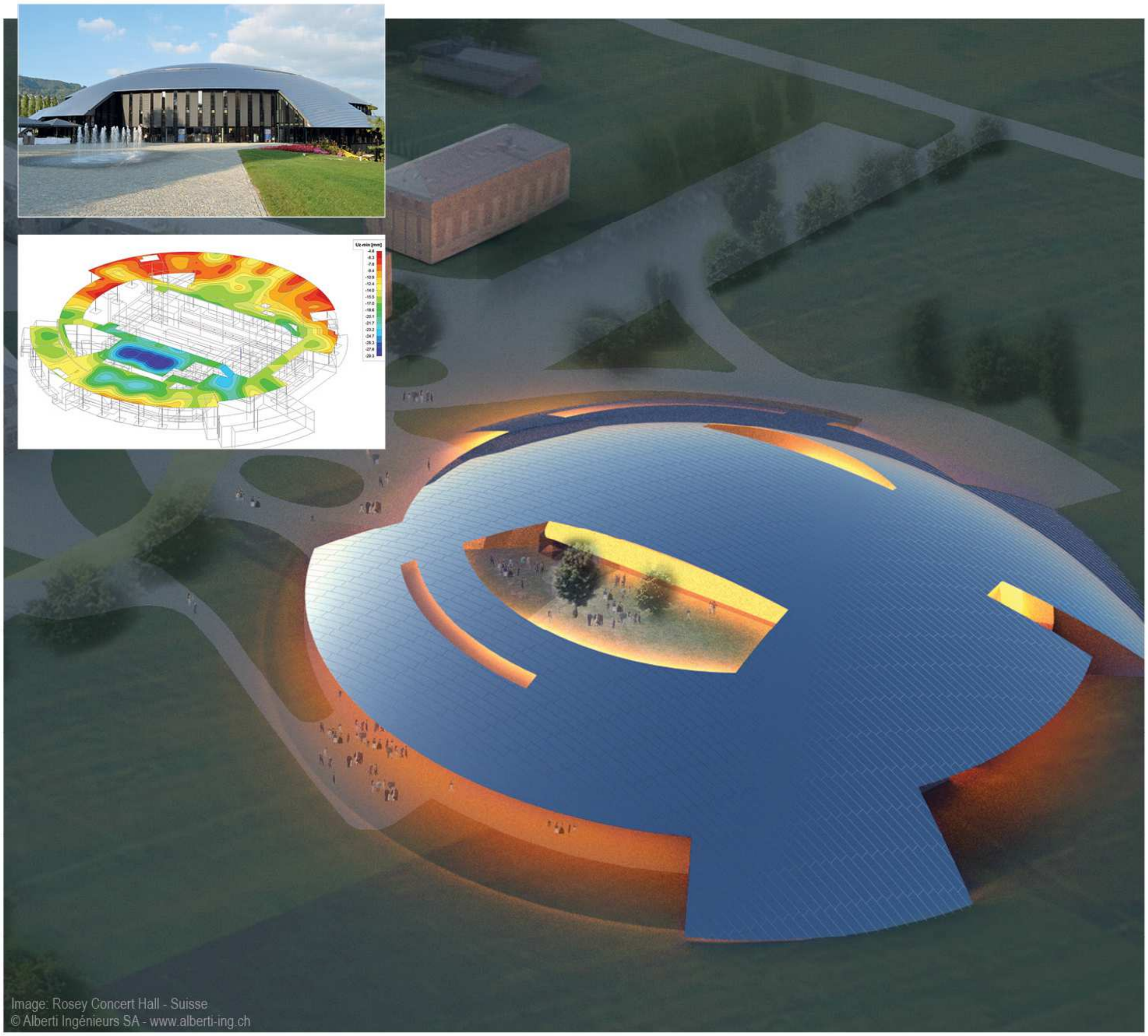
SCIA ENGINEER



Eurocode Training

EN 1993-1-1 & EN 1993-1-2
Steel structures

Table of contents

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

Eurocode 3 applies to the design of buildings and civil engineering works in steel. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990 – Basis of structural design.
Eurocode 3 is subdivided in various parts:

EN 1993-1	Design of Steel Structures: General rules and rules for buildings.
EN 1993-2	Design of Steel Structures: Steel bridges.
EN 1993-3	Design of Steel Structures: Towers, masts and chimneys.
EN 1993-4	Design of Steel Structures: Silos, tanks and pipelines.
EN 1993-5	Design of Steel Structures: Piling.
EN 1993-6	Design of Steel Structures: Crane supporting structures.

In this Eurocode workshop, the general rules and rules for buildings (EN 1993-1) are discussed. The EN 1993- compromises:

EN 1993-1-1	Design of Steel Structures: General rules and rules for buildings.
EN 1993-1-2	Design of Steel Structures: Structural fire design.
EN 1993-1-3	Design of Steel Structures: Cold-formed thin gauge members and sheeting.
EN 1993-1-4	Design of Steel Structures: Stainless steels.
EN 1993-1-5	Design of Steel Structures: Plated structural elements.
EN 1993-1-6	Design of Steel Structures: Strength and stability of shell structures.
EN 1993-1-7	Design of Steel Structures: Strength and stability of planar plated structures transversely loaded.
EN 1993-1-8	Design of Steel Structures: Design of joints.
EN 1993-1-9	Design of Steel Structures: Fatigue strength of steel structures.
EN 1993-1-10	Design of Steel Structures: Selection of steel for fracture toughness and through-thickness properties.
EN 1993-1-11	Design of Steel Structures: Design of structures with tension components made of steel.
EN 1993-1-12	Design of Steel Structures: Supplementary rules for high strength steel.

In this manual EN 1993-1-1 and EN 1993-1-2 (“General rules and rules for buildings” and “Structural fire design”) are discussed.

EN 1993-1-1 Design of Steel Structures: General rules and rules for buildings

The following subjects are dealt with in EN 1993-1-1:

Section 1:	General
Section 2:	Basis of design
Section 3:	Materials
Section 4:	Durability
Section 5:	Structural analysis
Section 6:	Ultimate limit states
Section 7:	Serviceability limit states

Section 1: General

All kind of symbols are given in a list.
The following conventions for the member axis are given in the EN 1993-1-1 :

1.7 Conventions for member axes

(1) 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 steel 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 yy axis)
  - v-v - minor principal axis (where this does not coincide with the zz axis)

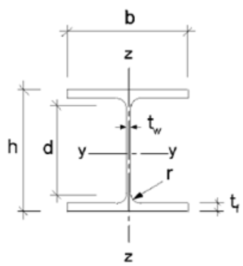
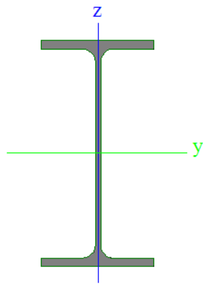
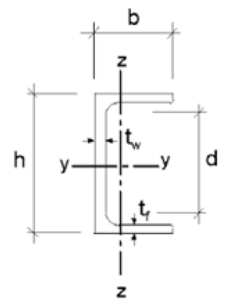
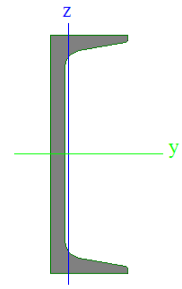
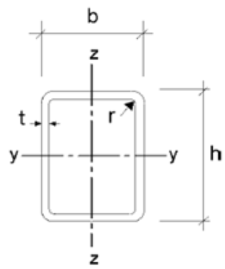
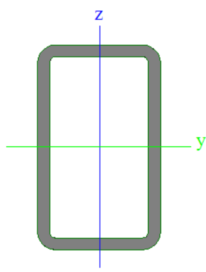
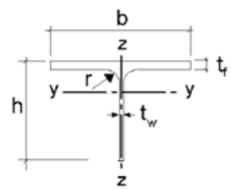
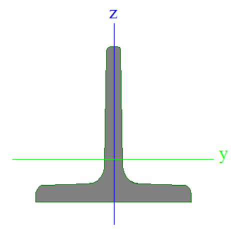
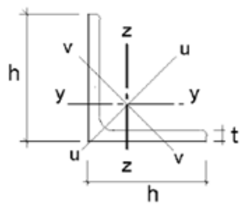
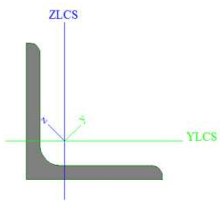
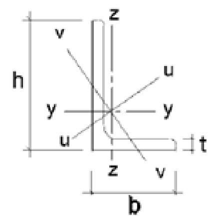
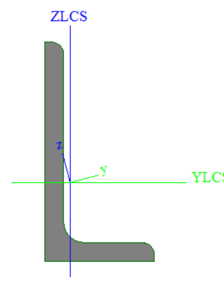
(3) The symbols used for dimensions and axes of rolled steel sections are indicated in Figure 1.1.

(4) The convention used for subscripts that 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 but for sections such as angles are defined by the axes u-u and v-v.

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

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

EN 1993-1-1	SCIA Engineer	EN 1993-1-1	SCIA Engineer
			
			
			

Section 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 steel structures equation (6.6c) or equation (6.6d) of EN 1990 applies:

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

where R_k is the characteristic value of the particular resistance determined with characteristic or nominal values for the material properties and dimensions

γ_M is the global partial factor for the particular resistance

Section 3: Materials

Material properties

EN 1993-1-1 **article 3.2.1**

For standard steel grades, the yield strength f_y and tensile strength f_u are defined according to the thickness of the element.

The standard steel grades are:

- S235
- S275
- S355
- S420
- S460

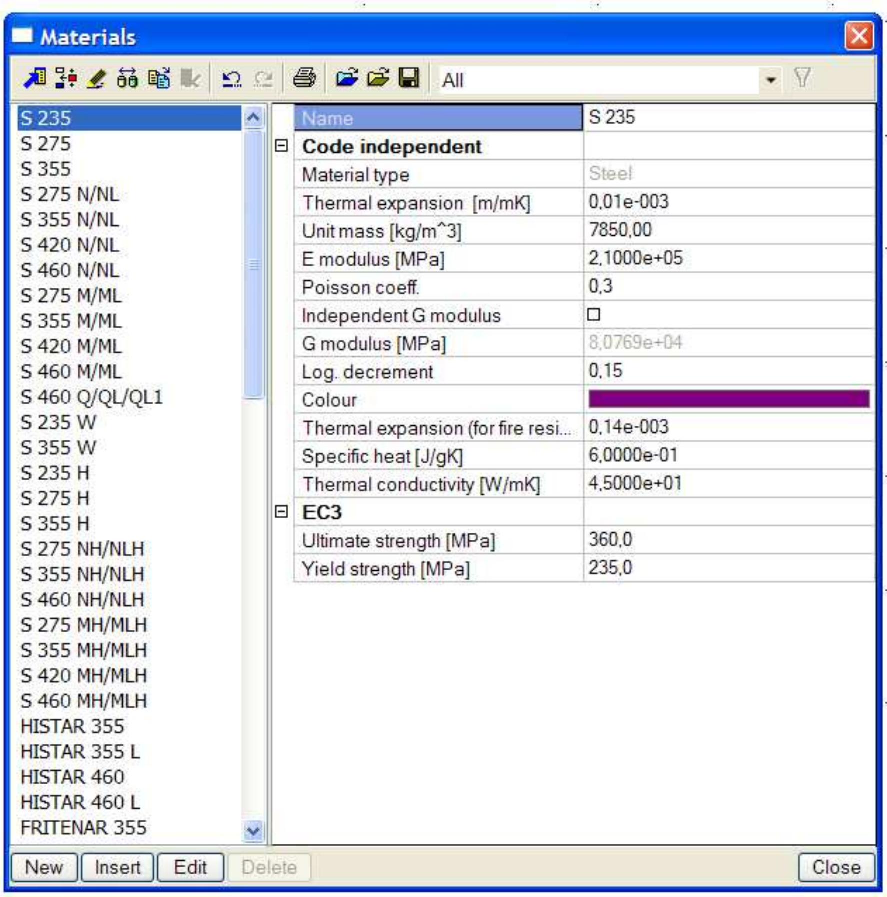
Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

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

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

Those materials are also inputted in SCIA Engineer:



The Histar and Fritenar steel grades have been implemented according to Arcelor.
For cold formed sections, the values for f_y and f_u are not influenced by the previous tables.

National annexes:

NF:

- Pour les structures de bâtiment, on utilise pour f_y et f_u les valeurs données dans le Tableau 3.1 (NF) ci-dessous (modification du Tableau 3.1 pour les valeurs f_u des nuances S355 et S355W).
- Pour les autres structures, lorsqu'elles ne sont pas traitées dans les parties applicatives 2 à 6 de l'EN 1993, on utilise pour f_y et f_u les valeurs minimales de la norme de produits.

Tableau 3.1 (NF) — Valeurs nominales de limite d'élasticité f_y et de résistance à la traction f_u pour les aciers de construction laminés à chaud

Norme et nuance d'acier	Épaisseur nominale t de l'élément [mm]			
	$t \leq 40$ mm		40 mm < $t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	490	335	470
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EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	490	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

NBN: Also tables in the national annex concerning the steel properties.

Design values of material coefficients

EN 1993-1-1 **article 3.2.6**

The material coefficients to be adopted in calculations for the structural steels covered by this Eurocode Part should be taken as follows:

- Modulus of elasticity: $E = 210000$ N/mm²
- Shear Modulus: $G = 81000$ N/mm²
- Poisson's ratio in elastic stage: $\nu = 0{,}3$
- Coefficient of linear thermal expansion: $\alpha = 12 \times 10^{-6}$ perK (for $T \leq 100$ °C)

Section 4: Durability

The basic requirements for durability are set out in EN 1990.

Parts susceptible to corrosion, mechanical wear or fatigue should be designed such that inspection maintenance and reconstruction can be carried out satisfactorily and access is available for in-service inspection and maintenance.

Corrosion protection does not need to be applied to internal building structures, if the internal relative humidity does not exceed 80%.

Section 5: Structural analysis

Global analysis

EN 1993-1-1 **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:

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

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 15 \quad \text{for plastic analysis} \qquad (5.1)$$

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: Voor raamwerken is een lagere limiet 10 voor α_{cr} gegeven voor plastische berekeningen, en dit overeenkomstig alle beschikbare studies. Voorts laat dit een volledige overeenkomst toe met de achtergrond van formule (5.7).

Voor andere types van constructies is een analoog concept van toepassing maar geschikte limietwaarden voor α_{cr} moeten gebaseerd zijn op wetenschappelijke grond.

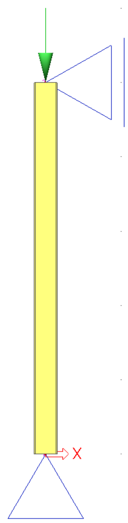
NEN: For Plastic analysis α_{cr} shall be taken as $\alpha_{cr} = F_{cr} / F_{Ed} \geq 10$ for all types of frames.

NF: l'Annexe Nationale ne donne pas de recommandation particulière.

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

Example: Calculation_Alpha_cr.esa

The column has a cross-section of type **IPE 300**, is fabricated from **S235** and has the following relevant properties:



$E = 210.000\ N/mm^2$
$f_y = 235\ N/mm^2$
$\gamma_{M1} = 1.00$
$L = 5000\ mm$
$A = 5380\ mm^2$
$I_y = 83560000 mm^4$
$W_{pl,y} = 628400\ mm^3$

Calculation of 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

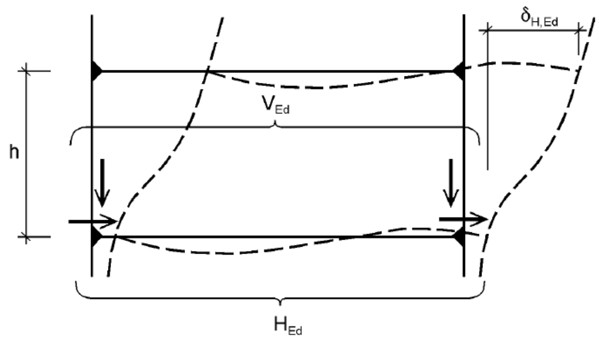
Critical load coefficients	
N	f
-	[]
Stability combination : S1	
1	6885,28

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

$$N_{cr} = \frac{\pi^2 EI}{l^2} = \frac{\pi^2 \cdot 210.000 \, {}^{N}\!/_{mm^2} \cdot 83560000 mm^4}{(5000 mm)^2} = 6927{,}51\ kN$$

For portal frames with shallow roof slopes and-column type plane frames in buildings, α_{cr} may be calculated using the following approximative formula, provided that the axial compression in the beams or rafters is not significant and if the criterium (5.1. – see above) is satisfied for each storey:

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right)$$



<u>NOTE:</u>

- Roof is shallow if it is not steeper than 1:2 = 26°
- Axial compression in the beams or rafters may be assumed to be significant if

$$\bar{\lambda} \geq 0{,}3\sqrt{\frac{A\,f_y}{N_{Ed}}}$$

EN 1993-1-1 **article 5.2.2**:

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 1993-1-1 **article 5.2.2 (7)**:

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.

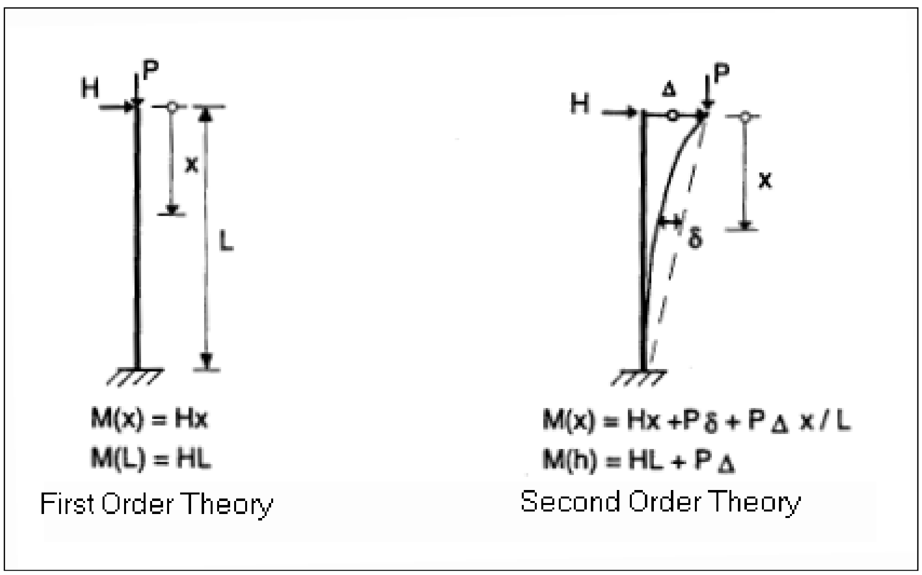
For single storey frames designed on the basis of elastic global analysis second order sway effects due to vertical loads may be calculated by increasing the horizontal loads H_{Ed} and equivalent loads V_{Ed} ϕ due to imperfections and other possible sway effects according to first order theory by the factor:

$$\frac{1}{1-\frac{1}{\alpha_{cr}}}$$ provided that $\alpha_{cr} \geq 3{,}0$.

