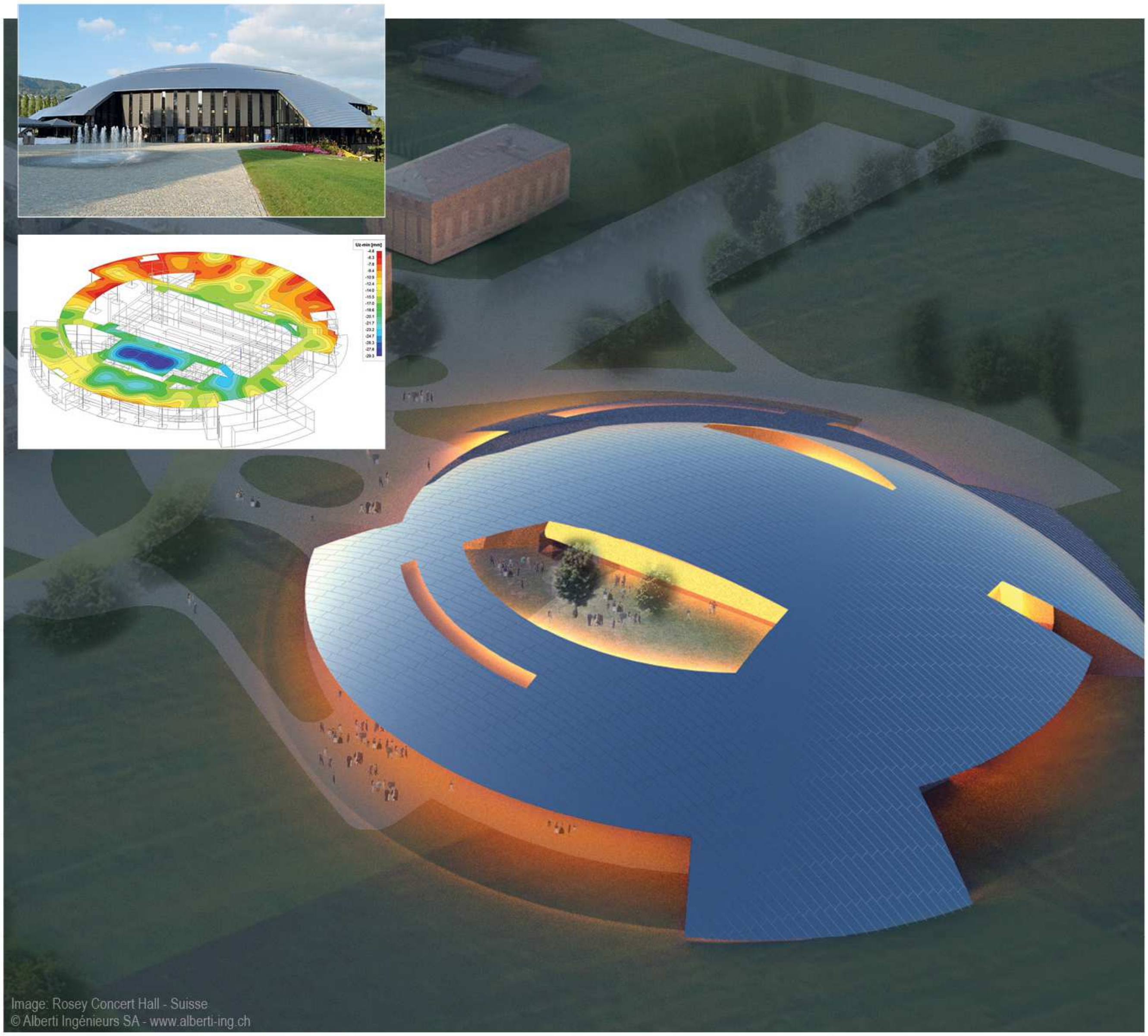
SCIA ENGINEER



Eurocode Training

EN 1999-1-1 in SCIA Engineer

Table of contents

Introduction

Edition

This course will explain the calculation and checking of aluminium profiles in SCIA Engineer. The aluminium code check is included in SCIA Engineer in a separate module:

esaad.01.01: Design of Aluminium Structures

This module is needed for the checks described in this document. Except this supplementary module, most of the other modules necessary for the options described in this manual, are included in the **Concept edition**. For some options a professional, an expert edition or an extra module is required. This will always be indicated in the corresponding paragraph.

Overview

The Structural Eurocode program comprises the following standards generally consisting of a number of Parts:

EN 1990	Eurocode:	Basis of structural design
EN 1991	Eurocode 1:	Action on structures
EN 1992	Eurocode 2:	Design of concrete structures
EN 1993	Eurocode 3:	Design of steel structures
EN 1994	Eurocode 4:	Design of composite steel and concrete structures
EN 1995	Eurocode 5:	Design of timber structures
EN 1996	Eurocode 6:	Design of masonry structures
EN 1997	Eurocode 7:	Geotechnical design
EN 1998	Eurocode 8:	Design of structures for earthquake resistance
EN 1999	Eurocode 9:	Design of aluminium structures

EN 1999 is intended to be applied to the design of aluminium structures

Eurocode 9 is subdivided in five parts:

EN 1999-1-1:	Design of Aluminium Structures: General structural rules.
EN 1999-1-2:	Design of Aluminium Structures: Structural fire design.
EN 1999-1-3:	Design of Aluminium Structures: Structures susceptible to fatigue.
EN 1999-1-4:	Design of Aluminium Structures: Cold-formed structural sheeting.
EN 1999-1-5:	Design of Aluminium Structures: Shell structures.

In this manual only EN 1999-1-1 (“General structural rules”) is discussed.

National annex for EN 1999-1-1
This standard gives alternative procedures and recommended values with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1997-1 should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1999-1-1 through:
- **1.1.2(1)**
- **2.1.2(3)**
- **2.3.1(1)**
- **3.2.1(1)**
- **3.2.2(1)**

- **3.2.2(2)**
- **3.2.3.1(1)**
- **3.3.2.1(3)**
- **3.3.2.2(1)**
- **5.2.1(3)**
- **5.3.2(3)**
- **5.3.4(3)**
- **6.1.3(1)**
- **6.2.1(5)**
- **7.1(4)**
- **7.2.1(1)**
- **7.2.2(1)**
- **7.2.3(1)**
- **8.1.1(2)**
- **8.9(3)**

- **A(6) (Table A.1)**

- **C.3.4.1(2)**
- **C.3.4.1(3)**
- **C.3.4.1(4)**

- **K.1(1)**
- **K.3(1)**

1. General

All kind of symbols are given in a list in EN 1999-1-1, art. 1.6.

The following conventions for the member axis are given in the EN 1999-1-1:

1.7 Conventions for member axes

(1) In general the convention for member axes is:

x-x - along the member

y-y - axis of the cross-section

z-z - axis of the cross-section

(2) For aluminium members, the conventions used for cross-section axes are:

- generally:

  y-y - cross-section axis parallel to the flanges

  z-z - cross-section axis perpendicular to the flanges

- for angle sections:

  y-y - axis parallel to the smaller leg

  z-z - axis perpendicular to the smaller leg

- where necessary:

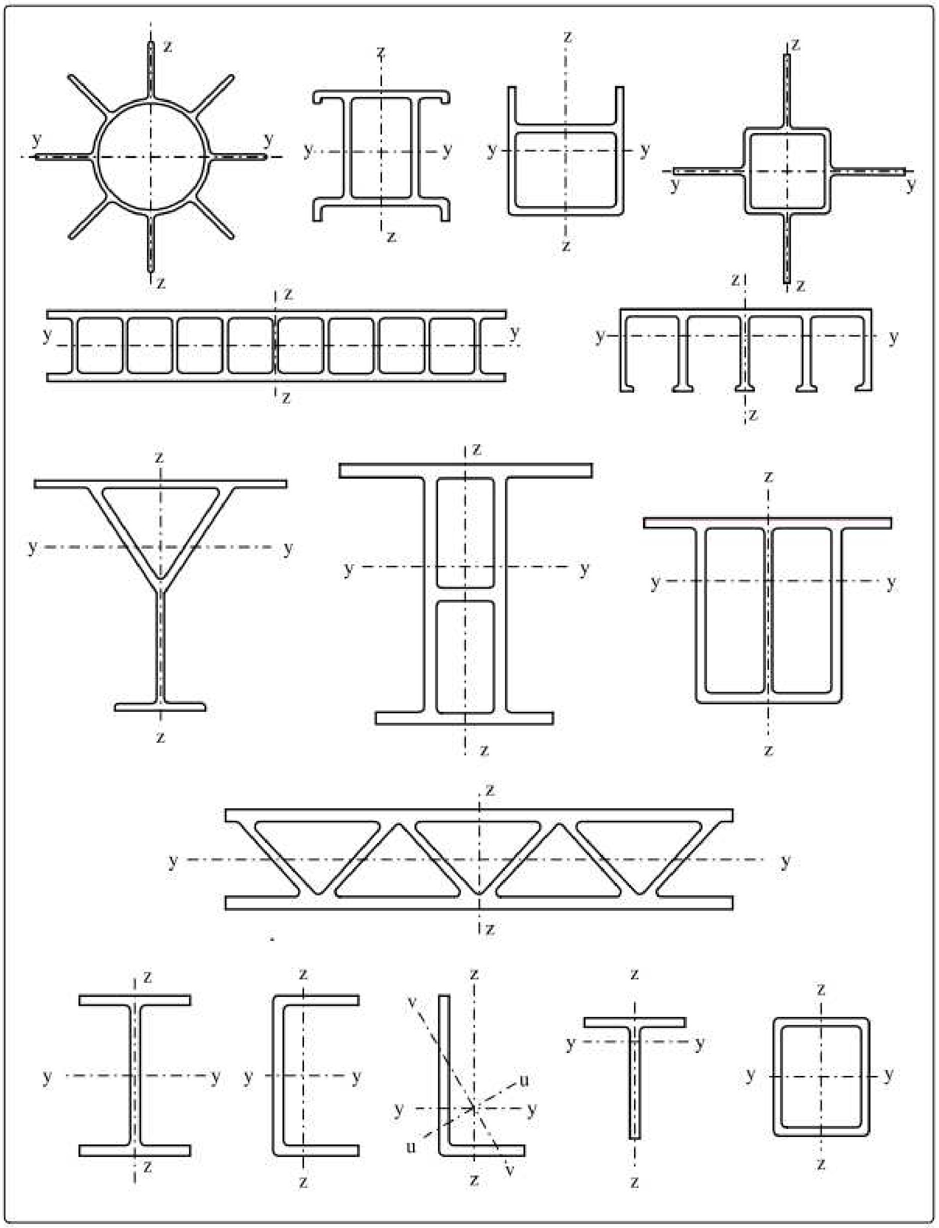
  u-u - major principal axis (where this does not coincide with the y-y axis)

v-v - minor principal axis (where this does not coincide with the z-z axis)

(3) The symbols used for dimensions and axes of aluminium sections are indicated in Figure 1.1.

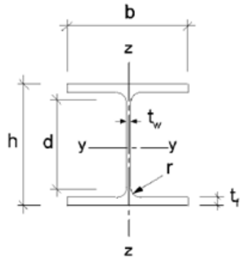
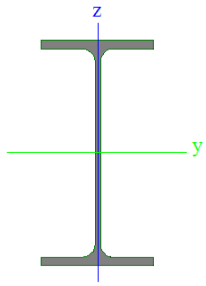
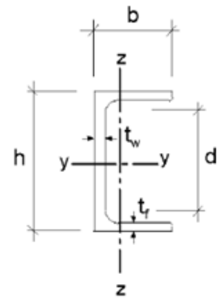
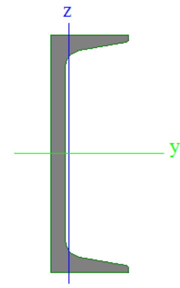
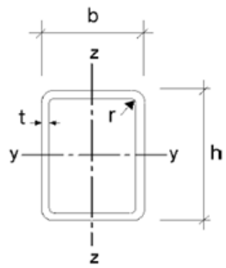
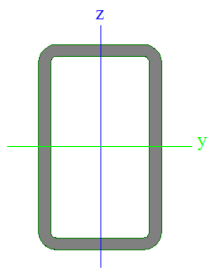
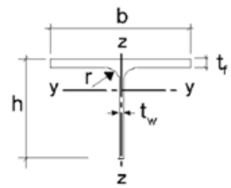
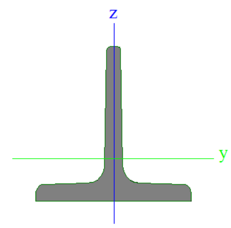
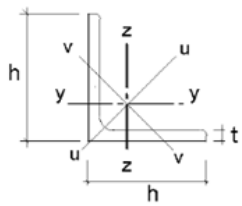
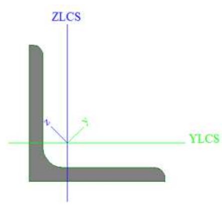
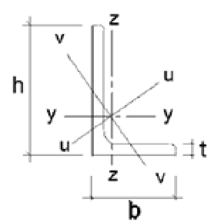
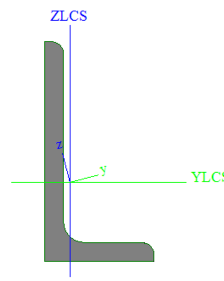
(4) The convention used for subscripts, which indicate axes for moments is: "Use the axis about which the moment acts."

NOTE All rules in this Eurocode relate to principal axis properties, which are generally defined by the axes y-y and z-z for symmetrical sections and by the u-u and v-v axis for unsymmetrical section such as angles.



