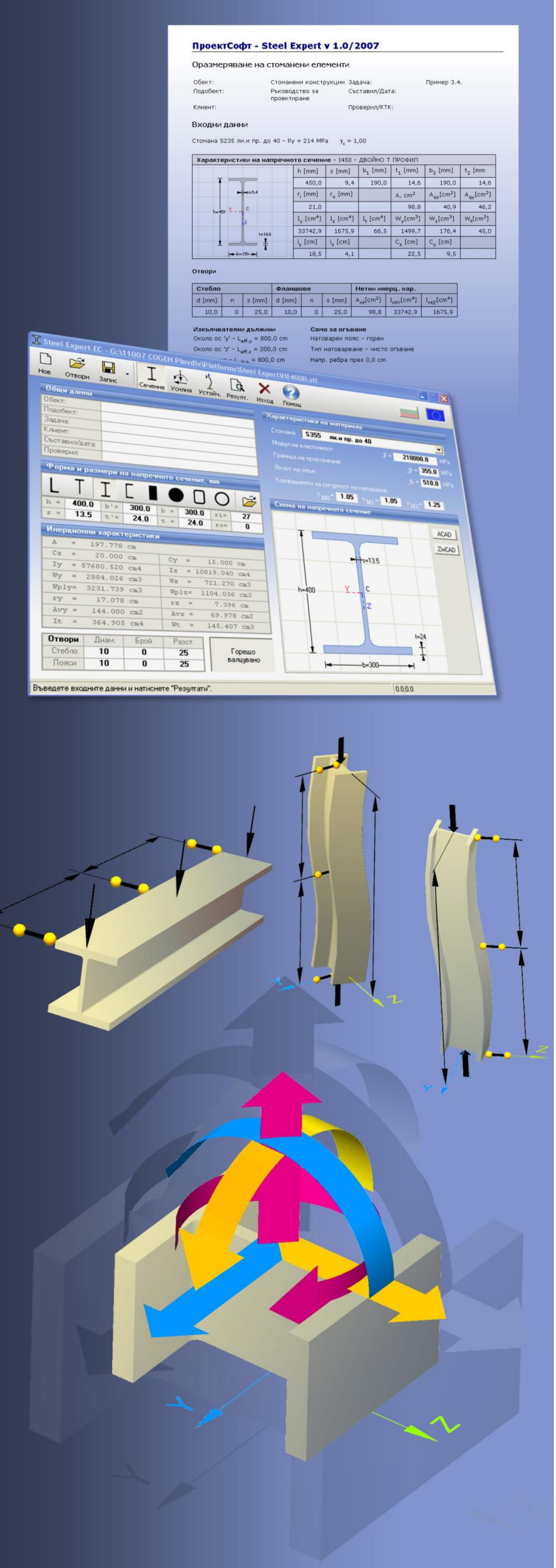

ПРОЕКТСОФТ

Software Package

Design Expert version 2.0

Steel Expert EC

Design of steel elements

User Manual

2011

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About the program

This program is created for design of steel elements according to Eurocode 3 (EN1993-1-1:2005) and Bulgarian Code for Steel Structures. Calculations include elastic and plastic section design and local and overall element buckling for combined action of axial forces (tension or compression), bending moments in two direction, shear forces and torsion. Eight different types of sections are available ⌊ ⊤ I [● ○ ▮ ▯. Program includes rich library of standard steel sections. An option to define holes in webs and flanges is also available. Results are presented in professional **html** report for viewing and printing.

Entering data

Program data is divided into several pages:

	Materials and Sections Data		Stability Data
	Loading Data		Design Results

Click the respective button to switch between pages. The "**Results**" button starts calculations and generates an **html** report, which is displayed on screen. If file is not saved, the user is prompted to do that. Input values are entered in tables or text fields on each page. You can move to the next field by mouse click or with the **Tab** key. You can go back to the previous field with **Shift+Tab** combination.

Files

Input data for each problem is saved in a file with ***.stl** extension, and the results are written to a ***.stl.html** file. File open and save are done with the respective "**Open**" and "**Save**" commands. A standard dialog is displayed, where you should select file name and path. If the file has already been saved, the "**Save**" command will use current file name without prompting. The "**New**" command clears current data. You can save the file under different name by clicking the arrow next to the "**Save As...**" button.