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 effect, 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 1993-1-1 **article 5.3.2(3)a)**:

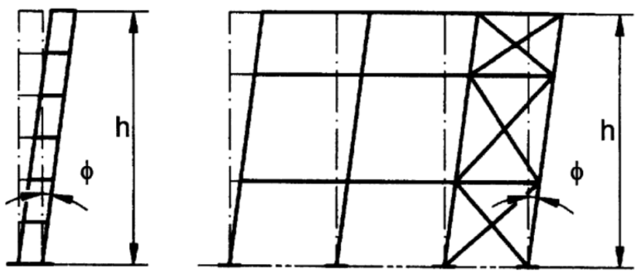
$$\varphi = \frac{1}{200} \cdot \alpha_h \cdot \alpha_m$$

$\alpha_h = \frac{2}{\sqrt{h}}$ 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

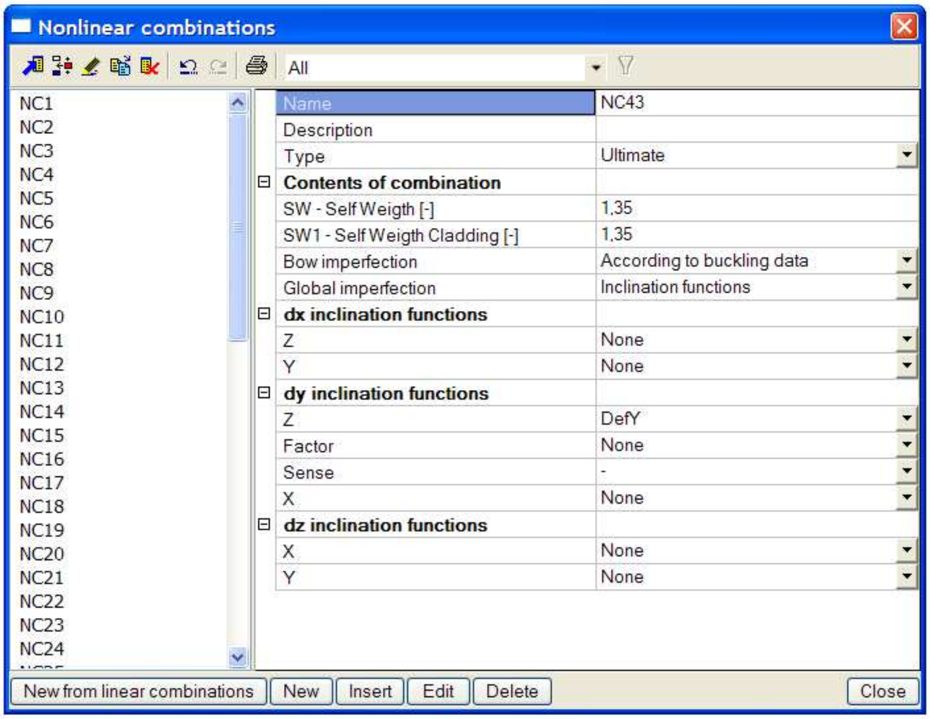
m The number of columns in a row including only those columns which carry a vertical load N_{Ed} not less than 50% of the average value of the vertical load per column in the plane considered.



This can be calculated automatically by SCIA Engineer:

Example: Imperfections.esa

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 1993-1-1 **article 5.3.2(3)b)**:

Recommended values are given in Table 5.1.

Table 5.1: Design values of initial local bow imperfection e_0 / L

Buckling curve acc. to Table 6.1	elastic analysis e_0 / L	plastic analysis e_0 / L
a_0	1 / 350	1 / 300
a	1 / 300	1 / 250
b	1 / 250	1 / 200
c	1 / 200	1 / 150
d	1 / 150	1 / 100

Where L is the member length.

National annex:

NBN: De waarden aanbevolen in de tabel 5.1 zijn niet gewijzigd. In deze tabel behoren de uitdrukkingen “elastische berekening” en “plastische berekening” gelezen te zijn als “elastische toets van de doorsnedeweerstand” en “plastische toets van de doorsnedeweerstand”.

NEN: The value for the maximum amplitude of the member imperfection e_0 shall be determined according to formula (12.3-9) of NEN 6771:

e_y^* is de imperfectieparameter, die het gecombineerde effect in rekening brengt van alle imperfecties zoals initiële vooruitbuiging, restspanningen, inhomogeniteiten:

$$e_y^* = \alpha_k\left(\lambda_{y;rel} - \lambda_0\right)\frac{M_{y;u;d}}{N_{c;u;d}}$$

NF : -

EN 1993-1-1 **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
- $\bar{\lambda} > 0{,}5\sqrt{\frac{A \cdot f_y}{N_{Ed}}}$

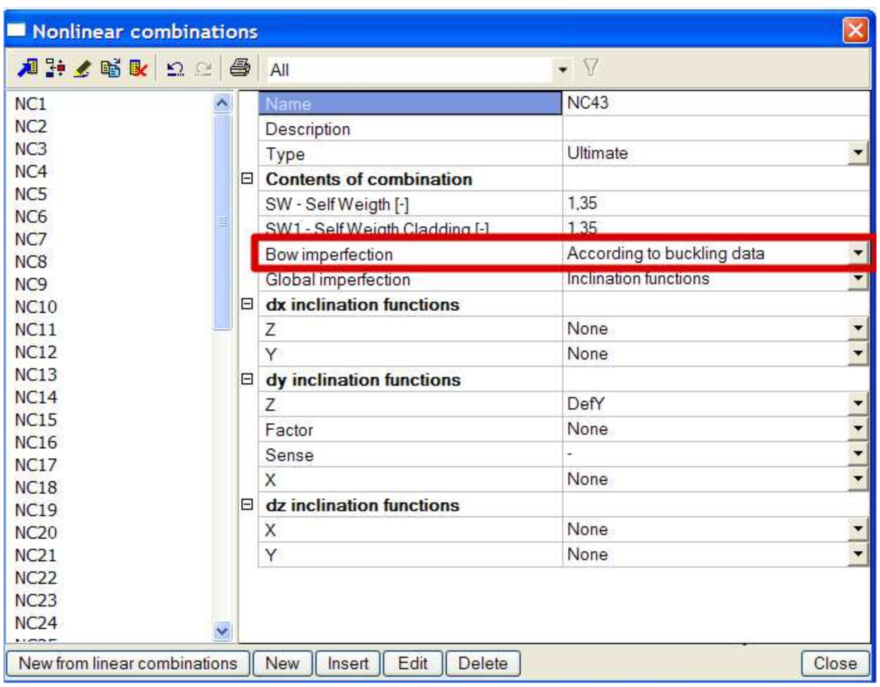
And $\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$

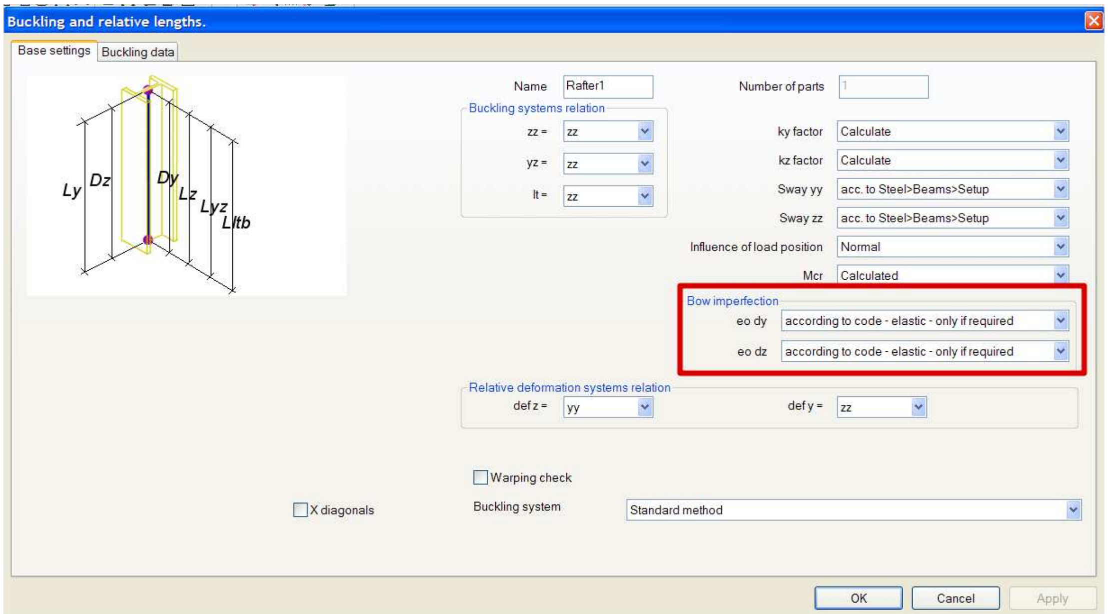
Thus: $\sqrt{\frac{A \cdot f_y}{N_{cr}}} > 0{,}5\sqrt{\frac{A \cdot f_y}{N_{Ed}}}$

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

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

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 η_{init} of this imperfection may be determined from:

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

$$e_0 = \alpha \cdot \left(\bar{\lambda} - 0{,}2\right) \cdot \frac{M_{Rk}}{N_{Rk}} \cdot \frac{1 - \dfrac{\chi \cdot \left(\bar{\lambda}\right)^2}{\gamma_{M1}}}{1 - \chi \cdot \left(\bar{\lambda}\right)^2} \qquad \text{for} \qquad \bar{\lambda} > 0{,}2$$

With: $\bar{\lambda} = \sqrt{N_{Rk}/N_{cr}}$ = The reduced slenderness

α = The imperfection factor for the relevant buckling curve.

χ = The reduction factor for the relevant buckling curve, depending on the relevant cross-section.

N_{Rk} = The characteristic resistance to normal force of the critical cross-section, i.e. $N_{pl,Rk}$.

N_{cr} = Elastic critical buckling load.

M_{Rk} = The characteristic moment resistance of the critical cross-section, i.e. $M_{el,Rk}$ or $M_{pl,Rk}$ as relevant.

η_{cr} = Shape of the elastic critical buckling mode.

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

National annex:

NF: Cette clause n'est pas applicable.

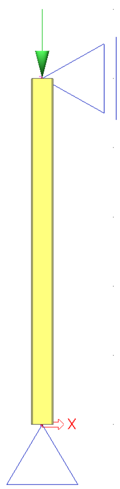
Le principe de cette méthode pourra toutefois être exploité pour des typologies de structures ou de composants structuraux pour lesquelles les conditions et limites d'application seront définies et justifiées, en fonction des caractéristiques du projet particulier.

NEN : There are no restrictions to the scope of application of this method.

NBN : 5.3.2(11) is beperkt tot vooruitbuigingen in enkel individuele elementen.

Example: Calculation_Alpha_cr.esa

The column has a cross-section of type **IPE 300**, is fabricated from **S235** and has the following relevant properties:



$E = 210.000\ N/mm^2$
$f_y = 235\ N/mm^2$
$\gamma_{M1} = 1.00$
$L = 5000\ mm$
$A = 5380\ mm^2$
$I_y = 83560000 mm^4$
$W_{pl,y} = 628400\ mm^3$

Calculation of α_{cr}

First a **Stability calculation** is done using a load of 1kN. This way, the elastic critical buckling load N_{cr} is obtained. In order to obtain precise results, the **Number of 1D elements** is set to **10**. 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:

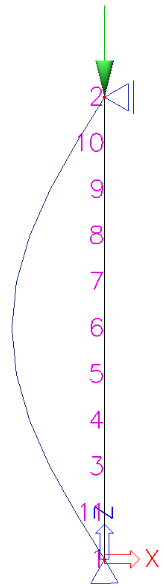
Critical load coefficients

Critical load coefficients	
N -	f []
Stability combination : S1	
1	6885,28

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

$$N_{cr} = \frac{\pi^2 EI}{l^2} = \frac{\pi^2 \cdot 210.000 \, N/_{mm^2} \cdot 83560000 mm^4}{(5000mm)^2} = 6927{,}51 kN$$

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

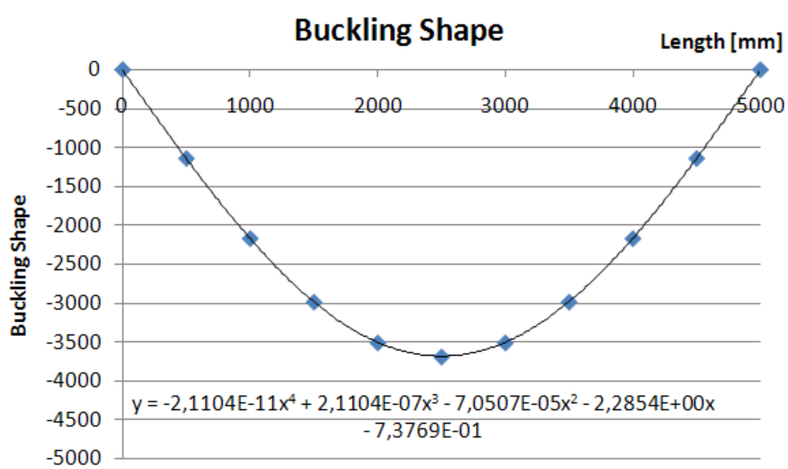


Displacement of nodes

Stability calculation, Extreme : No
Selection : All
Stability combinations : S1/1 - 6885,28
Values are mulitpied by 10000 for better numeric representation.

Node of mesh	Case	Ux [-]	Uz [-]	Fiy [-]
1	S1/1 - 6885,28	0,00	0,00	-2314,62
11	S1/1 - 6885,28	-1138,38	0,00	-2201,33
3	S1/1 - 6885,28	-2165,34	0,00	-1872,57
4	S1/1 - 6885,28	-2980,33	0,00	-1360,50
5	S1/1 - 6885,28	-3503,59	0,00	-715,26
6	S1/1 - 6885,28	-3683,89	0,00	0,00
7	S1/1 - 6885,28	-3503,59	0,00	715,26
8	S1/1 - 6885,28	-2980,33	0,00	1360,50
9	S1/1 - 6885,28	-2165,34	0,00	1872,57
10	S1/1 - 6885,28	-1138,38	0,00	2201,33
2	S1/1 - 6885,28	0,00	0,00	2314,62

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



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

$$\eta_{cr} = -2{,}1104 \cdot 10^{-11} \cdot x^4 + 2{,}1104 \cdot 10^{-7} \cdot x^3 - 7{,}0507 \cdot 10^{-5} \cdot x^2 + 2{,}2854 \cdot x - 7{,}3769 \cdot 10^{-1}$$

$$\eta''_{cr} = -2{,}5325 \cdot 10^{-10} \cdot x^2 + 1{,}2662 \cdot 10^{-6} \cdot x - 1{,}4101 \cdot 10^{-4}$$

Calculation of e_0

$$N_{Rk} = f_y \times A = 235 \, {}^{N}/_{mm^2} \times 5380 mm^2 = 1264300 \text{ N}$$

$$M_{Rk} = f_y \times W_{pl} = 235 \, {}^{N}/_{mm^2} \times 628400 mm^3 = 147674000 \text{ Nmm (class 2)}$$

$$\bar{\lambda} = \sqrt{N_{Rk} / N_{cr}} = \sqrt{1264300\,N / 6927510\,N} = 0{,}43$$

$\alpha = 0{,}21$ for buckling curve a

$$\chi = \frac{1}{0{,}5\left[1+\alpha\left(\bar{\lambda}-0.2\right)+\left(\bar{\lambda}\right)^2\right]+\sqrt{\left(0{,}5\left[1+\alpha\left(\bar{\lambda}-0.2\right)+\left(\bar{\lambda}\right)^2\right]\right)^2-\left(\bar{\lambda}\right)^2}} = 0{,}945$$

These intermediate results can be verified through SCIA Engineer when performing a Steel Code Check on the column:

Buckling parameters	yy
type	sway
Slenderness	40.12
Reduced slenderness	0.43
Buckling curve	a
Imperfection	0.21
Reduction factor	0.95
Length	5.00
Buckling factor	1.00
Buckling length	5.00
Critical Euler load	6927.51

$$\Rightarrow e_0 = \alpha \cdot \left(\bar{\lambda}-0{,}2\right) \cdot \frac{M_{Rk}}{N_{Rk}} \cdot \frac{1-\dfrac{\chi \cdot \left(\bar{\lambda}\right)^2}{\gamma_{M1}}}{1-\chi \cdot \left(\bar{\lambda}\right)^2} = 0{,}21 \cdot \left(0{,}43-0.2\right) \cdot \frac{147674000\,Nmm}{1264300\,N}$$

= **5,6416 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 mm

η_{cr} at mid section = -3681,8

$\eta''_{cr,\max}$ at mid section = $1{,}4418 \cdot 10^{-3} \cdot 1/mm^2$

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

$$= 5{,}6416\,mm \cdot \frac{6927510\,N}{210000\,N/mm^2 \cdot 83560000\,mm^4 \cdot 1{,}4418 \cdot 10^{-3} \cdot 1/mm^2} \cdot 3681{,}8$$

= **5,6528 mm**

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

Imperfection analysis of bracing systems

EN 1993-1-1 **article 5.3.3**.

In the analysis of bracing systems which are required to provide lateral stability within the length of beams or compression 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:

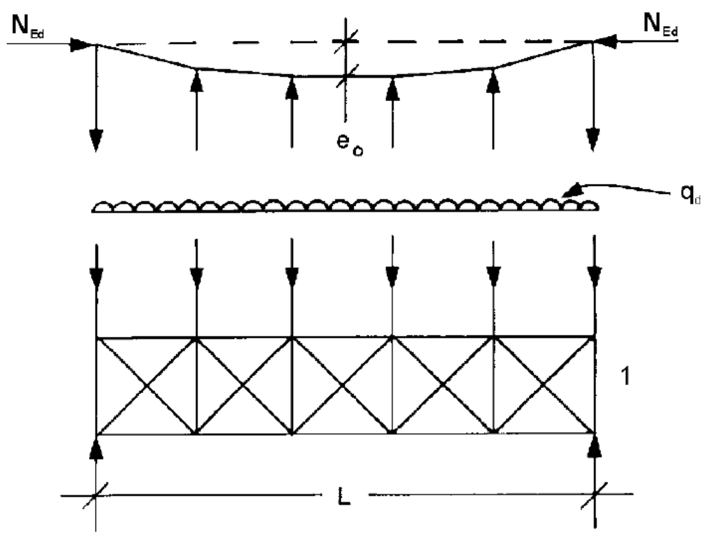
EN 1993-1-1 **Formula (5.12)**

$$e_0 = \alpha_m L/500$$

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

Where m is the number of members to be restrained.

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_d *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.

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

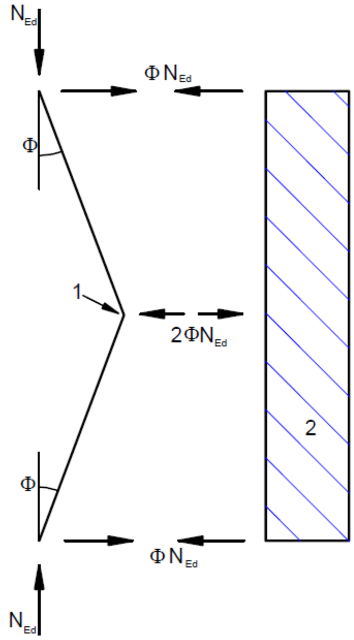
δ_q is the inplane deflection of the bracing system due to q plus any external loads calculated from first order analysis.

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 1993-1-1 **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.

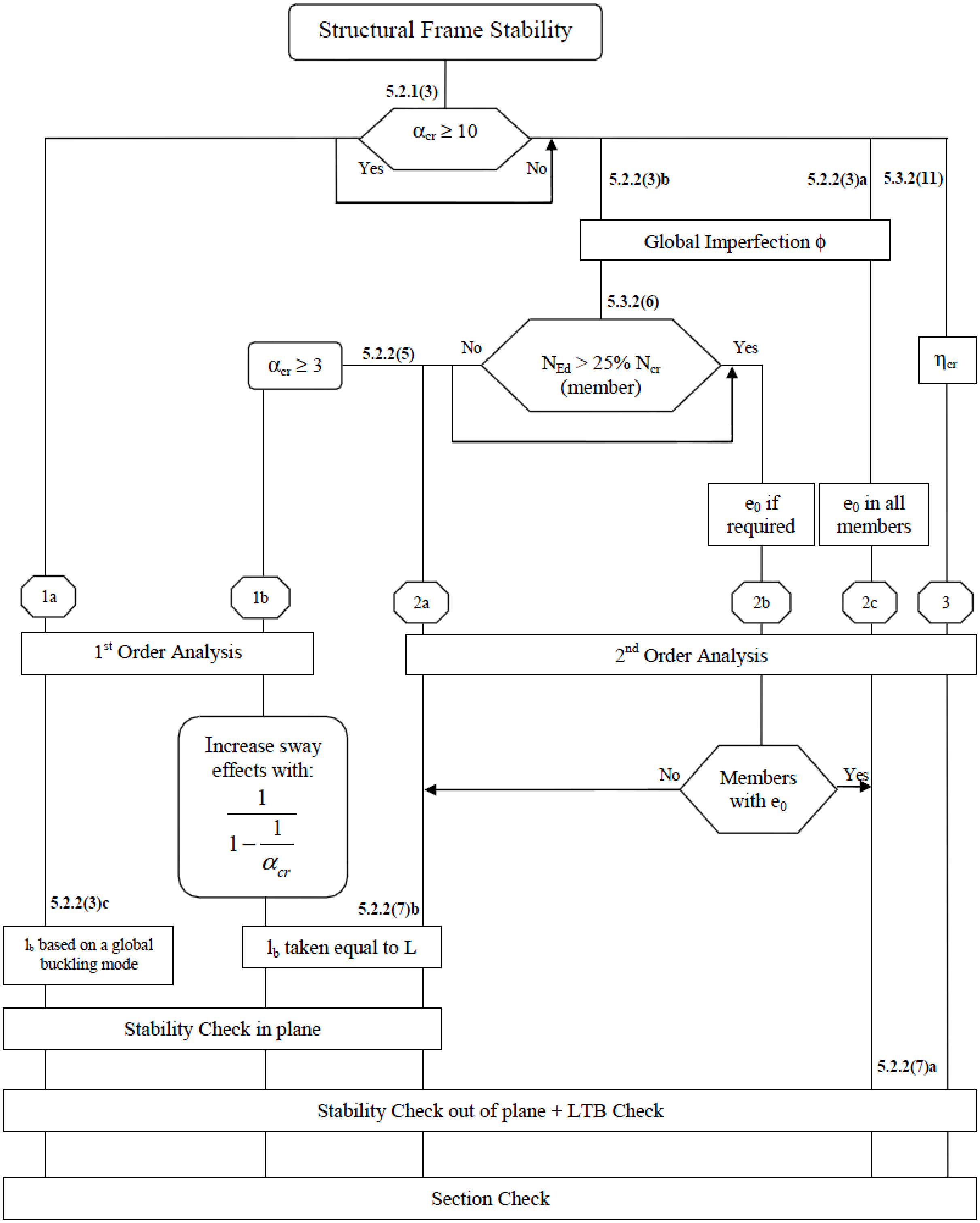
National annex

NBN: De aanbevolen waarde k = 0,5 is normatief

NF: La valeur k à utiliser est:
$k = 1 - 0{,}5\,\frac{b}{h} \geq 0{,}5$ en retenant la plus faible valeur du rapport *b/h* le long de la barre, où :

b est la largeur maximale de la partie comprimée de la section
h est la hauteur de la section

NEN: The value of k shall be taken as 0,5.