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

If the principal axes not coincide with the y-y and z-z axes following the EN 1999-1-1, also those axes are indicated in SCIA Engineer. Some examples comparing the local axes following EN 1999-1-1 and SCIA Engineer are given below:

EN 1999-1-1	SCIA Engineer	EN 1999-1-1	SCIA Engineer
			
			
			

2. Basis of design

Actions for the design of steel structures should be taken from EN 1991. For the combination for actions and partial factors of actions see Annex A to EN 1990.

For aluminium structures equation (6.6c) or equation (6.6d) of EN 1990 applies:

$$R_d = \frac{1}{\gamma_M} R_k \left(\eta_1 X_{k1}; \eta_i X_{ki}; a_d\right) \tag{2.1}$$

where:

R_k is the characteristic value of resistance of a cross section or member determined with characteristic or nominal values for the material properties and cross sectional dimensions

γ_M is the global partial factor for the particular resistance

3. Materials

Material properties

In this chapter a lot of tables for standard aluminium alloys are given. For example EN 1999-1-1, Table 3.2a is given below:

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)	Thickness 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 | H24/H34	≤ 6 | 3	180 | 170	220	1 | 3	75	155	0,42 | 0,44	0,70	B	23 | 18
	H16 | H26/H36	≤ 4 | 3	200 | 190	240	1 | 3			0,38 | 0,39	0,65	B	25 | 20
3005	H14 | H24	≤ 6 | 3	150 | 130	170	1 | 4	56	115	0,37 | 0,43	0,68	B	38 | 18
	H16 | H26	≤ 4 | 3	175 | 160	195	1 | 3			0,32 | 0,35	0,59	B	43 | 24
3103	H14 | H24	≤ 25 | 12,5	120 | 110	140	2 | 4	44	90	0,37 | 0,40	0,64	B	31 | 20
	H16 | H26	≤ 4	145 | 135	160	1 | 2			0,30 | 0,33	0,56	B	48 | 28
5005/ 5005A	O/H111	≤ 50	35	100	15	35	100	1	1	B	5
	H12 | H22/H32	≤ 12,5	95 | 80	125	2 | 4	44	100	0,46 | 0,55	0,80	B	18 | 11
	H14 | H24/H34	≤ 12,5	120 | 110	145	2 | 3			0,37 | 0,40	0,69	B	25 | 17
5052	H12 | H22/H32	≤ 40	160 | 130	210	4 | 5	80	170	0,50 | 0,62	0,81	B	17 | 10
	H14 | H24/H34	≤ 25	180 | 150	230	3 | 4			0,44 | 0,53	0,74	B	19 | 11
5049	O / H111	≤ 100	80	190	12	80	190	1	1	B	6
	H14 | H24/H34	≤ 25	190 | 160	240	3 | 6	100	190	0,53 | 0,63	0,79	B	20 | 12
5454	O/H111	≤ 80	85	215	12	85	215	1	1	B	5
	H14|H24/H34	≤ 25	220 | 200	270	2 | 4	105	215	0,48 | 0,53	0,80	B	22 | 15
5754	O/H111	≤ 100	80	190	12	80	190	1	1	B	6
	H14|H24/H34	≤ 25	190 | 160	240	3 | 6	100	190	0,53 | 0,63	0,79	B	20 | 12
5083	O/H111	≤ 50	125	275	11	125	275	1	1	B	6
		50<t≤80	115	270	14 3)	115	270			B	
	H12|H22/H32	≤ 40	250 | 215	305	3 | 5	155	275	0,62 | 0,72	0,90	B	22 | 14
	H14|H24/H34	≤ 25	280 | 250	340	2 | 4			0,55 | 0,62	0,81	A	22 | 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 | H24	≤ 12,5	110 | 100	125	2 | 3	37	85	0,34 | 0,37	0,68	B	37 | 22
	H16 | H26	≤ 4	130 | 120	145	1 | 2			0,28 | 0,31	0,59		33 | 33

1) If two (three) tempers are specified in one line, tempers separated by "|" 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.

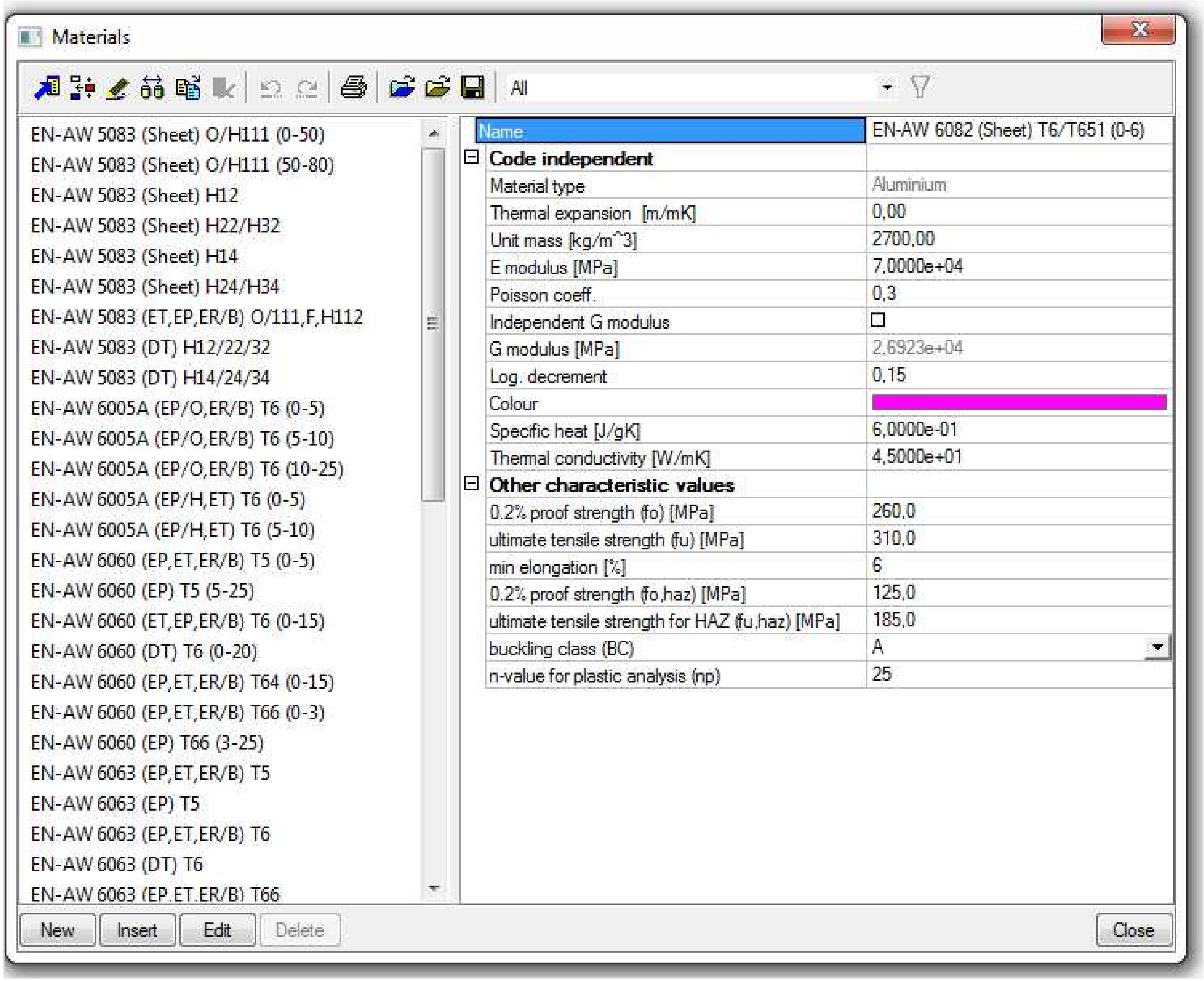
3) Based on A $(= A_{5,65\sqrt{A_o}})$, not A_{50}.

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.

Also in SCIA Engineer those materials are implemented:



<u>National annexes:</u>

NBN: De ANB geeft geen bijkomende informatie omtrent de andere aluminiumlegeringen of toestanden

NEN: No other aluminium alloys are allowed than those listed in Tables 3.1a and 3.1b.

Design values of material coefficients

EN 1999-1-1 **article 3.2.5**

The material constants to be adopted in calculations for the aluminium alloys covered by this European Standard should be taken as follows:

.

- Modulus of elasticity: $E = 70\ 000\ N/mm^2$
- Shear Modulus: $G = 27\ 000\ N/mm^2$
- Poisson's ratio in elastic stage: $\nu = 0,3$
- Coefficient of linear thermal expansion: $\alpha = 23 \times 10^{-6}$ perK (for $T \leq 100$ °C)
- Unit mass: $\rho = 2\ 700\ kg/m^3$

4. Durability

The basic requirements for durability are set out in EN 1990. For aluminium in contact with other material, recommendations are given in EN 1999-1-1, Annex D.

Under normal atmospheric conditions, aluminium structures made of alloys listed in Tables 3.1a and 3.1.b can be used without the need for surface protection to avoid loss of load-bearing capacity.

Components susceptible to corrosion and subject to aggressive exposure, mechanical wear or fatigue should be designed such that inspection, maintenance and repair can be carried out satisfactorily during the design life. Access should be available for service inspection and maintenance.

5. Structural analysis

Global analysis

EN 1999-1-1 **article 5.2**

The internal forces and moments may generally be determined using either:

- First order analysis, using the initial geometry of the structure or
- Second-order analysis, taking into account the influence of the deformation of the structure.

First order analysis may be used for the structure, if the relevant internal forces or moments or any other change of structural behaviour caused by deformations can be neglected. This condition may be assumed to be fulfilled, if the following criterion is satisfied:

$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10$ for elastic analysis (5.1)

Where:
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.

National annexes:

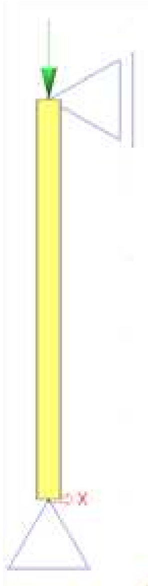
NBN: De ANB geeft geen verschillend criterium voor de grenswaarde α_{cr} om de invloed van de tweedeorde-effecten te verwaarlozen.

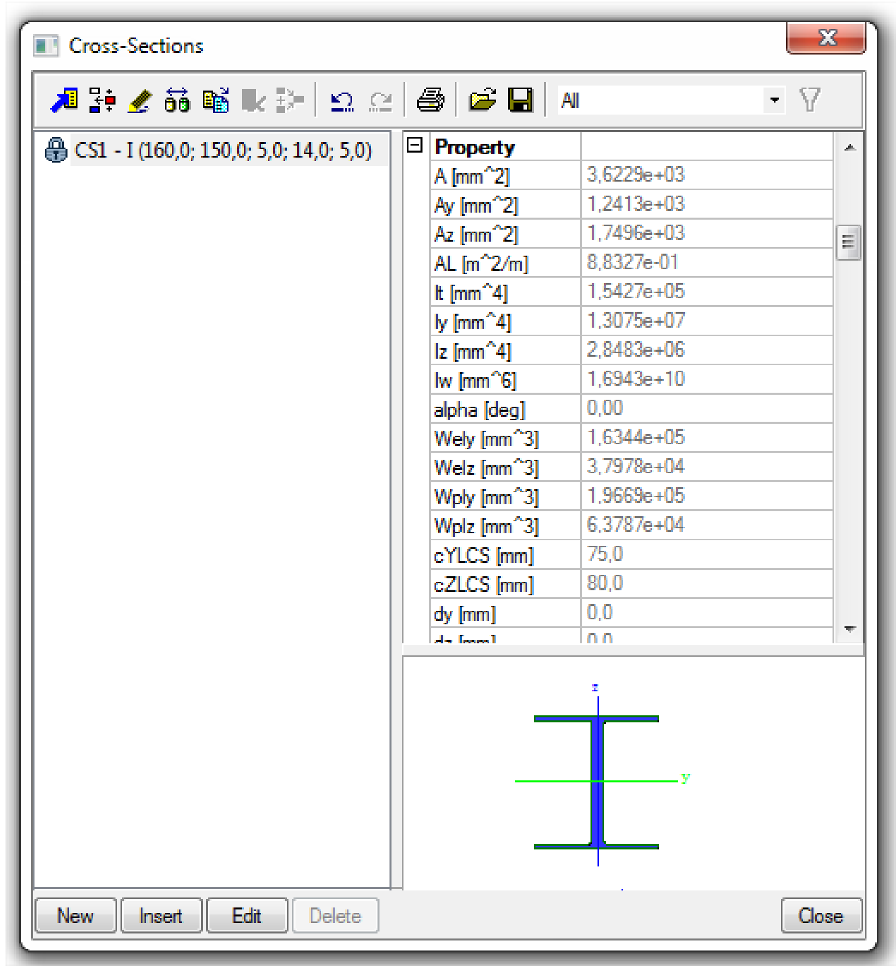
NEN: No lower limits for α_{cr} are allowed than given with equation (5.1).

With SCIA Engineer the value for α_{cr} can be calculated using a stability calculation.

Example: Calculation_Alpha_cr.esa

The column has the cross-section that is shown below, and is fabricated from **EN-AW 6082 (Sheet) T6/T651 (6-12.5)** and has the following relevant properties:





Calculation of α_{cr}

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

Critical load coefficients

Critical load coefficients	
N -	f []
Stability combination : S1	
1	360,82

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

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I}{l^2} = \frac{\pi^2 \cdot 70000N/mm^2 \cdot 1{,}3075 \cdot 10^7 mm^4}{(5000mm)^2} = 361{,}33\ kN$$

EN 1999-1-1 **article 5.2.2 (3)+ (4)**:

According to the type of frame and the global analysis, second order effects and imperfections may be accounted for by one of the following methods:

a) Both totally by the global analysis

b) Partially by the global analysis and partially through individual stability checks of members according to 6.3.

c) For basic cases by individual stability checks of equivalent members according to 6.3 using appropriate buckling lengths according to the global buckling mode of the structure.

