3.2 and formula (6.59),(6.63).

Table of values		
Eta,c (6.61a)	0,80	
Xi,yc (6.61b)	0,80	
Xi,zc (6.61c)	0,80	
Gamma,c	1,00	
Alpha,y	1,00	
Alpha,z	1,00	
NRd	761,88	kN
My,Rd	44,28	kNm
Mz,Rd	6,26	kNm

Unity check y-y (6.59) = 0,00 + 0,04 = 0,04 -
Unity check z-z (6.59) = 0,00 + 1,28 = **1,28** -
Unity check (6.63) = 0,00 + 0,32 + 1,22 = **1,54** -

Table of values		
Method for xs,y	Buckling Loadcase	
Method for xs,z	Buckling Loadcase	
xs,y	0,995	m
xs,z	4,000	m
w0	1,000	
wx,y	1,000	
wx,z	1,000	
wxLT	1,000	

Shear Buckling

The shear buckling check is verified using art. 6.7.4 & 6.7.6 Ref.[1].
Distinction is made between two separate cases:

- No stiffeners are inputted on the member or stiffeners are inputted only at the member ends.
- Any other input of stiffeners (at intermediate positions, at the ends and intermediate positions …).

The first case is verified according to art. 6.7.4.1 Ref.[1]. The second case is verified according to art. 6.7.4.2 Ref.[1].

Note:
For shear buckling only transverse stiffeners are supported. Longitudinal stiffeners are not supported. In all cases rigid end posts are assumed.

Plate girders with stiffeners at supports

No stiffeners are inputted on the member or stiffeners are inputted only at the member ends. The verification is done according to 6.7.4.1 Ref.[1].

The check is executed when the following condition is met:

$$\frac{h_w}{t_w} > \frac{2{,}37}{\eta}\sqrt{\frac{E}{f_0}}$$

With:	h_w	Web height
	t_w	Web thickness
	η	Factor for shear buckling resistance in the plastic range
	E	Modulus of Young
	f_0	0,2% proof strength

The design shear resistance $\mathbf{V_{Rd}}$ for shear buckling consists of one part: the contribution of the web $\mathbf{V_{w,Rd}}$.

The slenderness λ_w is calculated as follows:

$$\lambda_w = 0{,}35\frac{h_w}{t_w}\sqrt{\frac{f_0}{E}}$$

Using the slenderness λ_w the factor for shear buckling ρ_v is obtained from the following table:

Ranges of λ_w	ρ_v for rigid stiffener
$\lambda_w \leq \frac{0{,}83}{\eta}$	η
$\frac{0{,}83}{\eta} < \lambda_w < 0{,}937$	$\frac{0{,}83}{\lambda_w}$
$0{,}937 \leq \lambda_w$	$\frac{2{,}3}{1{,}66+\lambda_w}$

In this table, the value of η is taken as follows:

$$\eta = 0{,}7 + 0{,}35\, f_{uw} / f_{0w} \quad \text{but} \quad \eta \leq 1{,}2$$

With: $\mathbf{f_{uw}}$ Ultimate strength of the web material
$\mathbf{f_{0w}}$ Yield strength of the web material

The contribution of the web $\mathbf{V_{w,Rd}}$ can then be calculated as follows:

$$V_{w,Rd} = \rho_v t_w h_w \frac{f_0}{\sqrt{3}\gamma_{M1}}$$

For interaction see paragraph “ Interaction ”.

Plate girders with intermediate web stiffeners

Any other input of stiffeners (at intermediate positions, at the ends and intermediate positions …). The verification is done according to 6.7.4.2 Ref.[1].

The check is executed when the following condition is met:

$$\frac{h_w}{t_w} > \frac{1{,}02}{\eta}\sqrt{\frac{k_\tau E}{f_0}}$$

With:	h_w	Web height
	t_w	Web thickness
	η	Factor for shear buckling resistance in the plastic range
	k_τ	Shear buckling coefficient for the web panel
	E	Modulus of Young
	f_0	0,2% proof strength

The design shear resistance $\mathbf{V_{Rd}}$ for shear buckling consists of two parts: the contribution of the web $\mathbf{V_{w,Rd}}$ and the contribution of the flanges $\mathbf{V_{f,Rd}}$.

