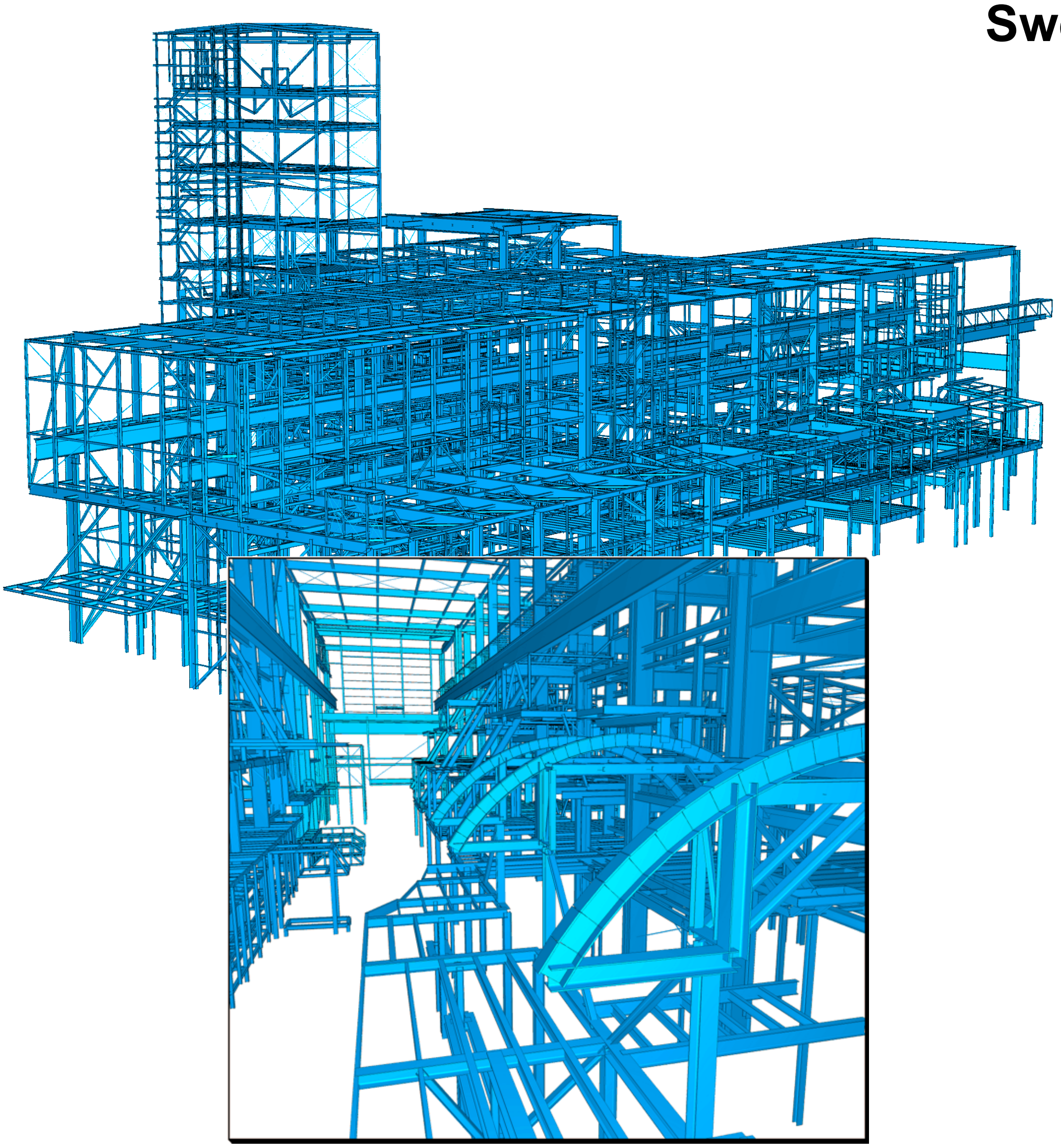
Steel Checks

according to EN 1993-1-1 with National Annexes Austria Germany Great Britain Sweden



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EN 1993-1-1 Steel Checks

Basics

The steel checks according to EN 1993-1-1 (Eurocode 3) can be used for buildings and engineering constructions under observance of the following standards:

- EN 1993-1-1:2005/A1:2014 as the base document
- DIN EN 1993-1-1:2014 with the National Annex Germany 2022-10
- OENORM EN 1993-1-1:2014 with the National Annex Austria B 1993-1-1:2017-11
- SS EN 1993-1-1:2014 with the National Annex Sweden 2019-01 (EKS 11)
- BS EN 1993-1-1:2005 with the National Annex Great Britain 2008-12

The desired rule is selected in the *Settings* dialog in the *Analysis* menu. When selecting the material the following alternatives are available:

- *S235-EN* to *S500-EN* for construction steel as per EN 1993-1-1, Table 3.1 or EN 10025-2
- *Stahl* for a free definition of the material characteristics

The design is carried out after the static calculation. To do so, you need to assign the calculated load cases to the actions in accordance with EN 1991-1-1:2002/AC:2009 (Eurocode 1). The program will take into account the preset safety factors and combination coefficients defined in EN 1990:2021 (Eurocode 0) for the desired design situations to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements. The stress checks are performed for all combinations in the ultimate limit state including the *special combination*. For every set of internal forces of these design combinations the cross-section class according to EN 1993-1-1, Chapter 5.5, is automatically determined and optionally the elastic or plastic cross-section resistance in accordance with Chapter 6.2 is checked. The shear buckling check according to EN 1993-1-5 is not carried out.

The predefined steel sections, project-specific parameter and polygon sections or sections from the user database can be used as cross-sections. Beams with the *Beam* section type are not checked as the section geometry is not known for them.

For area sections, the extremal internal forces for the defined action combinations are determined without performing a check and saved in the database for graphical representation.

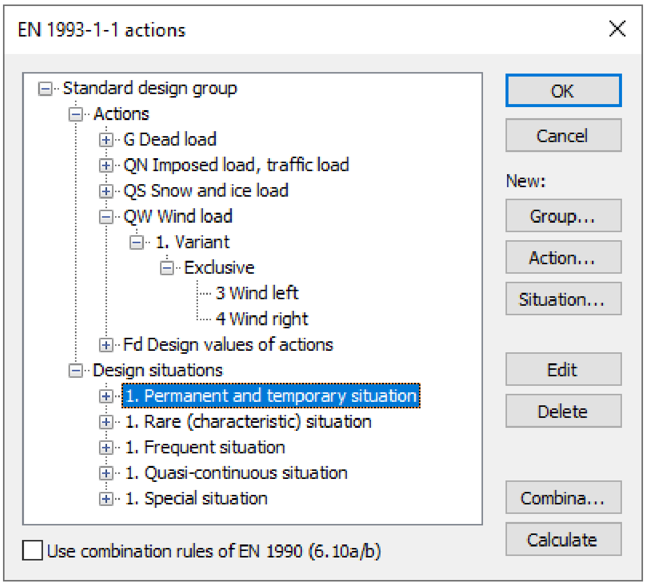
The buckling and lateral torsional buckling check is carried out on the fork supported equivalent beam with an independent program, which can be opened from the *Infograph* folder in the start menu.

The EN 1993-1-1 guidelines are primarily cited for the following descriptions. Reference to the relevant national settings is only made if they contain different or complementary rules. The passages in question are marked by a vertical line left of the text.

Input for Checks on the Entire Structure

Actions and Design Situations

The load design values are calculated based on the internal forces of individual load cases and load case combinations. To do so, the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the desired design situations. The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



Action...

Open the dialog for entering new actions:

- Permanent actions (G, GE, GH)
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Actions due to earthquakes (AE)
- Design values of actions (Fd)
  The assigned load cases should contain a design-relevant set of loads with combination coefficients and partial safety factors for actions and material such as for example a load group for the stability check on the entire structure according to EN 1993-1-1, Clause 5.2.2 (3). The selected load cases are combined exclusively.

Group...

Open the dialog for entering a new design group. According to e.g. standard EN 1991-1-1, Chapter 6.2.2 (2), certain components (sections) may be designed with reduced imposed loads. Therefore, variable actions (Q) and design situations can be changed here.

Situation...

Open the dialog for entering new design situations.

Edit

Open the Edit dialog for the selected action or situation.

Delete

Delete the selected action or situation.

Combinations...

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis or nonlinear analysis.

The following example shows the total variants of the *permanent and temporary situation* according to Eq. (6.10) to be examined with the load cases (L1...L6) involved and their weighting factors.

Actions	Load cases	γ_{sup}	γ_{inf}	ψ_0
Dead load	1	1.35	1.0	-
Imposed load, traffic load	2, 3 (inclusive)	1.5	0	0.7
Wind load	4	1.5	0	0.6
F_d Design values of actions	5, 6	1.0	1.0	-

1. Permanent and temporary situation

Combinations (24 of 24):

No.	L1	L2	L3	L4	L5	L6
1	1.35					
2	1.35	1.50	1.50	0.90		
3	1.35	1.50	1.50			
4	1.35	1.50		0.90		
5	1.35	1.50				
6	1.35		1.50	0.90		
7	1.35		1.50			
8	1.35	1.05	1.05	1.50		
9	1.35	1.05		1.50		
10	1.35		1.05	1.50		
11	1.35			1.50		
12	1.00					

OK Cancel Load group...

1. Permanent and temporary situation

Combinations (24 of 24):

No.	L1	L2	L3	L4	L5	L6
13	1.00	1.50	1.50	0.90		
14	1.00	1.50	1.50			
15	1.00	1.50		0.90		
16	1.00	1.50				
17	1.00		1.50	0.90		
18	1.00		1.50			
19	1.00	1.05	1.05	1.50		
20	1.00	1.05		1.50		
21	1.00		1.05	1.50		
22	1.00			1.50		
23					1.00	
24						1.00

OK Cancel Load group...

Calculate

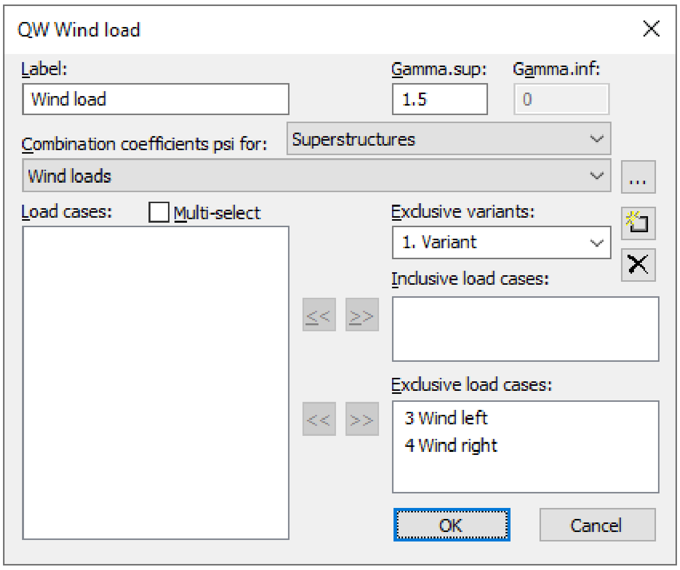
Calculate the defined design situations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all situations in the database. This allows you to evaluate the results without having to open the checking module. Each time you open the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

Use combination rules of EN 1990 (6.10a/b)

Optionally the Eq. (6.10a/b) are used for the combination of the permanent and temporary situation, otherwise Eq. (6.10).

Definition of an Action

The illustration below shows an example of the dialog field for entering a variable action. The dialog fields for other action types are of a similar appearance.



Label

User-defined label for the action.

Gamma.sup, **Gamma.inf**

Partial safety factors γ_{sup} and γ_{inf}.

SS EN 1990:

The program suggests the partial safety factors as they result in accordance with Section A, Article 11, for safety class 3 from $\gamma_d \cdot \gamma_{sup}$ with the reduction factor $\gamma_d = 1.0$ as per Article 14. If required, lower safety classes can be taken into account entering lower values.

Combination coefficients psi for:

Input fields for selecting the combination coefficients for variable actions. The ... button allows you to view and change the selected combination coefficients ψ_0, ψ_1 and ψ_2.

Load cases

List of possible load cases or load case combinations. You can choose an item from the list by selecting it and clicking the >> button or by using drag & drop.

Multi-select

Load cases and combinations can be added to the actions more than once.

Exclusive variants

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants with the or buttons.