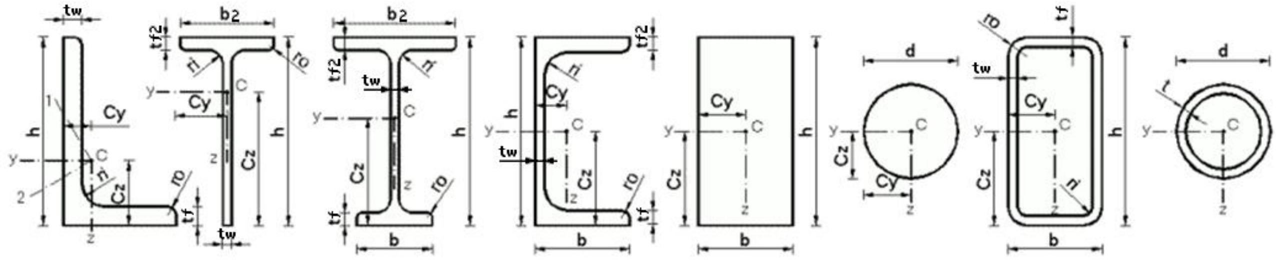
Input Data

Materials

Select steel grade from "**Steel**" combo box. Program automatically fills in the respective strength properties f_{yk} and f_{uk}. You can also input custom values for f_y and f_u, if steel grade is not available in the list. Partial safety factors are also required. Default values are: $\gamma_{M0} = 1.05$, $\gamma_{M1} = 1.05$, $\gamma_{M2} = 1.25$.

Cross sections

Eight different types of shapes are included. Sections can be standard - hot-rolled, cold formed, built-up, welded etc. For non-standard cross sections the section type is selected from the respective button (⌊ ⊤ I [▮ ● ▯ ○) and dimensions are entered by the user. Notations for dimensions and axes for different types of steel sections are as follows:



Section area properties are calculated precisely including fillets. **There is no option for tapered flanges, so they should be entered as parallel ones by entering the average thickness.** In this case section properties are approximate. Torsional properties are calculated by approximate formulas with precision of 1% – 2%.

Standard cross sections are selected from the "**Steel Sections Library**" tables. Library is opened with the button which can be found next to section types. Select section type according to the respective standard - European, Russian or Bulgarian.

European

- Hot rolled equal angles to EN 10056-1
- Hot rolled unequal angles to EN 10056-1
- Hot rolled normal channels to NF A 45-202
- Hot rolled I-sections IPE to EN 89
- Hot rolled wide flange I-sections HE to EN 53-62
- Hot rolled circular hollow sections CHS to EN 10210-2
- Hot rolled square hollow sections SHS to EN 10210-2
- Hot rolled rectangular hollow sections RHS to EN 10210-2
- Cold formed square hollow sections SHS to EN 10219-2
- Cold formed rectangular hollow sections RHS to EN 10219-2

Russian

- Hot rolled unequal angles to GOST 8510-72
- Hot rolled I-sections with paralel flanges to GOST 2620-83
- Hot rolled wide flange I-sections to GOST 2620-83
- Hot rolled I-columns to GOST 2620-83

Bulgarian

- Hot rolled equal angles to BDS 2612-73
- Hot rolled channels with tapered flanges to BDS 6176-75
- Hot rolled channels with paralel flanges to BDS 6176-75
- Hot rolled I-sections to BDS 5951-75

Section dimensions and properties are displayed in tables. Select a section with the mouse, and click the “**Load**” button. Section dimensions are loaded in the program, and area properties are calculated as for non-standard sections. Presented table properties are only for information. Library sections can be additionally modified after loading. For example with this method you can enter a T-section made by cutting a standard I-section in two. Select the I-section from the library, load it into the program and modify section type and height.

Holes/openings in sections

Holes in webs and flanges are entered by specifying diameter, number and distance between holes. Section preview is updated with the specified holes. Holes outside the section are colored in red. Holes with spacing and edge distance smaller than required are colored in yellow. This is a warning to the user that distances are non-compliant to code requirements for bolt connections, but this does not stop further input because holes may have another purpose. Specified holes are considered in section analysis. Holes are not considered in element buckling checks.

Loadings

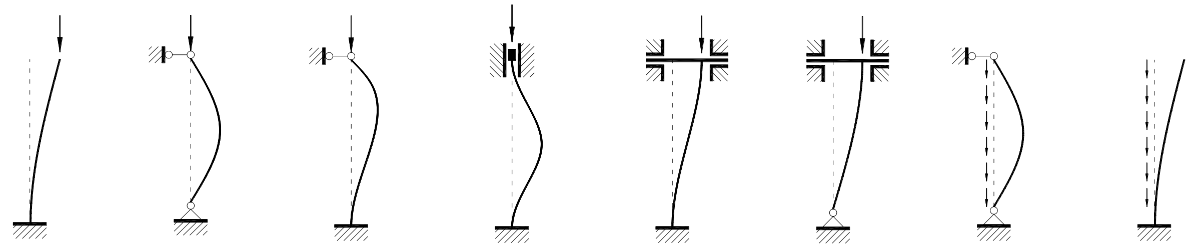
Axial force ***N***, bending moments M_y, M_z, shear forces Q_y, Q_z and torsional moment ***T*** are entered. You can have one, several or all of the above loads acting separately or simultaneously. Different combinations of design loads are possible. Positive axial force is tension.