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)
$\alpha_{cr} \geq 3$
5.2.2(5)
No
$N_{Ed} > 25\% \, N_{cr}$ (member)
Yes
η_{cr}
e_0 if required
e_0 in all members
1a
1b
2a
2b
2c
3
1st Order Analysis
2nd Order Analysis
Increase sway effects with: $\frac{1}{1-\frac{1}{\alpha_{cr}}}$
No
Members with e_0
Yes
5.2.2(3)c
5.2.2(7)b
l_b based on a global buckling mode
l_b taken equal to L
Stability Check in plane
5.2.2(7)a
Stability Check out of plane + LTB Check
Section Check

Material non-linearities

EN 1993-1-1 **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.
When plastic hinges occurs in a member:

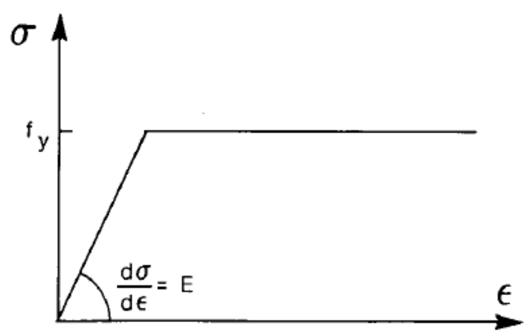
- Double symmetric or single symmetric
- Requirements of EN 1993-1-1 article 5.6

Simplified:

- Internal forces and moments remain in equilibrium with applied pads
- All members: Classification = Class 1 or Class 2 (EN 1993-1-1 article 5.5)
- Lateral torsional buckling of the members is prevented

Plastic analysis allows for the effects of material non-linearity in calculating the action effects of a structural system.

The bi-linear stress-strain relationship may be used for the grades of structural steel. Alternatively, a more precise relationship may be adopted (see 1993-1-5).

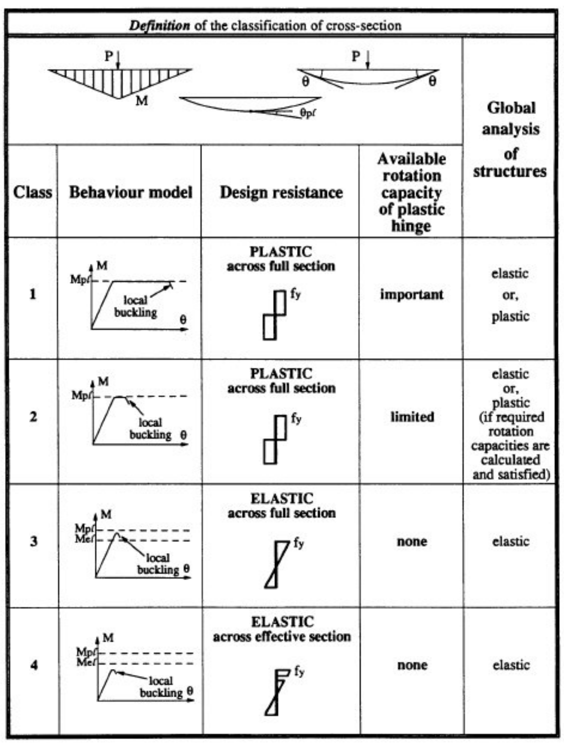


Classification of cross-sections

EN 1993-1-1 **article 5.5**.

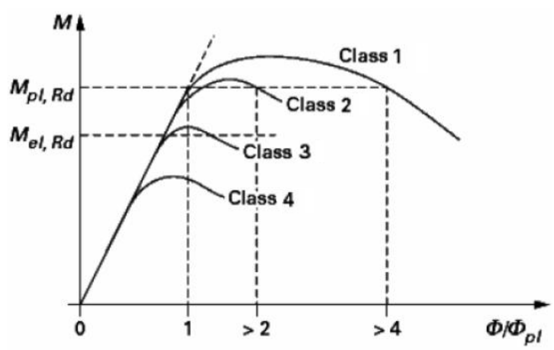
Four classes of cross sections are defined:

- Class 1 (EC3, NEN) or PL-PL (DIN) section
  Cross sections which can form a plastic hinge with the rotation capacity required for plastic analysis
- Class 2 (EC3, NEN) or EL-PL (DIN) section
  Cross sections which can develop their plastic moment resistance, but have limited rotation capacity.
- Class 3 (EC3, NEN) or EL-EL (DIN) section
  Cross sections in which the calculated stress in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 (EC3, NEN) or Slender section
  Cross sections in which it is necessary to make explicit allowance for the effects of local buckling when determining their moment resistance or compression resistance.



Definition of the classification of cross-section

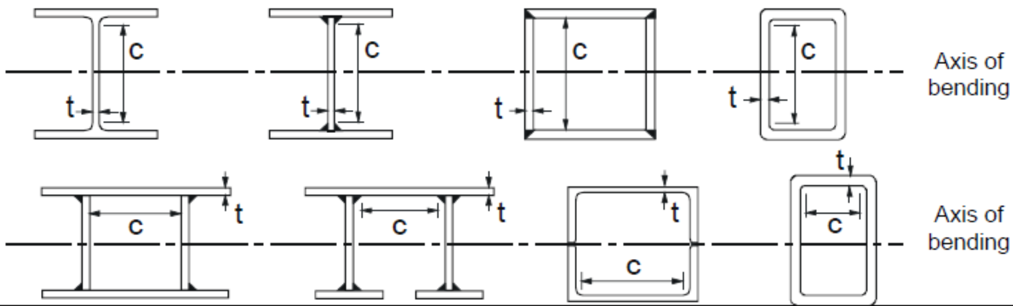
Class	Behaviour model	Design resistance	Available rotation capacity of plastic hinge	Global analysis of structures
1	local buckling	**PLASTIC** across full section	important	elastic or, plastic
2	local buckling	**PLASTIC** across full section	limited	elastic or, plastic (if required rotation capacities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

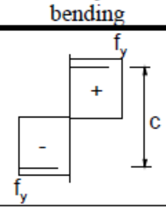
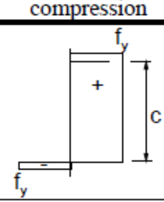
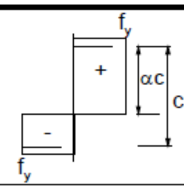
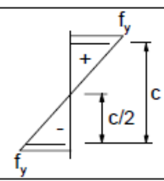
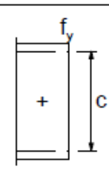
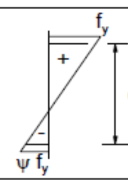


The limiting proportion for Class 1, 2 and 3 compression parts should be obtained from Table 5.2. A part which fails to satisfy the limits for Class 3 should be taken as Class 4.
Classification table 5.2. EN 1993-1-1:

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

Internal compression parts



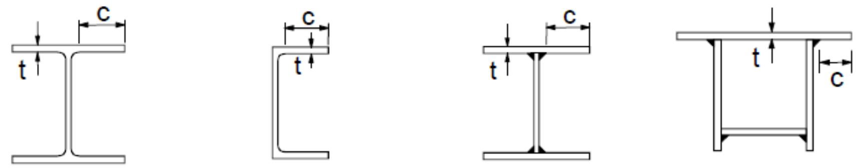
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \leq 72\varepsilon$	$c/t \leq 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{36\varepsilon}{\alpha}$
2	$c/t \leq 83\varepsilon$	$c/t \leq 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 124\varepsilon$	$c/t \leq 42\varepsilon$	when $\psi > -1$: $c/t \leq \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \leq -1^{*)}$: $c/t \leq 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

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

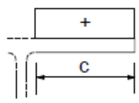
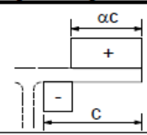
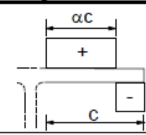
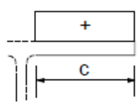
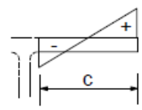
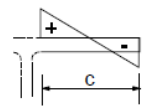
*) $\psi \leq -1$ applies where either the compression stress $\sigma \leq f_y$ or the tensile strain $\varepsilon_y > f_y/E$

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

Outstand flanges



Rolled sections — Welded sections

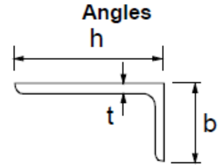
Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \dfrac{9\varepsilon}{\alpha}$	$c/t \le \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$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

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

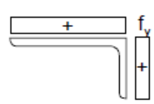
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

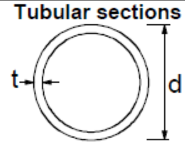
Refer also to "Outstand flanges" (see sheet 2 of 3)



Does not apply to angles in continuous contact with other components

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

Tubular sections



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

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

Example: Hall.esa

This classification is also inputted in SCIA Engineer:

Look at beam B44:

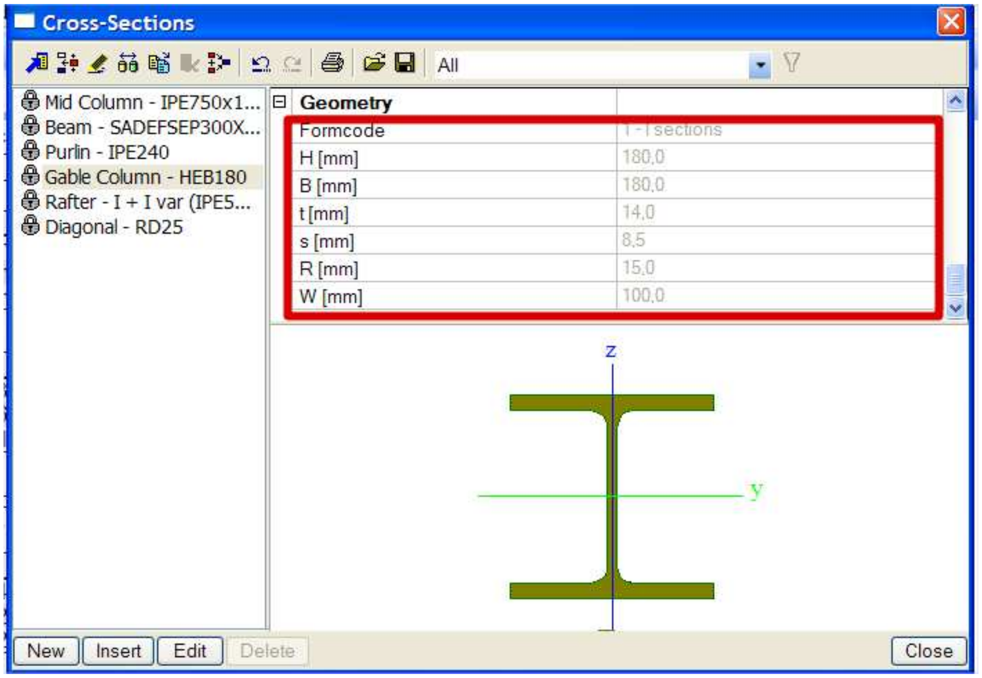


Table 5.2 (sheet 1):
c/t = (180-2*14-2*15)/8,5 = 122/8,5 = **14,35**

Table 5.2 (sheet 2):
c/t = (180/2 - 8,5/2 - 15)/14 = 70,75/14 = **5,05**

Width-to-thickness ratio for internal compression parts (EN 1993-1-1 : Tab.5.2. sheet 1).
ratio 14.35 on position 12.06 m

ratio		
maximum ratio	1	54.55
maximum ratio	2	62.81
maximum ratio	3	128.66

==> Class cross-section 1
Width-to-thickness ratio for outstand flanges (EN 1993-1-1 : Tab.5.2. sheet 2).
ratio 5.05 on position 12.06 m

ratio		
maximum ratio	1	9.00
maximum ratio	2	10.00
maximum ratio	3	14.03

==> Class cross-section 1

Look at beam B26:

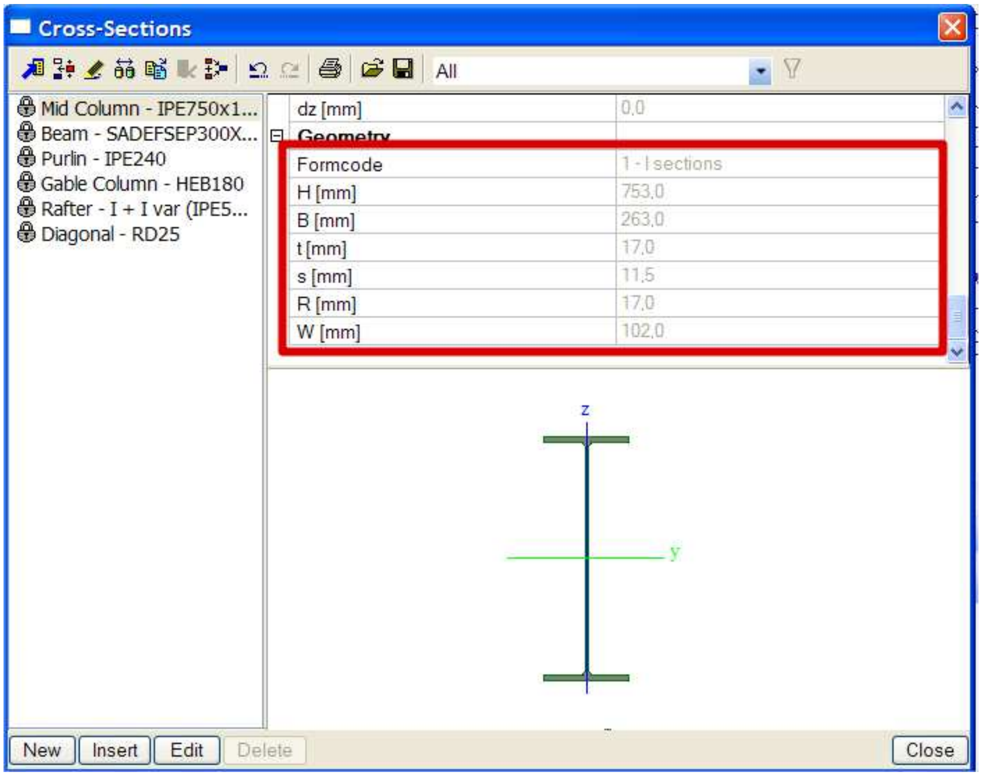


Table 5.2 (sheet 1):
c/t = (753 - 2*17 - 2*17)/11,5 = 685/11,5 = **59,57**

Table 5.2 (sheet 2):
c/t = (263/2 - 11,5/2 - 17)/17 = 108,75/14 = **6,40**

Width-to-thickness ratio for internal compression parts (EN 1993-1-1 : Tab.5.2. sheet 1).
ratio 59.57 on position 0.00 m

ratio		
maximum ratio	1	26.85
maximum ratio	2	30.92
maximum ratio	3	34.17

==> Class cross-section 4
Width-to-thickness ratio for outstand flanges (EN 1993-1-1 : Tab.5.2. sheet 2).
ratio 6.40 on position 0.00 m

ratio		
maximum ratio	1	7.32
maximum ratio	2	8.14
maximum ratio	3	11.39

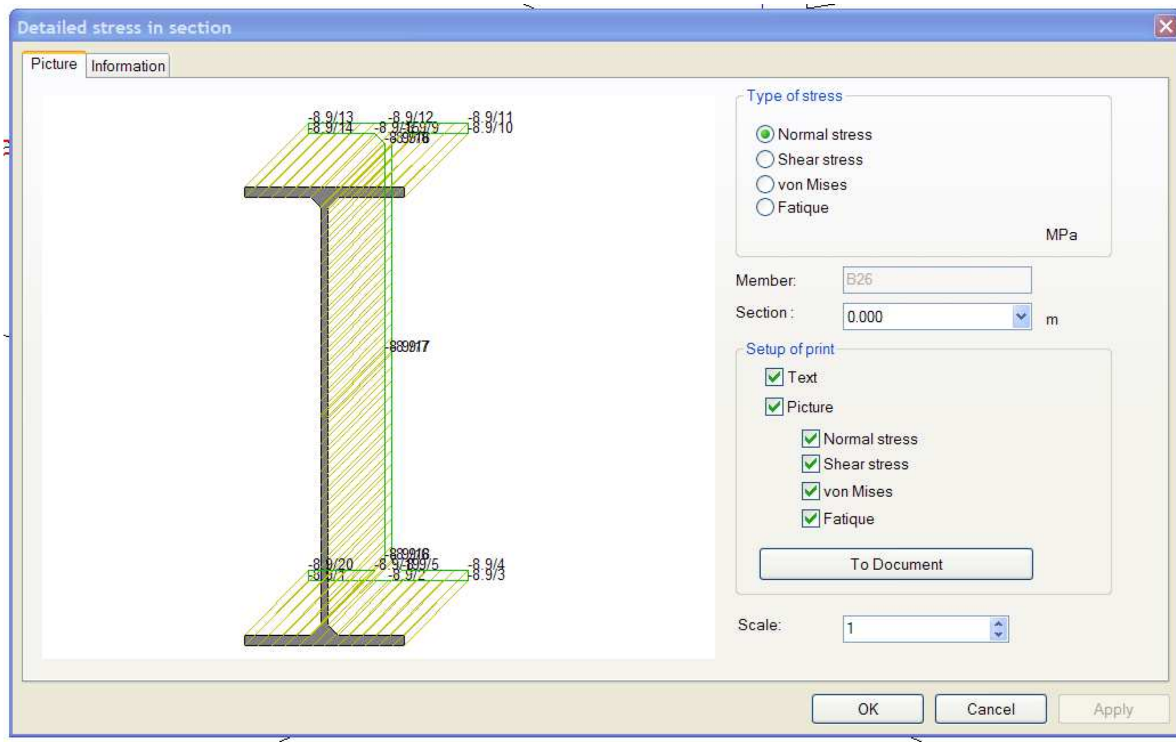
==> Class cross-section 1

This cross-section has as classification class 4, so effective properties have to be calculated.