Second order effects may be calculated by using an analysis appropriate to the structure. For frames where the first sway buckling mode is predominant first order analysis should be carried out with subsequent amplification of relevant action effects by appropriate factors.

EN 1999-1-1 **article 5.2.2 (5)**:

The stability of individual members should be checked according to the following:

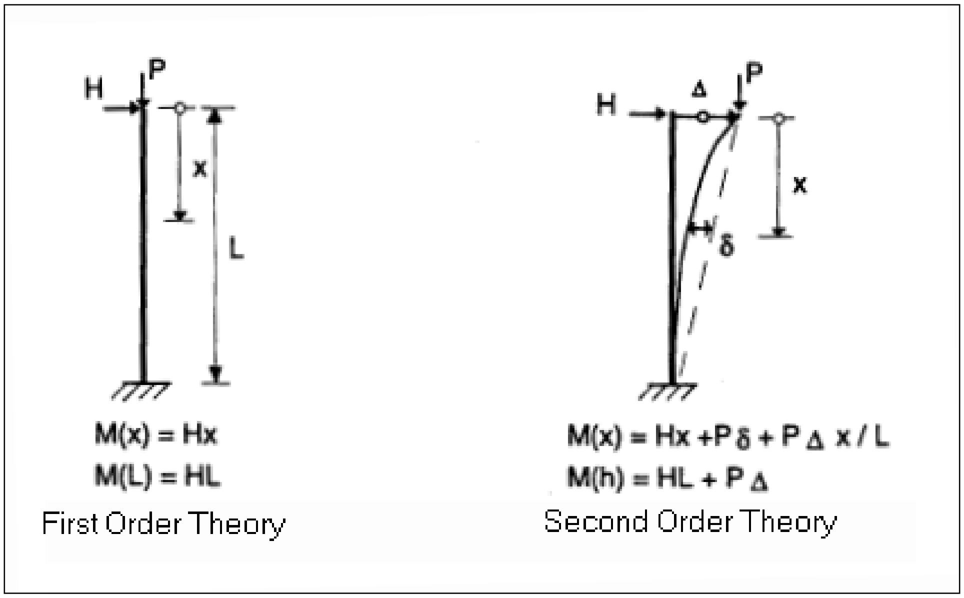
a) If second order effects in individual members and relevant member imperfections are totally accounted for in the global analysis of the structure, no individual stability check for the members according to 6.3 is necessary.

b) If second order effects in individual members or certain individual member imperfections are not totally accounted for in the global analysis, the individual stability of members should be checked according to the relevant criteria in 6.3 for the effect not included in the global analysis of the structure, including global second order effects and global imperfections when relevant and may be based on a buckling length equal to the system length.

Imperfections

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.



The following imperfections should be taken into account:

- Global imperfections for frames and bracing systems
- Local imperfections for individual members

The assumed shape of global imperfections an local imperfections may be derived from the elastic buckling mode of a structure in the plane of buckling considered.

Both in an out of plane buckling including torsional buckling in a sway mode the effect of imperfections should be allowed for in frame analysis by means of an equivalent imperfection in the form of an initial sway imperfection and individual bow imperfections of members. The imperfections may be determined from:

a) Global initial sway imperfections:

EN 1999-1-1 **article 5.3.2(3)a)**:

$$\varphi = \varphi_0 \cdot \alpha_h \cdot \alpha_m$$

$$\varphi_0 = \frac{1}{200}$$

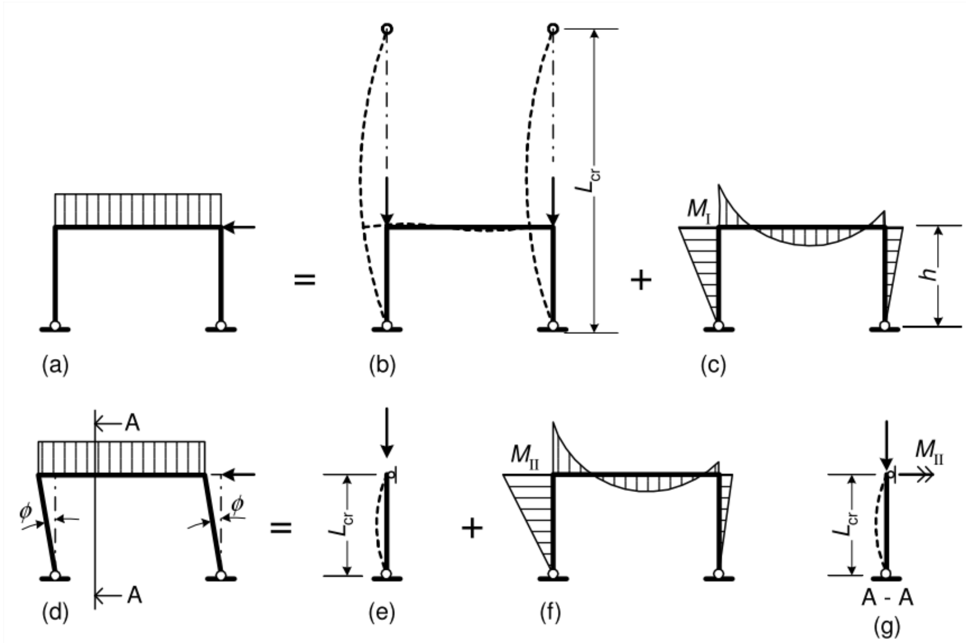
$\alpha_h = \frac{2}{\sqrt{h}}$ but $\frac{2}{3} \leq \alpha_h \leq 1{,}0$

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

With: h The height of the structure in meters

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

This can be illustrated as follows:

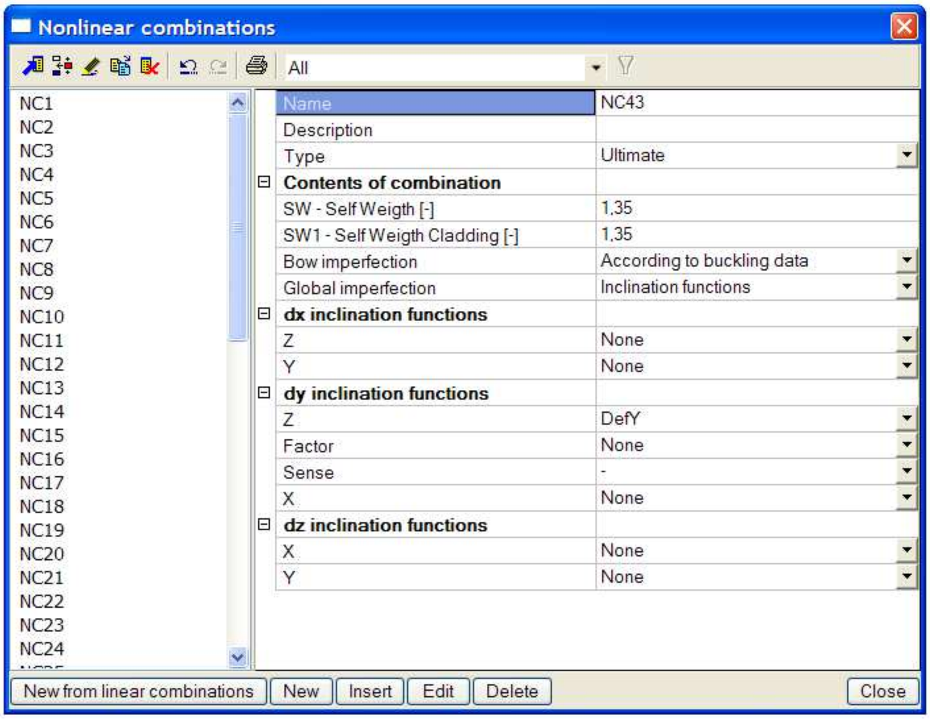


The equivalent column method is illustrated by (a), (b) and (c), where (a) is system and load, (b) is equivalent column length and (c) is the first order moment.

The equivalent sway method is illustrated by (d), (e), (f) and (g), where (d) is system, load and displacement, (e) is initial local bow and buckling length for flexural buckling, (f) is second order moment including moment from sway imperfection and (g) is initial local bow and buckling length for lateral-torsional buckling.

This can be calculated automatically by SCIA Engineer:

Name	DefY
Type	according to code
Basic imperfection value : 1...	200,00
Height of structure : [m]	8,400
Number of columns per pla...	6
Fi :	0,00263523124158382
alfa h : [-]	0,69
alfa m : [-]	0,76



b) Relative initial local bow imperfections of members for flexural buckling: e_0/L

EN 1991-1-1 **article 5.3.2(3)b):**
Recommended values are given in Table 5.1

Table 5.1 - Design values of initial bow imperfection e_0 / L

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

<u>National annexes:</u>

NBN: De aanbevolen waarden van de tabel 5.1 hieronder zijn normatief.
(Table given in this National Annex is identical on the table given above)

NEN: Under b) the values of member imperfection e_0/L shall be applied as given in Table 5.1. This table shall be read as normative.

EN 1999-1-1 **article 5.3.2(6)**:

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

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

- At least one moment resistant joint at one member end
- $\bar{\lambda} > 0{,}5 \sqrt{\frac{A \cdot f_0}{N_{Ed}}}$

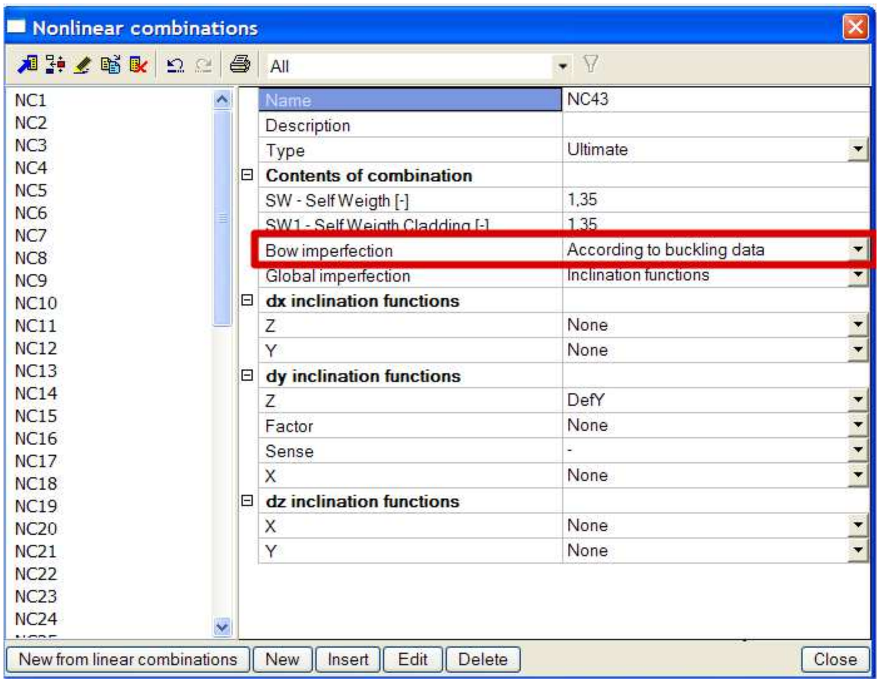
And $\bar{\lambda} = \sqrt{\frac{A \cdot f_0}{N_{cr}}}$

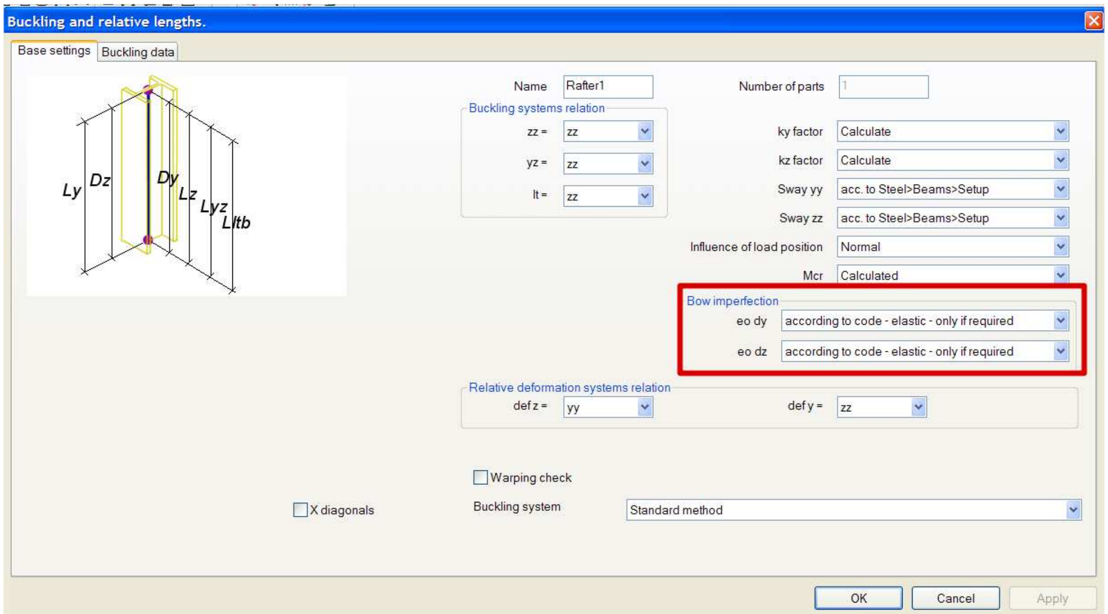
Thus: $\sqrt{\frac{A \cdot f_0}{N_{cr}}} > 0{,}5 \sqrt{\frac{A \cdot f_0}{N_{Ed}}}$

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

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

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





For buildings frame sway imperfections may be disregarded where $H_{Ed} \geq 0{,}15\ V_{Ed}$.