Effective lengths

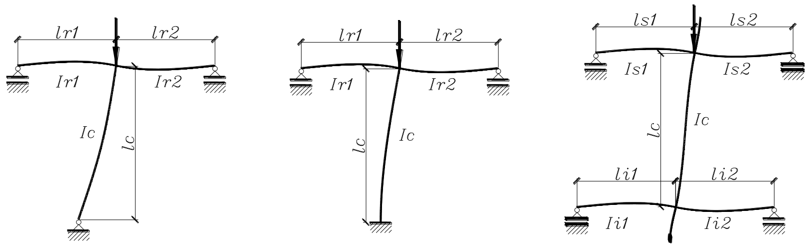
Effective lengths are required for buckling analysis of steel and reinforced concrete elements.

Buckling factors for the two main planes μ_y, μ_z shall be entered and the program calculates the respective effective lengths L_{efy}, L_{efz} .You can also input distances between restraints. Recommendations for buckling factor values for different types of structural elements are given in the respective design codes.

The button opens the "Buckling calculator" dialog, where μ is calculated depending on the selected support conditions. The procedure for columns in frames is also implemented.



μ =2.0 μ =1.0 μ =0.7 μ =0.5 μ =1.0 μ =2.0 μ =0.725 μ =1.12



Effective length for lateral-torsional buckling is defined as the distance between lateral restraints of compressed flange. Load position (top flange, neutral or bottom flange) and load type (end moments, distributed, concentrated) are also required as well as the shapes of bending moment diagram. Local buckling of beam webs is also checked. Stiffening ribs can be defined and rib spacing ***a*** should be entered.

Design to Bulgarian code

All required checks for strength, global and local buckling for the web and flanges are performed. Design procedures are developed in accordance with "Design Code for Steel Structures" - 1987, for spatial frame elements. Design checks are preformed independently for all load combinations. Detailed design results are displayed in a professional **html** report.

Design checks for strength

Strength design checks are performed in elastic state with no allowance for plastic deformations. Stress is calculated for specific points based on the internal forces and section properties.

$$\sigma_x(\boldsymbol{y},\boldsymbol{z}) = \mathbf{N}/\boldsymbol{A}_{nt} \pm \boldsymbol{M}_y \cdot \boldsymbol{z}/\boldsymbol{I}_{y,nt} \pm \boldsymbol{M}_z \cdot \boldsymbol{y}/\boldsymbol{I}_{z,nt}$$

$$\tau_{xz}(\boldsymbol{y}) = \alpha \cdot \boldsymbol{Q}_z \cdot \boldsymbol{S}_z(\boldsymbol{y})/\boldsymbol{b}(\boldsymbol{y}) * \boldsymbol{I}_y \qquad \alpha = \boldsymbol{s}_w / (\boldsymbol{s}_w - \boldsymbol{d}_w)$$

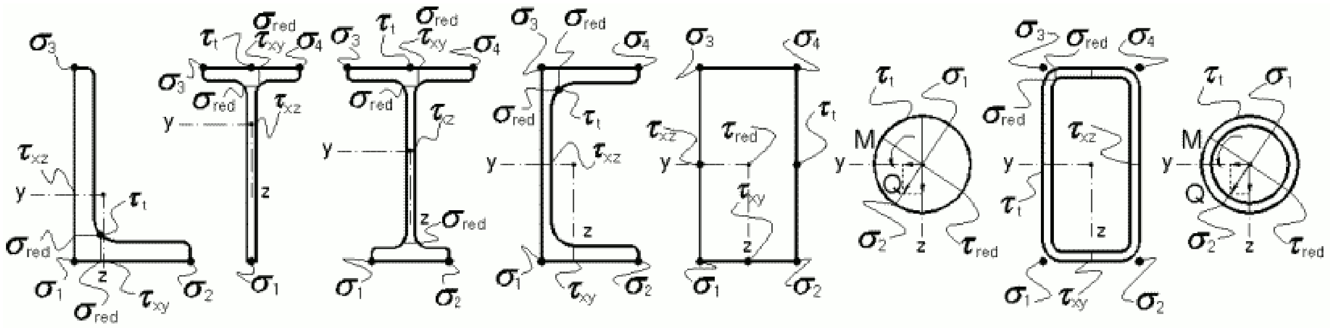
$$\tau_{xy}(\boldsymbol{z}) = \alpha \cdot \boldsymbol{Q}_y \cdot \boldsymbol{S}_y(\boldsymbol{z})/\boldsymbol{b}(\boldsymbol{z}) * \boldsymbol{I}_z \qquad \alpha = \boldsymbol{s}_f / (\boldsymbol{s}_f - \boldsymbol{d}_f)$$

$$\tau_{red} = \sqrt{\tau_{xy}^2 + \tau_{xz}^2}$$

$$\sigma_{red} = \sqrt{\sigma_x^2 + 3\tau_{red}^2}$$

$$\tau_t = \boldsymbol{T}/\boldsymbol{W}_t$$

If there are holes (openings), strength design checks are performed for the effective section properties. Stress is calculated in the following points depending on section type:



Maximum normal, shear and effective stresses are calculated for all points:

$$\sigma_x = \max(|\sigma_1|,|\sigma_2|,|\sigma_3|,|\sigma_4|) \qquad \tau_x = \max(|\tau_{xy}|,|\tau_{xz}|,|\tau_t|,|\tau_{red}|)$$

For unsymmetrical I sections shear force $\boldsymbol{Q}_y$ is distributed between top and bottom flange as follows:

$$\boldsymbol{Q}_{y1} = \boldsymbol{Q}_y \cdot \boldsymbol{I}_f/(\boldsymbol{I}_f + \boldsymbol{I}_f') \qquad \boldsymbol{Q}_{y2} = \boldsymbol{Q}_y \cdot \boldsymbol{I}_f/(\boldsymbol{I}_f + \boldsymbol{I}_f')$$

Final result is presented by the following bearing capacity factor:

$$K = \frac{\text{Design stress}}{\text{Material resistance}}$$

Checks are displayed in tabular form:

STRESS	σ_X	τ_{XY}	τ_{XZ}	τ_{MAX}	σ_{RED}
check	$\sigma_x < \gamma_c R_y$	$\tau_{xy} < 0.58\gamma_c R_y$	$\tau_{xz} < 0.58\gamma_c R_y$	$\tau_{max} < 0.58\gamma_c R_y$	$\sigma_{red} < 1.15\gamma_c R_y$
K factor	$\frac{\sigma_x}{\gamma_c \cdot R_y}$	$\frac{\tau}{\gamma_c \cdot 0.58 \cdot R_y}$			$\frac{\sigma_{red}}{\gamma_c \cdot 1.15 \cdot R_y}$

Global Buckling

Global buckling checks are performed for separate or combined compression and bending. The following buckling factors are calculated:

- $\varphi = \min(\varphi_y, \varphi_z)$ – for compression only;
- φ_{ey} – for compression and bending about **y** axis;
- φ_{zc} – buckling about **z** axis for compression and bending about **y** axis;
- $\varphi_{e,z}$ – for compression and bending about **z** axis;
- $\varphi_{e,yz}$ – for compression with biaxial bending;
- φ_b – for lateral-torsional buckling.

Global buckling checks are displayed in the form of - $\{\sigma\} < \gamma_c R_y$

$\{\sigma\}=$	$\frac{N}{\varphi \cdot A}$	$\frac{N}{\varphi_{EY} \cdot A}$	$\frac{N}{\varphi_Z \cdot C \cdot A}$	$\frac{N}{\varphi_{EZ} \cdot A}$	$\frac{N}{\varphi_{EYZ} \cdot A}$	$\frac{M}{\varphi_B \cdot W_C}$
K - factor	$\frac{\{\sigma\}}{\gamma_c \cdot \boldsymbol{R}_y}$					

Local Buckling

Local buckling design for web and flanges is performed for both bending and compression and more conservative results are used. Local buckling checks are not performed for rectangular or circular solid sections. The equation $\boldsymbol{r}/\boldsymbol{t} \le 3.14\sqrt{(\boldsymbol{E}/\boldsymbol{R}_y)}$ is used for circular tubes. Local transverse load σ_{loc} is not considered for buckling of beam webs. If "Allow web local buckling" option is active and web buckling checks are not satisfied, then calculations are repeated, considering only effective zones with width $0{,}65.\boldsymbol{t}.\sqrt{\boldsymbol{E}/\boldsymbol{R}_y}$ at both ends of the web. Middle web zone that has buckled is excluded.

Design results are the maximum ratios $\boldsymbol{b}_f/\boldsymbol{t}_f$ and $\boldsymbol{h}/\boldsymbol{t}_w$. From these values minimum thicknesses of flanges and webs are determined and compared to the real values $\boldsymbol{t}_{f,min}<\boldsymbol{t}_f$ и $\boldsymbol{t}_{w,min}<\boldsymbol{t}_w$.

Examples to Bulgarian code

Example 3.1.

Calculate bearing capacity of roof truss compressed chord, made from equal angles ⅂Г100.100.8 with in-plane and out-plane buckling lengths $\boldsymbol{l}_x = \boldsymbol{l}_y$ = 3m. Thickness of the gusset plates is 10 mm. Steel grade is ВСт3пс5.