Inclusive load cases

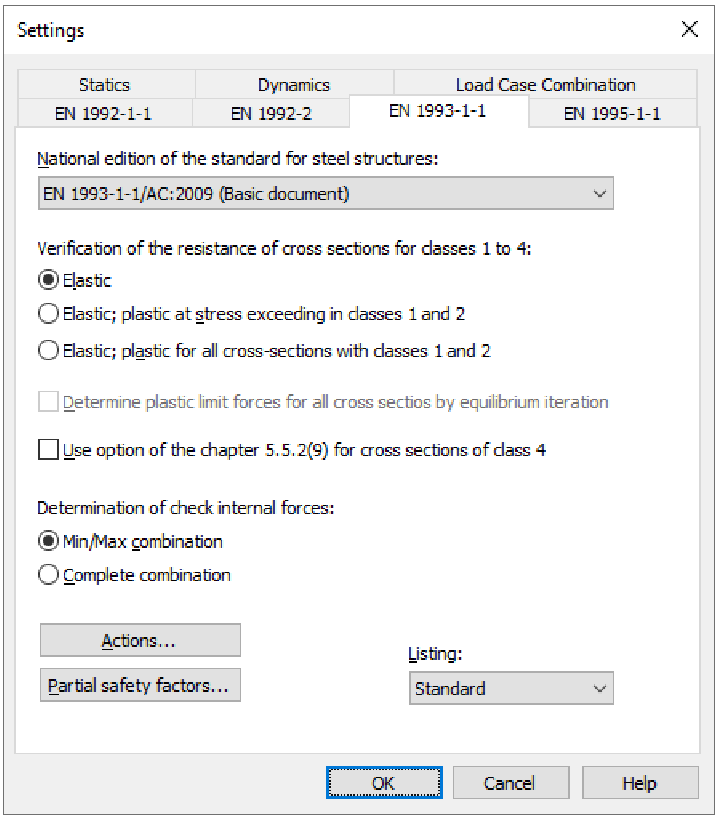
Selected load cases and combinations that can act simultaneously.

Exclusive load cases

Selected load cases and combinations that exclude each other.

Analysis Settings

The *EN 1993-1-1* dialog field can be accessed using the *Settings* option of the *Analysis* menu.



National edition of the standard

The edition you select will be used for all subsequent entries and calculations.

The following checks can be chosen:

Elastic

In accordance with Chapter 6.2.1 of the standard, the elastic cross-section resistance is verified for classes 1 to 4. For class 4 the check is carried out with effective cross-section properties as per 1993-1-5, Clause 4.3.

Elastic; plastic at stress exceeding in classes 1 and 2

In accordance with Chapter 6.2.1 of the standard, the elastic cross-section resistance is verified for classes 1 to 4. If the comparison stress in classes 1 and 2 exceeds the permissible limit, the plastic cross-section resistance will be verified.

Elastic; plastic for all cross-sections with classes 1 and 2

Generally the plastic cross-section resistance will be verified for class 1 and 2, also if the comparison stress does not exceed the permissible limit.

Determine plastic limit forces for all cross-sections by equilibrium iteration

When selected, for verifying the cross-section resistance, the plastic limit forces are determined by stress integration and equilibrium iteration. Otherwise, for double symmetrical I and H profiles with constant flange thickness as well as for rectangular box sections, the check is performed using the interaction equations according to EN 1993-1-1, Chapter 6.2.3 to 6.2.10. Equilibrium iteration is always used for all other cross sections.

Use option of the chapter 5.5.2(9) for cross-sections of class 4

Cross-sections of class 4 will be treated like cross-sections of class 3, if the c/t ratio does not exceed the limits of class 3 increased by a factor according to Chapter 5.5.2 (9).

Determination of the check internal forces

- *Min/Max combination*
  The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*
  To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The calculation effort increases exponentially with the number of inclusive load cases.

The differences between the two methods are explained in more detail in the section *Check internal forces*.

Actions...

Open the dialog for describing actions.

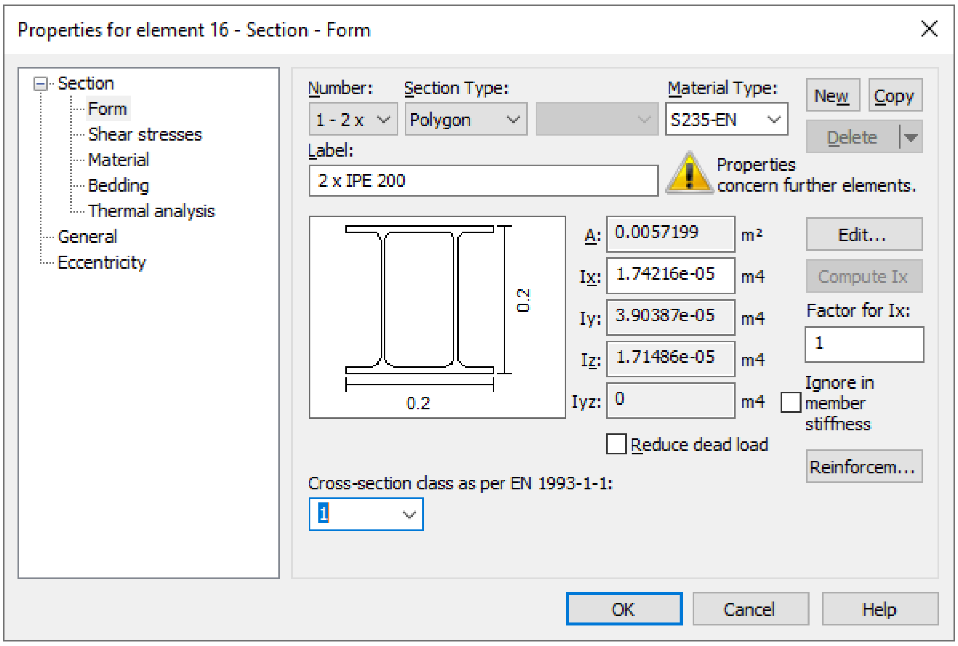
Partial safety factors...

Open the dialog for modifying the partial safety factor γ_{M0}. He is preset to 1.0 according to Chapter 6.1 (1).

Listing

- *No*: No log is generated by the design program.
- *Standard*: During analysis a log with a tabular output of the calculated utilizations is created.
- *Detailed*: This log differs from the standard log by additionally offering information on the analyzed internal force combinations.
- *Standard>permissible*: Standard log of the check locations at which the permissible limits are exceeded.
- *Detailed>permissible*: Detailed log of the check locations at which the permissible limits are exceeded.

Section Inputs

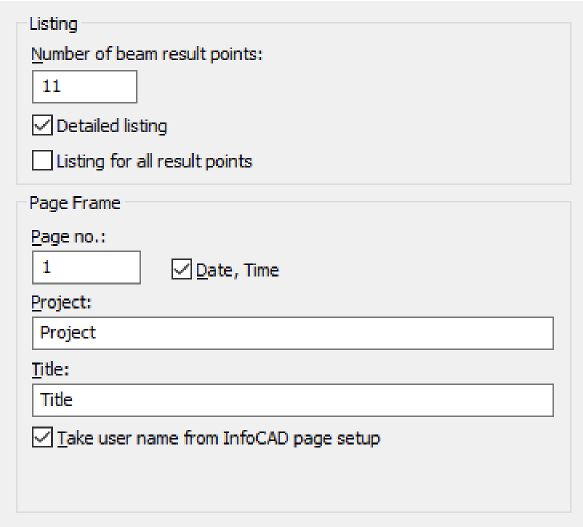


Cross-section class

For polygonal sections and sections from the user database the classification into the cross-section class according to EN 1993-1-1, Table 5.2, is carried out by the user. The check of the plastic cross-section resistance is always performed by equilibrium iteration, regardless of the calculation settings.

Input for Checks on the Equivalent Beam

Settings



Number of beam result points

The internal forces are calculated at equidistant points in the beam. For each result location the checks are performed with the corresponding internal forces and the corresponding M_{cr}.

Detailed listing

A detailed log with all necessary values is generated during calculation.

Listing for all result points

In addition to the determinant result location, the results for all result locations are documented.

Page no.

Page number used for the 1st side to be printed. The number is increased incrementally for each printing operation. You can enter '0' to suppress the numbering.

Date, time

This information appears automatically in the standard page frame.

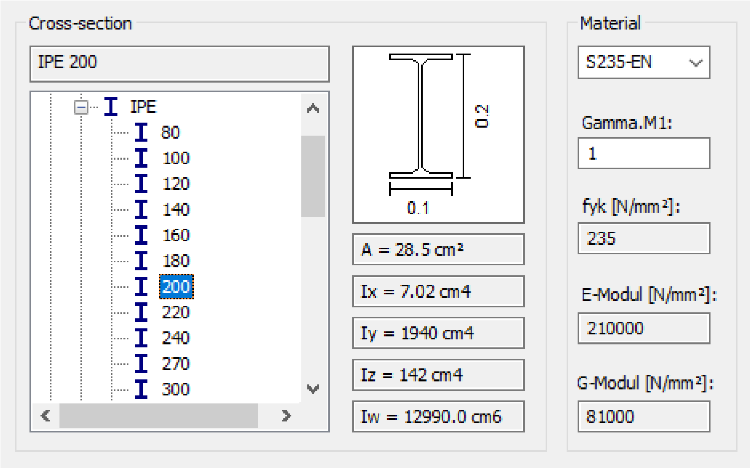
Project, title

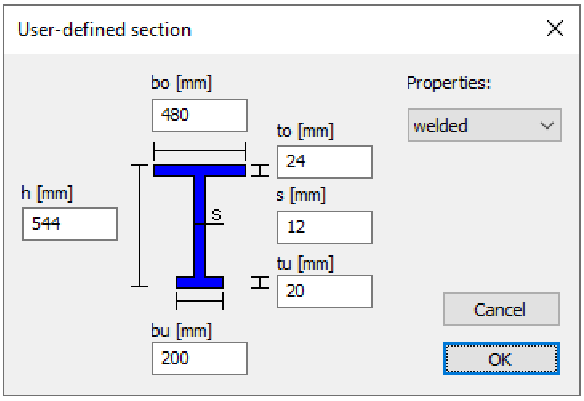
This text appears automatically in the standard page frame.

Take user name from InfoCAD page setup

If the mark is set, the user name is automatically retrieved from the *Igraph.dat* file and included in the standard page frame. By removing the mark a different user name can be defined.

Section & Material





Section

Selection of a steel section from the section library or input of a user-defined profile by specification of given dimensions.

User-defined profile

For the defined I profile the property *welded* or *rolled* for the assignment of buckling curves according to EN 1993-1-1, Tables 6.2 and 6.5 can be specified.

Material

- *S235-EN* to *S500-EN*: Construction steel according to EN 1993-1-1, Tab. 3.1 or EN 10025-2.
- *Steel*: User-defined steel.

Gamma.M1

The material-specific coefficient γ_{M1} is used to obtain the design value for the strength $f_{y,d}$.

fyk

Characteristic yield strength [N/mm²] of construction steel *S235-EN* to *S450-EN* according to EN 1993-1-1, Table 3.1 for product thicknesses $t \leq 40$ mm and of *S460-EN* and *S500-EN* according to EN 10025-2 for product thicknesses $t \leq 16$ mm. Other values can be taken into account selecting the material type *Steel*.

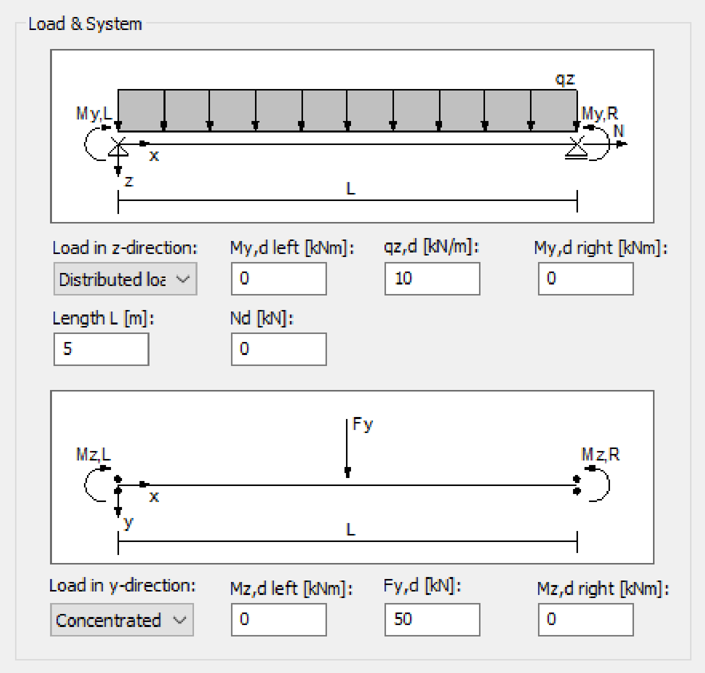
E-Modulus

Modulus of elasticity [N/mm²].

G-Modulus

Shear modulus [N/mm²].

Load & System



Load in z-direction

- *Distributed load*: A uniformly distributed load $q_{z,d}$ [kN/m] is applied to the equivalent beam.
- *Concentrated load*: A point load $F_{z,d}$ [kN/m] is applied to the equivalent beam. The load lever arm z_p is always set to zero for this selection.

My,d left, My,d right

Fixed end moments $M_{y,d}$ [kNm] including the partial safety factors γ_F. The fixed end moments can be determined, if required, using the second-order theory.

qz,d or Fz,d

Distributed load $q_{z,d}$ [kN/m] or point load $F_{z,d}$ [kN] including the partial safety factors γ_F.

Nd

Normal force N_d [kN] including the partial safety factors γ_F (negative compression). Only compressive forces can be considered. The normal force can be determined, if required, using the second-order theory.

Length L

Equivalent beam length [m].