The effects of initial sway imperfection and local bow imperfections may be replaced by systems of equivalent horizontal forces, introduced for each column.

EN 1993-1-1 **article 5.3.2(11)**:

As an alternative the shape of the elastic buckling mode η_{cr} of the structure may be applied as an unique global and local imperfection. The amplitude of this imperfection may be determined from:

$$\eta_{init}(x) = e_{0,d} \frac{N_{cr,m}}{E \cdot I_m \cdot |\eta''_{cr}|_{max}} \eta_{cr}(x)$$

$$e_{0,d} = \alpha \cdot (\bar{\lambda}_m - \bar{\lambda}_0) \frac{M_{Rk,m}}{N_{Rk,m}} \cdot \frac{1 - \frac{\chi \cdot \bar{\lambda}_m^2}{\gamma_{M1}}}{1 - \chi \cdot \bar{\lambda}_m^2} \qquad \text{for} \qquad \bar{\lambda}_m > \bar{\lambda}_0$$

Where:

m denotes the cross-sections where "$|\eta''_{cr}|$" reaches its maximum

$$\bar{\lambda}_m = \sqrt{N_{Rk}/N_{cr}}$$

$\bar{\lambda}_0$ is the limit given in Table 6.6

α = The imperfection factor for the relevant buckling curve, see Table 6.6

χ = The reduction factor for the relevant buckling curve, see 6.3.1.2

$N_{cr,m}$ = $\alpha_{cr} N_{Ed,m}$ is the value of the axial force in cross-section m when the elastic critical buckling was reached.

α_{cr} = the minimum force amplifier for the axial force configuration N_{Ed} in members to reach the elastic critical buckling

N_{Rk} = The characteristic resistance to normal force of the critical cross-section m according to (6.22) 6.2.4

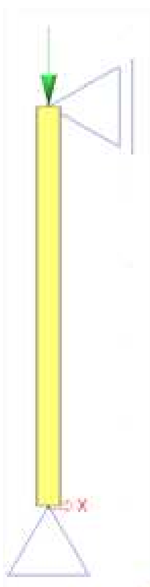
M_{Rk} = The characteristic moment resistance of the critical cross-section m according to (6.25) 6.2.5.1

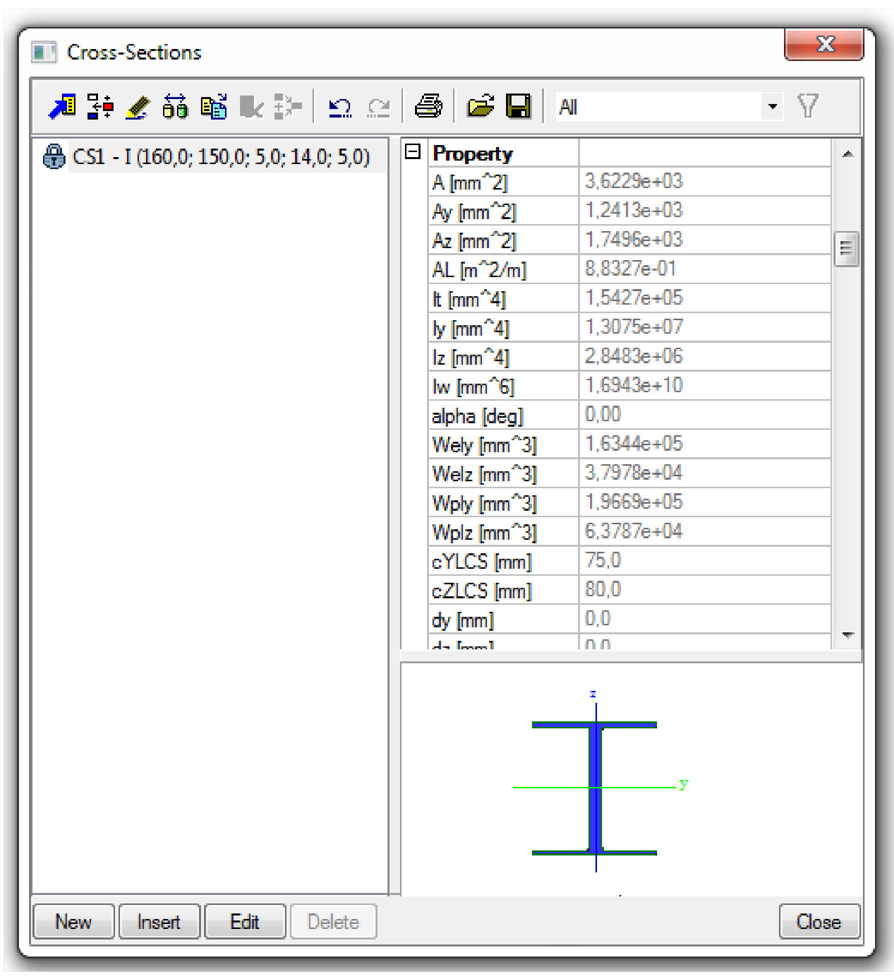
$E \cdot I_m \cdot |\eta''_{cr}|_{max}$ is the bending moment due to η

$\eta''_{cr,\max}$ = Maximal second derivative of the elastic critical buckling mode.

Example: Calculation_Alpha_cr.esa

The column has the cross-section that is shown below, and is fabricated from **EN-AW 6082 (Sheet) T6/T651 (6-12.5)** and has the following relevant properties:





Calculation of α_{cr}

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

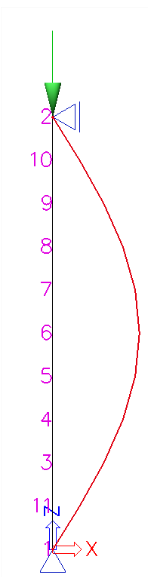
Critical load coefficients

Critical load coefficients	
N -	f []
Stability combination : S1	
1	360,82

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

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I}{l^2} = \frac{\pi^2 \cdot 70000\ N/mm^2 \cdot 1{,}3075 \cdot 10^7 mm^4}{(5000mm)^2} = 361{,}33\ kN$$

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

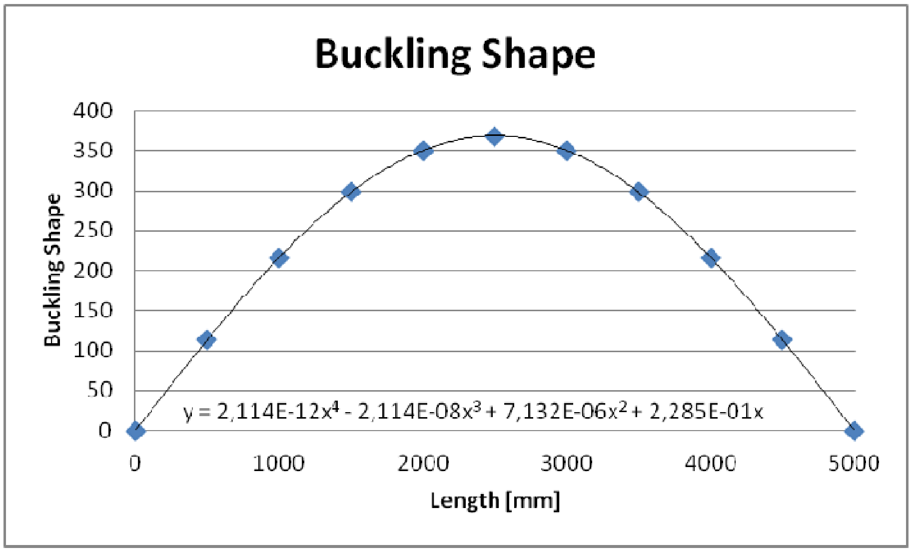


Displacement of nodes

Stability calculation, Extreme : No
Selection : All
Stability combinations : S1/1 - 360,82
Buckling shapes are dimensionless, units are printed for consistency purposes.

Node of mesh	Case	Ux [mm]	Uz [mm]	Fiy [mrad]
1	S1/1 - 360,82	0,0	0,0	231,5
11	S1/1 - 360,82	113,8	0,0	220,1
3	S1/1 - 360,82	216,5	0,0	187,3
4	S1/1 - 360,82	298,0	0,0	136,1
5	S1/1 - 360,82	350,4	0,0	71,5
6	S1/1 - 360,82	368,4	0,0	0,0
7	S1/1 - 360,82	350,4	0,0	-71,5
8	S1/1 - 360,82	298,0	0,0	-136,1
9	S1/1 - 360,82	216,5	0,0	-187,3
10	S1/1 - 360,82	113,8	0,0	-220,1
2	S1/1 - 360,82	0,0	0,0	-231,5

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



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

$$\eta_{cr} = 2{,}114 \cdot 10^{-12} \cdot x^4 - 2{,}114 \cdot 10^{-8} \cdot x^3 + 7{,}132 \cdot 10^{-6} \cdot x^2 + 2{,}2854 \cdot 10^{-1} \cdot x$$

$$\eta_{cr}'' = 2{,}537 \cdot 10^{-11} \cdot x^2 - 1{,}268 \cdot 10^{-7} \cdot x + 1{,}426 \cdot 10^{-5}$$

Calculation of e₀

$$N_{Rk} = f_0 \cdot A = 255 \frac{N}{mm^2} \cdot 3{,}62 \cdot 10^3 mm^2 = 923100\ N$$

$$M_{Rk} = f_0 \cdot W_{pl} = 255 \frac{N}{mm^2} \cdot 1{,}97 \cdot 10^5 mm^3 = 5{,}02\ \cdot 10^7 Nmm$$

$$\bar{\lambda}_m = \sqrt{\frac{N_{Rk}}{N_{cr}}} = \sqrt{\frac{923100 N}{360820\ N}} = 1{,}60$$

Table 6.6 - Values of α and $\bar{\lambda}_0$ for flexural buckling

Material buckling class according to Table 3.2	α	$\bar{\lambda}_0$
Class A	0,20	0,10
Class B	0,32	0,00

$$\bar{\lambda}_0 = 0{,}10$$
$$\alpha = 0{,}20$$

$$\phi = 0{,}5\left(1 + \alpha\left(\bar{\lambda} - \bar{\lambda}_0\right) + \bar{\lambda}^2\right) = 0{,}5(1 + 0{,}20(1{,}60 - 0{,}10) + (1{,}6)^2) = 1{,}93$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{1{,}93 + \sqrt{(1{,}93)^2 - (1{,}6)^2}} = 0{,}332$$

⇨ $$e_{0,d} = \alpha \cdot \left(\bar{\lambda}_m - \bar{\lambda}_0\right) \frac{M_{Rk,m}}{N_{Rk,m}} \cdot \frac{1 - \frac{\chi \cdot \bar{\lambda}_m^2}{\gamma_{M1}}}{1 - \chi \cdot \bar{\lambda}_m^2} = 0{,}2 \cdot (1{,}60 - 0{,}10) \frac{5{,}02 \cdot 10^7 Nmm}{923100\ N} \cdot \frac{1 - \frac{0{,}332 \cdot 1{,}60^2}{1{,}10}}{1 - 0{,}332 \cdot 1{,}60^2}$$

⇨ $$e_{0,d} = 24{,}73\ mm$$

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

Calculation of η_{init}

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

η_{cr} at mid section = 368,4

$\eta''_{cr,\max}$ at mid section = 1,443E-04 $1/mm^2$

⇨ $\eta_{init}(x) = e_{0,d}\frac{N_{cr,m}}{E\cdot I_m\cdot|\eta''_{cr}|_{max}}\eta_{cr}(x) = 24{,}73mm\frac{360820N}{\frac{70000N}{mm^2}\cdot 1{,}31\cdot 10^7 mm^4\cdot 1{,}443\cdot 10^{-4}}368{,}09$

⇨ $\eta_{init}(x) = 24{,}87mm$

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

Imperfection analysis of bracing systems

EN 1999-1-1 **article 5.3.3**.

In the analysis of bracing systems which are required to provide lateral stability within the length of beams or compressions members the effects of imperfections should included by means of an equivalent geometric imperfection of the members to be restrained, in the form of an initial bow imperfection:

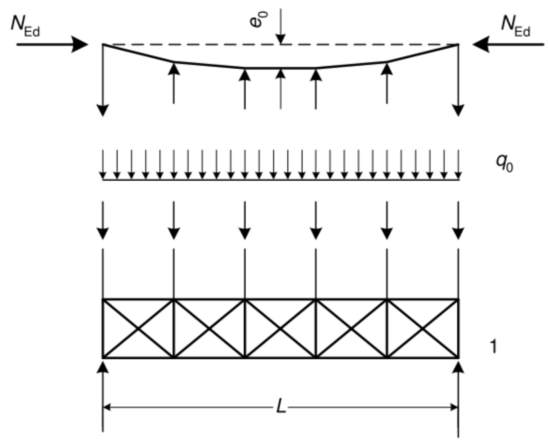
EN 1999-1-1 **Formula (5.9)**

$e_0 = \alpha_m L/500$

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

In which m is the number of members to be restrained and L is the span of the member.

For convenience, the effects of the initial bow imperfections of the members to be restrained by a bracing system, may be replaced by the equivalent stabilizing force as shown below:



e_0 *imperfection*
q_0 *equivalent force per unit length*
1 *bracing system*

The force N_{Ed} is assumed uniform within the span L of the bracing system. For non-uniform forces this is slightly conservative.

$$q_d = \sum N_{Ed}\, 8\, \frac{e_0 + \delta_q}{L^2}$$

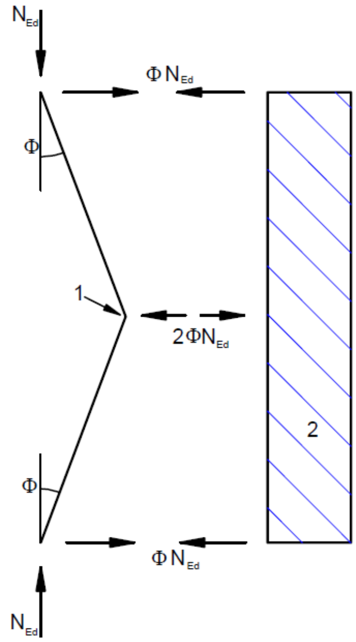
δ_q is the inplane deflection of the bracing system due to q plus any external loads calculated from first order analysis.