Solution

Steel strength:	$\boldsymbol{R}_y$ = 225 MPa	$\gamma_c \cdot R_y = 0.95 \cdot 225 = 214$ MPa
Cross section area:	$\boldsymbol{A} = 2 \cdot \boldsymbol{A}_1 = 31.2$ cm^2	
Radius of gyration:	$\boldsymbol{i}_y = 3.07$ cm, $\boldsymbol{i}_z = 4.47$ cm	
Maximum element slenderness:	$\lambda_{max} = 300/3.07 = 98 < \lambda_u = 180$	
Effective slenderness:	$\bar{\lambda} = \lambda \cdot (\boldsymbol{R}_y/\boldsymbol{E})^{1/2} = 98 \cdot (214/2.06 \cdot 10^5)^{1/2} = 3{,}16$	
Buckling factor:	$\varphi = 1.47 - 13 \cdot (214/2.06 \cdot 10^5) - (0.371 - 27.3 \cdot (214/2.06 \cdot 10^5)) \cdot 3.16 + (0.0275 - 5.53 \cdot (214/2.06 \cdot 10^5)) \cdot 3.16^2 = 0.59$	

Bearing capacity of the element is $\boldsymbol{F} = \varphi \cdot \boldsymbol{A} \cdot \gamma_c \cdot \boldsymbol{R}_y = 0.59 \cdot 31.2 \cdot 21.4 = 394$ kN

This example is checked with Steel Expert. Bearing capacity is entered as external loading and capacity ratio shall be close to 1.0. There is no option for built-up section form pair of angles. That is why section is defined as T section.

Proektsoft - Steel Expert EC 2.0/2010

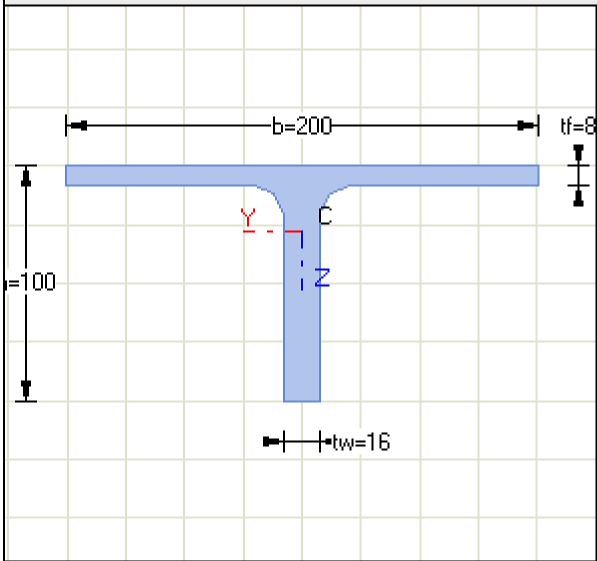
Steel Element Design To NPSK'87

Project:	Element:
Building:	Author/Date:
Client:	Checked By:

Input Data

Steel - Ry = 225 MPa γ_c = 0.95

Section Dimensions And Properties - L0 - TEE



h [mm]	t_w [mm]			b [mm]	t_f [mm]
100.0	16.0			200.0	8.0
r_i [mm]	r_o [mm]	A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
12.0		31.3	16.0	11.1	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
298.2	537.2	41.2	53.7	83.8	86.5
i_y [cm]	i_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
3.1	4.1	7.2	10.0	8.4	21.6

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 300.0cm

About axis "z" - $L_{eff,z}$ = 300.0cm

LT buckling - $L_{eff,b}$ = 0.0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - End moments

Web stiffeners at 0.0cm

Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	-394.0	0.0	0.0	0.0	0.0	0.0

Design Results

Stress Checks

Case	σ_x	τ_{xy}	τ_{xz}	τ_{max}	σ_{red}
1	125.7	0.0	0.0	0.0	125.7

Case	$\sigma_x / \gamma_c R_y$	$\tau_{xy} / 0.58\gamma_c R_y$	$\tau_{xz} / 0.58\gamma_c R_y$	$\tau_{max} / 0.58\gamma_c R_y$	$\sigma_{red} / 1.15\gamma_c R_y$
1	0.59	0.00	0.00	0.00	0.51

Element Buckling - $\{\sigma\} < \gamma_c R_y$

λ_y	λ_z	φ	φ_{ey}	$\varphi_z c$	$\varphi_{e,z}$	$\varphi_{e,yz}$	φ_b
97.3	72.5	0.58	0.00	0.00	0.00	0.00	1.00

Case	$\{\sigma\}$	$\frac{N}{\varphi A}$	$\frac{N}{\varphi_{ey} A}$	$\frac{N}{\varphi_z c A}$	$\frac{N}{\varphi_{e,z} A}$	$\frac{N}{\varphi_{e,yz} A}$	$\frac{M}{\varphi_b W_c}$
1	[MPa]	216.4	0.0	0.0	0.0	0.0	0.0
	Ratio	1.01	0.00	0.00	0.00	0.00	0.00

Local Buckling

max b/t_f = 21.0

$t_{f,min}$ = 3.8 mm < t_f

max h/t_f = 18.9

$t_{w,min}$ = 4.2 mm < t_w

Design checks are not satisfied: K = 1.01

Difference to manual calculations is 1.3%.