Load in y-direction

- *Distributed load*: A uniformly distributed load $q_{y,d}$ [kN/m] is applied to the equivalent beam.
- *Concentrated load*: A point load $F_{y,d}$ [kN/m] is applied to the equivalent beam.

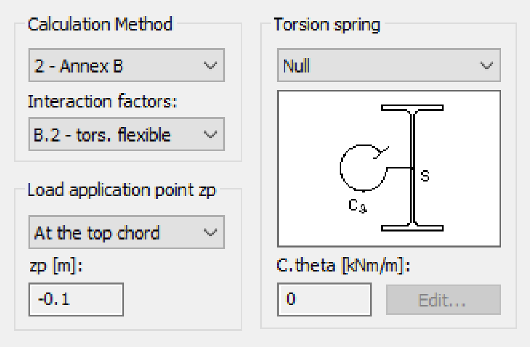
Mz,d left, Mz,d right

Fixed end moments $M_{z,d}$ [kNm] including the partial safety factors γ_F. The fixed end moments can be determined, if required, using the second-order theory.

qy,d or Fy,d

Distributed load $q_{y,d}$ [kN/m] or point load $F_{y,d}$ [kN] including the partial safety factors γ_F.

Lateral Torsional Buckling Parameters



Calculation method

- *1 - Annex A*: Determination of interaction factors k_{ij} with method 1 according to EN 1993-1-1, Annex A.
- *2 - Annex B*: Determination of interaction factors k_{ij} with method 2 according to EN 1993-1-1, Annex B.

Only method 1 can be applied for tube sections.

OENORM B 1993-1-1:
Method 2 is to be used. Tube sections can therefore only be tested for buckling.

SS EN 1993-1-1:
Method 1 is to be used.

Interaction factors

When selecting method 2, it is also necessary to specify whether the component is susceptible to torsional deformations or not according to EN 1993-1-1, Section 6.3.3 (1).

- *B.1 - tors. rigid*: The interaction factors k_{ij} for components that are not susceptible to torsional deformations (torsional rigid) are calculated according to Table B.1.
- *B.2 - tors. flexible*: The interaction factors k_{ij} for components that are susceptible to torsional deformations (torsional flexible) are calculated according to Table B.2.

Rectangular hollow sections are automatically assumed to be torsional rigid.

Load application point zp

- *Top chord*: The load acts at the top chord.
- *Shear center*: The load acts at the shear center.
- *Center of gravity*: The load acts at the center of gravity.
- *Bottom chord*: The load acts at the bottom chord.
- *User-defined*: User-defined load application point.

zp

The load lever arm z_p is calculated for the corresponding load application point.

Correction factor kc

DIN EN 1993-1-1, BS EN 1993-1-1:

- *Automatic*: Calculation corresponding to the moment distribution according to Tab. 6.6.
- *User-defined*: Factor $0 \le k_c \le 1$, e.g. as per DIN EN 1993-1-1, Eq. (NA.4) or BS EN 1993-1-1, Chapter NA.2.18.

Torsion spring

- *User-defined*: The torsion spring $C_{\vartheta,k}$ entered by the user is taken into account.
- *Compute*: The torsion spring $C_{\vartheta,k}$ is calculated from the input data (see the elastic rotational bedding dialog). For U profiles the elastic rotational bedding from the profile deformation of the supported girder is not taken into account.
- *Null*: No torsion spring is taken into account.

Edit...

Open the dialog to enter the torsion spring $C_{\vartheta,k}$ [kNm/m].

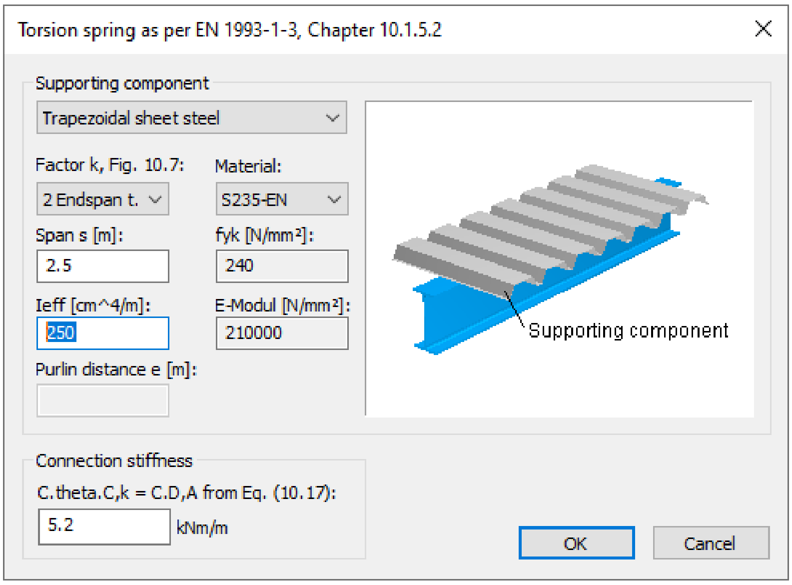
C.theta

The torsion spring $C_{\vartheta,k}$ is taken into account using an ideal second-degree torsional area moment $I_{T,ideal}$.

$I_{T,ideal} = I_T + C_{\vartheta,k} \cdot L^2 / (\pi^2 \cdot G)$

Using $I_{T,ideal}$, M_{cr} is calculated for girders without elastic rotational bedding. This approximation can only be applied for small values of elastic rotational bedding $C_{\vartheta,k}$ for instance for values of elastic rotational bedding for trapezoidal profiles, calculated according to EN 1993-1-1, Chapter 10.1.5.2; for larger values of $C_{\vartheta,k}$ the lateral torsional buckling moments calculated with $I_{T,ideal}$ can be unreliable (see Meister).

Torsion Spring



Supporting component

- *Trapezoidal sheet steel*
- *Purlins*: The connection strength is not taken into account.

Factor k, Fig. 10.7

Factor for the determination of $C_{D,C}$ according to EN 1993-1-3, Eq. (10.16):

- *2 Endspan t.*: k = 2; Endspan and rotation according to EN 1993-1-3, Fig. 10.7 top.
- *3 Endspan t.*: k = 3; Endspan and rotation according to EN 1993-1-3, Fig. 10.7 bottom.
- *4 Inside span t.*: k = 4; Inside span and rotation according to EN 1993-1-3, Fig. 10.7 top.
- *6 Inside span t.*: k = 6; Inside span and rotation according to EN 1993-1-3, Fig. 10.7 bottom.

Span s

Span [m] of the supporting component.

Ieff

Effective second-degree area moment [cm^4/m] of the supporting component.

Purlin distance e

Distance between the purlins perpendicular to the direction of load [m].

Material

Material of the supporting component.

- *S235-EN*, *S275-EN*, *S355-EN*, *S450-EN*: Construction steel according to EN 1993-1-1, Tab. 3.1.
- *Steel*: User-defined steel.

fyk

Characteristic yield strength of the supporting component [N/mm²].

E-Modulus

E-modulus [N/mm²] of the supporting component.

C.theta,C,k = C.D,A from Eq. (10.17)

Connection strength [kNm/m] for trapezoidal sheet steel according to EN 1993-1-3, Eq. (10.17).

Check Internal Forces

The calculation of load cases results in a set of internal forces for each load case at the check location (e.g. Nx, My). The check internal forces are then determined from the results of the load cases with the combination rules relevant for the check situations. One of the following methods can be selected in the analysis settings:

- *Min/Max combination*
  The results of a load case are added to the set of internal forces with the minimum or maximum of an internal force, if this increases the amount of the extreme value. Result sets from traffic actions in which the control variable is less than the threshold 10^{-3} are not combined. The min/max combination delivers a constant number of sets regardless of the number of load cases and thus represents a particularly economical solution for the checks.
- *Complete combination*
  To determine the evidence internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The number of records increases exponentially with the number of inclusive load cases and can therefore result in high time and memory requirements for the checks.

The internal forces relevant for the checks are documented in the detailed check listing. Regardless of the selection made, the results of the min/max combination are saved for the graphical representation. The load cases involved in the combination can be displayed using the *Combination information* context function.

The differences between the two combination methods mentioned before can be seen from the following example of a uniaxially stressed beam. The load cases 2, 3 and 4 shown can act simultaneously (inclusive). All safety and combination factors are assumed to be 1 for the example.

Action	Nx	My	Load case
G - permanent	-15	40	1
Q - variable	0	20	2
	5	10	3
	0	-10	4

Internal forces of the load cases

Extreme value	Nx	My	Combination
min Nx	-15	40	L1
max Nx	-10	50	L1+L3
min My	-15	30	L1+L4
max My	-10	70	L1+L2+L3

Results of min/max combination

Set	Nx	My	Combination
1	-15	40	L1
2	-15	60	L1+L2
3	-10	50	L1+L3
4	-15	30	L1+L4
5	-10	70	L1+L2+L3
6	-15	50	L1+L2+L4
7	-10	40	L1+L3+L4
8	-10	60	L1+L2+L3+L4

Results of complete combination

Ultimate Limit States

Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.4.3, the following combinations are taken into account in the ultimate limit states:

- For the combination of the permanent and temporary design situation either Equation (6.10) or the most unfavorable equation from (6.10a) and (6.10b) is permitted.

$$\sum_{j\geq1}\gamma_{G,j}\cdot G_{k,j}\text{"+"}\gamma_P\cdot P\text{"+"}\gamma_{Q,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \tag{6.10}$$

$$\sum_{j\geq1}\gamma_{G,j}\cdot G_{k,j}\text{"+"}\gamma_P\cdot P\text{"+"}\gamma_{Q,1}\cdot\psi_{0,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \tag{6.10a}$$

$$\sum_{j\geq1}\xi_j\cdot\gamma_{G,j}\cdot G_{k,j}\text{"+"}\gamma_P\cdot P\text{"+"}\gamma_{Q,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \tag{6.10b}$$

For the coefficient ξ the value of $\xi = 0.85$ results from Table A.1.2(B).

DIN EN 1990, OENORM B 1990:
Equation (6.10) is used for the combination.

SS EN 1990:
Equations (6.10a) and (6.10b) apply with following modifications:

$$\sum_{j\geq1}\gamma_d\cdot\gamma_{G,j}\cdot G_{k,j}\text{"+"}\gamma_P\cdot P \tag{6.10aSS}$$

$$\sum_{j\geq1}\xi_j\cdot\gamma_d\cdot\gamma_{G,j}\cdot G_{k,j}\text{"+"}\gamma_P\cdot P\text{"+"}\gamma_d\cdot\gamma_{Q,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_d\cdot\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \tag{6.10bSS}$$

Assuming reliability class 3, factor γ_d is set to 1. (see Section A, Article 11 and 14). The coefficient ξ is set to the value of $\xi = 0.89$.

BS EN 1990:
The coefficient ξ in Equation (6.10b) is set to the value of $\xi = 0.925$.

- Combination for accidental design situations

$$\sum_{j\geq1}G_{k,j}\text{"+"}P\text{"+"}A_d\text{"+"}(\psi_{1,1}\,\text{or}\,\psi_{2,1})\cdot Q_{k,1}\text{"+"}\sum_{i>1}\psi_{2,i}\cdot Q_{k,i} \tag{6.11b}$$

$\psi_{1,1}\cdot Q_{k,1}$ is used by the program for this combination.

OENORM B 1990-1:
$\psi_{2,1}\cdot Q_{k,1}$ is decisive.

- Combination for design situations caused by earthquakes

$$\sum_{j\geq1}G_{k,j}\text{"+"}P\text{"+"}A_{Ed}\text{"+"}\sum_{i\geq1}\psi_{2,i}\cdot Q_{k,i} \tag{6.12b}$$

Additionally, for the results of a stability check according to the second-order theory the

- Special combination

$$F_{d,1};\dots;F_{d,n}$$

is available. In this combination the action (F_d) are superposed exclusively.

For each combination you can define different design situations for the construction stages and final states. Each check is performed for all situations of a combination.

Design Values According to the Second-Order Theory

The calculation according to the second-order theory as per Clause 5.2.2 is carried out as part of the internal forces calculation. For this you need to create – with the help of load groups – appropriate load cases from decisive loads (see also function *Combinations* in the action dialog).

As the partial safety factors are already to be taken into account for the internal forces calculation, within the load groups the respective actions must be multiplied with their corresponding partial safety factor γ_G or γ_Q as well as with γ_{M0} according to Clause 6.1 (1).

DIN EN 1993-1-1:

Instead of γ_{M0} the value γ_{M1} = 1.1 is to be used.

In the check the load cases calculated according to the second-order theory are assigned to an action F_d and superposed in the *Special Combination* as an exclusive selection. The results are checked against the characteristic values of the cross-section resistances.

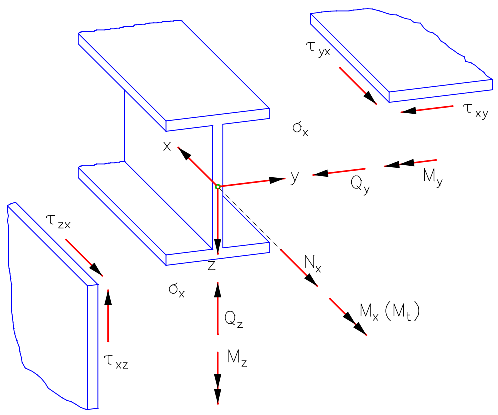
Characteristic Values

For sections made out of construction steel *S235-EN* to *S450-EN*, the yield strength is taken from EN 1993-1-1, Table 3.1 for product thicknesses $t \leq 40$ mm. For *S460-EN* and *S500-EN* the yield strength according to EN 10025-2 for product thicknesses $t \leq 16$ mm is assumed. If other sizes are used, the general material type *Steel* is to be chosen as it allows for freely defining all properties.