δ_q may be taken as 0 if second order theory is used

Where the bracing system is required to stabilize the compression flange of a beam of constant height, the force N_{Ed} may be obtained from:

$N_{Ed} = M_{Ed} / h$

At points where beams or compression members are spliced:



$\Phi = \alpha_m \Phi_0 \quad : \Phi_0 = 1 / 200$

$2\Phi N_{Ed} = \alpha_m N_{Ed} / 100$

1 splice
2 bracing system

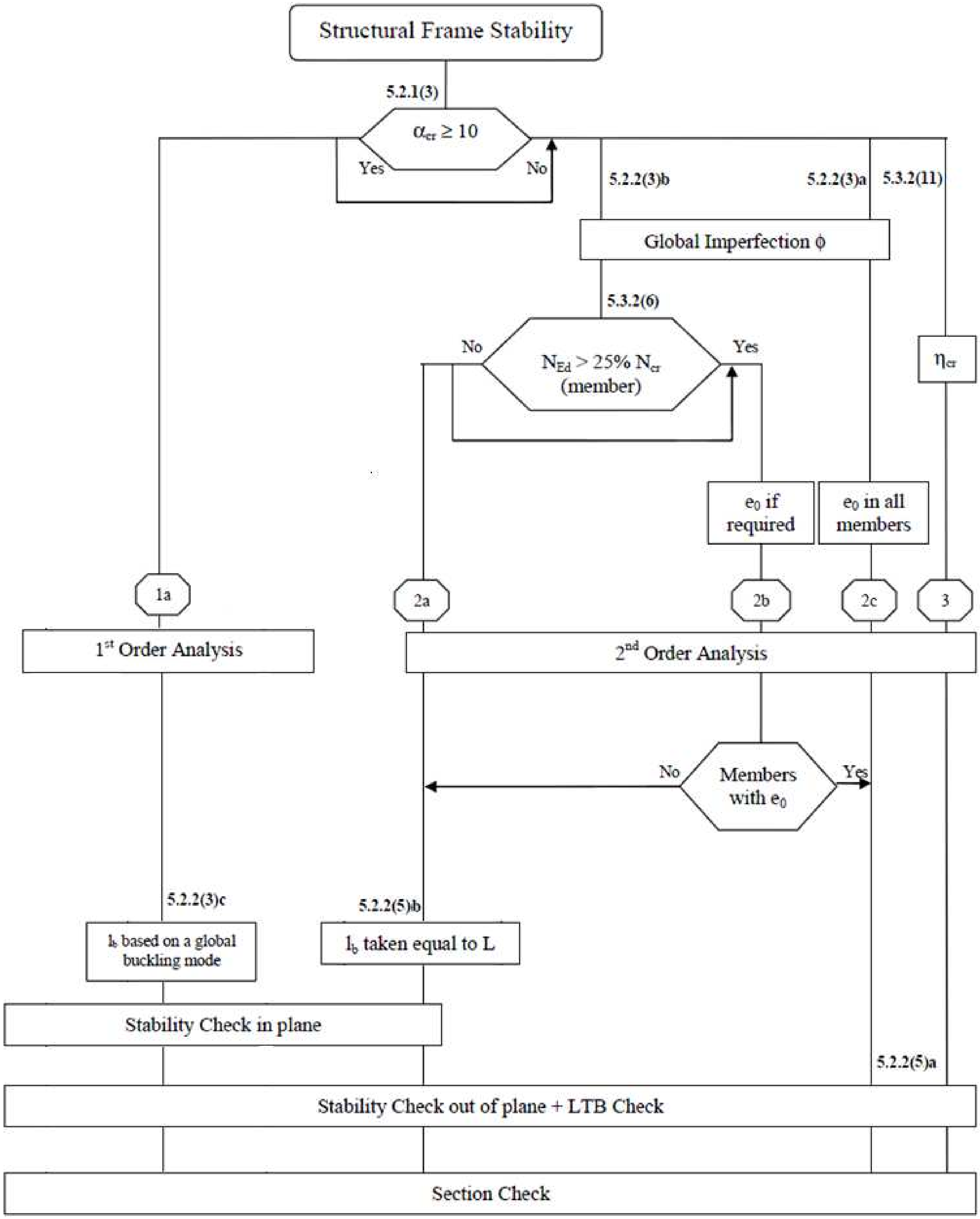
The principle of imperfection is summarized in the table on the next page:

EN 1999-1-1 **article 5.3.4**:

For a second order analysis taking account of lateral torisional buckling of a member in bending the imperfections may be adopted as $ke_{0,d}$, where $e_{0,d}$ is the equivalent initial bow imperfection of the weak axis of the profile considered.

The value of k = 0,5 is recommended.

National annexes
NEN: The value of k shall be taken as 0,5.
NBN: De aanbevolen waarde k=0,5 is normatief.



Structural Frame Stability
5.2.1(3)
$\alpha_{cr} \geq 10$
Yes
No
5.2.2(3)b
5.2.2(3)a
5.3.2(11)
Global Imperfection ϕ
5.3.2(6)
No
$N_{Ed} > 25\% \, N_{cr}$ (member)
Yes
η_{cr}
e_0 if required
e_0 in all members
1a
2a
2b
2c
3
1st Order Analysis
2nd Order Analysis
No
Members with e_0
Yes
5.2.2(3)c
5.2.2(5)b
l_b based on a global buckling mode
l_b taken equal to L
Stability Check in plane
5.2.2(5)a
Stability Check out of plane + LTB Check
Section Check

Methods of analysis

EN 1999-1-1 **article 5.4**.

The internal forces and moments may be determined using either

a) Elastic global analysis

b) Plastic global analysis

Elastic global analysis

May be used in all cases
Linear stress-strain behavior

Plastic global analysis

This analysis may be used only where the structure has sufficient rotation capacity at the actual locations of the plastic hinges.
Plastic global analysis should not be used for beams with transverse welds on the tension side of the member at the plastic hinge locations.

6. Ultimate limit state for members

Basis

Partial safety factors

EN 1999-1-1 **article 6.1**.

The following safety factors are taken into account:

γ_{M1} = 1,10 Resistance of members to instability accessed by member checks
γ_{M0} = 1,25 Resistance of cross-section in tension to fracture

National Annex:

NEN: γ_{M1} shall be taken as 1,10 and γ_{M2} shall be taken as 1,25.
For structures not covered by NEN-EN 1991-1-2 to NEN-EN 1999-1-5 the partial safety factor of NEN-EN 1999-1-1 shall be taken.

NBN: De aanbevolen waarden γ_{M1} = 1,10 en γ_{M2} = 1,25 zijn normatief.

Classification of cross-sections

Four classes of cross-sections are defined, as follows:

- 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.

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:

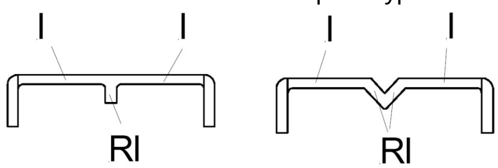
none	Not considered as reinforcement
RI	Reinforced Internal (intermediate stiffener)

RUO	Reinforced Unsymmetrical Outstand (edge stiffener)

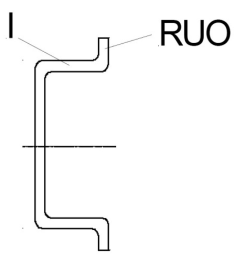
In case a part is specified as reinforcement, a reinforcement ID can be inputted. Parts having the same reinforcement ID are considered as one reinforcement.

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.



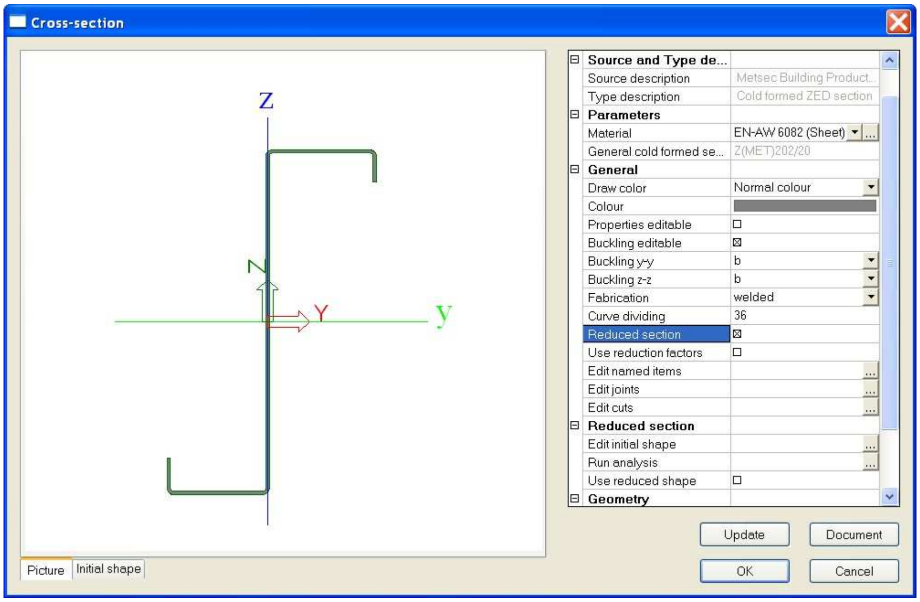
For standard cross-sections, the default type and reinforcement can be found in (Ref.[1]). For non standard section, the user has to evaluate the different parts in the cross-section.

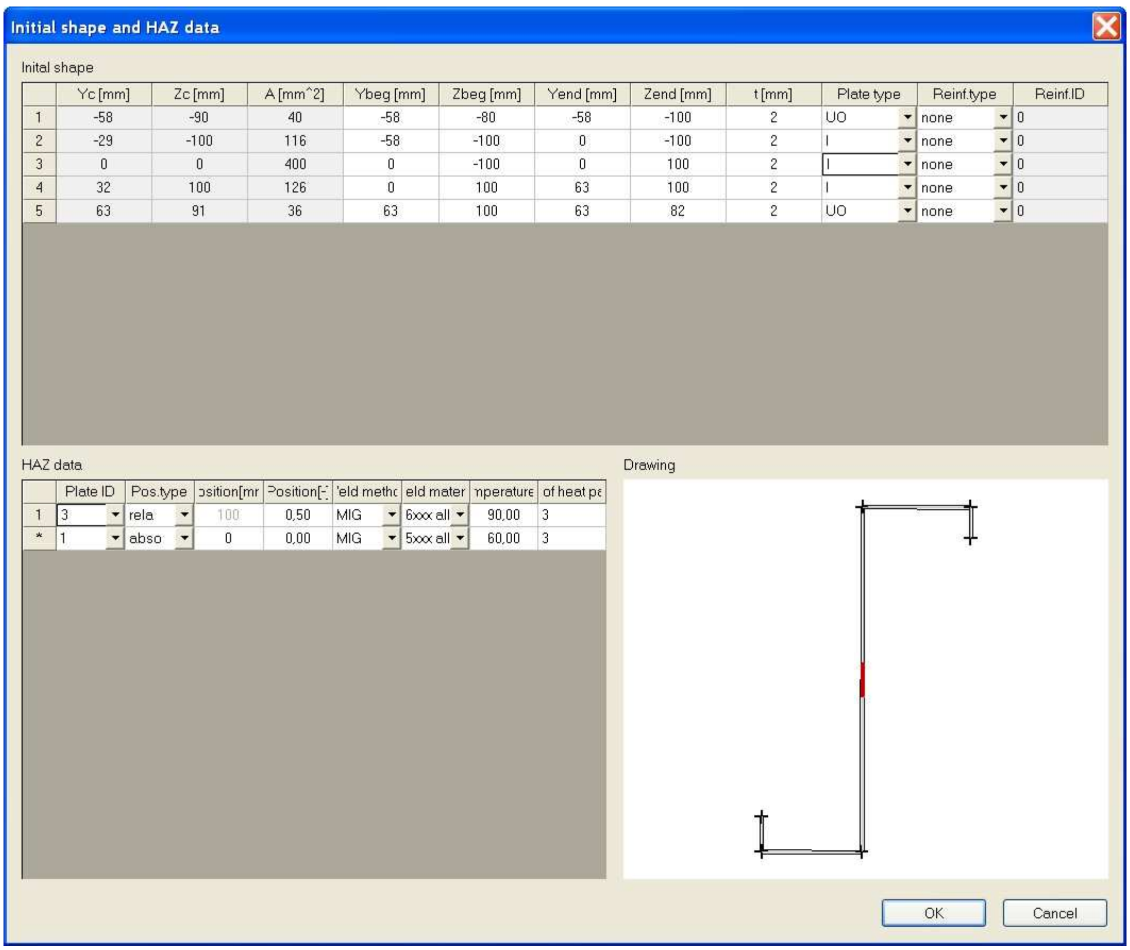
The Initial Shape can be inputted using ‘Cross-sections’ > ‘Edit’ > ‘Initial shape’. When this option is activated, the user can select ‘Edit initial shape’. In this box also welds (HAZ – Heath Affected Zone) can be inputted.

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 of the cross-section parts

See also EN 1999-1-1, art. 6.1.4.4

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						

Slenderness parameters

See also EN 1999-1-1, art. 6.1.4.3.

a) flat internal parts with no stress gradient or flat outstands with no stress gradient or peak compression at toe: β = b/t

b) internal parts with a stress gradient that results in a neutral axis at the center β = 0,40 b/t

c) internal parts with stress gradient and outstands with peak compression at root β = η b/t

Other rules for outstanding parts are also given in art. 6.1.4.3.