Contribution of the web

Using the distance **a** between the stiffeners and the height of the web $\mathbf{h_w}$ the shear buckling coefficient $\mathbf{k_\tau}$ can be calculated:

$$k_\tau = 5{,}34 + 4{,}00\left(\frac{h_w}{a}\right)^2 \quad \text{if} \quad \frac{a}{h_w} \geq 1$$

$$k_\tau = 4{,}00 + 5{,}34\left(\frac{h_w}{a}\right)^2 \quad \text{if} \quad \frac{a}{h_w} < 1$$

The value $\mathbf{k_\tau}$ can now be used to calculate the slenderness $\boldsymbol{\lambda_w}$.

$$\lambda_w = \frac{0{,}81}{\sqrt{k_\tau}}\frac{h_w}{t_w}\sqrt{\frac{f_0}{E}}$$

Using the slenderness λ_w the factor for shear buckling ρ_v is obtained from the following table:

Ranges of λ_w	ρ_v for rigid stiffener
$\lambda_w \leq \frac{0{,}83}{\eta}$	η
$\frac{0{,}83}{\eta} < \lambda_w < 0{,}937$	$\frac{0{,}83}{\lambda_w}$
$0{,}937 \leq \lambda_w$	$\frac{2{,}3}{1{,}66 + \lambda_w}$

In this table, the value of η is taken as follows:

$\eta = 0{,}7 + 0{,}35\, f_{uw} / f_{0w}$ but $\eta \leq 1{,}2$

With: f_{uw} Ultimate strength of the web material
f_{0w} Yield strength of the web material

The contribution of the web $\mathbf{V_{w,Rd}}$ can then be calculated as follows:

$$V_{w,Rd} = \rho_v t_w h_w \frac{f_0}{\sqrt{3}\, \gamma_{M1}}$$

Contribution of the flanges

First the design moment resistance of the cross-section considering only the flanges $\mathbf{M_{f,Rd}}$ is calculated.

When $M_{Ed} \geq M_{f,Rd}$ then $\mathbf{V_{f,Rd}} = 0$

When $M_{Ed} < M_{f,Rd}$ then $\mathbf{V_{f,Rd}}$ is calculated as follows:

$$V_{f,Rd} = \frac{b_f t_f^2 f_{0f}}{c\, \gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right)$$

With: $\mathbf{b_f}$ and $\mathbf{t_f}$ the width and thickness of the flange leading to the lowest resistance.

$b_f \leq 15\, t_f$ On each side of the web.

$$c = a \left(0{,}08 + \frac{4{,}4\, b_f t_f^2 f_{0f}}{t_w b_w^2 f_{0w}} \right)$$

With: f_{0f} Yield strength of the flange material
f_{0w} Yield strength of the web material

If an axial force $\mathbf{N_{Ed}}$ is present, the value of $\mathbf{M_{f,Rd}}$ is be reduced by the following factor:

$$\left(1 - \left(\frac{N_{Ed}}{(A_{f1} + A_{f2}) \frac{f_{0f}}{\gamma_{M1}}} \right) \right)$$

With: $\mathbf{A_{f1}}$ and $\mathbf{A_{f2}}$ the areas of the top and bottom flanges.

The design shear resistance $\mathbf{V_{Rd}}$ is then calculated as follows:

$$V_{Rd} = V_{w,Rd} + V_{f,Rd}$$

For interaction see paragraph "

Interaction ".

Interaction

If required, for both above cases the interaction between shear force, bending moment and axial force is checked according to art. 6.7.6.1 Ref.[1].

If $M_{Ed} > M_{f,Rd}$ the following two expressions are checked:

$$\frac{M_{Ed}+M_{f,Rd}}{2M_{pl,Rd}}+\frac{V_{Ed}}{V_{w,Rd}}\left(1-\frac{M_{f,Rd}}{M_{pl,Rd}}\right)\leq 1{,}00$$

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

With: $M_{c,Rd}=W_{eff}\,f_0/\gamma_{M1}$

$M_{f,Rd}$ design moment resistance of the cross-section considering only the flanges
$M_{pl,Rd}$ Plastic design bending moment resistance

If an axial force **N_{Ed}** is also applied, then **$M_{pl,Rd}$** is replaced by the reduced plastic moment resistance **$M_{N,Rd}$** given by:

$$M_{N,Rd}=M_{pl,Rd}\left(1-\left(\frac{N_{Ed}}{(A_{f1}+A_{f2})\frac{f_{0f}}{\gamma_{M1}}}\right)^2\right)$$

With: **A_{f1}** and **A_{f2}** the areas of the top and bottom flanges.