Example 3.2.

Design a steel column of pipe rack with height 720 cm, loaded with compressive force ***F***=1000 KN. Column is fixed to the foundation about its stronger axis and is hinged about the other axis. Top of the column is with hinged supports in both directions. There are additional lateral supports at 1/3 and 2/3 of column height about the weaker axis.

Solution

Column section is selected to be an **I** section from steel ВСт2пс6 - ГОСТ 380-71,

Steel strength is R_y = 235 MPa for thickness 4-20 mm.

Buckling lengths: $l_{ef,y} = \mu_x \cdot h = 2 \cdot 720 = 1440$ cm

$l_{ef,z} = h / 3 = 720/3 = 240$ cm

Try section I40 with the following properties:

Cross section area: $A = 72.6$ cm^2

Radius of gyration: $i_y = 16.2$ cm, $i_z = 3.03$ cm

Maximum element slenderness: $\lambda_{max} = 1440/16.2 = 89 < \lambda_u = 180$

Effective slenderness: $\bar{\lambda} = \lambda \cdot (R_y/E)^{1/2} = 89 \cdot (235/2.06 \cdot 10^5)^{1/2} = 3.00$

Buckling factor: $\varphi = 1.47 - 13 \cdot (235/2.06 \cdot 10^5) - (0.371 - 27.3 \cdot (235/2.06 \cdot 10^5)) \cdot 3.00 + (0.0275 - 5.53 \cdot (235/2.06 \cdot 10^5)) \cdot 3.00^2 = 0.63$

Normal stresses are $\sigma = F/(\varphi \cdot A) = 1000/(0.63 \cdot 72.6) = 21.86 < \gamma_c \cdot R_y = 23.5$ kN

Bearing capacity of the element is $k = \sigma/\gamma_c \cdot R_y = 21.86/23.5 = 0.93$

This example is checked with Steel Expert. Since there is no option to define tapered flanges, section is defined approximately with parallel flanges with mean thickness.

Proektsoft - Steel Expert EC 2.0/2010

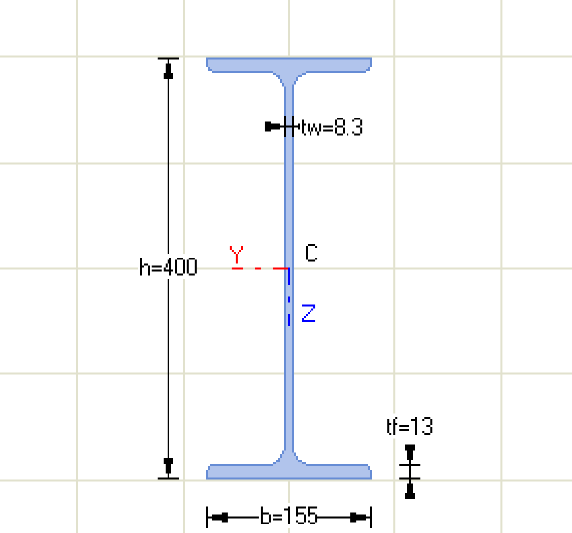
Steel Element Design To NPSK'87

Project:	Element:
Building:	Author/Date:
Client:	Checked By:

Input Data

Steel - Ry = 235 MPa γ_c = 1.00

Section Dimensions And Properties - I 40 - I-SECTION



h [mm]	t_w [mm]	b [mm]	t_f [mm]	b_2 [mm]	t_{f2} [mm]
400.0	8.3	155.0	13.0	155.0	13.0
r_i [mm]	r_o [mm]	A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
15.0	6.0	73.0	32.1	33.6	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
19255.2	791.9	962.8	102.2	1099.7	161.7
i_y [cm]	i_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
16.2	3.3	20.0	7.8	35.9	26.7

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 1440.0cm

About axis "z" - $L_{eff,z}$ = 240.0cm

LT buckling - $L_{eff,b}$ = 240.0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - Uniformly distributed

Web stiffeners at 0.0cm

Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	-1000.0	0.0	0.0	0.0	0.0	0.0

Design Results

Stress Checks

Case	σ_x	τ_{xy}	τ_{xz}	τ_{max}	σ_{red}
1	137.1	0.0	0.0	0.0	137.1

Case	$\sigma_x / \gamma_c R_y$	$\tau_{xy} / 0.58\gamma_c R_y$	$\tau_{xz} / 0.58\gamma_c R_y$	$\tau_{max} / 0.58\gamma_c R_y$	$\sigma_{red} / 1.15\gamma_c R_y$
1	0.58	0.00	0.00	0.00	0.51