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]:

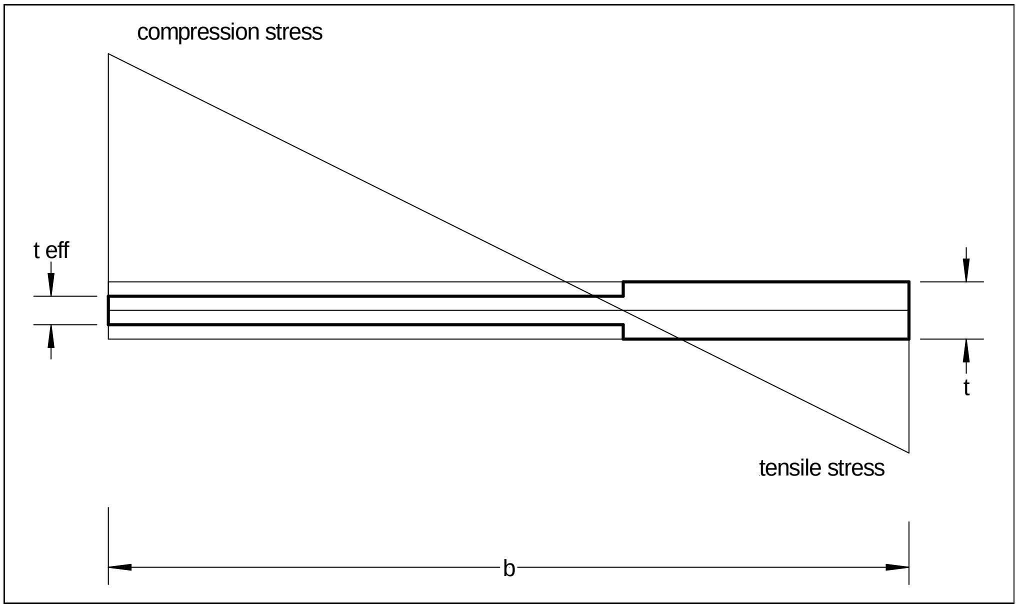
$$\rho_c = \frac{C_1}{(\beta/\varepsilon)} - \frac{C_2}{(\beta/\varepsilon)^2}$$

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.



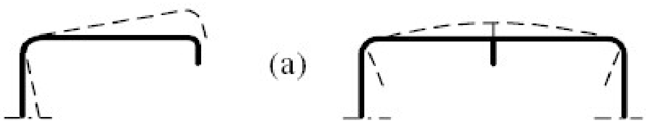
Reduction factor χ (Kappa) for distortional buckling

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.

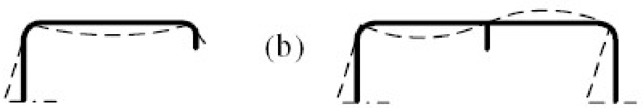
The effect of 'Local and Distortional Buckling' is explained as follows (Ref.[1]):
When considering the susceptibility of a reinforced flat part to local buckling, three possible buckling modes should be considered.

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.

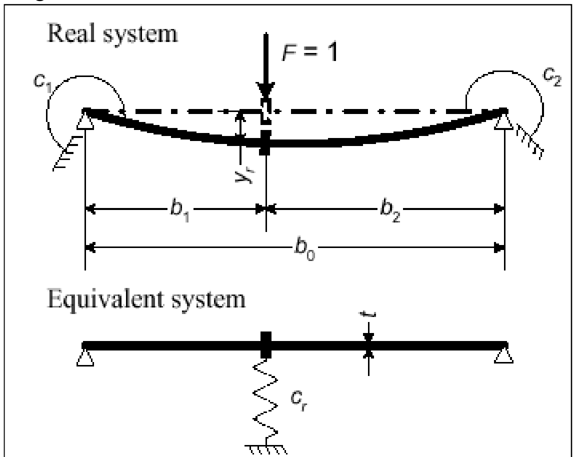
The following procedure is applied for calculating the reduction factor for an intermediate stiffener (RI) or edge stiffener (RUO):

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:



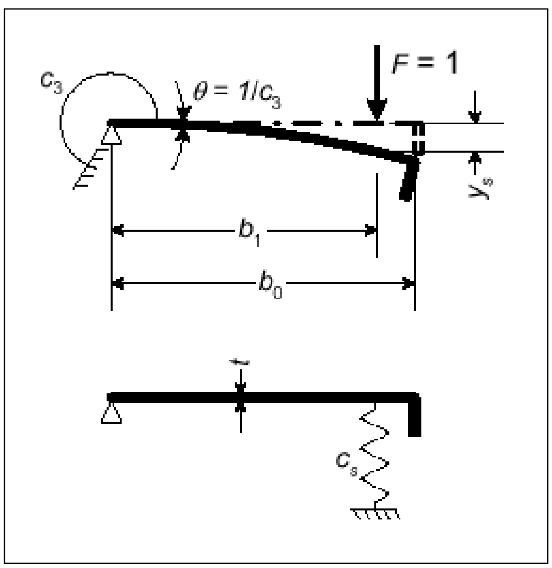
Spring stiffness

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

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

if $c_1 = c_2 = 0$

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



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

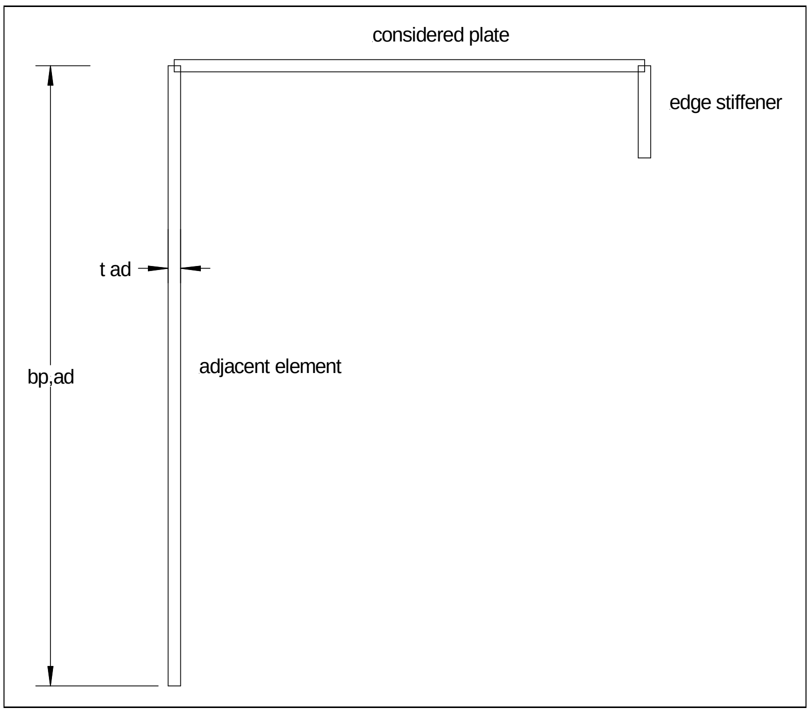
$$y_s = \frac{4(1-\nu^2)b_1^3}{Et^3} + \frac{b_1^2}{c_3}$$

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

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

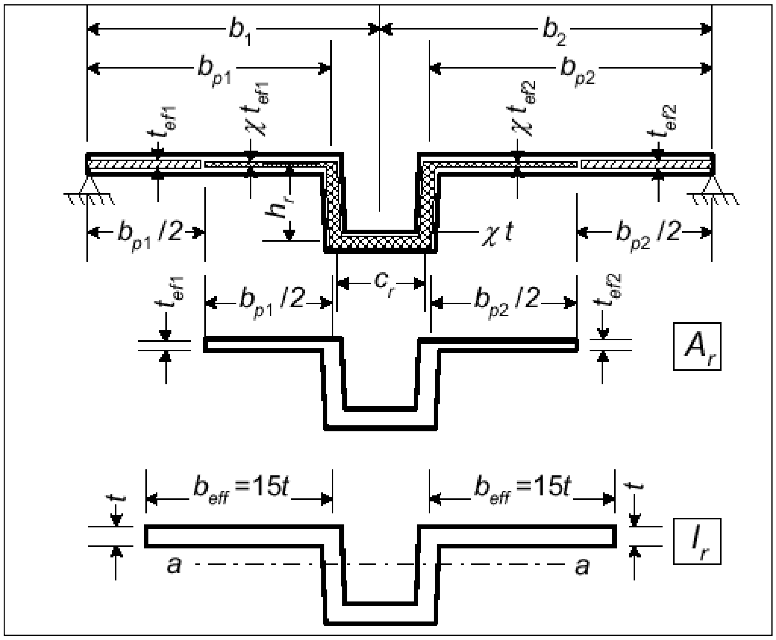
After calculating the spring stiffness the area **Ar** and Second moment of area **Ir** are calculated.

With:

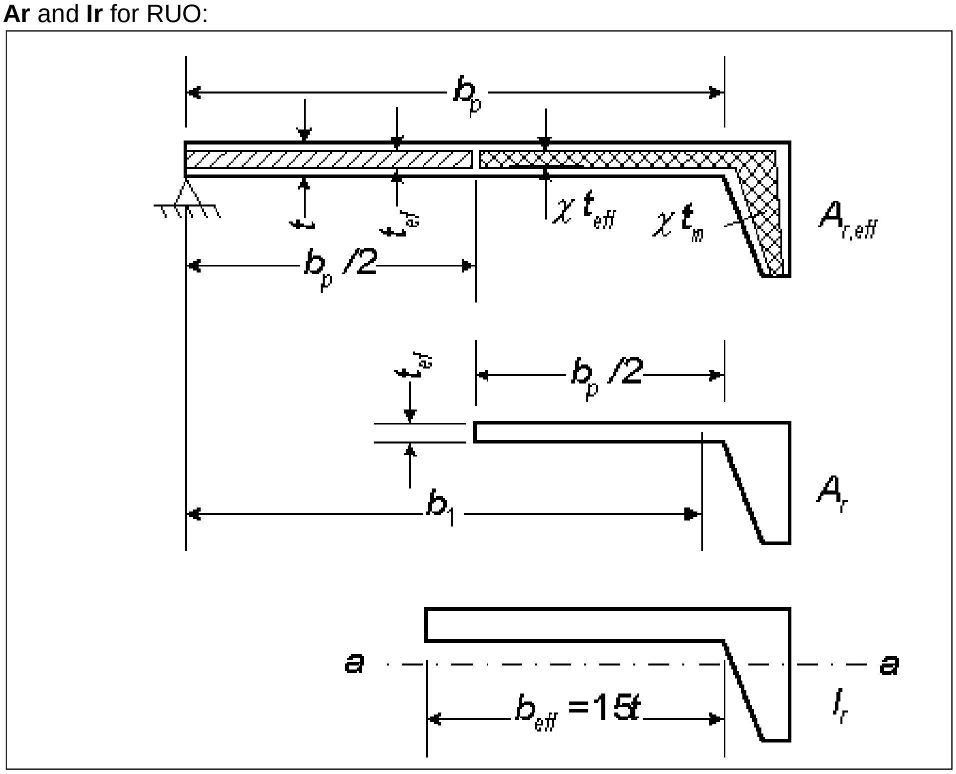
Ar	the area of the effective cross section (based on $t_{eff} = \rho_c$ t) composed of the stiffener area and half the adjacent plane elements
Ir	the second moment of area of an effective cross section composed of the (unreduced) stiffener and part of the adjacent plate elements, with thickness t and 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:

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

$$\alpha = 0.20$$

$$\lambda_0 = 0.60$$

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

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

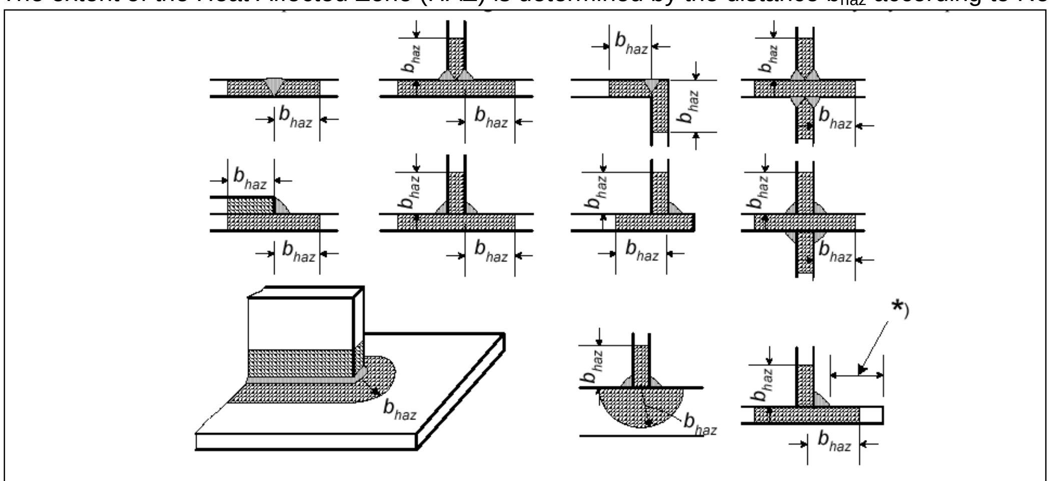
$$if \quad \lambda_c \geq \lambda_0 \quad => \chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}} \leq 1.00$$

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 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

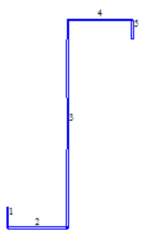
Note:
The variations in numbers of heath paths 3/n is specifically intended for fillet welds. In case of a butt weld the parameter n should be set to 3 (instead of 2) to negate this effect.