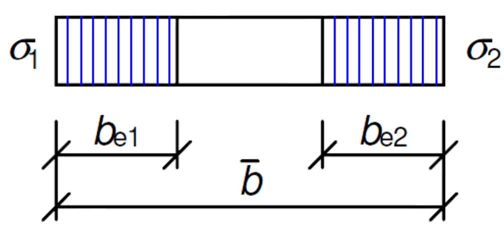
These properties are also given in SCIA Engineer:

Properties					
sectional area A eff	14401.5	mm^2			
Shear area Vy eff	8942.0	mm^2	Vz eff	5459.5	mm^2
radius of gyration iy eff	329.9	mm	iz eff	59.9	mm
moment of inertia Iy eff	1567384430.2	mm^4	Iz eff	51633558.9	mm^4
elastic section modulus Wy eff	4163039.7	mm^3	Wz eff	392650.6	mm^3
Eccentricity eny	-0.0	mm	enz	0.0	mm

The calculation of the sectional area A_{eff} is given below:



The used diagram is the following (EN 1993-1-5:2006 – Tabel 4.1 and EN 1993-1-5:2006/AC:2009 article 9):



$\underline{\psi = 1}$:

$b_{\text{eff}} = \rho\ \bar{b}$

$b_{\text{e1}} = 0{,}5\ b_{\text{eff}}$ $\qquad$ $b_{\text{e2}} = 0{,}5\ b_{\text{eff}}$

$\overline{\lambda_p} \leq 0{,}5 + \sqrt{0.085 - 0.055\psi}\ = 0{,}673 \qquad \rho = 1$

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

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

ε = 0,81 (S355)

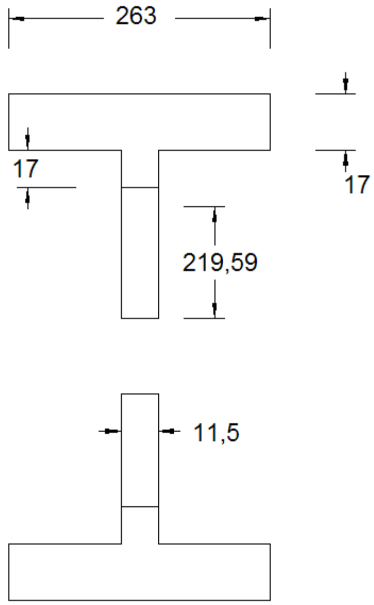
k_σ = 4,0 (Tabel 4.1 of EN 1993-1-5:2006)

$\overline{\lambda_p} = 1{,}29$

$$\rho = \frac{\overline{\lambda_p} - 0.22}{\overline{\lambda_p}^2} = 0{,}64$$

be1 = be2 = 0,5* 0,64 * 685 mm = 219,59 mm

Aeff = [263 x 17 + 219,59 x 11,5 + 17 x 11,5] x 2 = 13384 mm²



Section 6: Ultimate limit state

General

EN 1993-1-1 **article 6.1**.

The following safety factors are taken into account:

γ_{M0} = 1,00 Resistance of cross-section
γ_{M1} = 1,00 Resistance of members to instability accessed by member checks
γ_{M2} = 1,25 Resistance of cross-section in tension to fracture

National Annex:

NEN: Clause 14 of NEN 6770 shall be applied.

NBN: De aanbevolen waarden zijn normatief.

NF: Les valeurs à utiliser sont les valeurs recommandées.

Dans le cas d'une analyse au second ordre avec imperfections locales dans les barres, la résistance des sections est vérifiée en utilisant le coefficient γ_{M1} en lieu et place de γ_{M0} dans les formules de résistance.

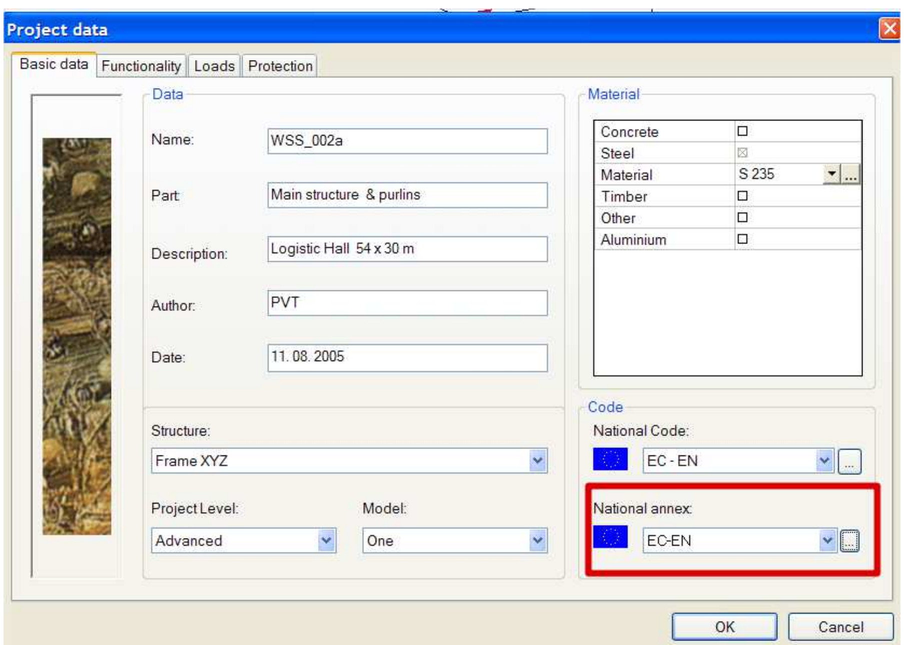
NOTE : For other recommended values see EN 1993 part 2 to part 6 the national Annex may define the partial factors γ_{Mi}; it is recommended to take the partial factors γ_{Mi} from EN 1993-2.

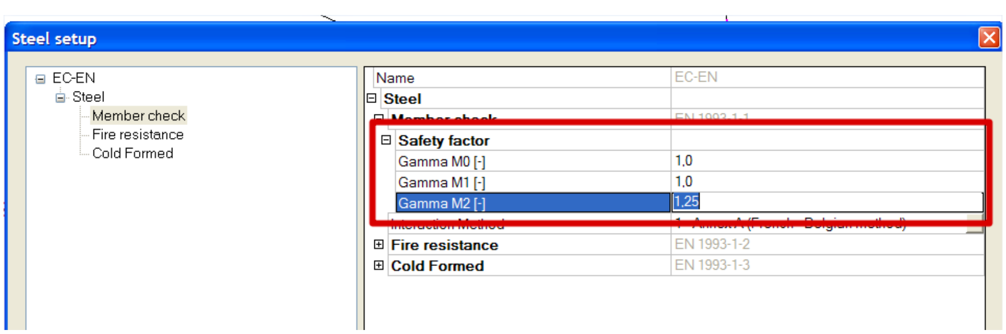
National Annex

NEN: For structures not covered by parts 2 to 6 of NEN-EN 1993, γ_{Mi} shall be taken from NEN-EN 1993-2.
For buildings γ_{M0} and γ_{M1} shall be taken as 1,00 and γ_{M2} shall be taken as 1,25.

NBN: Voor constructies die niet gedekt zijn door EN 1993 Deel 2 tot Deel 6, behoren de numerieke waarden voor de partiële factoren γ_{Mi} opgegeven te zijn door de opdrachtgever.
Indien geen waarde is opgegeven door de opdrachtgever, zijn de aanbevolen waarden bevestigd.

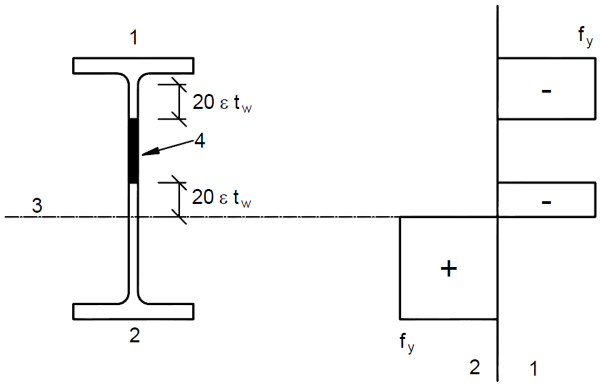
NF: Pour les structures non couvertes dans les parties applicatives 2 à 6 de l'EN 1993, il appartient aux documents du marché de préciser les valeurs des coefficients partiels à appliquer.





NOTE: Following EN 1993-1-1 **article 6.2.2.4**.

Where cross-sections with a class 3 web and class 1 or 2 flanges are classified as effective Class 2 cross-sections, see 5.5.2(11), the proportion of the web in compression should be replaced by a part of $20\varepsilon t_w$ adjacent to the compression flange with another part of $20\varepsilon t_w$ adjacent to the plastic neutral axis of the effective cross-section in accordance with figure 6.3:



1 compression
2 tension
3 plastic neutral axis
4 neglect

Figure 6.3: Effective class 2 web

NOTE: Following EN 1993-1-1 **article 6.2.2.5 (4)**.

Where a class 4 cross section is subjected to an axial compression force, the method given in EN 1993-1-5 should be used to determine the possible shift e_N of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross section and the resulting additional moment: $\Delta M_{Ed} = N_{Ed} e_N$

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**):

$$\left(\frac{\sigma_{x,Ed}}{\frac{f_y}{\gamma_{M0}}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{\frac{f_y}{\gamma_{M0}}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{\frac{f_y}{\gamma_{M0}}}\right)^2 \left(\frac{\sigma_{z,Ed}}{\frac{f_y}{\gamma_{M0}}}\right)^2 + 3\left(\frac{\tau_{Ed}}{\frac{f_y}{\gamma_{M0}}}\right)^2 \leq 1$$

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

As a conservative approximation for all cross-section classes a linear summation of the utilization ratios for each stress resultant may be used. For class 1, class 2 or class 3 cross sections subjected to the combination of N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ this method may be applied using the following criteria:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1$$

For class 4 sections:

EN 1993-1-1 **formula (6.44)**:

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

Where A_{eff} is the effective area of the cross-section when subjected to uniform compression
$W_{eff, min}$ is the effective section modulus of the cross-section when subjected only to moment about the relevant axis
e_N is the shift of the relevant centroidal axis when the cross-section is subjected to compression only.

Section properties

Effective cross-section properties of Class 4 cross-sections
EN 1993-1-1 **article 6.2.2.5**.

- The effective cross-section properties of Class 4 cross-sections should be based on the effective widths of the compression parts
- For cold formed thin walled sections see EN 1993-1-3
- The effective widths of planar compression parts should be obtained from EN 1993-1-5

(see also example “Classification of cross-section”)

Tension

EN 1993-1-1 **article 6.2.3**.

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

Where $N_{t,Rd}$ should be taken as the smaller of:

- $N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ the design plastic resistance of the gross cross-section
- $N_{u,Rd} = \frac{0{,}9 \cdot A_{net} \cdot f_u}{\gamma_{M2}}$ the design ultimate resistance of the net gross cross-section at holes for fasteners.

Example: Hall.esa

This classification is also inputted in SCIA Engineer:

Take a look at B150 (Non linear combination NC1):

The critical check is on position 7.29 m

Internal forces		
NEd	45.10	kN
Vy,Ed	-0.00	kN
Vz,Ed	-0.00	kN
TEd	-0.00	kNm
My,Ed	-0.00	kNm
Mz,Ed	-0.00	kNm

Normal force check
according to article EN 1993-1-1 : 6.2.3. and formula EN 1993-1-1 : (6.5)

Table of values		
Nt.Rd	174.17	kN
unity check	0.26	

Compression

EN 1993-1-1 **article 6.2.4**.

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

Where

- $N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ For class 1, 2 or 3 cross-sections
- $N_{c,Rd} = \frac{A_{eff} \cdot f_y}{\gamma_{M0}}$ For class 4 cross-sections

NOTE:
In the case of unsymmetrical Class 4 cross-sections, the method given in 6.2.9.3 should be used to allow for the additional moment ΔM_{Ed} due to the eccentricity of the centroidal axis of the effective section, see 6.2.2.5(4).

Example: Hall.esa

This classification is also inputted in SCIA Engineer:

B26 (Non linear combination NC1):

The critical check is on position 3.18 m

Internal forces		
NEd	-149.48	kN
Vy,Ed	0.16	kN
Vz,Ed	100.76	kN
TEd	0.08	kNm
My,Ed	328.29	kNm
Mz,Ed	0.52	kNm

Warning: Torsion is not taken into account for this cross-section!
Warning: Torsion is not taken into account for this cross-section!
Compression check
according to article EN 1993-1-1 : 6.2.4 and formula EN 1993-1-1 : (6.9)
Section classification is 2.

Table of values		
Nc.Rd	6212.50	kN
unity check	0.02	

Bending moment

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

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

Where

- $M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}}$ For class 1 or 2 cross-sections
- $M_{c,Rd} = M_{el,Rd} = \frac{W_{el,min} \cdot f_y}{\gamma_{M0}}$ For class 3 cross-sections
- $M_{c,Rd} = \frac{W_{eff} \cdot f_y}{\gamma_{M0}}$ For class 4 cross-sections

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.

Example: Hall.esa

This check is also inputted in SCIA Engineer:

Look at beam B26 (Non linear combination NC1):

Bending moment check (My)
according to article EN 1993-1-1 : 6.2.5. and formula EN 1993-1-1 : (6.12)
Section classification is 2.

Table of values		
Mc,Rd	1727.08	kNm
unity check	0.19	

Bending moment check (Mz)
according to article EN 1993-1-1 : 6.2.5. and formula EN 1993-1-1 : (6.12)
Section classification is 2.

Table of values		
Mc,Rd	218.01	kNm
unity check	0.00	

Shear

EN 1993-1-1 **article 6.2.6**.

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

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

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

Where A_v is the shear area. The formula for A_v depends on the cross-section (see EN 1993-1-1 article 6.2.6(3)).

For elastic design, $V_{c,Rd}$ is the design elastic shear resistance. The following criterion for a critical point of the cross-section may be used unless the buckling verification in section 5 of EN 1993-1-5 applies:

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

Where $\tau_{Ed} = \frac{V_{Ed}\cdot S}{I\,t}$

For I- or H-section, the shear stress in the web may be taken as:

$\tau_{Ed} = \frac{V_{Ed}}{A_w}$ if $\frac{A_f}{A_w} \geq 0{,}6$

Where A_f is the area of one flange
A_w is the area of the web: $A_w = h_w t_w$

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.

Where the shear force is combined with a torsional moment, the plastic resistance $V_{pl,Rd}$ should be reduced as specified in 6.2.7(9).

Example: Hall.esa

This check is also inputted in SCIA Engineer.

Look at beam B26 (Non linear combination NC1):

Shear check (Vy)
according to article EN 1993-1-1 : 6.2.6. and formula EN 1993-1-1 : (6.17)

Table of values		
Vc,Rd	1892.08	kN
unity check	0.00	

Shear check (Vz)
according to article EN 1993-1-1 : 6.2.6. and formula EN 1993-1-1 : (6.17)

Table of values		
Vc,Rd	2033.65	kN
unity check	0.05	

Torsion

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

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

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

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

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

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

As a simplification, in the case of a member with a closed hollow cross-section, such as structural hollow section, it may be assumed that the effects of torsional warping can be neglected.
Also 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.

For combined shear force and torsional moment the plastic shear resistance accounting for torsional effects should be reduced from $V_{pl,Rd}$ to $V_{pl,T,Rd}$ and the shear force should satisfy:

$$\frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1$$

In which $V_{pl,T,Rd}$ should be derived with formulas (6.26), (6.27) or (6.28) depending on the cross-section.

Example: Hall.esa

This check is also inputted in SCIA Engineer:

Look at beam B28 (Non linear combination NC1):

Torsion check
according to article EN 1993-1-1 : 6.2.7. and formula EN 1993-1-1 : (6.23)

Table of values		
tau t,Rd	205.90	MPa
tau t, Ed	2.30	MPa
unity check	0.01	

Bending and shear

EN 1993-1-1 **article 6.2.8**.

Where the shear force is less than half the plastic shear resistance its effect on the moment resistance $M_{c,Rd}$ may be neglected except where shear buckling reduces the section resistance.

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

$$(1-\rho)f_y$$

Where

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

And

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

When torsion is present ρ should be obtained from

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

But ρ should be taken as 0 for $V_{Ed} \leq 0{,}5\, V_{pl,T,Rd}$

Also this check has been implemented in SCIA Engineer.

National annex

Note in EN 1993-1-1 article 6.2.8(6): For the interaction of bending, shear and transverse laods see section 7 of EN 1993-1-5.

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

NEN: Read after the full text of (6): Clauses 11.3.1.1 and 11.3.1.3 of NEN 6770 shall be applied.

Bending and axial force

Class 1 and 2 cross-sections

EN 1993-1-1 **article 6.2.9.1**.

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

The formula for $M_{N,Rd}$ depends on the cross-section of the member.

Bi-axial bending

For bi-axial bending the following criterion may be used:

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

in which α and β are defined as follows:

- I and H sections:

  $\alpha = 2$; $\beta = 5n$ but $\beta \geq 1$
- Circular hollow sections:

  $\alpha = 2$; $\beta = 2$
- Rectangular hollow sections:

  $\alpha = \beta = \frac{1{,}66}{1-1{,}13n^2}$ but $\alpha = \beta \leq 6$

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

Class 3 cross-sections

EN 1993-1-1 **article 6.2.9.2**.

In the absence of shear force, for Class 3 cross-sections the maximum longitudinal stress $\sigma_{x,Ed}$ should satisfy the criterion:

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

Class 4 cross-sections

EN 1993-1-1 **article 6.2.9.3**.

In the absence of shear force, for Class 4 cross-sections the maximum longitudinal stress $\sigma_{x,Ed}$ calculated using the effective cross-sections should satisfy the criterion:

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

The following criterion should be met:

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

Where A_{eff} is the effective area of the cross-section when subjected to uniform compression
$W_{eff,min}$ is the effective section modulus of the cross-section when subjected only to moment about the relevant axis
e_N is the shift of the relevant centroidal axis when the cross-section is subjected to compression only.

Bending, shear and axial force

EN 1993-1-1 **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.

<u>National annex</u>

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

NEN: Read after the full text of (3): Clauses 11.3.1.1 to 11.3.1.3 and 11.3.2.1 to 11.3.2.3 of NEN 6770 and clause 11.3 of NEN 6771 shall be applied.

Example: Hall.esa

This check is also inputted in SCIA Engineer:

Look at beam B26 (Non linear combination NC1):

This beam has been checked on position 3,18m on the beam. On this place the classification = Class2.

For this cross-section (IPE 750), the following formula is given:

Compression check
according to article EN 1993-1-1 : 6.2.4 and formula EN 1993-1-1 : (6.9)
Section classification is 2.

Table of values		
Nc.Rd	6212.50	kN
unity check	0.02	

⇨ n = 0,02

Combined bending, axial force and shear force check
according to article EN 1993-1-1 : 6.2.9.1. and formula EN 1993-1-1 : (6.41)
Section classification is 2.

Table of values		
MNVy.Rd	1727.08	kNm
MNVz.Rd	218.01	kNm

alfa	2.00	beta	1.00
unity check	0.04		

Buckling resistance of members

Uniform members in compression

EN 1993-1-1 **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 $N_{b,Rd}$ is the design buckling resistance of the compression member

- $N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M0}}$ For class 1, 2 and 3 cross-sections
- $N_{b,Rd} = \frac{\chi \cdot A_{eff} \cdot f_y}{\gamma_{M0}}$ For class 4 cross-sections

Where 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 $\Phi = 0{,}5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right]$
$\bar{\lambda}$ is the non-dimensional slenderness
α is an imperfection factor, depending on the buckling curve

For members with non-symmetric Class 4 sections, allowance should be made for the additional moment ΔM_{Ed} due to the eccentricity of the centroidal axis of the effective section, see also 6.2.2.5(4), and the interaction should be carried out to 6.3.4 or 6.3.3.

Slenderness for buckling

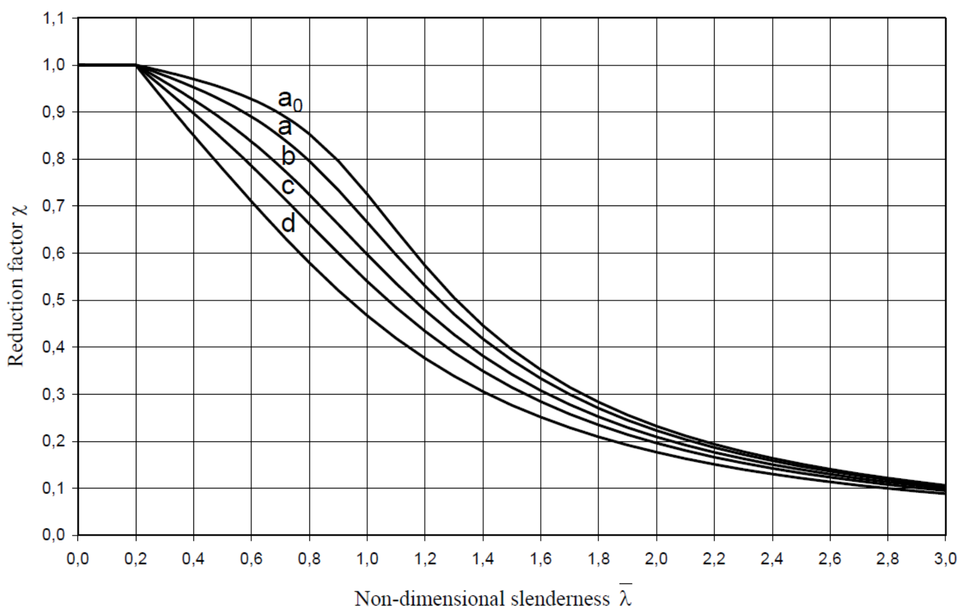
- $\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$ For class 1, 2 and 3 cross-sections
- $\bar{\lambda} = \sqrt{\frac{A_{eff} \cdot f_y}{N_{cr}}}$ For class 4 cross-sections

Buckling curves
The buckling curve for a cross-section should be obtained from Table 6.2 (see next page).