Element Buckling - $\{\sigma\} < \gamma_c R_y$

λ_y	λ_z	φ	φ_{ey}	$\varphi_z c$	$\varphi_{e,z}$	$\varphi_{e,yz}$	φ_b
88.6	72.8	0.63	0.00	0.00	0.00	0.00	1.00

Case	$\{\sigma\}$	$\frac{N}{\varphi A}$	$\frac{N}{\varphi_{ey} A}$	$\frac{N}{\varphi_z c A}$	$\frac{N}{\varphi_{e,z} A}$	$\frac{N}{\varphi_{e,yz} A}$	$\frac{M}{\varphi_b W_c}$
1	[MPa]	218.4	0.0	0.0	0.0	0.0	0.0
	Ratio	0.93	0.00	0.00	0.00	0.00	0.00

Local Buckling

max b/t_f = 20.3

$t_{f,min}$ = 2.9 mm $< t_f$

max h/t_f = 75.5

$t_{w,min}$ = 4.6 mm $< t_w$

Design checks are satisfied: K = 0.93

Difference from manual calculation is (218.6–218.4)/218.6·100 = 0.09%

Example 3.3.

Design a secondary beam from industrial platform under following conditions:

Span length: $\boldsymbol{l}$ = 6 m; spacing between secondary beams: $\boldsymbol{a}$ = 1.5 m; dead load: $\boldsymbol{g}_n$ = 2 kN/m^2; live load: $\boldsymbol{v}_n$ = 5 kN/m^2; steel grade: ВСт2кп , $\boldsymbol{R}_y$ = 215 MPa, thickness: $\boldsymbol{t}$ < 16 mm.

Solution

Characteristic load: $\boldsymbol{q}_n = (\boldsymbol{g}_n+\boldsymbol{v}_n)\cdot\boldsymbol{a} = (2.0+5.0)\cdot1.5 = 10.5$ kN/m

Ultimate load: $\boldsymbol{q} = (\boldsymbol{g}_n\cdot\gamma_{fg}+\boldsymbol{v}_n\cdot\gamma_{fv})\cdot\boldsymbol{a} = (2.0\cdot1.05+5.0\cdot1.2)\cdot1.5=12.15$ kN/m

Bending moment: $\boldsymbol{M} = \boldsymbol{q}\cdot\boldsymbol{l}^2/8 = 12.15\cdot6^2/8 = 54.68$ kN·m

Selected section is **I24** with the following properties

Section modulus: $\boldsymbol{W}$ = 289 cm^3

Bearing capacity of the element is $\sigma = \boldsymbol{M}/\boldsymbol{W} = 5468/289 = 18.9 < \gamma_c\cdot\boldsymbol{R}_y = 21.5$ kN

$k = \sigma/\gamma_c\cdot\boldsymbol{R}_y = 18.9/21.5 = 0.879$

This example is checked with Steel Expert. Since there is no option to define tapered flanges, section is defined approximately with parallel flanges with mean thickness.

Proektsoft - Steel Expert EC 2.0/2010

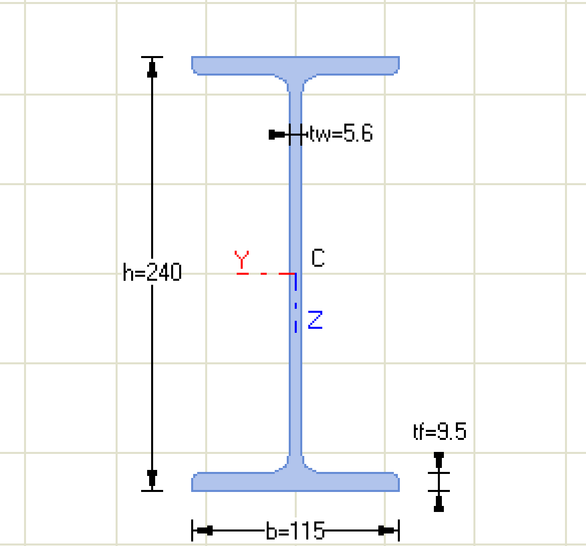
Steel Element Design To NPSK'87

Project:	Element:
Building:	Author/Date:
Client:	Checked By:

Input Data

Steel - Ry = 215 MPa γ_c = 1.00

Section Dimensions And Properties - I 24 - I-SECTION



h [mm]	t_w [mm]	b [mm]	t_f [mm]	b_2 [mm]	t_{f2} [mm]
240.0	5.6	115.0	9.5	115.0	9.5
r_i [mm]	r_o [mm]	A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
11.0	4.0	35.1	12.9	18.2	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
3511.8	237.1	292.7	41.2	329.9	64.3
i_y [cm]	i_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
10.0	2.6	12.0	5.8	9.5	9.6