The reduction factor for the HAZ is given by:

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

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

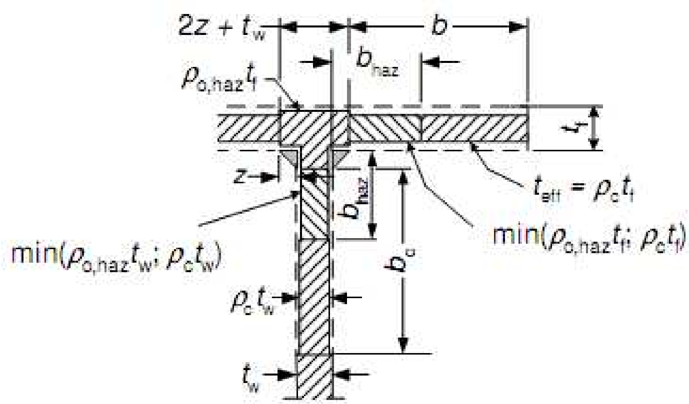
By editing a profile in SCIA Engineer, the user can evaluate for each component (N, My and Mz) the determined classification and reduction factors via the option 'Run analysis'.



Parts	Id	Psi	Sigma Beg [kN/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	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

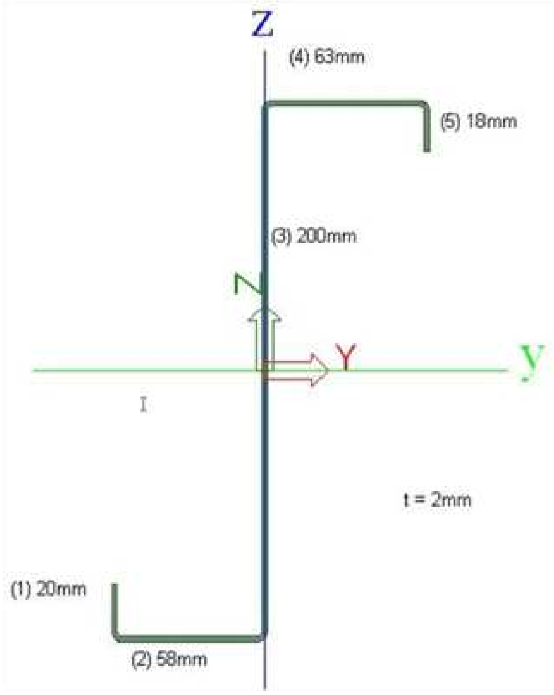
For each part the final thickness reduction ρ is determined as the minimum of $\chi.\rho_c$ and ρ_{haz}.



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	none	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	none	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)

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

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

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

				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).

		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 ρc 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):

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

	β			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

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

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

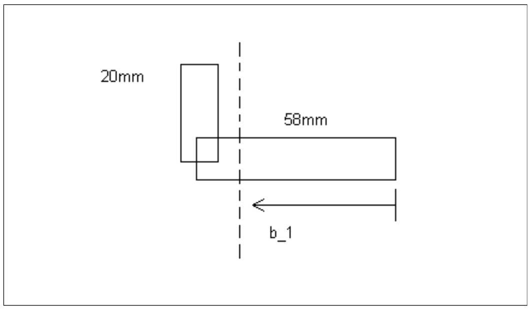
$$c_3 = \sum \frac{\alpha E t_{ad}^3}{12(1-\nu^2)b_{p,ad}}$$

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:

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



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

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

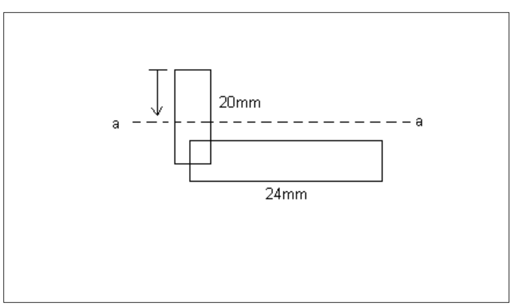
$$c = c_s = \frac{1}{y_s} = \frac{1}{2{,}903} = 0{,}344 N/mm^2$$

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

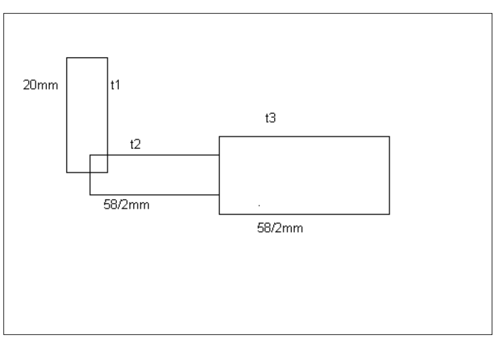
Calculation of effective thickness

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

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

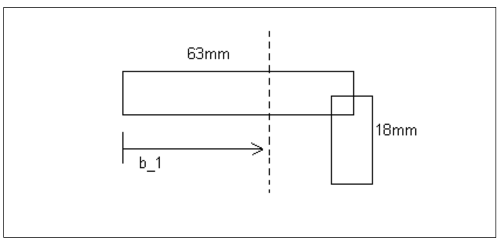
$$y_s = \frac{4(1-\nu^2)b_1^3}{Et^3} + \frac{b_1^2}{c_3}$$

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

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

Thus this gives:

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



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

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

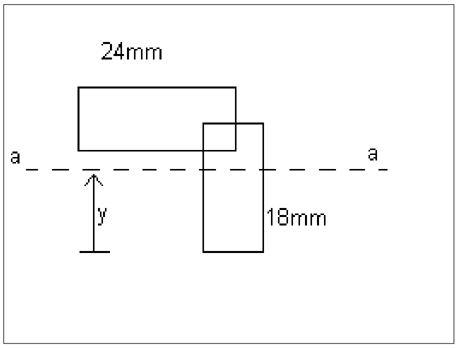
$$c = c_s = \frac{1}{y_s} = \frac{1}{3{,}26} = 0{,}3066 N/mm^2$$

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

$$y = \frac{(24\times2)\times18+(18\times2)\times\frac{18}{2}}{(24\times2)+(18\times2)} = 14{,}14mm$$

$$I_r = \frac{24\times2^3}{12} + (24\times2)\times(18-14{,}14)^2$$
$$+\frac{2\times18^3}{12} + (18\times2)\times(14{,}14-\frac{18}{2})^2 = 2654{,}29mm^4$$

Step3: calculation of stiffener buckling load

$$N_{r,cr} = 2\times\sqrt{c\times E\times I_r} = 2\times\sqrt{0{,}3066\times70000\times2654{,}29} = 15095{,}8N$$

$$\lambda_c = \sqrt{\frac{f_0\times A_r}{N_{r,cr}}} = \sqrt{\frac{205\times89{,}613}{15095{,}8}} = 1{,}103$$

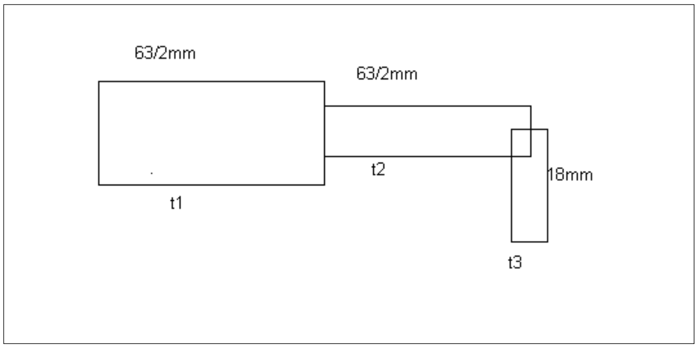
$$\alpha = 0{,}2$$
$$\lambda_0 = 0{,}60$$
$$\Rightarrow \lambda_0 > \lambda_0$$
$$\Rightarrow \phi = 0{,}50\times(1+0{,}2\times(1{,}103-0{,}6)+1{,}103^2) = 1{,}159$$
$$\Rightarrow \chi = \frac{1}{\phi+\sqrt{\phi^2-\lambda_c^2}} = 0{,}661$$

Kappa = reduction factor for distortional buckling

Calculation of effective thickness

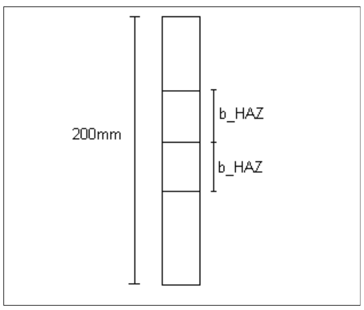
t_1, t_2 and t_3 are the thicknesses Part (4) and (5)

$$t_1 = 2\times\rho_c = 2\times0{,}851 = 1{,}702mm$$
$$t_2 = 2\times\rho_c\times\chi = 2\times0{,}851\times0{,}661 = 1{,}125mm$$
$$t_3 = 2\times\rho_c\times\chi = 2\times0{,}866\times0{,}661 = 1{,}145mm$$



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

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

Part (3): $75 \times 0{,}592 = 44{,}4 mm^2$

$50 \times 0{,}592 = 29{,}6 mm^2$

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

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

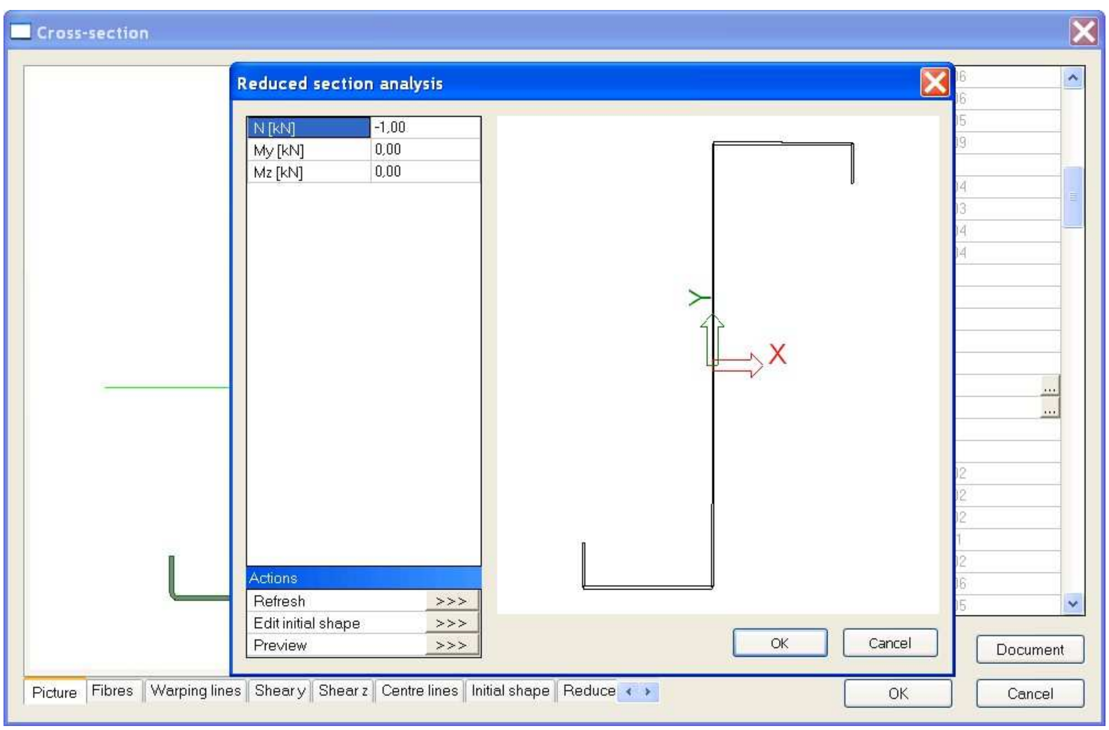
$\frac{63}{2} \times 1{,}064 = 35{,}4 mm^2$

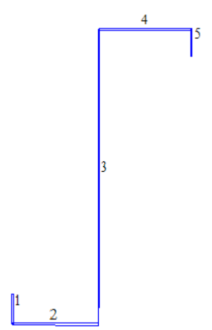
Part (5): $18 \times 1{,}145 = 20{,}6 mm^2$

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

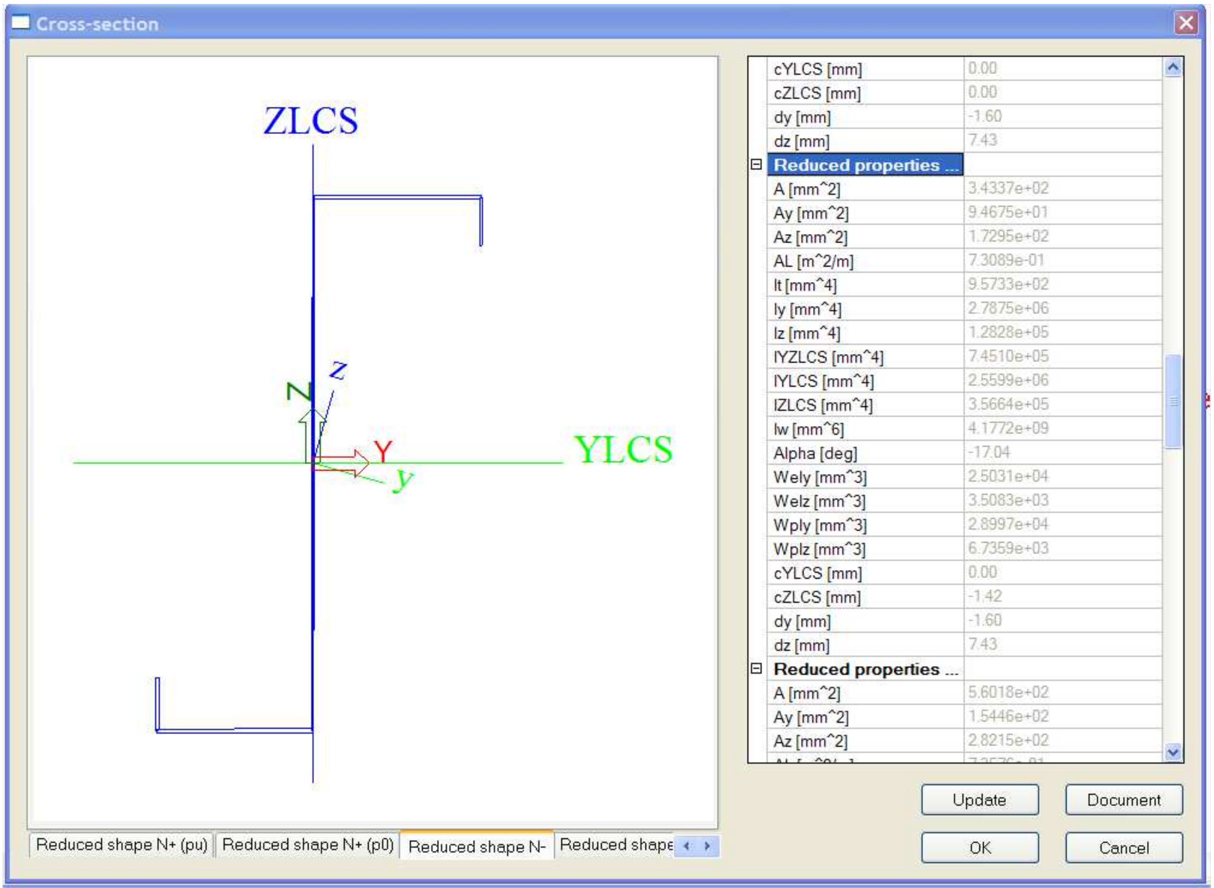
Comparison with SCIA Engineer

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





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	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	0.743	1.000	0.603	0	92.17	3531.14
	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	0.743	1.000	0.669	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	0.661	1.000	0.562			
	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	0.661	1.000	0.572	0	89.64	2654.29



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

Resistance of cross-sections

General

EN 1993-1-1 **article 6.2.1**.