The imperfection factor α corresponding to the appropriate buckling curve:

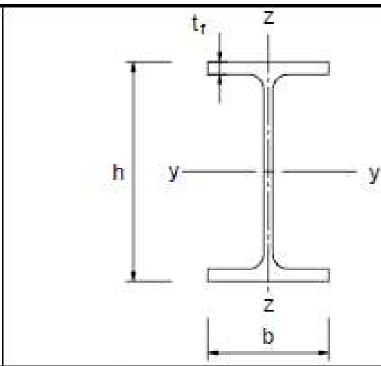
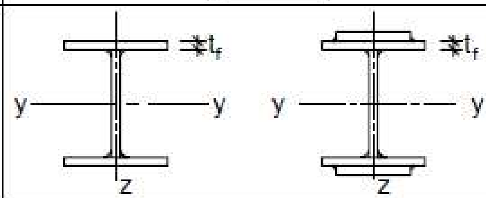
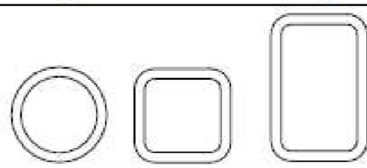
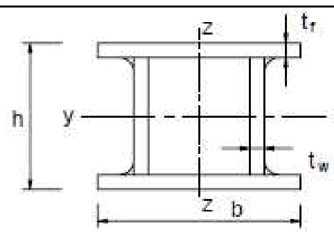
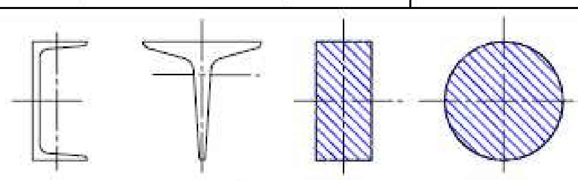
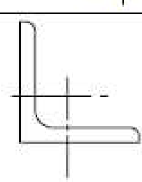
Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Values of the reduction factor χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ may be obtained from the following figure with buckling curves:



For slenderness $\bar{\lambda} \leq 0{,}2$ or for $\frac{N_{Ed}}{N_{cr}} \leq 0{,}04$ the buckling effects may be ignored (EN 1993-1-1 article 6.3.1.2 (4)).

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

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

Slenderness for flexural buckling

- $\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_1}$ For class 1, 2 and 3 cross-sections
- $\bar{\lambda} = \sqrt{\frac{A_{eff} \cdot f_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda_1}$ For class 4 cross-sections

Where:

L_{cr} is the buckling length
i is the radius of gyration about the relevant axis

$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93{,}3\ \varepsilon$ For class 1, 2 and 3 cross-sections

$\varepsilon = \sqrt{\frac{235}{f_y}}$ (f_y in N/mm²)

For flexural buckling the appropriate buckling curve may be determined from Table 6.2.

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

NBN: Hoe rekening te houden met de belemmering van de hoekverdraaiing van L_{cr} is gegeven in Bijlage F van deze nationale bijlage.

NEN: Read after the full text of (2): Clauses 12.1.1.3 and 12.1.5.3.2 of NEN 6770 and clause 12.1.1.3 of NEN 6771 shall be applied.

Slenderness for torsional and torsional-flexural buckling

- $\bar{\lambda}_T = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$ For class 1, 2 and 3 cross-sections
- $\bar{\lambda}_T = \sqrt{\frac{A_{eff} \cdot f_y}{N_{cr}}}$ For class 4 cross-sections

Where:

$N_{cr} = N_{cr,TF}$ but $N_{cr} < N_{cr,T}$

$N_{cr,TF}$ is the elastic torsional-flexural buckling force
$N_{cr,T}$ is the elastic torsional buckling force

For torsional or torsional-flexural buckling the appropriate buckling curve may be determined from Table 6.2 considering the one related to the z-axis.

National Annex

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

NBN: Hoe men N_{cr}, $N_{cr,T}$ *en* $N_{cr,TF}$ bepaalt, is gegeven in Bijlage E van deze nationale bijlage.

NEN: Read after full text of (3): Clauses 12.1.2 and 12.1.3 of NEN 6770 shall be applied.

Example: Hall.esa

Look at beam B26 (Non linear combination NC1):
The buckling check is given in SCIA Engineer:

Buckling parameters	yy	zz	
type	non-sway	non-sway	
Slenderness	20.14	122.24	
Reduced slenderness	0.24	1.45	
Buckling curve	a	b	
Imperfection	0.21	0.34	
Reduction factor	0.99	0.36	
Length	6.90	6.90	m
Buckling factor	0.88	0.96	
Buckling length	6.09	6.64	m
Critical Euler load	89415.28	2427.26	kN

Buckling check
according to article EN 1993-1-1 : 6.3.1.1. and formula EN 1993-1-1 : (6.46)

Table of values		
Nb.Rd	1844.45	kN
unity check	0.08	

This beam will buckle first around the weak axis (z-axis). So Nb,Rd is calculated with the reduction factor around this axis.

Automatic calculation of the buckling length in SCIA Engineer

For the calculation of the buckling ratios *k*, some approximate formulas are used. These formulas are treated in SCIA Engineer's Theoretical Background (Ref.[10]).

The following formulas are used for the buckling ratios:

– for a *non sway* structure:

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

– for a *sway* structure:

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

With
l the buckling length
L the system length
E the modulus of Young
I the moment of inertia
C_i the stiffness in node i
M_i the moment in node i
ϕ_i the rotation in node i

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

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

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

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

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.

Attention: 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, and might overwrite them manually.

Uniform members in bending

EN 1993-1-1 **article 6.3.2**.

Buckling resistance

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

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

$W_y = W_{pl,y}$ for class 1 or 2 cross-sections
$W_y = W_{el,y}$ for class 3 cross-sections
$W_y = W_{eff\,y}$ for class 4 cross-sections

Lateral torsional buckling curves

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

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

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

National Annex:

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

NBN: Hoe men Mcr bepaalt, is gegeven in bijlage D van deze ANB.
Hoe men $\bar{\lambda}_{LT}$ benadert op een eenvoudige manier, is gegeven in bijlage G van deze ANB.

NF: l'Annexe MCR de la présente norme fournit une formulation du calcul de M_{cr} pour des barres uniformes à sections indéformables doublement symétriques, simplement fléchies et maintenues au déversement à leurs deux extrémités.

Recommended buckling curves:

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

Recommended values for imperfection factors:

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0,21	0,34	0,49	0,76

National Annex:

NEN: The values for α_{LT} shall be taken from Table 6.3.

NBN: De aanbevolen waarden α_{LT} voor zijn normatief.

NF: Les valeurs à utiliser sont les valeurs recommandées.

For slendernesses $\bar{\lambda}_{LT} \leq \bar{\lambda}_{LT,0}$ or for $\frac{M_{Ed}}{M_{cr}} \leq \bar{\lambda}_{LT,0}{}^2$ lateral torsional buckling effects may be ignored.

EN 1993-1-1 **article 6.3.2.3**: $\bar{\lambda}_{LT,0}$= 0,4 (maximum value) – see also the next paragraph for the adapted value following the National Annex.

Example: Hall.esa

Look at beam B26 (Non linear combination NC1):
The lateral torsional buckling check is given in SCIA Engineer:

Lateral Torsional Buckling Check
According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	4.163e+06	mm^3
Elastic critical moment Mcr	1682.58	kNm
Relative slenderness Lambda,LT	0.94	
Limit slenderness Lambda,LT,0	0.40	
LTB curve	b	
Imperfection Alpha,LT	0.34	
Reduction factor Chi,LT	0.64	
Buckling resistance Mb.Rd	941.76	kNm
Unity check	0.29	-

Mcr Parameters		
LTB length	6.900	m
k	1.00	
kw	1.00	
C1	1.74	
C2	0.01	
C3	1.00	

Note: C Parameters according to ECCS 119 2006 / Galea 2002
load in center of gravity

Lateral torsional buckling for rolled sections or equivalent welded sections

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

But: $\chi_{LT} \leq 1{,}0$

$\chi_{LT} \leq \frac{1}{\bar{\lambda}_{LT}^2}$

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

and

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

β= 0,75 (minimum value)

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

National Annex

NF: Les valeurs de $\bar{\lambda}_{LT,O}$ et de α_{LT} sont calculées au moyen des expressions suivantes :

- Sections laminées en I (doublement symétriques) :

  $\bar{\lambda}_{LT,O} = 0{,}2 + 0{,}1b/h$ et $\alpha_{LT} = 0{,}4 - 0{,}2\frac{b}{h}\bar{\lambda}_{LT}^2 \geq 0$

- Sections soudées en I (doublement symétriques) :

  $\bar{\lambda}_{LT,O} = 0{,}3b/h$ et $\alpha_{LT} = 0{,}5 - 0{,}25\frac{b}{h}\bar{\lambda}_{LT}^2 \geq 0$

- Autres sections

  $\bar{\lambda}_{LT,O} = 0{,}2$ et $\alpha_{LT} = 0{,}76$

Et pour toutes les sections : β = 1,0.

Pour l'application de cette clause, on admet comme sections soudées équivalentes des sections reconstituées soudées en I dont les caractéristiques respectent les dispositions suivantes :

- La section est symétrique par rapport à l'âme
- Le rapport des inerties des semelles dans leur plan n'excède pas 1,2
- $t_{f,max} / t_w \leq 3$

NBN : $\bar{\lambda}_{LT,O}$= 0,2 en β= 1,0

Als alternatief mogen voor liggers in gebouwen met steunen waarden $\bar{\lambda}_{LT,O}$= 0,4 en β= 0,75 toegepast zijn op voorwaarde dat deze steunen volledig verwaarloosd zijn bij de bepaling van M_{cr}.

NEN: $\bar{\lambda}_{LT,O}$= 0,4 en β= 0,75 (normative)

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

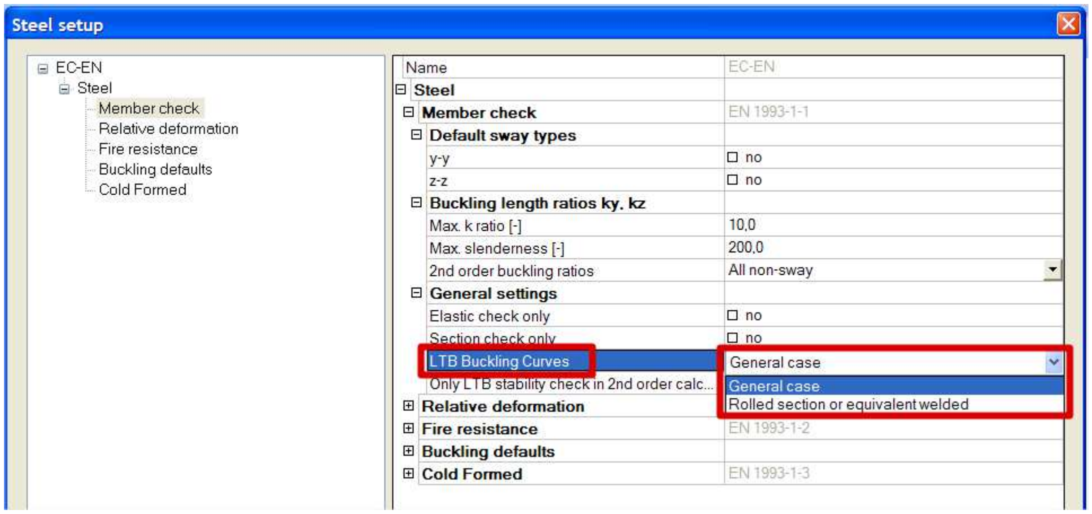
$$\chi_{LT,mod} = \frac{\chi_{LT}}{f}$$

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

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

But $f \leq 1{,}0$

This option can be chosen in SCIA Engineer's in the Steel setup:



National annex

NF: La valeur de *f* à utiliser est la valeur recommandée en vérifiant également que $\chi_{LT} \leq \frac{1}{\bar{\lambda}_{LT}^2}$.

Le coefficient *f* n'est pas applicable qu'à des barres sans maintiens latéraux intermédiaires et en adoptant une hypothèse de gauchissement libre aux extrémités pour le calcul de $\bar{\lambda}_{LT}$.

NBN: f is calculated as indicated above.

NEN: f is calculated as indicated above.

In paragraph 6.3.2.4 a simplified assessment method for beams with restraints in buildings is explained.

Uniform members in bending and axial compression

EN 1993-1-1 **article 6.3.2**

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

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

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

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

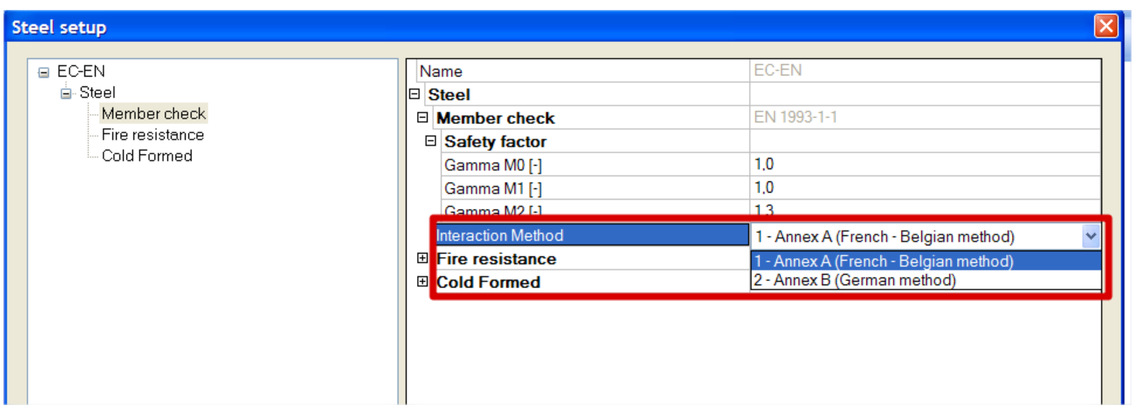
$\boldsymbol{\chi_y}$ **and** $\boldsymbol{\chi_z}$ are the reduction factors due to flexural buckling

$\boldsymbol{\chi_{LT}}$ is the reduction factor due to lateral torsional buckling

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

The interaction factors $\mathbf{k_{yy}}$, $\mathbf{k_{yz}}$, $\mathbf{k_{zy}}$ **and** $\mathbf{k_{zz}}$ have been derived from Annex A (alternative method1) or from Annex B (alternative method 2)

The choice between Interaction Method 1 or 2 can be made in SCIA Engineer in the National Annex parameters:



<u>National annex:</u>

NF: La méthode de référence choisie est la méthode 1 (**Annexe A** de l'EN 1993-1-1). In this annexe some remarks concerning this method are described.

NBN: De alternatieve **methode 1** is normatief. Ten behoeve van de duidelijkheid zijn de formules (6.61) en (6.62) alsook de bijlage A herschreven op een meer gebruiksvriendelijke manier (Zie bijlage C ANB: "Prismatische, op buiging en druk belaste staven"). Deze bijlage moet gebruikt worden in zijn geheel, zonder nog te verwijzen naar de formules (6.61) en (6.62) en naar bijlage A.

NEN: For determination of the values of k_{yy}, k_{yz}, k_{zy} and k_{zz} **Annex B** shall be applied. Read after the full text of (5): Clause 12.3.1.2.3 of NEN 6770 shall be applied.

All the calculated parameters are given in SCIA Engineer:

Example: Hall.esa

Look at beam B26 (Non linear combination NC1):
The combined compression and bending check is given in SCIA Engineer:

Compression and bending check
according to article EN 1993-1-1 : 6.3.3. and formula EN 1993-1-1 : (6.61) (6.62)
Interaction Method 1

Table of values		
kyy	1.055	
kyz	1.024	
kzy	1.012	
kzz	0.983	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	14401.54	mm^2
Wy	4163039.66	mm^3
Wz	392650.64	mm^3
NRk	5112.55	kN
My,Rk	1477.88	kNm
Mz,Rk	139.39	kNm
My,Ed	692.02	kNm
Mz,Ed	1.07	kNm
Interaction Method 1		
Mcr0	965.57	kNm
reduced slenderness 0	1.24	
Cmy,0	0.999	
Cmz,0	0.961	
Cmy	1.000	
Cmz	0.961	
CmLT	1.053	
muy	1.000	
muz	0.960	
wy	1.146	
wz	1.500	
npl	0.029	
aLT	0.999	
bLT	0.002	
cLT	0.987	
dLT	0.002	
eLT	0.282	
Cyy	0.987	
Cyz	0.483	
Czy	0.945	
Czz	0.974	

unity check = 0.03 + 0.75 + 0.01 = 0.79
unity check = 0.08 + 0.72 + 0.01 = 0.81

Uniform built-up compression members

EN 1993-1-1: 2005 **article 6.4**

Uniform built-up compression members with hinged ends that are laterally supported:

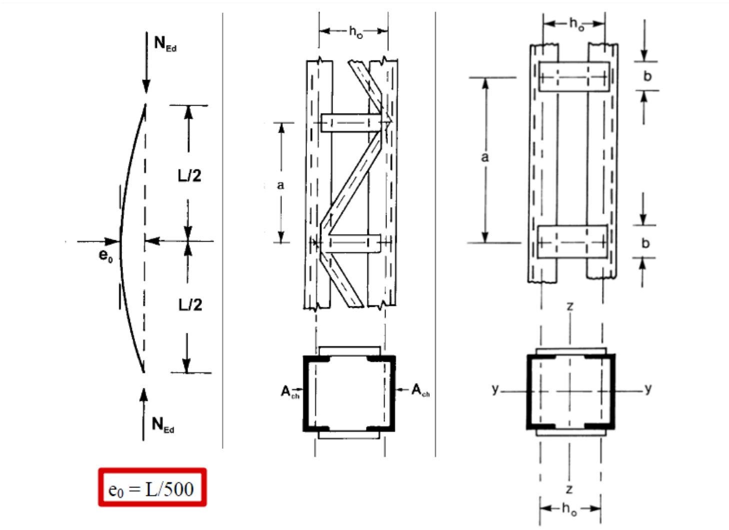


Figure 6.7: Uniform built-up columns with lacings and battenings

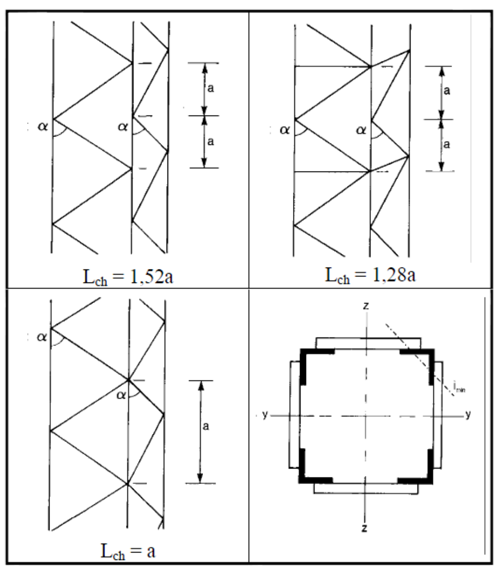


Figure 6.8: Lacings on four sides and buckling length L_{ch} of chords

Checks:

$$N_{ch,Ed} = 0{,}5N_{Ed} + \frac{M_{Ed}h_0A_{ch}}{2I_{eff}}$$

And shear force in the battened members

$$V_{Ed} = \pi\frac{M_{Ed}}{L}$$

Where

$$M_{Ed} = \frac{N_{Ed} e_0 + M_{Ed}^I}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}}$$

$N_{cr} = \frac{\pi^2 E I_{eff}}{L^2}$ is the effective critical force of the built-up member

N_{Ed} is the design value of the compression force to the built-up member

M_{Ed} is the design value of the maximum moment in the middle of the built-up member considering second order effects

M_{Ed}^I is the design value of the maximum moment in the middle of the built-up member without second order effects

h_0 is the distance between the centroids of chords

A_{ch} is the cross-sectional area of one chord

I_{eff} is the effective second moment of area of the built-up member

S_v is the shear stiffness of the lacings or battened panel

Laced compression members

The chords and diagonal lacings subject to compression should be designed for buckling.