Stress Determination

Coordinate Systems

The following illustration shows the definition of the coordinate systems for internal forces, loads and stresses using the example of a beam with an I profile. The section edge is defined as a closed polygon.



The longitudinal force N_x runs through the centroid, the lateral forces Q_y and Q_z run through the shear center of the section which, in this case, coincides with the centroid. The vectors of the moments M_x, M_y, M_z and stresses σ_x, τ_{xy}, τ_{xz} run parallel to the coordinate axes.

Longitudinal Stresses

The necessary section properties for determination of the longitudinal stresses from longitudinal force and bending are determined elementarily with the segmentation method from the coordinates of the polygon. They are to be regarded as gross section properties in the sense of Clause 6.2.2.1 of the standard.

More specifically, these are:

y_s, z_s	Centroid coordinates.
A	Area of the section.
I_y, I_z	Moments of inertia in relation to the coordinate axes.
I_{yz}	Deviation moment of inertia.
I_1, I_2	Moments of inertia in relation to the main axes.
φ	Twisting angle of the main axes.
W_y, W_z	Moments of resistance for the calculation of the extremal longitudinal stresses from M_y, M_z.

For the analysis of a section, the stresses are determined at the automatically defined calculation points. The longitudinal stress σ for a point (y,z) of the section results from

$$\sigma(y,z) = N_x \frac{1}{A} + M_y \frac{\Delta z I_z - \Delta y I_{yz}}{I_y \cdot I_z - I_{yz}^2} + M_z \frac{\Delta y I_y - \Delta z I_{yz}}{I_y \cdot I_z - I_{yz}^2}$$

with $\Delta y = y - y_s$ and $\Delta z = z - z_s$.

Shear Stresses

Lateral force

For load as a result of lateral force, the differential equation of the so-called shear warpage ω_τ

$$\frac{\partial^2 \omega_\tau}{\partial y^2} + \frac{\partial^2 \omega_\tau}{\partial z^2} = -\frac{Q_z\, z}{G\, I_y} \quad \text{or} \quad -\frac{Q_y\, y}{G\, I_z}$$

is solved numerically with the help of the boundary element method. From this the following shear characteristics are determined:

$\tau_{Qy,1}$	Shear stress for the lateral force $Q_y = 1$.
$\tau_{Qz,1}$	Shear stress for the lateral force $Q_z = 1$.
W_{qy}, W_{qz}	Area values for calculation of the extremal shear stresses from Q_y and Q_z with $\tau_{Qy} = Q_y / W_{qy}$ and $\tau_{Qz} = Q_z / W_{qz}$.

If selected in the Section dialog, the shear stresses across the section width will be averaged.

Torsion

For load as a result of St. Venant torsion, the differential equation of the unit warpage ω

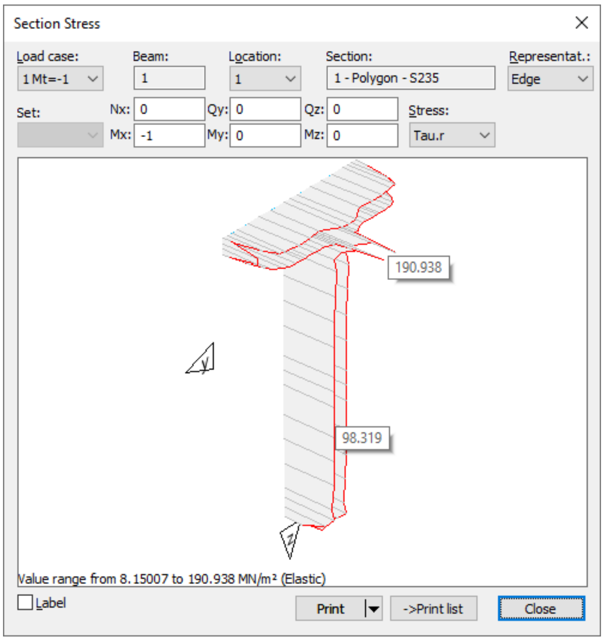
$$\frac{\partial^2 \omega}{\partial y^2} + \frac{\partial^2 \omega}{\partial z^2} = 0$$

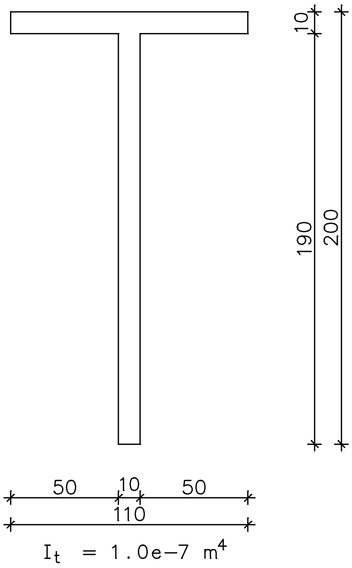
is decisive. Its solution leads to the following characteristics:

I_t	Torsion moment of inertia.
$\tau_{Mx,1}$	Shear stress for the torsional moment $M_x = 1$.
W_t	Moment of resistance for the calculation of the extremal shear stress from M_x.

As a result, in the fillet area of rolled sections or in blunt corners of welded profiles larger torsional shear stresses occur than those according to the theory of thin-walled profiles. As an option in the section dialog, specifying the maximum sheet thickness (open profiles) or the torsion resistance moment (box profiles) allows you to limit the stresses to the maximum value according to the theory of thin-walled profiles.

The following figure shows the stress curve for a load of $M_x = 1$ (*Section Stress* function):





The numerically determined value for the web in the example corresponds to the theoretical shear stress for open thin-walled sections $\tau = M_x \cdot t \,/\, I_t = 100$ MN/m². The peak value of 190 MN/m² results from a singularity in the blunt corners.

All unit shear stresses are calculated at discrete points on the section edges. The discretization is automatically carried out by the program. The unit shear stresses are edge stresses which run along the section edge. Their sign depends on the direction of the edge. The resultant shear stress τ from lateral force and torsion is then determined by the following equation:

$$\tau = Q_y \cdot \tau_{Qy,1} + Q_z \cdot \tau_{Qz,1} + M_x \cdot \tau_{Mx,1}$$

Classification of Cross-Sections

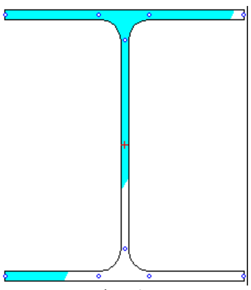
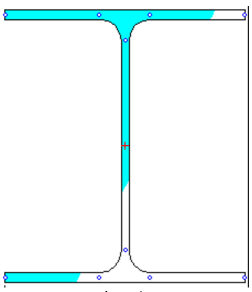
The classification is performed for standard profiles and parameterized steel sections according to EN 1993-1-1, Table 5.2. To this end, the stress distribution for simultaneous stress from biaxial bending and normal force in the center line of the section parts is used. T-profiles are treated like outstand flanges according to table 5.2. Angle sections are not classified as flanges supported on one side, but according to the separate table section for angle sections of Tab. 5.2 (see Kuhlmann et al. 2016, note to Tab. 5.2). User defined rectangle sections are assigned to class 3 without further analysis. For polygonal sections and sections from the user database, the class can be specified in the cross-section dialog, whereby the classification according to Table 5.2 must be carried out in advance by the user.

A cross-section is generally classified by the most unfavorable class of its compressed section parts. The permissible exceptions described in the Chapters 5.5.2 (11), 5.5.2 (12), 6.2.1 (10) and 6.2.2.4 (1) are not used in the program.

Example

Cross-section: HEAA 360 – S355

Design load: N = –400 kN, My = 175 kNm, Mz = 65 kNm

Strain state			Web	F1	F2	F3	F4
F1 F2  F4 elastic F3	F1 F2  F4 plastic F3	ε	0.81	0.81	0.81	0.81	0.81
		α	0.70	1	0.66		0.78
		ψ	-0.45	0.58	-0.15		-0.52
		c/t	29	9.88	9.88		9.88
		1: c/t ≤	39.95	7.32	13.63		9.35
		2: c/t ≤	46.01	8.14	15.14		10.39
		3: c/t ≤	65.56	11.73	28.73		14.28
		Class	**1**	**3**	**1**	**1**	**2**

Under the given load, resulting from the classification of flange F1, the whole cross-section is categorized as class 3 and only the elastic check applies.

Elastic Cross-Section Resistance

The elastic cross-section resistance is verified for all cross-sections of the classes 1 to 4. Cross-sections of class 4 are treated like class 3 if selected by the user and if the c/t ratio does not exceed the limits of class 3 increased by a factor according to Chapter 5.5.2 (9). Otherwise the check is carried out with effective cross-section properties as per EN 1993-1-5, Clause 4.3, taking into account the additional moments according to EN 1993-1-1, Eq. (6.4). The effective cross-section properties are determined under simultaneous effect of N_{Ed} and M_{Ed} without iteration. Circular hollow sections classified in class 4 cannot be checked, because the effective cross section area A_{eff} cannot be determined according to Chapter 4.3. For angle sections, the effective cross-section values are determined for the unfavorable case of a completely compressed cross-section (cf. Kuhlmann et al. 2016, note to Tab. 5.2).

Check

The permissibility of the comparison stress is proofed with the yield criterion according to equation (6.1).

$$\left(\frac{\sigma_{x,\mathrm{Ed}}}{f_y/\gamma_{M0}}\right)^2+\left(\frac{\sigma_{z,\mathrm{Ed}}}{f_y/\gamma_{M0}}\right)^2-\left(\frac{\sigma_{x,\mathrm{Ed}}}{f_y/\gamma_{M0}}\right)\cdot\left(\frac{\sigma_{z,\mathrm{Ed}}}{f_y/\gamma_{M0}}\right)+3\left(\frac{\tau_{\mathrm{Ed}}}{f_y/\gamma_{M0}}\right)^2\leq 1 \qquad (6.1)$$

Where

$\sigma_{x,\mathrm{Ed}}$	is the design value of the longitudinal stress at the point of consideration.
$\sigma_{z,\mathrm{Ed}}$	is the design value of the transverse stress at the point of consideration. $\sigma_{z,\mathrm{Ed}} = 0$.
τ_{Ed}	is the design value of the shear stress at the point of consideration.
f_y	is the nominal value of the yield strength as per Table 3.1.
γ_{M0}	is the partial safety factor for the resistance of cross-sections as per Clause 6.1 (1).

Stresses resulting from the *Special Combination* are checked against the characteristic value of the yield strength f_y, because the partial safety factor γ_{M0} has already been taken into account during the calculation of the internal forces.

Plastic Cross-Section Resistance

The plastic cross-section resistance will be verified for all cross-sections of the classes 1 and 2 if selected by the user and if the elastic cross-section resistance of the contemplated set of internal forces is exceeded. For this, the following methods apply:

- Interaction equations for double symmetrical I and H profiles as well as for rectangular box sections.
- Equilibrium iteration of absorbable internal forces for the remaining cross-sections.

Optionally, the user can select to determine the plastic resistance by equilibrium iteration for **all** cross-sections.

Interaction Equations

For double symmetrical I and H profiles with constant flange thickness as well as for rectangular box sections, the plastic cross-section resistance can be determined according to EN 1993-1-1, Chapter 6.2.3 to 6.2.10. If none of the single actions exceeds its plastic resistance, the interaction of internal forces is examined using the interaction equations given in Chapter 6.2. For this, according to Chapter 6.2.10 (2), the load bearing capacity of cross-sections stressed by bending and normal force is not reduced if the effect of lateral force does not exceed half of the plastic lateral force resistance. The following special conditions also apply:

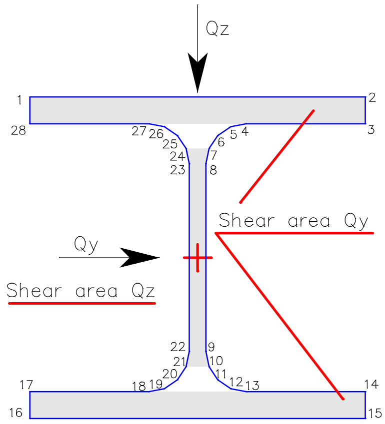
- The coefficient η as per Chapter 6.2.6 (3) is assumed to be 1.
- In the case of lateral force interaction according to Chapter 6.2.8 (3) and 6.2.10 (3), the reduced yield strengths for rolled I and H profiles are applied to the shear area A_{Vz} for lateral force in z direction and A - A_{Vz} for lateral force in y direction.
- The simplification according to Chapter 6.2.8 (5) is not applied.
- If the simultaneous interaction of bending, normal force and lateral force is taken into account, the factors a, a_w and a_f from Chapter 6.2.9.1 (5) are calculated with the correspondingly reduced shear areas. This is based on the interpretation of the NABau to DIN EN 1993-1-1 of April 2017.