For the elastic verification the following yield criterion for a critical point of the cross section may be used unless other interaction formulae apply (see EN1993-1-1 **article 6.2.8 to 6.2.10**):

$$\left(\frac{\sigma_{x,Ed}}{\frac{f_0}{\gamma_{M1}}}\right)^2+\left(\frac{\sigma_{z,Ed}}{\frac{f_0}{\gamma_{M1}}}\right)^2-\left(\frac{\sigma_{x,Ed}}{\frac{f_0}{\gamma_{M1}}}\right)^2\left(\frac{\sigma_{z,Ed}}{\frac{f_0}{\gamma_{M1}}}\right)^2+3\left(\frac{\tau_{Ed}}{\frac{f_0}{\gamma_{M1}}}\right)^2\leq C$$

$\frac{\sigma_{x,Ed}}{\frac{f_0}{\gamma_{M1}}} \leq 1$, $\frac{\sigma_{z,Ed}}{\frac{f_0}{\gamma_{M1}}} \leq 1$ and $\frac{\tau_{Ed}}{\frac{f_0}{\gamma_{M1}}} \leq 1$

Where $\sigma_{x,Ed}$ is the design value of the local longitudinal stress at the point of consideration
$\sigma_{z,Ed}$ is the design value of the local transverse stress at the point of consideration
τ_{Ed} is the design value of the local shear stress at the point of consideration
C ≥ *1* is a constant and may be defined in the National Annex. The numerical value *C= 1,2* is recommended.

National Annex:

NBN : De aanbevolen numerieke waarde C = 1,2 is normatief.

NEN : The value of C shall be taken as 1,2

Tension

EN 1999-1-1 **article 6.2.3**.

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

Where $N_{t,Rd}$ should be taken as the lesser of $N_{0,Rd}$ and $N_{u,Rd}$ where:
-

- $N_{0,Rd} = \frac{A \cdot f_0}{\gamma_{M1}}$ the general yielding along the member
- $N_{u,Rd} = \frac{0{,}9 \cdot A_{net} \cdot f_u}{\gamma_{M2}}$ local failure at section with holes
- $N_{u,Rd} = \frac{A_{eff} \cdot f_u}{\gamma_{M2}}$ local failure at section with HAZ

Compression

EN 1999-1-1 **article 6.2.4**.

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

Where $N_{c,Rd}$ should be taken as the lesser of $N_{u,Rd}$ and $N_{c,Rd}$ where:

- $N_{u,Rd} = \frac{A_{net} \cdot f_u}{\gamma_{M2}}$ in sections with unfilled holes
- $N_{c,Rd} = \frac{A_{eff} \cdot f_0}{\gamma_{M1}}$ others sections

Bending moment

EN 1993-1-1 **article 6.2.5**.

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

Where

- $M_{c,Rd} = M_{pl,Rd} = \frac{W_{net} \cdot f_u}{\gamma_{M2}}$ in a net section
- $M_{c,Rd} = M_{el,Rd} = \alpha \frac{W_{el} \cdot f_0}{\gamma_{M1}}$ at each cross-sections

With α the shape factor:

Table 6.4 - Values of shape factor α

Cross-section class	Without welds	With longitudinal welds
1	W_{pl} / W_{el} *)	$W_{pl,haz} / W_{el}$ *)
2	W_{pl} / W_{el}	$W_{pl,haz} / W_{el}$
3	$\alpha_{3,u}$	$\alpha_{3,w}$
4	W_{eff} / W_{el}	$W_{eff,haz} / W_{el}$
*) NOTE These formulae are on the conservative side. For more refined value, recommendations are given in Annex F		

For bending about both axes, the methods given in EN 1993-1-1 article 6.2.9 ("Bending and axial force" => see further) should be used.

Shear

EN 1999-1-1 **article 6.2.6**.

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

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

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

Where A_v is the shear area. The formula for A_v depends on the cross-section (see EN 1999-1-1 article 6.2.6(2)).

Torsion

EN 1999-1-1 **article 6.2.7**.

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

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

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

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

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

As a simplification, in the case of a member with open cross-section, such as I or H, ot may be assumed that the effects for St. Venant torsion can be neglected.

Bending and shear

EN 1999-1-1 **article 6.2.8**.

Where the shear force is less than half the plastic shear resistance its effect on the moment resistance may be neglected except where shear buckling reduces the section resistance.

Otherwise the moment resistance should be calculated using a reduced yield strength:

$$f_{0,V} = f_0\left(1-\left(\frac{2\,V_{Ed}}{V_{pl,Rd}}-1\right)^2\right)$$

When torsion is present V_{Rd} in the expression above is replaced by $V_{T,Rd}$ but $f_{0,V} = f_0$ for $V_{Ed} \leq 0,5\ V_{T,Rd}$

Bending and axial force

Open cross-sections

EN 1999-1-1 **article 6.2.9.1**.

For doubly symmetric cross-sections (except soled sections, see next paragraph), the following two criterions should be satisfied:

$$\left[\frac{N_{Ed}}{\omega_0 \cdot N_{Rd}}\right]^{\xi_0} + \frac{M_{y,Ed}}{\omega_0 \cdot M_{y,Rd}} \leq 1{,}00$$

$$\left[\frac{N_{Ed}}{\omega_0 \cdot N_{Rd}}\right]^{\eta_0} + \left[\frac{M_{y,Ed}}{\omega_0 \cdot M_{y,Rd}}\right]^{\gamma_0} + \left[\frac{M_{z,Ed}}{\omega_0 \cdot M_{z,Rd}}\right]^{\xi_0} \leq 1{,}00$$

All the coefficients above are explained in paragraph 6.2.9.1.

Hollow sections and solid cross-sections

EN 1999-1-1 **article 6.2.9.2**.

Hollow sections and solid cross-sections should satisfy the following criterion

$$\left[\frac{N_{Ed}}{\omega_0 \cdot N_{Rd}}\right]^{\psi} + \left[\left[\frac{M_{y,Ed}}{\omega_0 \cdot M_{y,Rd}}\right]^{1,7} + \left[\frac{M_{z,Ed}}{\omega_0 \cdot M_{z,Rd}}\right]^{1,7}\right]^{0,6} \leq 1{,}00$$

All the coefficients above are explained in paragraph 6.2.9.2.

Members containing localized welds

EN 1999-1-1 **article 6.2.9.3**.

If a section is affected by HAZ softening with a specified location along the length and if the softening does not extend longitudinally a distance greater than the least width of the member, then the limiting stress should be taken as the design *ultimate strength* $\rho_{u,haz}$ f_u /γ_{M2} of the reduced strength material:

$$\omega_0 = (\rho_{u,haz} f_u/\gamma_{M2})/(f_0/\gamma_{M1})$$

If the softening extend longitudinally a distance greater than the least width of the member the limiting stress should be taken as the strength $\rho_{0,haz}$ f_0 for *overall yielding* of the reduced strength material, thus $\omega_0 = \rho_{0,haz}$

Bending, shear and axial force

EN 1999-1-1 **article 6.2.10**.

Where shear and axial force are present, allowance should be made for the effect of both shear force and axial force on the resistance moment.

Buckling resistance of members

Uniform members in compression

EN 1999-1-1 **article 6.3.1**.

National annex

Remark: Following EN 1993-1-1 no National Annex can be applied on this article.

NEN: Read before 6.3.1: Clauses 12.1.2.2, 12.1.3.2 and 12.1.4.2 of NEN 6771 shall be applied.

A compression member should be verified against buckling as follows:

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

Where

$$N_{b,Rd} = \kappa \frac{\chi \cdot A_{eff} \cdot f_0}{\gamma_{M1}}$$

χ is the reduction factor for the relevant buckling mode

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

where

$$\Phi = 0,5\left[1 + \alpha\left(\bar{\lambda} - \bar{\lambda}_0\right) + \bar{\lambda}^2\right]$$

- $\bar{\lambda} = \sqrt{\frac{A_{eff} \cdot f_0}{N_{cr}}}$

The imperfection factor α correspondents with the appropriate buckling curve and $\bar{\lambda}_0$ is the limit of horizontal plateau

Table 6.6 - Values of α and $\bar{\lambda}_0$ for flexural buckling

Material buckling class according to Table 3.2	α	$\bar{\lambda}_0$
Class A	0,20	0,10
Class B	0,32	0,00

κ is a factor to allow for the weakening effects for welding. For longitudinally welded member κ is given in Table 6.5. for flexural buckling and κ =1 for torsional and torsional-flexural buckling. In case of transversally welded member $\kappa = \omega_x$ according to 6.3.3.3.

Table 6.5 - Values of κ factor for member with longitudinal welds

Class A material according to Table 3.2	Class B material according to Table 3.2
$\kappa = 1 - \left(1 - \frac{A_1}{A}\right) 10^{-\bar{\lambda}} - \left(0,05 + 0,1\frac{A_1}{A}\right)\bar{\lambda}^{1,3(1-\bar{\lambda})}$ with $A_1 = A - A_{haz}(1 - \rho_{o,haz})$ in which A_{haz} = area of HAZ	$\kappa = 1$ if $\bar{\lambda} \leq 0,2$ $\kappa = 1 + 0,04(4\bar{\lambda})^{(0,5-\bar{\lambda})} - 0,22\bar{\lambda}^{1,4(1-\bar{\lambda})}$ if $\bar{\lambda} > 0,2$

For slenderness $\bar{\lambda} \leq \bar{\lambda}_0$ or for $\frac{N_{Ed}}{N_{cr}} \leq \bar{\lambda}_0{}^2$ the buckling effects may be ignored and only cross-sectional check apply.

Slenderness for flexural buckling

- $\bar{\lambda} = \sqrt{\frac{A_{eff} \cdot f_0}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\pi}\sqrt{\frac{A_{eff} \cdot f_0}{A\,E}}$

-

Where:

L_{cr} is the buckling length

i is the radius of gyration about the relevant axis

Slenderness for torsional and torsional-flexural buckling

- $\bar{\lambda}_T = \sqrt{\frac{A_{eff} \cdot f_0}{N_{cr}}}$

<u>Calculation of the buckling length in SCIA Engineer</u>

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

- for a non sway structure :

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

- for a sway structure :

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

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

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

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

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

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

The following load cases are considered :

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

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

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

Uniform members in bending

EN 1999-1-1 **article 6.3.2**.

Buckling resistance

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

$$M_{b,Rd} = \chi_{LT} \alpha W_y \frac{f_0}{\gamma_{M1}}$$

Lateral torsional buckling curves

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

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

The value of α_{LT} and $\bar{\lambda}_{0,LT}$ should be taken as:

- α_{LT} = 0,10 and $\bar{\lambda}_{0,LT}$ = 0,6 for class 1 and 2 cross-sections
- α_{LT}= 0,20 and $\bar{\lambda}_{0,LT}$ = 0,4 for class 3 and 4 cross-sections.

The slenderness:

$$\bar{\lambda}_T = \sqrt{\frac{\alpha \cdot W_{el,y} \cdot f_0}{M_{cr}}}$$

Uniform members in bending and axial compression

EN 1999-1-1 **article 6.3.2**

The exact check depends on the kind of cross-section. For all types of cross sections, this is explained in paragraph EN 1991-1-1, article 6.3.2.

7. Serviceability limit states

The basic requirements for serviceability limit states are given in 3.4 of EN 1990.

Any serviceability limit state and the associated loading and analysis model should be specified for a project.

8. References

[1]	EN 1999-1-1 *Eurocode 9: Design of aluminium structures - Part 1-1: General structural rules* CEN, 2007.
[2]	NEN-EN 1999-1-1/NB *Eurocode 9: Design of aluminium structures - Part 1-1: General structural rules*
[3]	NBN EN 1999-1-1 ANB *Eurocode 9: Ontwerp en berekening van aluminiumconstructies – Deel 1-1: Algemene regels - Nationale bijlage*