Check for chords:

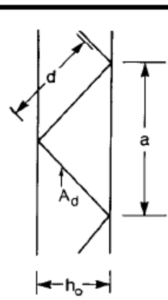
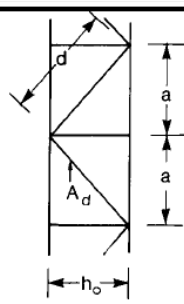
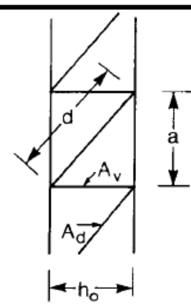
$$\frac{N_{ch,Ed}}{N_{b,Rd}} \le 1$$

$N_{ch,Ed}$: see above

$N_{b,Rd}$: buckling resistance of the chord taking the buckling length L_{ch} from above

$I_{eff} = 0{,}5 h_0^2 A_{ch}$

S_v : see EN 1993-1-1 – Figure 6.9:

System			
S_V	$\frac{nEA_d a h_0^2}{2d^3}$	$\frac{nEA_d a h_0^2}{d^3}$	$\frac{nEA_d a h_0^2}{d^3\left[1+\frac{A_d h_0^3}{A_V d^3}\right]}$

n is the number of planes of lacings
A_d and A_V refer to the cross sectional area of the bracings

Battened compression members

The chords and the battens and their joints to the chord should be checked for the actual moments and forces in an end panel and at mid-span as indicated below:

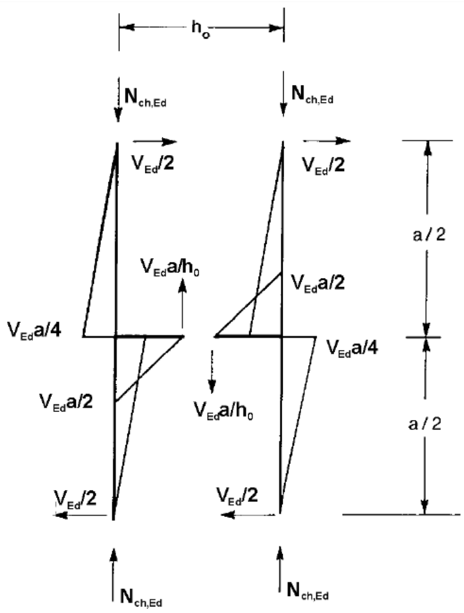


Figure 6.11: Moments and forces in an end panel of a battened built-up member

For simplicity the maximum chord forces $N_{ch,Ed}$ may be combined with the maximum shear force V_{Ed}.

$$S_v = \frac{24\,E\,I_{ch}}{a^2\left[1 + \dfrac{2\,I_{ch}}{n\,I_b}\dfrac{h_0}{a}\right]} \leq \frac{2\,\pi^2\,E\,I_{ch}}{a^2}$$

$$I_{eff} = 0{,}5\,h_0^2\,A_{ch} + 2\,\mu\,I_{ch}$$

Where:

I_{ch}	in plane second moment of area of one chord
I_b	in plane second moment of area of one batten
n	number of planes of lacings
μ	efficiency factor:

Table 6.8: Efficiency factor μ

Criterion	Efficiency factor μ
$\lambda \geq 150$	0
$75 < \lambda < 150$	$\mu = 2 - \dfrac{\lambda}{75}$
$\lambda \leq 75$	1,0
where $\lambda = \dfrac{L}{i_0}$; $i_0 = \sqrt{\dfrac{I_1}{2A_{ch}}}$; $I_1 = 0{,}5h_0^2A_{ch} + 2I_{ch}$	

Closely spaced built-up members

Built-up compression members with chords in contact or closely spaced and connected through paking plates or star battened angle members connected by pairs of battens in two perpendicular planes should be checked for buckling as a single integral member ignoring the effect of shear stiffness ($S_v = \infty$) when the conditions in Table 6.9 are met:

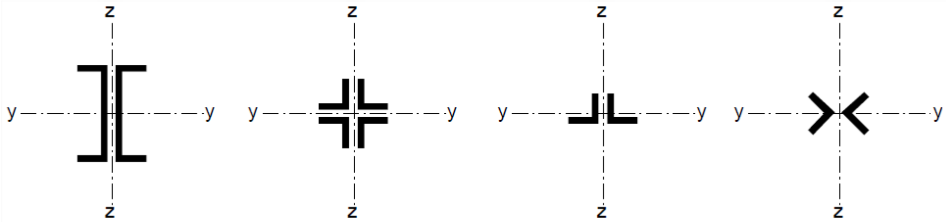


Figure 6.12: Closely spaced built-up members

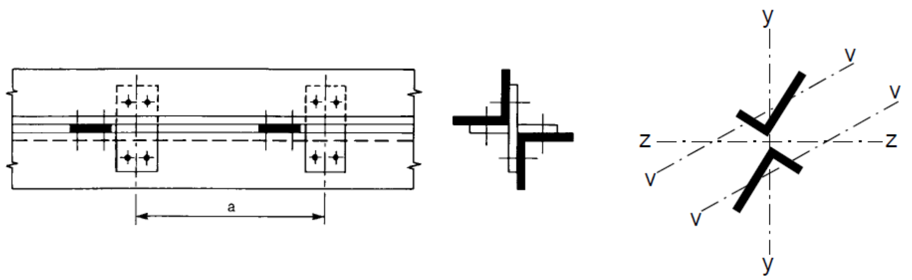


Figure 6.13: Star-battened angle members

Check:

Table 6.9: Maximum spacings for interconnections in closely spaced built-up or star battened angle members

Type of built-up member	Maximum spacing between interconnections *)
Members according to Figure 6.12 connected by bolts or welds	$15\ i_{min}$
Members according to Figure 6.13 connected by pair of battens	$70\ i_{min}$
*) centre-to-centre distance of interconnections i_{min} is the minimum radius of gyration of one chord or one angle	

Section 7: Serviceability limit state

Specified in EN 1990

National annex

NF: The principles for the serviceability limit state are described in the national annex.

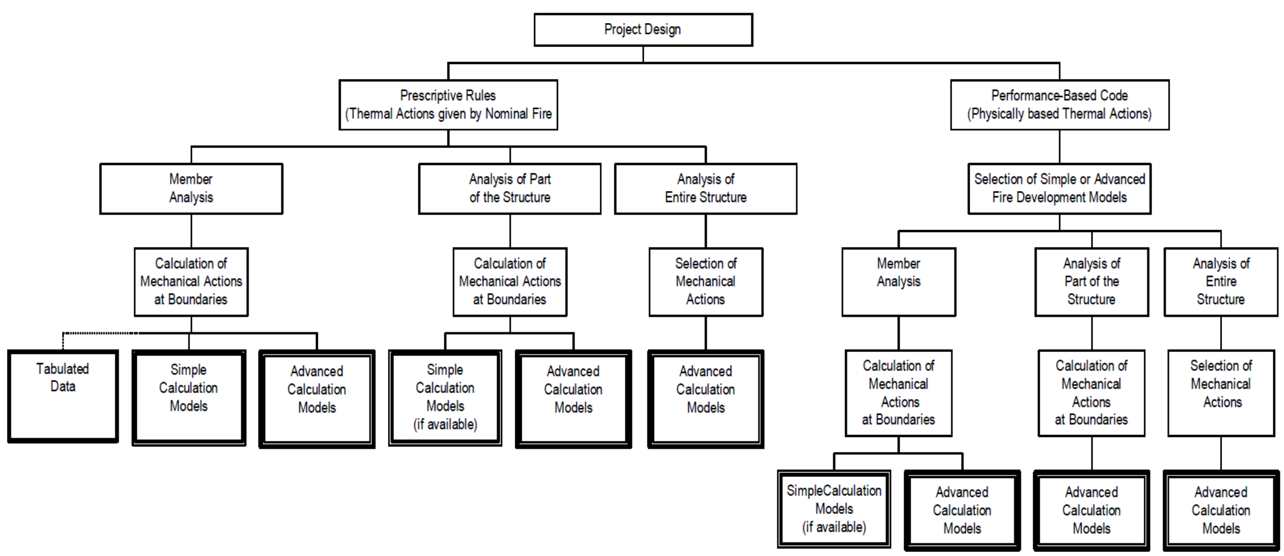
NBN: Refers to EN 1990

NEN: Refers to EN 1990

EN 1993-1-2 Design of Steel Structures: Structural fire design

The following subjects are dealt with in EN 1993-1-2:

Section 1: General
Section 2: Basis of design
Section 3: Material properties
Section 4: Structural fire design



General

In the Scope will be described for which members, steel grades, ... this check will be applicable.

Basis of design

Requirements:

EN 1993-1-2:2005 **article 2.1.2**:

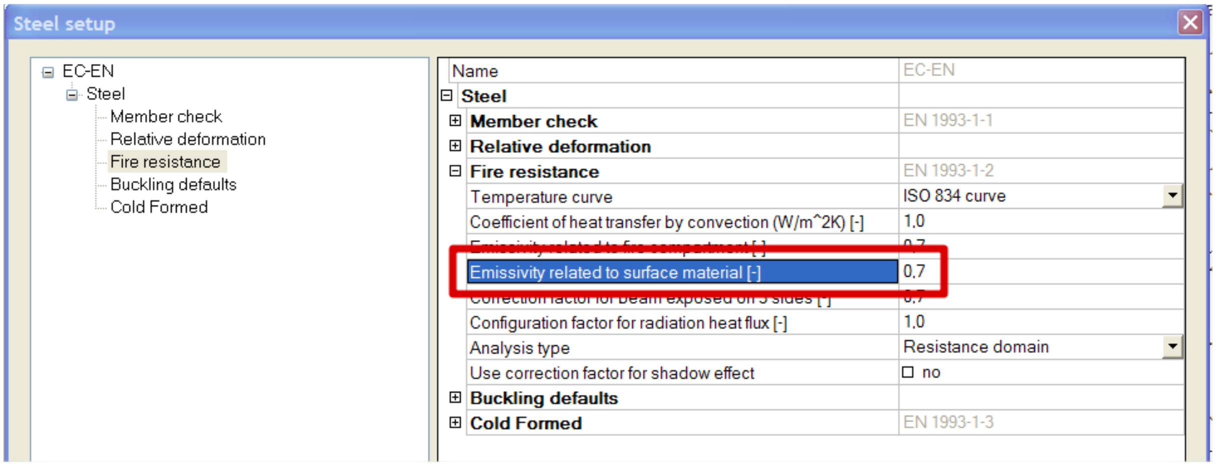
- Criterion R: load bearing only (mechanical resistance)
- Criterion HC: Hydrocarbon fire exposure

Actions:

EN 1993-1-2:2005 **article 2.2**:

Emissivity related to the steel surface should be equal to 0,7 for Carbon steel and equal to 0,4 for Stainless steel.

This can also be inputted in SCIA Engineer:



Design values of material properties:

EN 1993-1-2:2005 **article 2.3**:

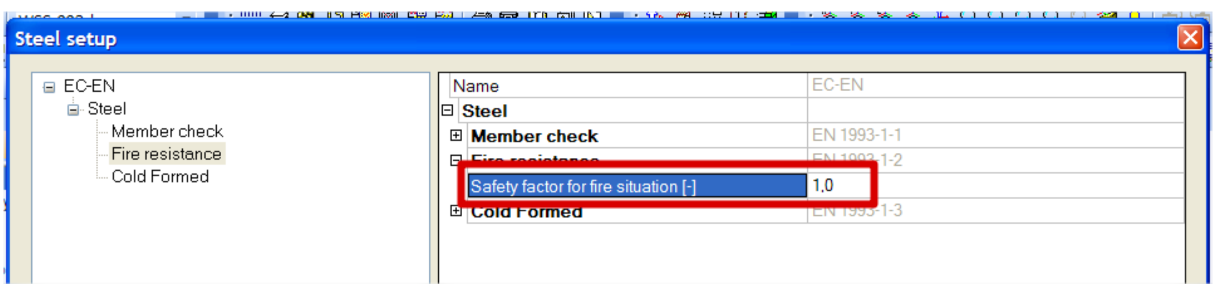
Design values of mechanical material properties:
$X_{d,fi} = k_{\theta} X_k / \gamma_{M,fi}$

Design values of thermal material properties:

If an increase of property is favorable for safety:
$X_{d,fi} = X_k / \gamma_{M,fi}$

If an increase of property is unfavorable for safety:
$X_{d,fi} = \gamma_{M,fi} X_k$

The use of $\gamma_{M,fi}$ = 1 is recommended but can be adapted following the national annexes. This can be adapted in SCIA Engineer also:



<u>National annex</u>

NBN, NEN, NF: $\gamma_{M,fi}$ = 1

Verification methods

EN 1993-1-2:2005 **article 2.4**:

General: $E_{fi,d} \leq R_{fi,d,t}$

The effect of actions should be determined for t=0 using combinations factors $\psi_{1,1}$ or $\psi_{2,1}$ according to EN 1991-1-2 clause 4.3.1.

As a simplification: use reduction factors: $E_{d,fi} = \eta_{fi} E_d$
With the following load reduction factors:

Combination (6.10):

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}}$$

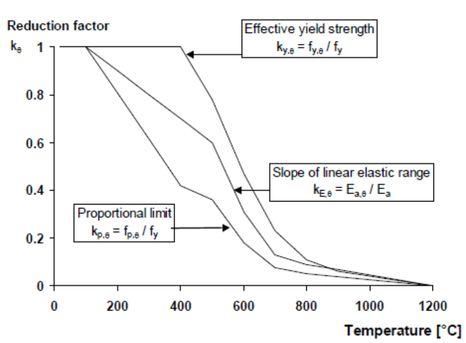
Combination (6.10a) and (6.10b):

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} \psi_{0,1} Q_{k,1}}$$

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\xi \gamma_G G_k + \gamma_{Q,1} Q_{k,1}}$$

Material properties

EN 1993-1-2005 **article 3** (Carbon Steel) + EN 1993-1-2:2005 **Annex C** (Stainless steel):

Carbon steel	**Stainless steel**
Effective yield strength Effective yield strength, relative to yield strength at 20°C: $k_{y,\theta} = f_{y,\theta}/f_y$ Proportional limit, relative to yield strength at 20°C: $k_{P,\theta} = f_{P,\theta}/f_y$ Slope of linear elastic range, relative to slope at 20°C: $k_{E,\theta} = E_{a,\theta}/E_a$	Effective yield strength Proof strength, relative to yield strength at 20°C: $k_{0.2p,\theta} = f_{0.2p,\theta}/f_y$ Tensile strength, relative to tensile strength at 20°C: $k_{u,\theta} = f_{u,\theta}/f_u$ Slope of linear elastic range, relative to slope at 20°C: $k_{E,\theta} = E_{a,\theta}/E_a$
Reduction factors: 	Reduction factors: See tables C.1 and C.2 (EN 1993-1-2)

<table>
<tr>
<td>

Unit mass

$$\rho_a = 7850\ kg/m^3$$

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<td>

Unit mass

$$\rho_a = 7850\ kg/m^3$$

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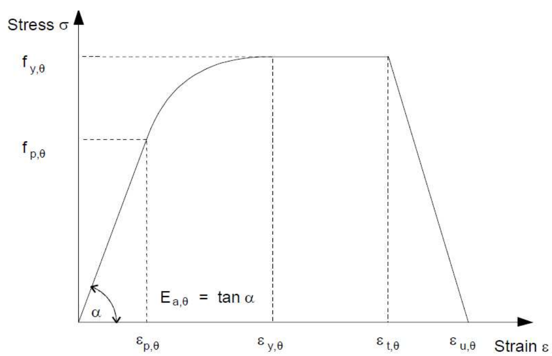
Stress-strain relationship



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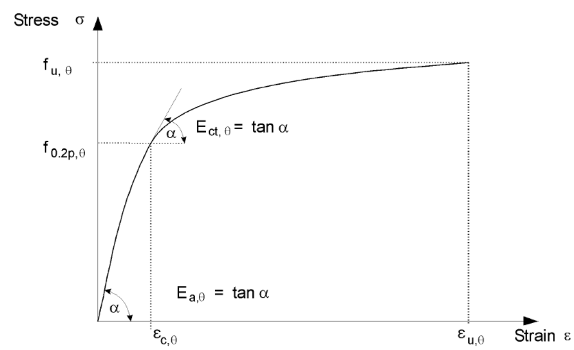
Stress-strain relationship



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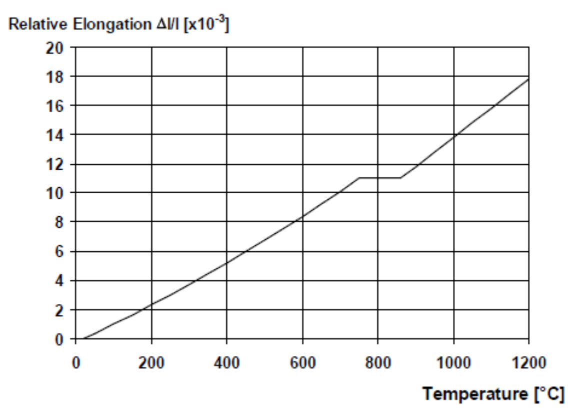
Thermal elongation

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



For $20°C \leq \theta_a < 750°C$

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

For $750°C \leq \theta_a < 860°C$

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

For $860°C \leq \theta_a < 1200°C$

$$\frac{\Delta l}{l} = 2\ x\ 10^{-5}\ \theta_a - \ 6{,}2\ x\ 10^{-3}$$

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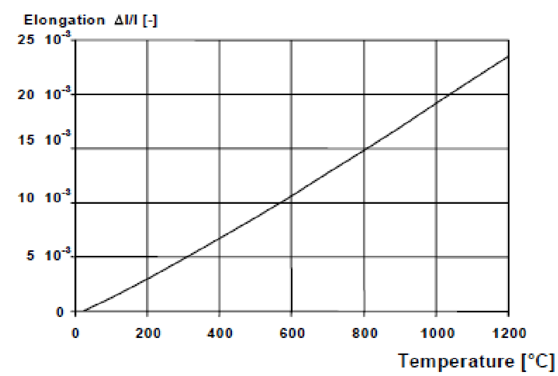
Thermal elongation

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



$$\frac{\Delta l}{l} = (16 + 4{,}79\ x\ 10^{-3}\theta_a - 1{,}243\ x\ 10^{-6}\theta_a^2)$$

$$x\ \ (\theta_a - 20\ )10^{-6}$$

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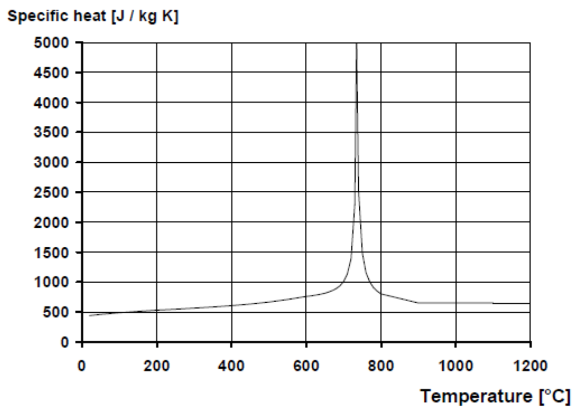
Specific heat



For $20°C \leq \theta_a < 600°C$

$$c_a = 425 + 7{,}73 \ x\ 10^{-1}\ \theta_a - 1{,}69\ x\ 10^{-3}\theta_a^2 + 2{,}22\ x\ 10^{-6}\theta_a^3 \quad J/kgK$$

For $600°C \leq \theta_a < 735°C$

$$c_a = 666 + \frac{13002}{738 - \theta_a} \quad J/kgK$$

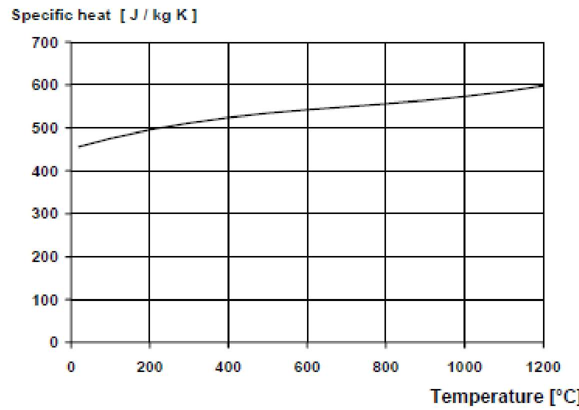
For $735°C \leq \theta_a < 900°C$

$$c_a = 545 + \frac{17820}{\theta_{a-731}} \quad J/kgK$$

For $900°C \leq \theta_a < 1200°C$

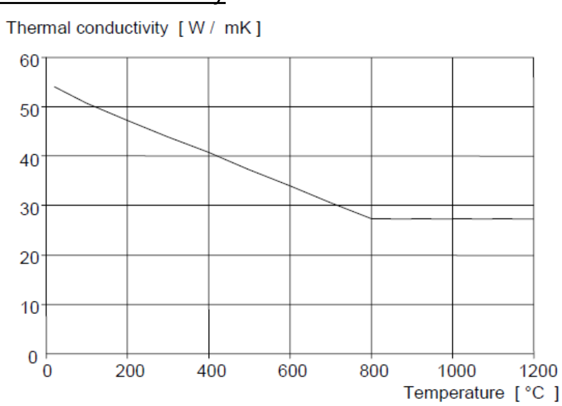
$$c_a = 650 \quad J/kgK$$

Specific heat



$$c_a = 450 + 0{,}280\ x\ \theta_a - 2{,}91\ x\ 10^{-4}\theta_a^2 + 1{,}34\ x\ 10^{-7}\theta_a^3 \quad J/kgK$$

Thermal conductivity



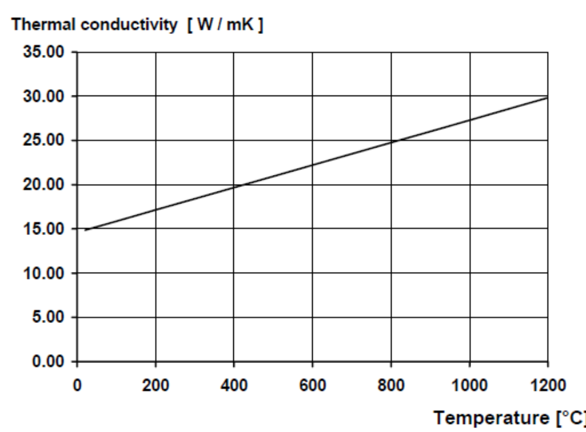
For $20°C \leq \theta_a < 800°C$

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

For $800°C \leq \theta_a < 1200°C$

$$\lambda_a = 27{,}3 \quad W/mK$$

Thermal conductivity



$$\lambda_a = 14{,}6 + 1{,}27 x\ 10^{-2}\ \theta_a \quad W/mK$$

In SCIA Engineer the properties for Carbon Steel are implemented.