Equilibrium Iteration

The verification of the plastic cross-section resistance is carried out by comparison of the absorbable internal forces (resistance) with the load internal forces of a section. Here it is to be checked that the limit internal forces in the plastic state are not exceeded. The calculation of the absorbable internal forces is performed by integration of the stresses on the section polygon and equilibrium iteration taking into account the following conditions:

- Linear-elastic ideal plastic stress-strain relationship.
- Section remains planar.
- Huber-von Mises yield criterion.

The interaction of all 6 internal forces N_x, Q_y, Q_z, M_x, M_y, M_z is to be taken into account. To ensure this, simplified assumptions with respect to the shear stress distribution from lateral force and torsion are to be made as a closed solution of the interaction problem is not available. Because in steel construction thin-walled sections are common, subareas can be defined as a good approximation and then used to dissipate the shear stresses:



The steel sections in the section library have predefined subareas which are listed in the section table of the log. The following assumptions apply:

- The lateral forces create constant shear stresses $\tau_{xy} = Q_y/A_{qy}$ or $\tau_{xz} = Q_z/A_{qz}$ in their corresponding subareas.
- The torsion moment $M_x$results in local, constant shear stress states in the subareas and the remaining area with the maximum edge shear stress τ_t acting on the area.

For polygon sections and sections from the user database, subareas cannot be taken into account. In this case the following assumptions apply for the entire section:

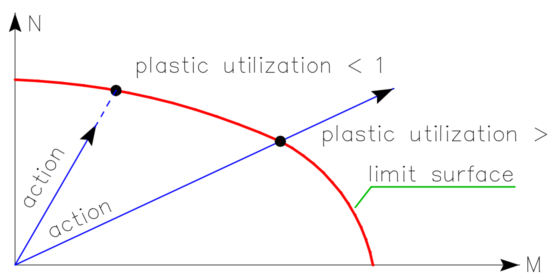
- The lateral forces create constant shear stresses $\tau_{xy} = Q_y / W_{qy}$ or $\tau_{xz} = Q_z / W_{qz}$.
- The torsion moment M_x creates the constant shear stress state $\tau_t = M_x / W_t$.

The design values of the strengths are each reduced locally by the shear stress described above. To allow calculation of the absorbable internal forces, the remaining available strength is used for each area. The calculation of the shear characteristics is carried out for determining the section properties as described above. The exception for effects of small lateral forces according to EN 1993-1-1, Chapter 6.2.10 (2), is not utilized.

The calculation approach described delivers limit internal forces that are always on the safe side; however, in individual cases it could be that not all reserves are used.

To help estimate the degree of plastic utilization of a section, the load vector is intersected with the limit area of the 6-dimensional internal force space. This can be used to calculate a factor for plastic utilization, which is ≤ 1 if the limit area is not exceeded and > 1 if it is exceeded.

The following illustration shows the iteration of the limit area using the example of an N-M interaction.



Check

The check evaluates if the load internal forces exceed the limit internal forces taking into account the interaction of all internal forces in the full plastic state. In the *Special Combination*, the plastic internal forces are calculated with the nominal value f_y of the yield strength whereas f_y / γ_{M0} is used in all other combinations.

Check Against Buckling and Lateral Torsional Buckling

The check for potentially unstable frameworks can be carried out according to EN 1993-1-1, Section 5.2.2 (7b) for the individual component. The program performs the buckling and the lateral torsional buckling check on the fork supported equivalent beam according to Chapter 6.3. The fixed end forces are to be determined in a static analysis of the entire structure, taking into account the second-order theory and global imperfections, and are to be applied at the equivalent beam. For the buckling length of the individual component the system length can be applied.

For the check against buckling and lateral torsional buckling on the equivalent beam the following requirements must be met:

- The calculation of the internal forces is carried out according to the theory of elasticity.
- The equivalent beam is assumed to be straight.
- Section and material are constant along the length of the beam.
- The dimensions of the section are small compared with the other dimensions.
- The mathematical curvature is linearized.
- The influence of shear deformations on the internal forces are not considered.
- The load is slowly increased to its final value and does not undergo any deviation in direction as a result of the system deformation.
- Actions in the z direction act at a distance zp from the shear center.
- The actions in the y direction act at the shear center.
- The equivalent beam rests on a fork support at both ends.
- Sheet buckling is not included.
- The effects from torsion are not accounted for.

The check against lateral torsional buckling can be carried out for I, U and user-defined steel sections, for other sections only the check against buckling according to Chapter 6.3.1 can be carried out.

Calculation of the resistance

Initially for each check location a cross-section classification according to Chapter 5.5 is carried out with the corresponding internal forces. Depending on the cross-section class the absorbable internal forces are determined according to Table 6.7.

For cross-sections of classes 1 and 2 the determination of the plastic internal forces is performed by integration of the stresses on the section polygon, as described above.

For cross-section class 4 the cross-section properties are determined with the effective area of the section parts under compression. If thereby a translation of the main axes of the effective section compared to the gross section occurs, furthermore the resulting additional moments ($\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$) are determined.

Calculation of N_{cr}

$N_{cr,y}$ and $N_{cr,z}$ are the elastic flexural buckling forces about the y and z axis. They are calculated by the program with the following formulas:

$$N_{cr,y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr,y}{}^2}$$

$$N_{cr,z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{cr,z}{}^2}$$

Calculation of the elastic critical moment M_{cr} for lateral torsional buckling

The elastic critical moment M_{cr} for lateral torsional buckling can be determined for example according to Petersen (1980) with the following equation:

$$M_{cr} = M_{Ki,y} = \zeta \cdot N_{Ki,z} \cdot \left(\sqrt{c^2 + 0.25 \cdot z_p^2} + 0.5 \cdot z_p \right)$$

This formula is only applicable for double-symmetric I profiles. Additionally, for a general moment curve, the moment coefficient ζ is quite difficult to determine. For single-symmetric profiles, it is no longer possible to determine M_{cr} using the abovementioned approach.

In order to be able to calculate any single-symmetric profile under general load, this program module contains a method for the direct determination of M_{cr}.

The solution is derived at by varying the elastic potential $\prod$ while neglecting the terms from the calculated bending w. For the unknown functions v and φ a multiple-term power series is used that fulfills the boundary conditions.

$$\prod = \frac{1}{2} \int_0^L \left\{ \left[G I_x + (r_y - 2 z_m)\, M_y(x)\, \eta_{cr} \right] \left(\frac{\partial \varphi}{\partial x} \right)^2 + E I_w \left(\frac{\partial^2 \varphi}{\partial x^2} \right)^2 + E I_z \left(\frac{\partial^2 v}{\partial x^2} \right)^2 + 2\, M_y(x)\, \eta_{cr} \left(\frac{\partial^2 v}{\partial x^2} \right) \varphi + q\, \eta_{cr}\, z_p\, \varphi^2 \right\} dx$$

The geometrically nonlinear share comprising actions in the z direction is included via a uniformly distributed load q_z. This is chosen such that the same maximum moment of span is created as through the given action.

The resulting eigenvalue problem provides the smallest positive critical load factor η_{cr} and thus the required lateral torsional buckling moment M_{cr} according to Martin (1996). This is calculated for each check location.

$$M_{cr} = \eta_{cr} \cdot M_{y,d}$$

The method described offers the advantage that the user does not need to specify the moment coefficient ζ.

Serviceability Limit States

Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.5.3, the following combinations are taken into account in the serviceability limit states:

- Combination for characteristic situations

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{0,i} \cdot Q_{k,i} \tag{6.14b}$$

- Combination for frequent situations

(6.15b)

- Combination for quasi-continuous situations

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \tag{6.16b}$$

Limiting Deformations

In accordance with EN 1993-1-1, Chapter 7.2, the vertical and horizontal deformations shall be limited in consultation with the client corresponding to EN 1990, Annex A.1.4. According to A.1.4.3 (3) the quasi-continuous situation is decisive for the check.

OENORM B 1990-1, Chapter 8.2:

Under the quasi-continuous situation, the deflection under long-term loading should not exceed 1/250th of the span. A limitation to 1/500th of the span under the frequent or the quasi-continuous situation can be necessary for the special cases described in Chapter 8.2.

SS EN 1993-1-1:

In thin-walled structures, according to Article 16 and 17 the deformations under the frequent situation with reversible limit load should not exceed 1/200th of the span.

The compliance with the aforementioned limitations can be proven in the graphical and numerical deformation representation. In the folder *Node Deformations* of the database tree, the deformations of all decisive situations can be accessed.

Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The detailed log also lists the decisive combination internal forces of all design situations for each result location.

Stresses

σ_x, σ_y	Extremal normal stresses from bending and normal force.
σ_{xy}	Extremal shear stresses from torsional moment and shear force.
τ_{xy}, τ_{xz}	Extremal shear stresses from lateral force and torsion.
σ_v	Maximum comparison stress.
σ_1, σ_2	Extremal principal stresses.

All stresses are given in [MN/m²].

Utilizations

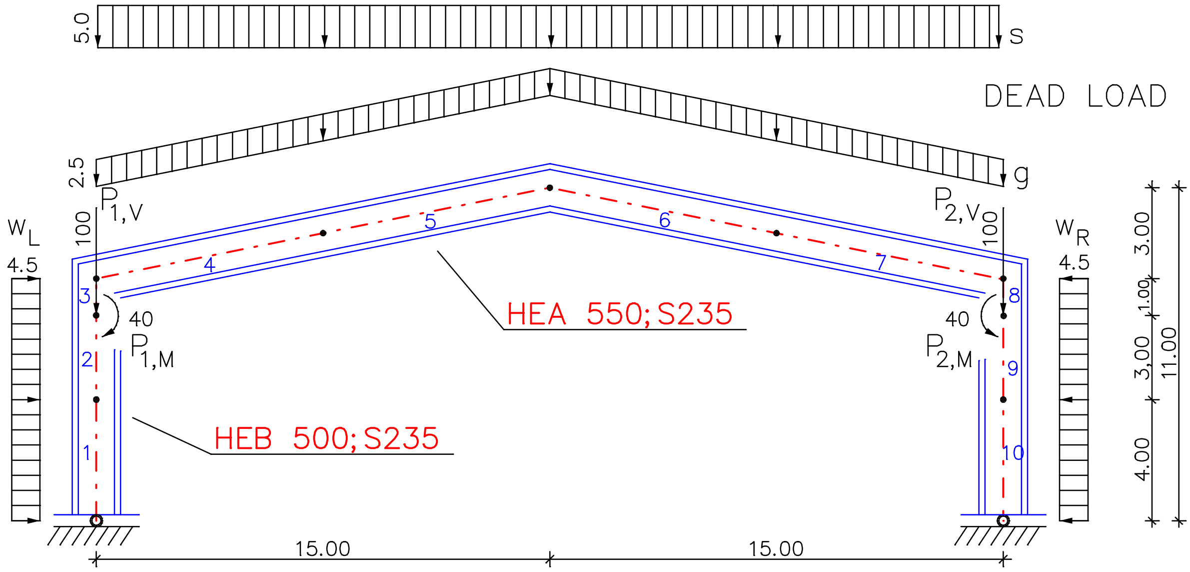
The utilization is defined as the ratio between the action E_d and, depending on the user selection, the elastic or plastic resistance R_d of a cross-section. In the folder *Stresses > Steel Checks* of the result tree the following results are available:

- Utilization of the beams for each situation.
- Maximal utilization of the beams of all situations.
- Maximal utilization of the sections of all situations.

Examples

Steel Checks on the Entire System

In the hall frame illustrated below a framework analysis and a stability check are carried out according to the second-order theory. The plastic cross-section resistance is verified for all cross-sections of classes 1 and 2, because exceedances occur in the elastic stress check.