Example

wsa_010 shear buckling - stiffeners
- B1, B2 and B3 loaded by line load 10kN/m - B4 loaded by line load of 10kN/m and normal compression force of 1200kN - B1: no stiffeners - B2: stiffeners at ends - B3: stiffeners at ends and interior - B4: stiffeners at ends and interior - input of result-sections at beginning of beams

Results for Beam B1
- using formula (6.122) and (6.147, Interaction) - V_{Rd} = $V_{w,Rd}$ = 905,61kN - u.c. = 0,03

Shear buckling check.
According to EN 1999-1-1 article 6.7.4.1 & 6.7.6.1 and formula (6.122), (6.147).
Rigid end posts

Table of values		
hw/tw	81,333	
Eta	1,18	
Lambda,w	1,541	
Rho,v	0,72	
Af1	3600	mm^2
Af2	3600	mm^2
Mf,Rd	662,86	kNm
VRd	905,61	kN
Mpl,Rd	1157,10	kNm
Mc,Rd	747,45	kNm
Unity check (6.122)	0,03	-
Unity check (6.147 curve 2)	-	
Unity check (6.147 curve 3)	-	

The member satisfies the stability check.

Results for Beam B2
- stiffeners at ends, idem as results for B1
- using formula (6.122) and (6.147, Interaction)
- V_{Rd} = $V_{w,Rd}$ = 905,61kN
- u.c. = 0,03

Results for Beam B3
- stiffeners at ends and intermediate stiffeners
- a = distance between stiffeners = 1,5m
- using formula (6.124) and (6.147, Interaction)
- V_{Rd} = $V_{w,Rd}$ + $V_{f,Rd}$ = 964,75 + 54,28 = 1019,03kN
- u.c. = 0,03

Shear buckling check.
According to EN 1999-1-1 article 6.7.4.2 & 6.7.6.1 and formula (6.124), (6.147).
Rigid end posts

Table of values		
hw/tw	81,333	
a	1500	mm
k,Tau	7,033	
Eta	1,18	

Table of values		
Lambda,w	1,344	
Rho,v	0,77	
Af1	3600	mm^2
Af2	3600	mm^2
c	145	mm
Mf,Rd	662,86	kNm
Vw,Rd	964,75	kN
Vf,Rd	54,28	kN
VRd	1019,03	kN
Mpl,Rd	1157,10	kNm
Mc,Rd	747,45	kNm
Unity check (6.124)	0,03	-
Unity check (6.147 curve 2)	-	
Unity check (6.147 curve 3)	-	

The member satisfies the stability check.

Results for Beam B4

- stiffeners at ends and intermediate stiffeners (+ extra normal force)
- a = distance between stiffeners = 1,5m
- using formula (6.124) and (6.147, Interaction)
- Normal force exist, $M_{f,Rd}$ so needs to be reduced
- $M_{Ed} > M_{f,Rd}$ → shear contribution of the flanges may not be taken into account
- $V_{Rd} = V_{w,Rd} + V_{f,Rd} = 964{,}75 + 0{,}00 = 964{,}75$kN
- u.c. = 0,03 (6.122)

- u.c. = 0,39 (6.147 curve (2))
- u.c. = 0,13 (6.147 curve (3))

Shear buckling check.

According to EN 1999-1-1 article 6.7.4.2 & 6.7.6.1 and formula (6.124), (6.147).
Rigid end posts

Table of values		
hw/tw	81,333	
a	1500	mm
k,Tau	7,033	
Eta	1,18	
Lambda,w	1,344	
Rho,v	0,77	
Af1	3600	mm^2
Af2	3600	mm^2
c	145	mm
Mf,Rd	70,06	kNm
Vw,Rd	964,75	kN
Vf,Rd	0,00	kN
VRd	964,75	kN
Mpl,Rd	231,67	kNm
Mc,Rd	747,45	kNm
Unity check (6.124)	0,03	-
Unity check (6.147 curve 2)	0,39	-
Unity check (6.147 curve 3)	0,13	-

The member does NOT satisfy the stability check!

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