Fire protection materials

The properties and performance of fire protection materials used in design should have been assessed using the test procedures given in ENV 13381-1, ENV 13381-2 or ENV 13381-4 as appropriate.

Structural fire design

General

EN 1993-1-2:2005 **article 4.1**

To determine the fire resistance the following design methods are permitted:

- Simplified calculation models
- Advanced calculation models
- Testing

⇨ The decision on use of advanced calculation models in a Country may be found in its National Annex.

National annex:

NEN: Advanced calculation models may be used in accordance with 4.3.
NBN: Het gebruik van geavanceerde berekeningsmodellen is aanvaard. Er dient verwezen te worden naar wettelijke voorschriften zoals vastgesteld door het Ministerie van Binnenlandse zaken.
NF: L'utilisation des modèles de calcul avancés est autorisée sous réserve que soient respectées les conditions rappelées dans la clause 9 de l'avant-propos.

⇨ The advanced calculation models may be used in Belgium, the Netherlands and France (under certain conditions).

Simple calculation models

General

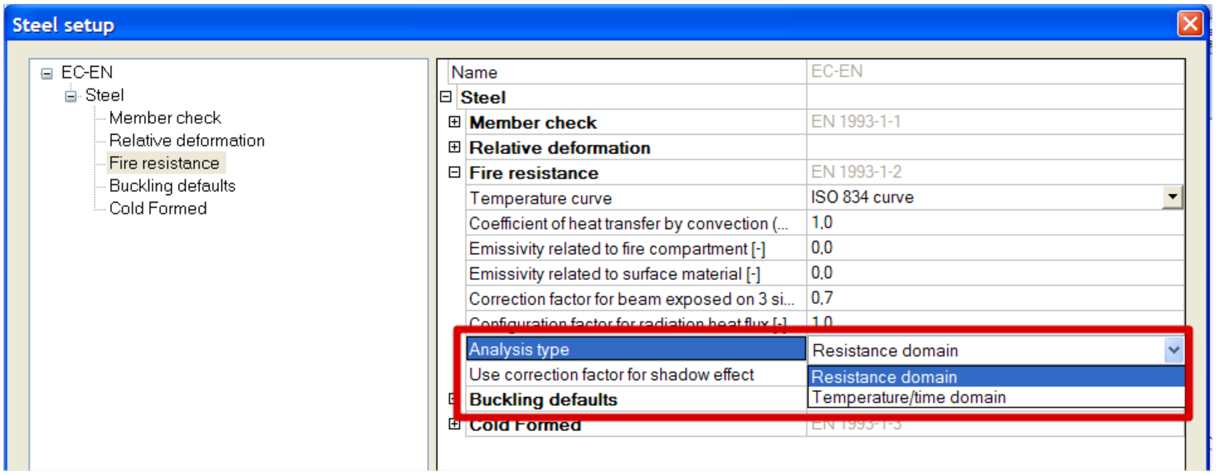
EN 1993-1-2:2005 **article 4.2.1**

Load-bearing function:

$E_{fi,d} \leq R_{fi,d,t}$ (design situation at time *t*)

Alternative to this function the verification may be carried out in the temperature domain (see further: EN 1993-1-2:2005 article 4.2.4.

The choice between those two methods can also be made in SCIA Engineer:



<u>Fire resistance of a bolted or welded joint</u>

This can be calculated with the conditions described in EN 1993-1-2:2005 article 4.2.1 (6)
Or alternative the method given in Annex D (EN 1993-1-2) can be used.

Classification

EN 1993-1-2:2005 **article 4.2.2**:

Change the value for ε into the following:

$$\varepsilon = 0{,}85\sqrt{235/f_y}$$

(= the original ε reduced with a factor 0,85 considering the influences due to increasing temperature)

Steel temperature development

EN 1993-1-2:2005 **article 4.2.5**:

<u>Unprotected internal steelwork</u>

Uniform temperature

$$\Delta\theta_{a,t} = k_{sh}\frac{A_m/V}{c_a\rho_a}\dot{h}_{net}\Delta t$$

Where:

k_{sh} Correction factor for shadow effect

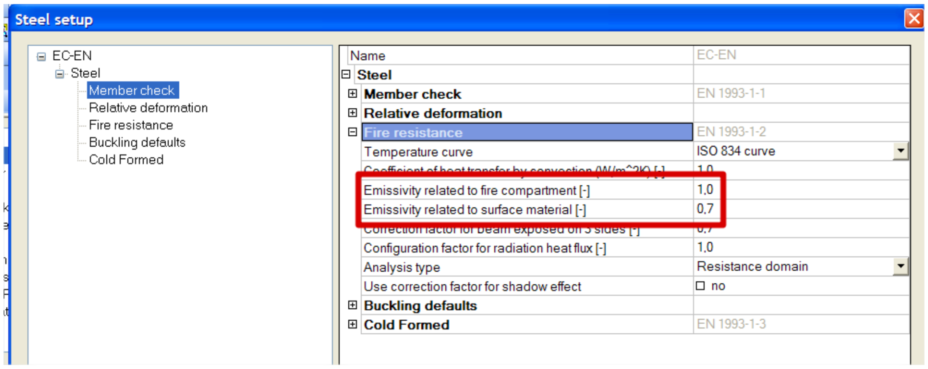
For I-sections under nominal fire actions: $k_{sh} = 0.9\,[A_m/V]_b/[A_m/V]$

All other cases: $k_{sh} = [A_m/V]_b/[A_m/V]$

A_m/V Section factor for unprotected steel members [1/m]
In table 4.2 (EN 1993-1-2) some section factors are calculated for unprotected steel members.

$[A_m/V]_b$ — the box value for the section factor

$\dot{h}_{net}$ — the design value on the net heat flux per unit area [W/m²]
This value should be obtained from EN 1991-1-2 with ε_f = 1,0 and ε_m = 0,7.



EN 1991-1-2 article 3.1:

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

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

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

With

α_c — Coefficient of heat transfer by convection [W/m²K]

Θ_g — The gas temperature in the vicinity of the fire exposed member [°C]
This temperature may be adopted as nominal temperature-time curves as given below

Θ_m — The surface temperature of the member [°C]

σ — Stephan Boltzmann constant (=$5{,}67 \cdot 10^{-8}$ W/m²K^4)

ε_m — Surface emissivity of fire = 0,7 (EN 1993-1-2)

ε_f — Emissivity of fire = 1

Φ — Configuration factor - Φ = 1,0. A lower value may be chosen to take account of so called position and shadow effects (calculation is given in EN 1991-1-2 – Annex G).

Θ_r — Is the effective radiation temperature of the fire environment [°C]
In case of fully fire engulfed members, the radiation temperature Θ_r may be represented by the gas temperature Θ_g around that member.

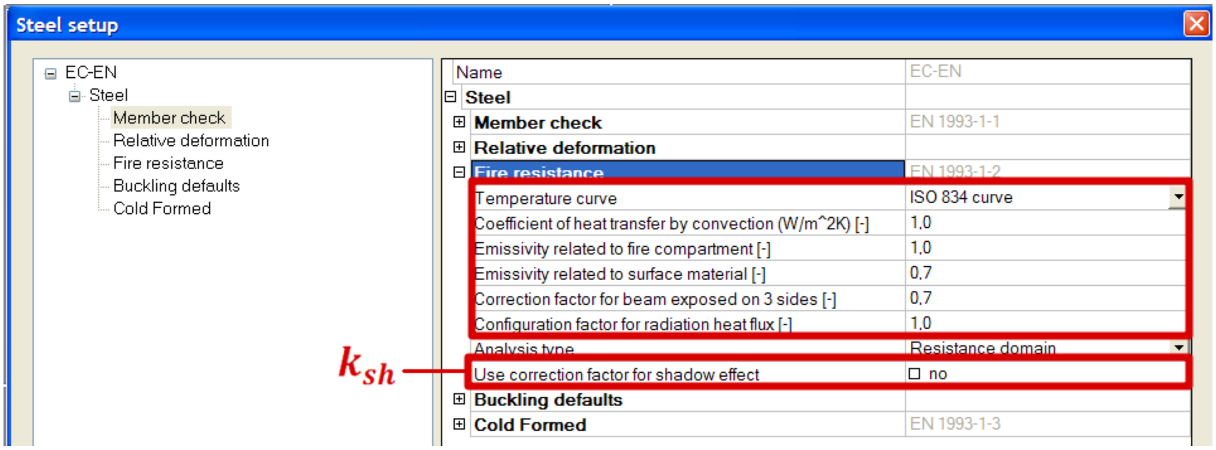
Nominal temperature-time curves
Following curves are given in the **EN 1991-1-2 article 3.2**: (t = time in minutes)

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

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

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

All those factors can be inputted in SCIA Engineer:



National annexes:

Remark: Following EN 1993-1-2 no National Annex can be applied on this article.

NBN: Table 4.5 (NA NBN) gives the steel temperature after 30 minutes in function of $k_{sh}\,A_m/V$

Internal steelwork insulated by fire protection

Uniform temperature:

$$\Delta\theta_{a,t} = \frac{\lambda_p A_p/V}{d_p c_a \rho_a} \cdot \frac{\theta_{g,t} - \theta_{a,t}}{(1 + \phi/3)} \Delta t - \left(e^{\frac{\phi}{10}} - 1\right) \Delta\theta_{g,t}$$

But $\Delta\theta_{a,t} \geq 0$ if $\Delta\theta_{g,t} > 0$

With:

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} \mathrm{d_p} \cdot A_p/V$$

Where:

A_p/V section factor – see also table 4.3. En 1993-1-2

Internal steelwork in a void that is protected by heat screens

Can be applied to both of the following cases:

- Steel members in a void that have a floor on top and by a horizontal heat screen below
- Steel members in a void that have vertical heat screens on both sides

Provided in both cases that there is a gap between the heat screen and the member.

$\Delta\theta_{a,t}$ should be based on the methods given above, taking the ambient gas temperature $\theta_{g,t}$ as equal to the gas temperature in the void.

Example: Fire Resistance.esa

Let's take a look at the fire resistance check in SCIA Engineer.

Look at beam B17 (Non linear combination NC1 Acc):

Following fire resistance properties are inputted:

Properties

Member (1)

Time resistance [sec]	1800,00
Buckling ratio ky	0
Buckling ratio kz	0
Fire exposure	3 sides
Covered flange	top flange
Protection	Yes
Insulation LIB	Fibre board
Thickness [mm]	5
k2	0,85
Temperature-Time curve	By setup
Member	B17

Fire resistance check:

First the safety factors and the material data are given:

EN 1993-1-1 Code Check
Fire resistance according to EN 1993-1-2

Member B17	**IPE750X137**	**S 355**	**NC1 Acc**	**0.64**

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

Material data		
yield strength fy	355.00	MPa
tension strength fu	510.00	MPa
fabrication	rolled	

Afterwards the fire resistance properties are displayed:

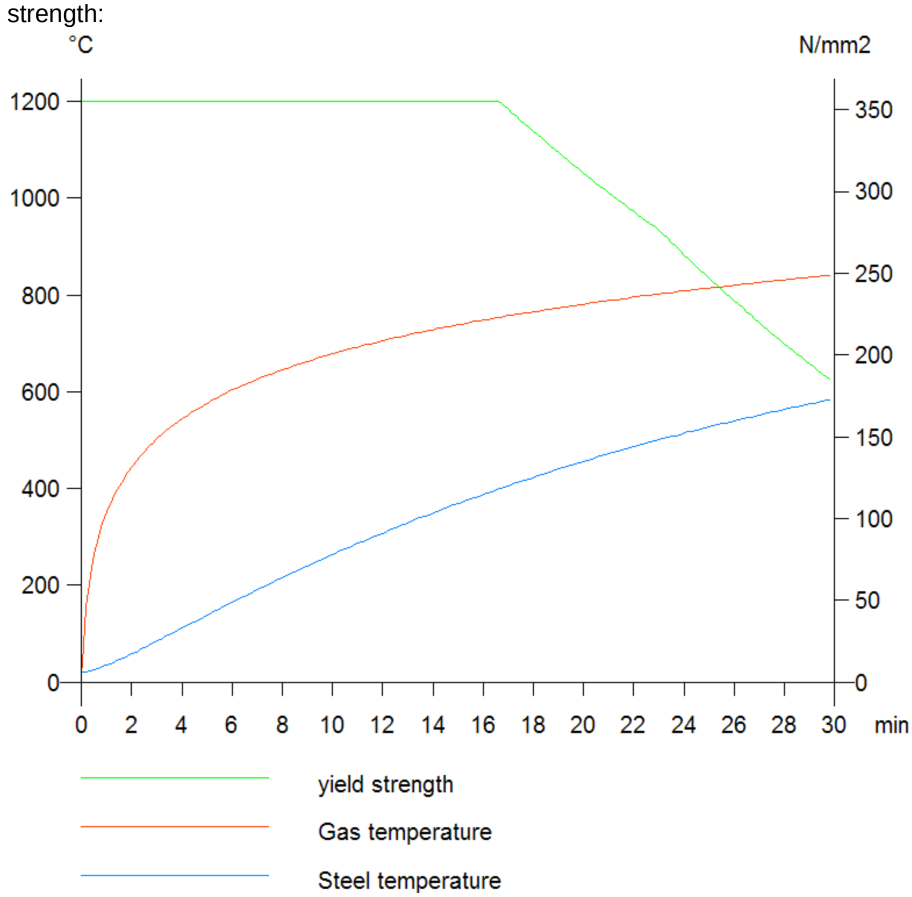
Fire resistance according to EN 1993-1-2 in resistance domain.
Results are given for checks at time t = 30.0 min

Fire resistance data		
Temperature-time curve	Standard temperature-time curve (ISO 834)	
Coefficient of heat transfer by convection Alfa,c	1.00	W/m2K
Emissitivity related to fire compartment Epsilon,f	0.70	
Emissitivity related to surface material Epsilon,m	1.00	
Configuration factor for radiation heat flux Fi	1.00	
Required fire resistance	30.00	min
Material temperature Teta a,t	585.67	°C
Gas temperature Teta,g	841.80	°C
Correction factor Kappa 1	0.70	
Correction factor Kappa 2	0.85	
Beam exposure	3 sides	
Covered flange	Upper flange	
ky,Teta	0.51	
kE,Teta	0.35	

And the insulation properties:

Insulation properties		
Name:	Fibre board	
Thickness	5.00	mm
Incasement type	Hollow encasement	
Insulation type	Board	
Unit mass	150.00	kg/m^3
Thermal conductivity	0.20	W/mK
Specific heat	1200.00	J/kgK
Ap/V	0.101	1/mm

And the calculated temperature (gas temperature and steel temperature) and the correspondent yield strength:



After those properties, the check will be given beginning with the classification:

..::SECTION CHECK::..

Width-to-thickness ratio for internal compression parts (EN 1993-1-1 : Tab.5.2. sheet 1).
ratio 59.57 on position 0.00 m

ratio		
maximum ratio	1	22.82
maximum ratio	2	26.28
maximum ratio	3	29.05

==> Class cross-section 4
Width-to-thickness ratio for outstand flanges (EN 1993-1-1 : Tab.5.2. sheet 2).
ratio 6.40 on position 0.00 m

ratio		
maximum ratio	1	6.22
maximum ratio	2	6.92
maximum ratio	3	9.68

==> Class cross-section 2

And afterwards the internal forces on the critical position are given:

The critical check is on position 6.90 m

Internal forces		
Nfi,Ed	-47.01	kN
Vy,fi,Ed	0.00	kN
Vz,fi,Ed	-31.04	kN
Mt,fi,Ed	-0.00	kNm
My,fi,Ed	-195.33	kNm
Mz,fi,Ed	0.01	kNm

Resistance

EN 1993-1-2:2005 **article 4.2.3**

Tension members:

Uniform temperature:

$$N_{fi,\theta,Rd} = k_{y,\theta} N_{Rd} \; [\gamma_{M,0}/\gamma_{M,fi}]$$

($k_{y,\theta}$is the reduction factor on the yield strength.

⇨ Formula EN 1993-1-2 (4.4) will give the formula for a tension member with a non-uniform temperature

Example: Fire Resistance.esa

Look at beam B196 (Non linear combination NC1 Acc):

The critical check is on position 3.00 m

Internal forces		
Nfi,Ed	7.14	kN
Vy,fi,Ed	0.00	kN
Vz,fi,Ed	0.00	kN
Mt,fi,Ed	0.00	kNm
My,fi,Ed	0.61	kNm
Mz,fi,Ed	-0.00	kNm

Normal force check
according to article EN 1993-1-2 : 4.2.3.1 and formula EN 1993-1-2 : (4.3)

Table of values		
Nfi,t,Rd	65.51	kN
unity check	0.11	

Compression members (Class1, Class2 and Class3 cross-sections)

Buckling resistance:

$$N_{b,fi,t,Rd} = \chi_{fi} A\, k_{y,\theta} f_y / \gamma_{M,fi}$$

With:

$$\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^2 - \bar{\lambda}_\theta^2}}$$ but $\chi \leq 1{,}0$

where

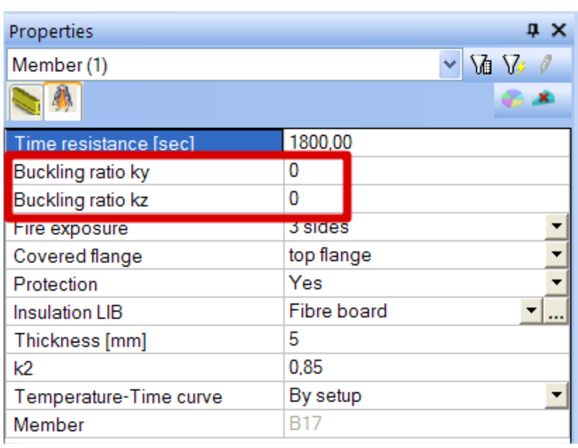
$$\varphi_\theta = 0{,}5\left[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}_\theta^2\right]$$

$$\alpha = 0{,}65\sqrt{235/f_y}$$

$$\bar{\lambda}_\theta = \bar{\lambda}\sqrt{k_{y,\theta}/k_{E,\theta}}$$

The buckling length l_{fi} of a column should generally be determined as for normal temperature design.