Static system and dimensions

Sum of installed loads and support reactions

LC	Label	Fx [kN]	Fz [kN]
1	Dead load Support reactions	0.000 -0.000	200.250 200.250
2	Snow Support reactions	0.000 -0.000	150.000 150.000
3	Wind left Support reactions	36.000 36.000	0.000 -0.000
4	Wind right Support reactions	-36.000 -36.000	0.000 0.000
5	Crane track left Support reactions	0.000 0.000	100.000 100.000
6	Crane track right Support reactions	0.000 -0.000	100.000 100.000
7	Crane track center:(LC5+LC6)/2 Support reactions	0.000 -0.000	100.000 100.000
11	Th2+Predef1; My min,right;PC.1 Support reactions	32.400 32.400	600.338 600.338
12	Th2+Predef2; My min,left;PC.1 Support reactions	-32.400 -32.400	600.338 600.338
13	Th2; My max,center;PC.1 Support reactions	0.000 0.000	495.338 495.338
14	Th2+Predef1; Nx min,right;PC.1 Support reactions	32.400 32.400	600.338 600.338
15	Th2+Predef2; Nx min,left;PC.1 Support reactions	-32.400 -32.400	600.338 600.338

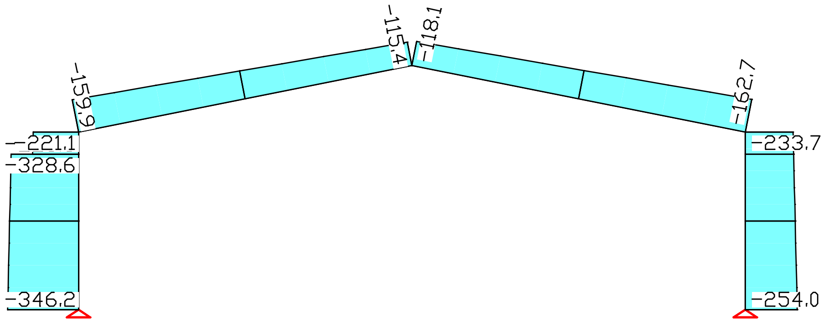
Material properties

No.	Material type	EModule [MN/m²]	alpha.t [1/K]	gamma [kN/m³]	kbx-a [MN/m³]	kbx-e [MN/m³]	kbz-a [MN/m³]	kbz-e [MN/m³]	bx [m]	bz [m]
1	S235-EN	210000	1.200e-05	78.500	0	0	0	0	1.00	1.00
2	S235-EN	210000	1.200e-05	78.500	0	0	0	0	1.00	1.00

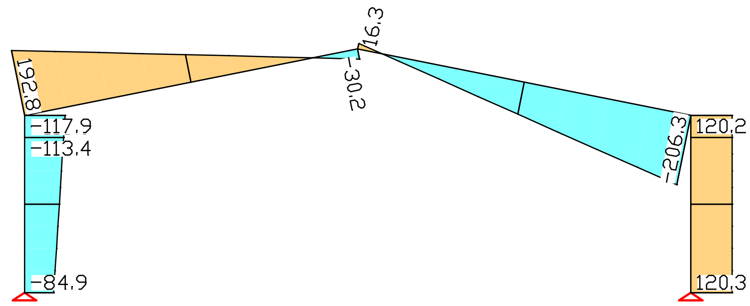
Section properties

```
No.   1 HEB 500
        A = 2.390e-02 [m²], Iy = 1.072e-03 [m4]
No.   2 HEA 550
        A = 2.120e-02 [m²], Iy = 1.119e-03 [m4]
```

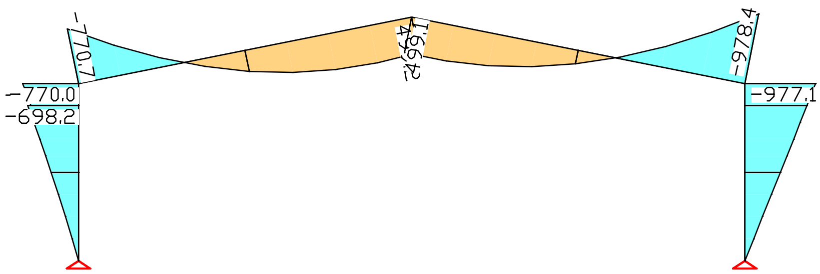
For stability checks according to the second-order theory, load groups have been composed in compliance with the combination information of the respective internal forces.
The following internal forces curves result from load case 11 composed as described before, considering a pre-deformation of l/200 to the right.



Load case 11, second-order *theory, internal forces* N_x



Load case 11, second-order *theory, internal forces* Q_z



Load case 11, second-order *theory, internal forces* M_y

To perform the checks the actions must be assigned as shown in the following log. The checking program performs the necessary internal force combinations and verifies the cross-section resistance.

Design per EN 1993-1-1:2005/A1:2014

According to Clause 6.2.1 of the standard, for classes 1 and 2 the plastic resistance, for class 3 the elastic resist. of the cross-section is verified.

For Class 4 cross-sections the check is carried out with effective properties as per EN 1993-1-5, Clause 4.3.

The actions are combined acc. to EN 1990, Eq. (6.10), using the partial and combination factors according to EN 1990:2021.

Designing occurs for all possible combinations of actions.

Terms in Printout

```
Sigma.x, Sigma.v: Normal stresses, comparison stresses acc. to Eq. (6.1).
Tau.xy, Tau.xz  : Shear stresses from lateral force Qz.
A               : Sectional area.
Iy, Iz, Iyz     : Moments of inertia.
Aqz             : Shear subarea of the polygon section for Qz.
Wqz             : Resistance momentum of the ext. shear stress of Qz.
y, z            : Location in the sectional polygon [m].
```

Stress Calculation

```
The normal stresses and shear stresses are calculated for homogeneous
polygon sections from normal force, bending moment and lateral force.
The shear characteristics are calculated acc. to the Boundary-Element-Method.
The calculation points for all stresses are edge points of the cross-section.

For the following cross-sections the shear stresses from lateral force
are averaged over the section width.

Cross-section        Cross-section        Cross-section
   1 HEB 500            2 HEA 550                -
```

Cross-Section Classification

```
The classification occurs for standard profiles and parameter cross-sections
acc. to EN 1993-1-1, Table 5.2. T-Profiles are handled as outstand flanges.
User-defined polygonal cross-sections are classified always into class 3.
```

Plastic Cross-Section Resistance

```
For double symmetrical I and H profiles with constant flange thickness as well
as for rectangular box sections, the plastic cross-section resistance is
determined according to EN 1993-1-1, Chapter 6.2.3 to 6.2.10. If none of the
single actions exceeds its plastic resistance, the interaction of internal
forces is examined. For this, according to Chapter 6.2.10 (2) the load bearing
capacity is not reduced if the effect of lateral force does not exceed half of
the plastic lateral force resistance. The coefficient eta as per Chapter
6.2.6 (3) is assumed to be 1.

For all further profiles, the absorbable internal forces are calculated by
integration of the stresses and equilibrium iteration on the section polygon.
For this, the interaction of internal forces is regarded and the exception for
effects of small lateral forces as per EN 1993-1-1, Chapter 6.2.10 (2),
is not utilized.

For simplification a constant shear stress Tau=Qz/Aqz is assumed
for the coresponding shear area during integration.

The shear characteristics are calculated with use of the Boundary-
Element-Method at the section edge.

The Huber-v. Mises-Condition as per Eq. (6.1) is used as yield criterion.
```

Partial Safety Factor for Steel

```
                                    gamma.M0
Permanent and temporary comb.         1.00
Accidental combination                1.00
Special combination                   1.00
```

Characteristic Material Properties

```
Yield stresses for steel acc. to EN 1993-1-1, Table 3.1 for t <=40 mm [MN/m²].
Limiting stresses according to Eq. (6.19) and Eq. (6.42).

              Perm. C.         Accid. C.        Special C.
Material fyk  Sigma.Rd Tau.Rd  Sigma.Rd Tau.Rd  Sigma.Rk Tau.Rk
S235-EN  235   235.00  135.68   235.00  135.68   235.00  135.68
```

Cross-Section Properties

```
Cross-section   Material  A[mm²] Aqz[mm²]     Iy[m4]      Iz[m4]      Iyz[m4]
   1 HEB 500    S235-EN    23900     5997  1.0720e-03  1.2620e-04  0.0000e+00
   2 HEA 550    S235-EN    21200     5776  1.1190e-03  1.0820e-04  0.0000e+00
```

EN 1993-1-1 actions

Standard design group

G - Dead load

```
Gamma.sup / gamma.inf = 1.35 / 1

Load cases
----------
1    Dead load
```

QN - Imposed load, traffic load

```
Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures
Traffic load - category G: 30 kN < vehicle weight <= 160 kN
Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.3

Load cases 1. Variant, exclusive
--------------------------------
5    Crane track left
6    Crane track right
7    Crane track center:(LC5+LC6)/2
```

QS - Snow and ice load

```
Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures
Snow load - Places in CEN member states with less than 1000 m above sea level
Psi.0 / Psi.1 / Psi.2 = 0.5 / 0.2 / 0

Load cases 1. Variant, exclusive
--------------------------------
2    Snow
```

QW - Wind load

```
Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures
Wind loads
Psi.0 / Psi.1 / Psi.2 = 0.6 / 0.2 / 0

Load cases 1. Variant, exclusive
--------------------------------
3    Wind left
4    Wind right
```

Fd - Design values of actions

```
Load cases
----------
11   Th2+Predef1; My min,right;PC.1
12   Th2+Predef2; My min,left;PC.1
13   Th2; My max,center;PC.1
14   Th2+Predef1; Nx min,right;PC.1
15   Th2+Predef2; Nx min,left;PC.1
```

1. Permanent and temporary situation

```
Final state

G  - Dead load
QN - Imposed load, traffic load
QS - Snow and ice load
QW - Wind load
```

1. Quasi-continuous situation

```
Final state

G  - Dead load
QN - Imposed load, traffic load
QS - Snow and ice load
QW - Wind load
```

1. Special situation

```
Final state

Fd - Design values of actions
```

Check of the Cross-Section Resistance for Beams

```
The results represent the extrema of all combinations.

Class       Decisive section class according to EN 1993-1-1, Table 5.2.
Utilization Relation between the action Ed and the resistance Rd.
            Plastic resistance acc. to Chapter 6.2.10 is marked with '!'.
PC,SC       Permanent and temporary comb., Special combination
```

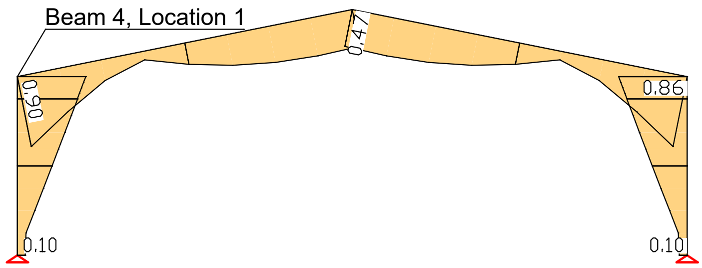
Beam	Se.	Lo.	Comb.	Class	Utilization elastic	Utilization plastic
1	1	1	SC.1	1	0.14	0.10!
		2	SC.1	1	0.20	0.11!
		3	SC.1	1	0.31	0.22!
		4	SC.1	1	0.43	0.33!
		5	SC.1	1	0.55	0.44!
2	1	1	SC.1	1	0.55	0.44!
		2	SC.1	1	0.64	0.52!
		3	SC.1	1	0.73	0.60!
		4	SC.1	1	0.82	0.68!
		5	SC.1	1	0.92	0.76!

Beam	Se.	Lo.	Comb.	Class	Utilization elastic	Utilization plastic
3	1	1	SC.1	1	0.89	0.76!
		2	SC.1	1	0.92	0.78!
		3	SC.1	1	0.95	0.81!
		4	SC.1	1	0.98	0.84!
		5	SC.1	1	1.01	0.86!
4	2	1	SC.1	1	1.04	0.90!
		2	SC.1	1	0.66	0.56!
		3	SC.1	1	0.33	0.27!
		4	SC.1	2	0.15	0.11!
		5	SC.1	1	0.33	0.27!
5	2	1	SC.1	1	0.33	0.27!
		2	SC.1	1	0.47	0.39!
		3	SC.1	1	0.54	0.46!
		4	SC.1	1	0.57	0.49!
		5	SC.1	1	0.55	0.47!
6	2	1	SC.1	1	0.55	0.47!
		2	SC.1	1	0.57	0.49!
		3	SC.1	1	0.54	0.46!
		4	SC.1	1	0.47	0.39!
		5	SC.1	1	0.33	0.27!
7	2	1	SC.1	1	0.33	0.27!
		2	SC.1	2	0.15	0.11!
		3	SC.1	1	0.33	0.27!
		4	SC.1	1	0.66	0.56!
		5	SC.1	1	1.04	0.90!
8	1	1	SC.1	1	1.01	0.86!
		2	SC.1	1	0.98	0.84!
		3	SC.1	1	0.95	0.81!
		4	SC.1	1	0.92	0.78!
		5	SC.1	1	0.89	0.76!
9	1	1	SC.1	1	0.92	0.76!
		2	SC.1	1	0.82	0.68!
		3	SC.1	1	0.73	0.60!
		4	SC.1	1	0.64	0.52!
		5	SC.1	1	0.55	0.44!
10	1	1	SC.1	1	0.55	0.44!
		2	SC.1	1	0.43	0.33!
		3	SC.1	1	0.31	0.22!
		4	SC.1	1	0.20	0.11!
		5	SC.1	1	0.14	0.10!

Max. Cross-Section Utilization

```
Class       Decisive section class according to EN 1993-1-1, Table 5.2.
Utilization Relation between the action Ed and the resistance Rd.
x           Distance from the beam startpoint [m].
PC,SC       Permanent and temporary comb., Special combination
```

Cross-section	Material	Beam	Loc.	x[m]	Comb.	Class	Utilization
1 HEB 500	S235-EN	3	5	1.00	SC.1	1	0.86
2 HEA 550	S235-EN	4	1	0.00	SC.1	1	0.90



Plastic utilization

An extract of the detailed listing of beam 4 is printed below. This shows that the cross-section resistance in this location is exceeded and therefore the plastic check becomes necessary.

Check of the Cross-Section Resistance for Beams

```
The results represent the extrema of all combinations.

Class       Section class according to EN 1993-1-1, Table 5.2.
W, F, A     Section part of standard profiles: Web, Flange, Angle.
A.eff,I.eff Effective properties for Class 4 as per EN 1993-1-5, Clause 4.3.
e.z         Shift of the centroid of A.eff relative to the centroid of the gross
            cross-section to determine the additional moment as per Eq. (6.4).
Utilization Relation between the action Ed and the resistance Rd.
x           Distance from the beam startpoint [m].

Beam 4: x = 0.00 m (Beam Length 7.65 m)
Cross-section 2: HEA 550, S235-EN, fyk=235 MN/m², A=21200 mm²
Web    : c = 438.0 mm, t = 12.5 mm, c/t = 35.04
Flange : c = 116.7 mm, t = 24.0 mm, c/t =  4.86
```

1. Permanent and temporary comb. (PC.1): G+QN+QS+QW

```
No set of internal forces in this combination was relevant.
```

1. Special combination (SC.1): Fd

```
Relevant values from 5 sets of internal forces
                                     Class
Set       Nx[kN]     Qz[kN]    My[kNm]  W F F F F
2    :  -162.73    206.31    -978.36   1 1 1 1 1 : 1

Load case combination for the relevant set of internal forces
Set    Combination
2    : L12
```

Normal Stresses [MN/m²]

```
Min. Sigma.x      :  -243.74    Class           : 1
     Sigma.x(Nx) :    -7.68    Situation       : SC.1,2
     Sigma.x(My) :  -236.07    Se.point y [m]  :  0.150
                                         z [m]  :  0.270

Max. Sigma.x      :   228.39    Class           : 1
     Sigma.x(Nx) :    -7.68    Situation       : SC.1,2
     Sigma.x(My) :   236.07    Se.point y [m]  : -0.150
                                         z [m]  : -0.270
```

Shear Stresses [MN/m²]

```
Ext. Tau.xy       :    -5.53    Class           : 1
     Tau.xy(Qz)  :    -5.53    Situation       : SC.1,2
                                Se.point y [m]  : -0.033
                                         z [m]  :  0.246

Ext. Tau.xz       :    34.05    Class           : 1
     Tau.xz(Qz)  :    34.05    Situation       : SC.1,2
                                Se.point y [m]  :  0.006
                                         z [m]  : -0.000
```

Comparison Stress [MN/m²]

```
Max. Sigma.v      :   243.90    Class           : 1
     Sigma.x(Nx) :    -7.68    Situation       : SC.1,2
     Sigma.x(My) :  -236.07    Se.point y [m]  :  0.045
     Tau.xy(Qz)  :     5.13             z [m]  :  0.270
     Tau.xz(Qz)  :     0.00
```

Check of the Crosss-Section Resistance and Utilisation

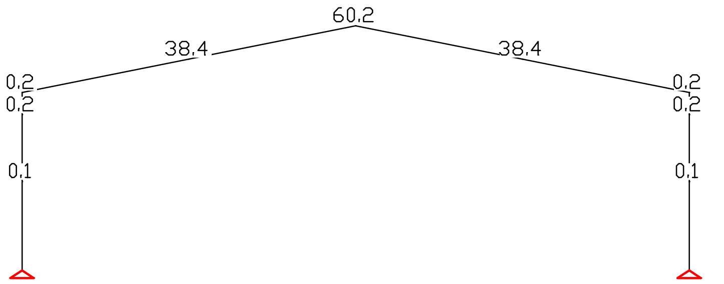
```
Elastic                  Ed        Rd
     Nx [kN]      :  -162.73   -156.79    Utilization:  1.04 > 1
     Qz [kN]      :   206.31    198.78    Class       : 1
     My [kNm]     :  -978.36   -942.65    Situation   : SC.1,2

Plastic                  Ed        Rd     Ch. 6.2.10
     Nx [kN]      :  -162.73   4982.00    Utilization:  0.90
     Qz [kN]      :   206.31   1139.15    Class       : 1
     My [kNm]     :  -978.36   1086.13    Situation   : SC.1,2

The plastic utilization is decisive.
```

Limitation of deformation

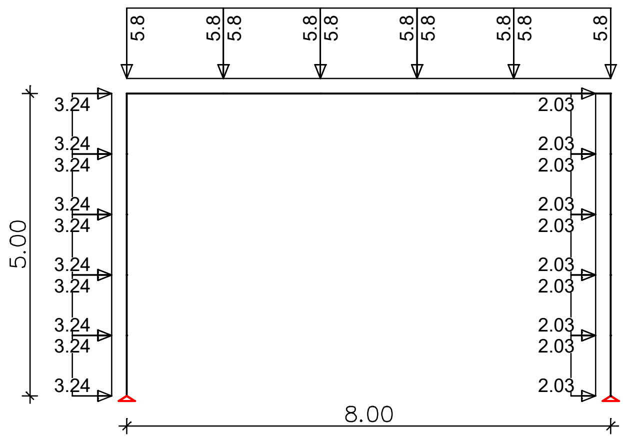
The maximal deformations *uz,max* under the quasi-continuous situation are presented in the following picture. The maximal deformation in the center of the frame is 1/498th of the span.

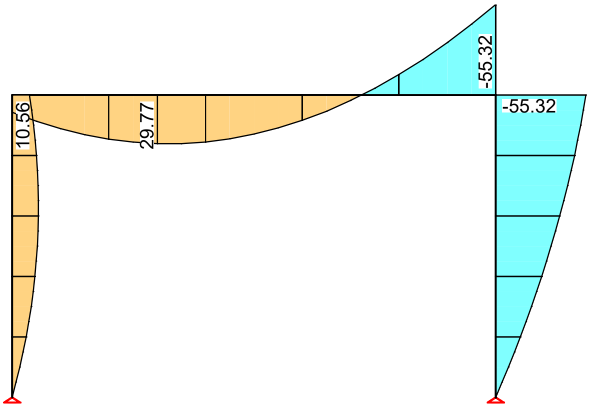


Maximal deformations uz,max under the 1st quasi-continuous situation [mm].

Lateral Torsional Buckling Check of a Two-Hinged Frame

The input values correspond to example 2b on page 79 from the essay of W. Martin (1996).

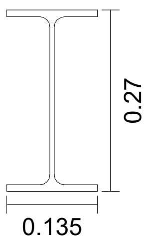




Log Printout:

Lateral Torsional Buckling as per EN 1993-1-1:2014-07

Cross section: IPE 270



h = 270.0 mm
b = 135.0 mm
s = 6.6 mm
t = 10.2 mm
r = 15.0 mm

$A = 45.9\ cm^2$	$I_y = 5790.0\ cm^4$	$I_z = 420.0\ cm^4$
$I_T = 16.0\ cm^4$	$I_\omega = 70580\ cm^6$	
$i_y = 11.23\ cm$	$i_z = 3.02\ cm$	$i_p = 11.63\ cm$

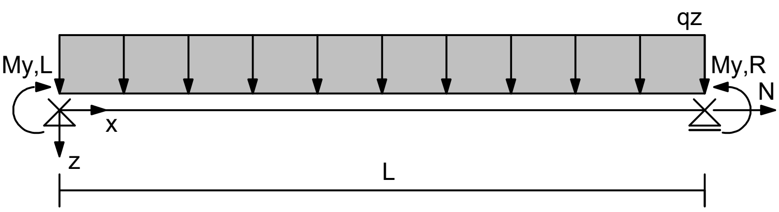
Material: S235-EN

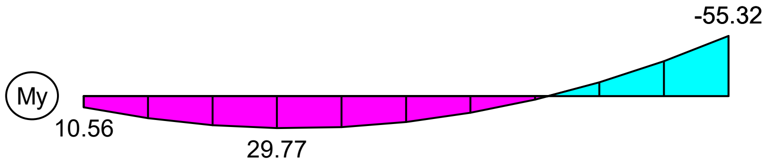
$f_{y,k} = 235\ N/mm^2$
E-Modulus = 210000 N/mm²
G-Modulus = 81000 N/mm²
$\gamma_{M1} = 1.00$

Load in z-direction

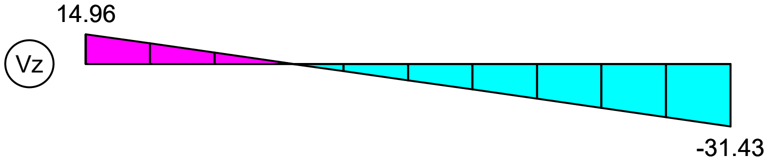
$L = 8.00\ m$	$N_d = 0.00\ kN$	$q_{z,d} = 5.80\ kN/m$
$M_{y,d,left} = 10.56\ kNm$	$M_{y,d,right} = -55.32\ kNm$	

The load application point is in the shear center $\Rightarrow$ $z_p = 0.00\ cm$
The ends of the beam are forked supported.

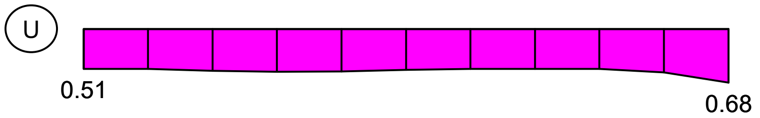




Max My,d = 29.77 kNm, Min My,d = -55.32 kNm



Max Vz,d = 14.96 kN, Min Vz,d = -31.43 kN



Maximum utilization = 0.68

Elastic flexural buckling force $N_{cr,y}$ about the y axis

$$N_{cr,y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr,y}^2}$$

E = 210000 N/mm² I_y = 5790.0 cm^4 $L_{cr,y}$ = 8.00 m

$N_{cr,y}$ = 1875.07 kN

Elastic flexural buckling force $N_{cr,z}$ about the z axis

$$N_{cr,z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{cr,z}^2}$$

E = 210000 N/mm² I_z = 420.0 cm^4 $L_{cr,z}$ = 8.00 m

$N_{cr,z}$ = 136.02 kN

Critical load factor η_{cr} for the lateral torsional buckling moment M_{cr}

η_{cr} = 1.97

Decisive check at location x = 8.00 m

Design loading

N_{Ed} = 0.00 kN
$V_{z,Ed}$ = -31.43 kN $V_{y,Ed}$ = 0.00 kN
$M_{y,Ed}$ = -55.32 kNm $M_{z,Ed}$ = 0.00 kNm

$\Rightarrow$ Section class 1

Plastic resistances as per Table 6.7

N_{Rk} = ±1078.65 kN
$M_{y,Rk}$ = ±113.74 kNm $M_{z,Rk}$ = ±22.78 kNm

Elastic critical moment M_{cr} for lateral torsional buckling

$$M_{cr} = \eta_{cr} \cdot M_{y,d}$$

M_{cr} = 1.97 · (-55.32) = -108.73 kNm

Reduction factor for lateral torsional buckling as per Chapter 6.3.2.3

$$\chi_{LT} = \frac{1}{\Theta_{LT} + \sqrt{\Theta_{LT}^2 - \beta\overline{\lambda}_{LT}^2}} \le \frac{1}{\overline{\lambda}_{LT}^2} \le 1 \qquad (6.57)$$

$$\Theta_{LT} = 0.5\left[1 + \alpha_{LT}\left(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}\right) + \beta\overline{\lambda}_{LT}^2\right]$$

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

with $\overline{\lambda}_{LT}$ = 1.02 $\overline{\lambda}_{LT0}$ = 0.40

β = 0.75 α_{LT} = 0.34 as per Tables 6.3 and 6.5

M_{cr} = -108.73 kNm $W_{pl,y}$ = 4.8400e-4 m^3 as per Table 6.7

χ_{LT} = 0.69

Modification as per Equation (6.58)

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} \le 1 \qquad (6.58)$$

$$f = 1 - 0.5 \cdot (1 - k_c) \cdot [1 - 2(\overline{\lambda}_{LT} - 0.8)^2] \le 1$$

f = 0.96 k_c = 0.91 as per Table 6.6

$\chi_{LT,mod}$ = 0.71

Lateral torsional buckling check as per Chapter 6.3.2

$$\frac{M_{Ed}}{M_{b,Rd}} \le 1 \qquad (6.54)$$

$$M_{b,Rd} = \chi_{LT} \cdot W_y \frac{f_y}{\gamma_{M1}} \qquad (6.55)$$

with M_{Ed} = -55.32 kNm W_y = 4.8400e-5 m^3

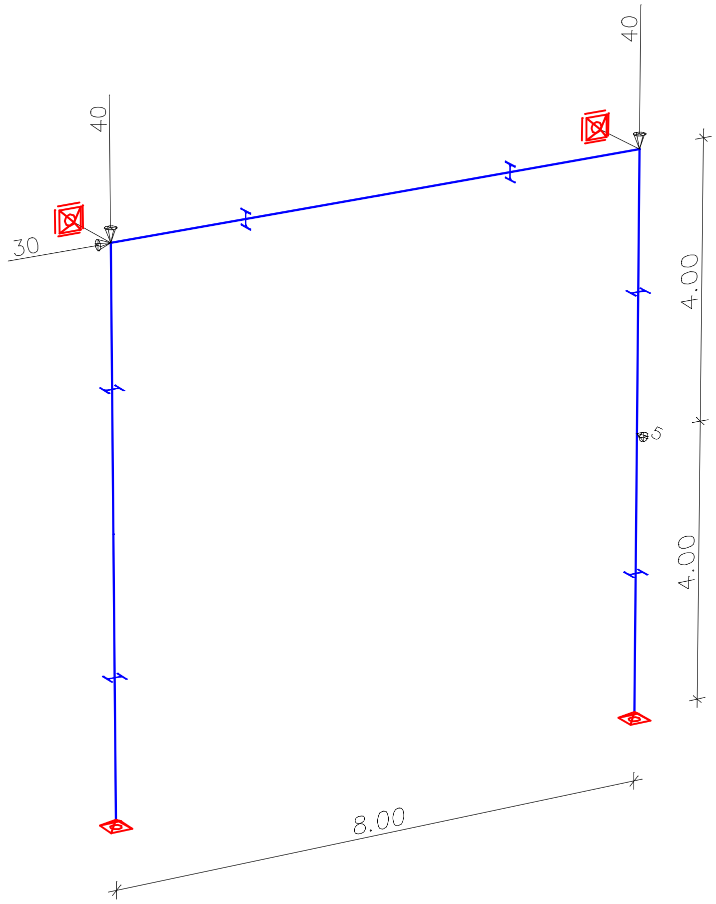
γ_{M1} = 1.00 f_y = 235 N/mm²

$\chi_{LT} = \chi_{LT,mod}$ = 0.71

M_{Ed} / $M_{b,Rd}$ = 0.68 ≤ 1 **Check ok !**

Lareral Torsional Buckling Check of a Frame Column With Two-Axis Bending and Normal Force

The input values correspond to example 7c (check with corresponding internal forces) on page 262 from the book of J. Meister (2002).