In article 4.2.3.2(3) and article 4.2.3.2(4) of EN 1993-1-2 is described that the buckling lengths l_{fi} may be changed. This can also be inputted in SCIA Engineer:



Properties

Member (1)

Time resistance [sec]	1800,00
Buckling ratio ky	0
Buckling ratio kz	0
Fire exposure	3 sides
Covered flange	top flange
Protection	Yes
Insulation LIB	Fibre board
Thickness [mm]	5
k2	0,85
Temperature-Time curve	By setup
Member	B17

Example: Fire Resistance.esa

Look at beam B17 (Non linear combination NC1 Acc):

Compression check
according to article EN 1993-1-2 : 4.2.3.2 and formula EN 1993-1-2 : (4.5)
Section classification is 3.

Table of values		
Nfi,t,Rd	3188.54	kN
unity check	0.01	

Beams with Class 1 or Class 2 cross-sections

Design moment resistance $M_{fi,\theta,Rd}$ with a uniform temperature θ_a should be determined from:

$$M_{fi,\theta,Rd} = k_{y,\theta}\,[\frac{\gamma_{M,0}}{\gamma_{M,fi}}]M_{Rd}$$

In formula (4.9) of EN1993-1-2 the design moment resistance will be given for a non-uniform temperature distribution across the cross-section.

An alternative for a non-uniform temperature distribution is given in formula (4.10) of EN 1993-1-2:

$$M_{fi,t,Rd} = M_{fi,\theta,Rd}/\kappa_1\kappa_2$$

With:

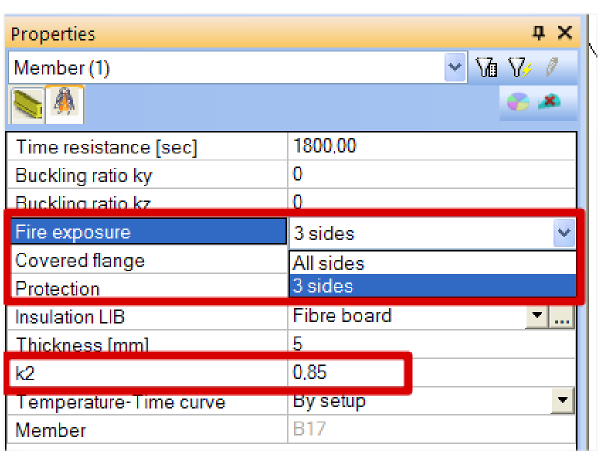
- κ_1 Is an adaptation factor for non-uniform temperature across the cross section
- κ_2 Is an adaptation factor for non-uniform temperature along the beam

Those factors should be taken as follows:

κ_1 For a beam exposed on all four sides $\kappa_1 = 1{,}0$
For a beam exposed on 3 sides, with a composite or concrete slab on side 4: $\kappa_1 = 0{,}70$
For a protected beam exposed on 3 sides, with a composite or concrete slab on side 4: $\kappa_1 = 0{,}85$

κ_2 At the supports of a statically indeterminate beam $\kappa_2 = 0{,}85$
In all other cases: $\kappa_2 = 1{,}00$

These values can also be inputted in SCIA Engineer:



Design lateral torsional resistance moment $M_{b,fi,t,Rd}$ at time *t* of a laterally unrestrained member should be determined from:

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{pl,y} k_{y,\theta,com}\, f_y/\gamma_{M,fi}$$

With:

$\chi_{LT,fi}$	Is the reduction factor for LTB in the fire design situation
$k_{y,\theta,com}$	Is the reduction factor from section 3 for the yield strength of steel at the maximum temperature in the compression flange.

⇨ The calculation formulas for $\chi_{LT,fi}$ can be found in article 4.2.3.3(5) from EN 1993-1-2.

National Annex:

Remark: Following EN 1993-1-2 no National Annex can be applied on this article.

NBN $\chi_{LT,fi,mod}$ mag aangepast worden als volgt: $\chi_{LT,fi,mod} = \frac{\chi_{LT,fi}}{f}$

Waarbij f = 1 – 0,5 (1 – k_c)
Met k_c :

Tabel 4.4 ANB: Correctiefactoren k_c

Momentenverdeling	k_c
M_1  ψM_1 $-1 \le \psi \le 1$	$0.6 + 0.3\psi + 0.15\psi^2$ maar $k_c \le 1$
	0.79
	0.91

Opmerking: voor andere diagrammen van buigende momenten $k_c = 1$.

En kip moet enkel getoest worden indien $\lambda_{LT,fi,mod}$ groter is dan de waarden opgesomd in tabel 4.5 van ANB (see national annex).

Design shear resistance $V_{fi,t,Rd}$ at time *t* should be determined from:

$$V_{fi,t,Rd} = k_{y,\theta,web}\, V_{Rd} \frac{\gamma_{M,0}}{\gamma_{M,fi}}$$

With:

$k_{y,\theta,web}$	Is the reduction factor for the yield strength of steel at the average temperature in the web of the section.

<u>Beams with Class 3 cross-sections</u>

Design moment resistance $M_{fi,\theta,Rd}$ with a uniform temperature should be determined from:

$$M_{fi,t,Rd} = k_{y,\theta} \left[\frac{\gamma_{M,0}}{\gamma_{M,fi}}\right] M_{Rd}$$

National Annex:
NBN: Voor de effecten van dwarskracht is het nodig om ρ te berekenen zoals gegeven 6.2.8 van EN 1993-1-1 op basis van de verhouding van de dwarskrachtn $V_{fi,Ed}$ en $V_{fi,Rd}$ voor de ontwerptoestand van brand.

And the design moment resistance for a non-uniform temperature distribution :

$$M_{fi,t,Rd} = k_{y,\theta} \left[\frac{\gamma_{M,0}}{\gamma_{M,fi}}\right] M_{Rd}/\kappa_1\kappa_2$$

With:

κ_1 See previous paragraph

κ_2 See previous paragraph

National Annex:
NBN: Voor de effecten van dwarskracht is het nodig om ρ te berekenen zoals gegeven 6.2.8 van EN 1993-1-1 op basis van de verhouding van de dwarskrachtn $V_{fi,Ed}$ en $V_{fi,Rd}$ voor de ontwerptoestand van brand.

Design lateral torsional resistance moment $M_{b,fi,t,Rd}$ at time *t* of a laterally unrestrained member should be determined from:

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{el,y} k_{y,\theta,com}\, f_y/\gamma_{M,fi}$$

Explanation of factors: see previous paragraph.

National Annex:

NBN : Zie ook Liggers met doorsneden in klasse 1 en 2, en de aanpassing voor $\chi_{LT,fi}$.

Design shear resistance $V_{fi,t,Rd}$ at time *t* should be determined from:

$$V_{fi,t,Rd} = k_{y,\theta,web}\, V_{Rd}\, \frac{\gamma_{M,0}}{\gamma_{M,fi}}$$

Members with Class 1, 2 or 3 cross-sections, subject to combined bending and axial compression

The design buckling resistance $R_{fi,t,d}$ at time *t* of a member subject to combined bending and axial compression should be verified by:

Class 1 or Class 2 cross-section:

$$\frac{N_{fi,Ed}}{\chi_{min,fi}\, A\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} + \frac{k_y M_{y,fi,Ed}}{W_{pl,y}\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} + \frac{k_z M_{z,fi,Ed}}{W_{pl,z}\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} \leq 1$$

$$\frac{N_{fi,Ed}}{\chi_{z,fi}\, A\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} + \frac{k_{LT} M_{y,fi,Ed}}{W_{pl,y}\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} + \frac{k_z M_{z,fi,Ed}}{W_{pl,z}\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} \leq 1$$

Class 3 cross section:

$$\frac{N_{fi,Ed}}{\chi_{min,fi}\, A\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} + \frac{k_y M_{y,fi,Ed}}{W_{el,y}\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} + \frac{k_z M_{z,fi,Ed}}{W_{el,z}\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} \leq 1$$

$$\frac{N_{fi,Ed}}{\chi_{z,fi}\, A\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} + \frac{k_{LT} M_{y,fi,Ed}}{W_{el,y}\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} + \frac{k_z M_{z,fi,Ed}}{W_{el,z}\, k_{y,\theta}\, \frac{f_y}{\gamma_{M,fi}}} \leq 1$$

The calculation of the factors can be found in article 4.2.3.5 (1) of EN 1993-1-2.

National Annex:

Remark: Following EN 1993-1-2 no National Annex can be applied on this article.

NF Les expression μ_y et μ_z pour les relations (4.21a), (4.21b), (4.21c) et (4.21d) doivent être remplacées par :

$$\mu_y = (2\,\beta_{M,y} - 5)\bar{\lambda}_{y,\theta} + 0{,}44\,\beta_{M,y} + 0{,}29 \leq 0{,}8 \quad \text{avec } \lambda_y \leq 1{,}1$$

$$\mu_z = (1{,}2\,\beta_{M,y} - 3)\bar{\lambda}_{z,\theta} + 0{,}71\,\beta_{M,y} - 0{,}29 \leq 0{,}8$$

NBN : Zie ook Liggers met doorsneden in klasse 1 en 2, en de aanpassing voor $\chi_{LT,fi}$.

Members with Class 4 cross-sections

For members with class 4-cross sections other than tension members it may be assumed that 4.2.1(1) [$E_{fi,d} \leq R_{fi,d,t}$] is satisfied if at time *t* the steel temperature θ_a at all cross-sections is not more than θ_{crit}.

The limit θ_{crit} may be chosen in the National Annex. The value θ_{crit} = 350°C is recommended.

National Annex

NEN: θ_{crit} = 350°C

NBN: De aanbevolen waarde θ_{crit} = 350°C is normatief

NF: La valeur à utiliser est la valeur recommandée. => θ_{crit} = 350°C

EN 1993-1-2: ANNEX E:

Advanced calculation models

May be used when all stability effects are taken into account.

Simple calculation models

The resistance of members with a class 4 cross section should be verified with the equations given in EN 1993-1-2 articles 4.2.3.2, 4.2.3.4 and 4.2.3.5 in which the area is replaced by the effective area and the section modulus is replaced by the effective section modulus.

The effective cross section => EN 1993-1-3 and EN 1993-1-5

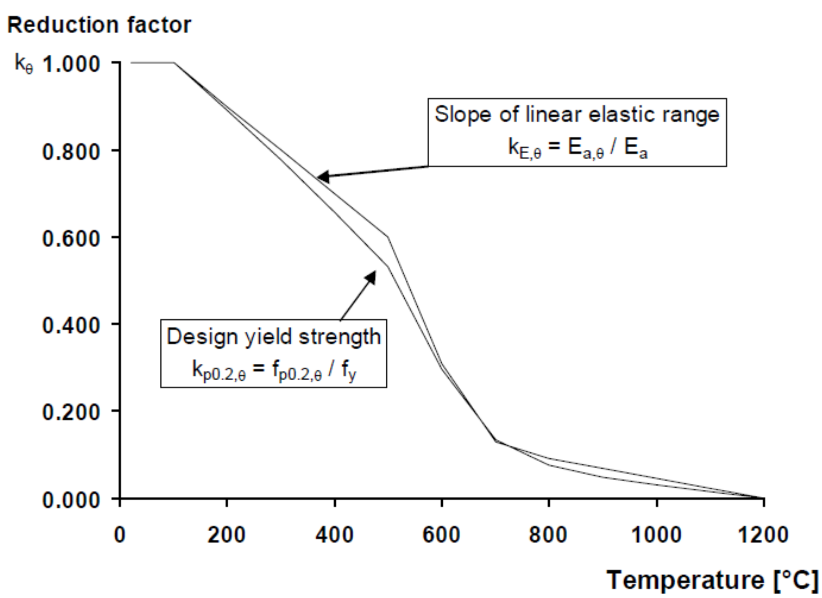
Reduction factors for stainless steel => EN 1993-1-2 – Annex C

Reduction factors for carbon steel:

Steel Temperature θ_a	Reduction factor (relative to f_y) for the design yield strength of hot rolled and welded class 4 sections $k_{p0,2,\theta} = f_{p0,2,\theta}/f_y$	Reduction factor (relative to f_{yb}) for the design yield strength of cold formed class 4 sections $k_{p0,2,\theta} = f_{p0,2,\theta}/f_{yb}$
20°C	1,00	
100°C	1,00	
200°C	0,89	
300°C	0,78	
400°C	0,65	
500°C	0,53	
600°C	0,30	
700°C	0,13	
800°C	0,07	
900°C	0,05	
1000°C	0,03	
1100°C	0,02	
1200°C	0,00	

NOTE 1: For intermediate values of the steel temperature, linear interpolation may be used.

NOTE 2: The definition for f_{yb} should be taken from EN 1993-1-3



Critical temperature

EN 1993-1-2:2005 **article 4.2.4**:

This is an alternative method to the resistance method (from article 4.2.3 EN 1993-1-2).

Except when considering deformation criteria or when stability phenomena have to be taken into account, the critical temperature $\theta_{a,cr}$ of carbon steel at time *t* for a uniform temperature distribution in a member may be determined for any degree of utilization μ_0 at time *t=0* using:

$$\theta_{a,cr} = 39{,}19\ ln\left[\frac{1}{0{,}9674\ \mu_0^{3{,}833}} - 1\right] + 482$$

$\mu_O = E_{fi,d}/R_{fi,d,0}$ And $\mu_O \geq 0{,}013$

National annexes:

NBN: 2 remarks:

- stability phenomena = instability phenomena
- Tables for the critical temperature Table 4.1 in function of $\lambda_{fi,0}$ and μ_{pl}

With $\mu_{pl} = \frac{N_{fi,Ed}}{A_a \cdot f_y}$

NEN: No default values for critical temperatures are given.

NF: Les valeurs par défaut de température critique $\theta_{a,cr}$ pour les éléments en acier de classe 1 à 3 dans les bâtiments courants de catégories A à D, peuvent être prises égales à :

- 540°C pour des poutres isostatiques ou des éléments tendus
- 570°C pour des poutres hyperstatiques
- 500°C pour des éléments comprimés ou des éléments comprimés et fléchis

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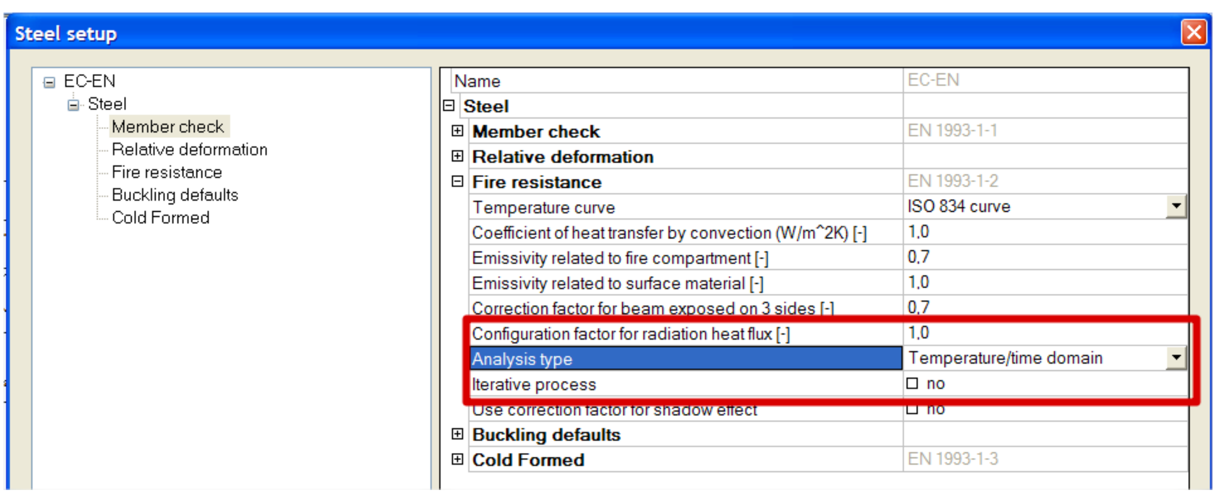
Example: Fire Resistance.esa

In the steel setup can be chosen for the method of the critical temperature:



For this method, the user can choose between the formula for $\theta_{a,cr}$ described in the Eurocode (an estimation formula for the critical temperature) or can choose for an iterative process. The last method will increase the temperature and calculate the resistance (unity check) for the member until this unity check will become 1. On this moment, the critical temperature is known.

Calculation for Beam B17 with the Temperature/Time domain (no iteration):

Fire resistance according to EN 1993-1-2 in time/temperature domain.
Results are given for checks at time t = 0 min.

Fire resistance data		
Temperature-time curve	Standard temperature-time curve (ISO 834)	
Coefficient of heat transfer by convection Alfa,c	1.00	W/m2K
Emissivity related to fire compartment Epsilon,f	0.70	
Emissivity related to surface material Epsilon,m	1.00	
Configuration factor for radiation heat flux Fi	1.00	
Required fire resistance	30.00	min
Material temperature Teta a,t	585.67	°C
Gas temperature Teta,g	841.80	°C
Critical temperatur Teta a,cr	678.25	°C
Fire resistance	40.40	min
Correction factor Kappa 1	0.70	
Correction factor Kappa 2	0.85	
Beam exposure	3 sides	
Covered flange	Upper flange	
Degree of utilization Mu0	0.27	
ky,Teta	1.00	
kE,Teta	1.00	

The unity check = $\frac{585{,}67°C}{678{,}25°C} = 0{,}86$

Calculation for Beam B17 with the Temperature/Time domain (with iterative process):

Fire resistance according to EN 1993-1-2 in time/temperature domain.
Results are given for checks at critical material temperature Teta a,cr = 654.9 °C

Fire resistance data		
Temperature-time curve	Standard temperature-time curve (ISO 834)	
Coefficient of heat transfer by convection Alfa,c	1.00	W/m2K
Emissivity related to fire compartment Epsilon,f	0.70	
Emissivity related to surface material Epsilon,m	1.00	
Configuration factor for radiation heat flux Fi	1.00	
Required fire resistance	30.00	min
Material temperature Teta a,t	585.67	°C
Gas temperature Teta,g	841.80	°C
Critical temperatur Teta a,cr	654.94	°C
Fire resistance	37.40	min
Correction factor Kappa 1	0.70	
Correction factor Kappa 2	0.85	
Beam exposure	3 sides	
Covered flange	Upper flange	
ky,Teta	0.34	
kE,Teta	0.21	

The unity check = $\frac{585{,}67°C}{654{,}94°C} = 0{,}89$

Advanced calculation models

EN 1993-1-2:2005 **article 4.3**

Advanced calculation models should include separate calculation models of the determination of:

- The development and distribution of the temperature within structural members (thermal response model)
- The mechanical behavior of the structure or of any part of it (mechanical response model)

References

[1] EN 1993-1-1: 2005, Design of steel structures – Part 1-1: General rules and rules for buildings

[2] NEN-EN 1993-1-1/NA, National Annex to NEN-EN 1993-1-1 Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings

[3] NBN EN 1993-1-1 ANB – Eurocode 3: Ontwerp en berekening van staalconstructies – Deel 1-1: Algemene regels en regels voor gebouwen – Nationale bijlage

[4] NF EN 1993-1-1/NA – Calcul des structures en acier – Partie 1-1: Règles générales et règles pour les bâtiments

[5] EN 1993-1-2: 2005, Design of steel structures – Part 1-2: General rules – Structural fire design

[6] EN 1991-1-2: 2005, Actions on structures - Part 1-2: General actions - Actions on structures exposed to fire

[7] NBN EN 1993-1-2 ANB – Eurocode 3 – Ontwerp en berekening van staalconstructies – Deel 1-2: Algemene regels – Ontwerp en berekening van constructies bij brand – Nationale bijlage

[8] NF EN 1993-1-2/NA – Calcul des structures en acier – Partie 1-2: Règles générales – Calcul du comportement au feu

[9] NEN-EN 1993-1-2/NB, National Annex to NEN-EN 1993-1-2 Eurocode 3 : Design of steel structures – Part 1-2 : General rules – Structural fire design

[10] SCIA Engineer Steel Code Check
Theoretical Background
04/2011