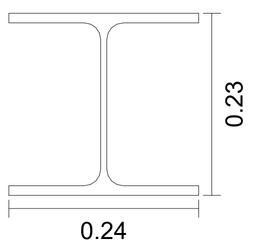


The right frame column is checked.
γ_M= 1.00 (simplified assumption for the example)

Log Printout:

Lateral Torsional Buckling as per EN 1993-1-1:2014

Cross section: HEA 240



h = 230.0 mm
b = 240.0 mm
s = 7.5 mm
t = 12.0 mm
r = 21.0 mm

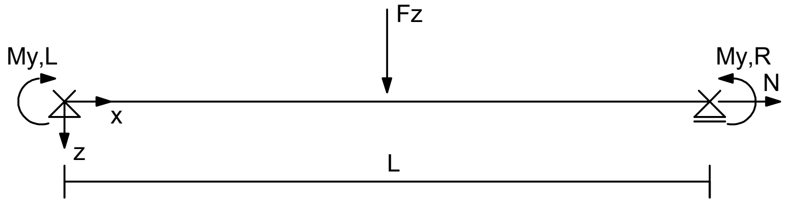
A = 76.8 cm^2	I_y = 7760.0 cm^4	I_z = 2770.0 cm^4
I_T = 41.7 cm^4	I_ω = 328500 cm^6	
i_y = 10.05 cm	i_z = 6.01 cm	i_p = 11.71 cm

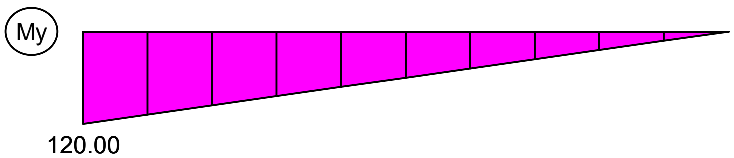
Material: S235-EN

$f_{y,k}$ = 235 N/mm²
E-Modulus = 210000 N/mm²
G-Modulus = 81000 N/mm²
γ_{M1} = 1.00

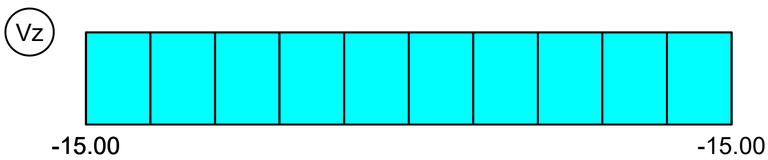
Load in z-direction

L = 8.00 m N_d = -70.00 kN $F_{z,d}$ = 0.00 kN
$M_{y,d,left}$ = 120.00 kNm $M_{y,d,right}$ = 0.00 kNm
The load application point is in the shear center $\Rightarrow$ z_p = 0.00 cm
The ends of the beam are forked supported.

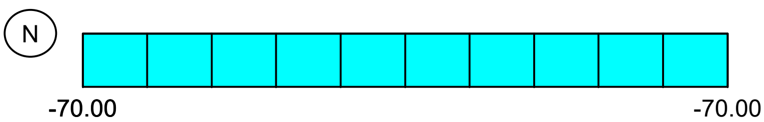




Max My,d = 120.00 kNm



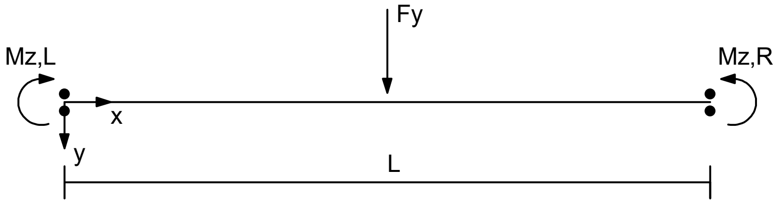
Max Vz,d = -15.00 kN

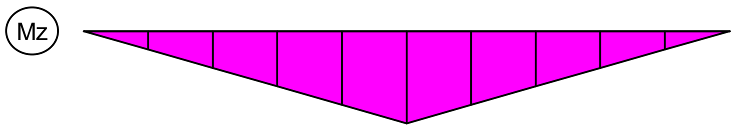


Min N,d = -70.00 kN

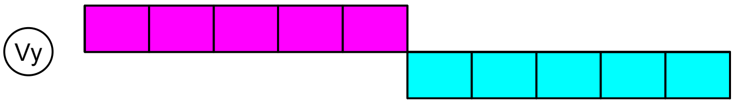
Load in y-direction

L = 8.00 m $F_{y,d}$ = 5.00 kN
$M_{z,d,left}$ = 0.00 kNm; $M_{z,d,right}$ = 0.00 kNm

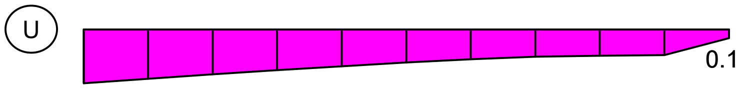




Max Mz,d = 10.00 kNm



Max Vy,d = 2.50 kN, Min Vy,d = -2.50 kN



Maximum utilization = 0.69

Elastic flexural buckling force $N_{cr,y}$ about the y axis

$$N_{cr,y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr,y}^2}$$

E = 210000 N/mm² I_y = 7760.0 cm^4 $L_{cr,y}$ = 8.00 m

$N_{cr,y}$ = 2513.05 kN

Elastic flexural buckling force $N_{cr,z}$ about the z axis

$$N_{cr,z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{cr,z}^2}$$

E = 210000 N/mm² I_z = 2770.0 cm^4 $L_{cr,z}$ = 8.00 m

$N_{cr,z}$ = 897.05 kN

Critical load factor η_{cr} for the lateral torsional buckling moment M_{cr}

η_{cr} = 3.03

Decisive check at location x = 0.00 m

Design loading

N_{Ed} = -70.00 kN
$V_{z,Ed}$ = -15.00 kN $V_{y,Ed}$ = 2.50 kN
$M_{y,Ed}$ = 120.00 kNm $M_{z,Ed}$ = 0.00 kNm

$\Rightarrow$ Section class 1

Plastic resistances as per Table 6.7

N_{Rk} = ±1804.80 kN
$M_{y,Rk}$ = ±174.99 kNm $M_{z,Rk}$ = ±82.65 kNm

Reduction factors for buckling as per Chapter 6.3.1

$$\chi = \frac{1}{\Theta + \sqrt{\Theta^2 - \overline{\lambda}^2}} \leq 1 \qquad (6.49)$$

$$\Theta = 0.5\left[1 + \alpha\left(\overline{\lambda} - 0.2\right) + \overline{\lambda}^2\right]$$

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

with $\overline{\lambda}_y = 0.85$ $\quad\overline{\lambda}_z = 1.42$
$\alpha_y = 0.34$ $\quad\alpha_z = 0.49$ $\quad$ as per Tables 6.1 and 6.2
$N_{cr,y} = 2513.05$ kN $\quad N_{cr,z} = 897.05$ kN

$\chi_y = 0.69$
$\chi_z = 0.34$

Lateral buckling check as per Chapter 6.3.1

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \qquad (6.46)$$

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \qquad (6.47)$$

with $N_{Ed} = -70.00$ kN $\quad f_y = 235$ N/mm²
$A = 76.80$ cm^2 $\quad\gamma_{M1} = 1.00$
$\chi_y = 0.69$ $\quad\chi_z = 0.34$

N_{Ed} / $N_{b,y,Rd}$ = 0.06 ≤ 1
N_{Ed} / $N_{b,z,Rd}$ = 0.11 ≤ 1 **Check ok !**

Elastic critical moment M_{cr} for lateral torsional buckling

$$M_{cr} = \eta_{cr} \cdot M_{y,d}$$

$M_{cr} = 3.03 \cdot 120.00 = 363.60$ kNm

Reduction factor for lateral torsional buckling as per Chapter 6.3.2.3

$$\chi_{LT} = \frac{1}{\Theta_{LT} + \sqrt{\Theta_{LT}^2 - \beta\overline{\lambda}_{LT}^2}} \leq \frac{1}{\overline{\lambda}_{LT}^2} \leq 1 \qquad (6.57)$$

$$\Theta_{LT} = 0.5\left[1 + \alpha_{LT}\left(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}\right) + \beta\overline{\lambda}_{LT}^2\right]$$

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

with $\overline{\lambda}_{LT} = 0.69$ $\quad\overline{\lambda}_{LT0} = 0.40$
$\beta = 0.75$ $\quad\alpha_{LT} = 0.34$ $\quad$ as per Tables 6.3 and 6.5
$M_{cr} = 363.60$ kNm $\quad W_{pl,y} = 7.4462\text{e-}4$ m^3 $\quad$ as per Table 6.7

$\chi_{LT} = 0.87$

Modification as per Equation (6.58)

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} \leq 1 \qquad (6.58)$$

$$f = 1 - 0.5 \cdot (1 - k_c) \cdot [1 - 2(\bar{\lambda}_{LT} - 0.8)^2] \leq 1$$

$f = 0.88$ $\quad k_c = 0.75$ $\quad$ as per Table 6.6

$\chi_{LT,mod} = 0.99$

Lateral torsional buckling check as per Chapter 6.3.2

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1 \qquad (6.54)$$

$$M_{b,Rd} = \chi_{LT} \cdot W_y \frac{f_y}{\gamma_{M1}} \qquad (6.55)$$

with $M_{Ed} = 120.00$ kNm $\quad W_y = 7.4462\text{e-}4\ m^3$
$\gamma_{M1} = 1.00$ $\quad f_y = 235$ N/mm²
$\chi_{LT} = \chi_{LT,mod} = 0.99$

M_{Ed} / $M_{b,Rd}$ = 0.69 $\leq$ 1 $\quad$ **Check ok !**

Interaction factors as per Annex B

Equivalent moment factors as per Table B.3

$\psi_y = 0.00$ $\quad \psi_z = 0.00$
$\quad \alpha_{h,z} = 0.00$
$C_{my} = 0.60$ $\quad C_{mz} = 0.90$
$C_{mLT} = 0.60$

Interaction factors as per Table B.1 for members not susceptible to torsional deformations
For I-sections under axial compression and uniaxial bending $M_{y,Ed}$ the value $k_{zy} = 0$ may be assumed
$k_{yy} = 0.62$ $\quad k_{yz} = 0.63$
$k_{zy} = 0.00$ $\quad k_{zz} = 1.04$

Lateral torsional buckling check as per Chapter 6.3.3

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \qquad (6.61)$$

$$\frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \qquad (6.62)$$

with $N_{Ed} = -70.00$ kN $\quad M_{y,Ed} = 120.00$ kNm $\quad M_{z,Ed} = 0.00$ kNm
$\quad \Delta M_{y,Ed} = 0.00$ kNm $\quad \Delta M_{z,Ed} = 0.00$ kNm
$N_{Rk} = -1804.80$ kN $\quad M_{y,Rk} = 174.99$ kNm $\quad M_{z,Rk} = 82.65$ kNm
$\chi_y = 0.69$ $\quad k_{yy} = 0.62$ $\quad k_{yz} = 0.63$
$\chi_z = 0.34$ $\quad k_{zy} = 0.00$ $\quad k_{zz} = 1.04$
$\gamma_{M1} = 1.00$ $\quad \chi_{LT} = \chi_{LT,mod} = 0.99$

0.06 + 0.43 + 0.00 = 0.49 $\leq$ 1 $\quad$ (6.61)
0.11 + 0.00 + 0.00 = 0.11 $\leq$ 1 $\quad$ (6.62) $\quad$ **Check ok !**

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