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 0.0cm

About axis "z" - $L_{eff,z}$ = 0.0cm

LT buckling - $L_{eff,b}$ = 0.0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - Uniformly distributed

Web stiffeners at 0.0cm

Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	0.0	54.7	0.0	0.0	0.0	0.0

Design Results

Stress Checks

Case	σ_x	τ_{xy}	τ_{xz}	τ_{max}	σ_{red}
1	186.9	0.0	0.0	0.0	167.0

Case	$\sigma_x / \gamma_c R_y$	$\tau_{xy} / 0.58\gamma_c R_y$	$\tau_{xz} / 0.58\gamma_c R_y$	$\tau_{max} / 0.58\gamma_c R_y$	$\sigma_{red} / 1.15\gamma_c R_y$
1	0.87	0.00	0.00	0.00	0.68

Element Buckling - $\{\sigma\} < \gamma_c R_y$

λ_y	λ_z	φ	φ_{ey}	$\varphi_z C$	$\varphi_{e,z}$	$\varphi_{e,yz}$	φ_b
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00

Case	$\{\sigma\}$	$\frac{N}{\varphi A}$	$\frac{N}{\varphi_{ey} A}$	$\frac{N}{\varphi_z C A}$	$\frac{N}{\varphi_{e,z} A}$	$\frac{N}{\varphi_{e,yz} A}$	$\frac{M}{\varphi_b W_c}$
1	[MPa]	0.0	0.0	0.0	0.0	0.0	0.0
	Ratio	0.00	0.00	0.00	0.00	0.00	0.00

Local Buckling

max b/t_f = 16.6

$t_{f,min}$ = 2.6 mm $< t_f$

max h/t_f = 0.0

$t_{w,min}$ = 0.0 mm $< t_w$

Design checks are satisfied: K = 0.87

Difference from manual calculation is (0.879-0.869)/0.879·100 = 1.1%

Example 3.4.

Design an eccentrically loaded column with IPE section. Column height is 8.0 m, axial force is ***F*** = 1080 kN and bending moment along strong axis is ***M*** = 120 KN.m. Column is hinged in both ends. Column is laterally restrained at each 2m out of plane of bending. Steel class is S235 to EN-1993-1-1.

Solution

Steel yield strength: $\boldsymbol{R}_y = 235/1.1 = 214$ MPa; $\gamma_c = 1.0$;

Section **IPE 450** is selected with the following properties:

Cross section area: $\boldsymbol{A} = 98.8$ cm2;

Section elastic modulus: $\boldsymbol{W}_y = 1500$ cm2;

Radius of gyration: $\boldsymbol{i}_y = 18.5$ cm; $\boldsymbol{i}_z = 4.12$ cm

Slenderness: $\lambda_y = \boldsymbol{l}_y/\boldsymbol{i}_y = 800/18.5 = 43.2 < \lambda_u = 180$

Effective slenderness: $\bar{\lambda}_y = \lambda_y \cdot (\boldsymbol{R}_y/\boldsymbol{E})^{1/2} = 43.2 \cdot (214/206000)^{1/2} = 1.39$

Effective eccentricity: $\boldsymbol{m} = \boldsymbol{M} \cdot \boldsymbol{A}/(\boldsymbol{F} \cdot \boldsymbol{W}_y) = 12000 \cdot 98.8/(1080 \cdot 1500) = 0.732$

Factor η is calculated trough linear interpolation for ratio:

$\boldsymbol{A}_f/\boldsymbol{A}_w = 190.4 \cdot 14.6/((450-2 \cdot 14.6) \cdot 9.4) = 0.7$

For $\boldsymbol{A}_f/\boldsymbol{A}_w = 0.5$ $\eta = (1.75-0.1 \cdot \boldsymbol{m}) - 0.02 \cdot (5-\boldsymbol{m}) \cdot \bar{\lambda}_y =$

$= (1.75-0.1 \cdot 0.732) - 0.02 \cdot (5-0.732) \cdot 1.39 = 1.558$

For $\boldsymbol{A}_f/\boldsymbol{A}_w$ = 1.0 $\qquad \eta = (1.90-0.1\cdot \boldsymbol{m}) - 0.02\cdot (6-\boldsymbol{m})\cdot \bar{\lambda}_y =$

$= (1.90-0.1\cdot 0.732) - 0.02\cdot (6-0.732)\cdot 1.39 = 1.680$

$\eta = 1.558 + (1.680-1.558)\cdot (0.7-0.5)/(1.0-0.5) = 1.61$

$\boldsymbol{m}_{ef} = \eta\cdot \boldsymbol{m} = 1.61\cdot 0.732 = 1.18$

Factor φ_e is defined with linear interpolation table 61, Art. 114

$\bar{\lambda}_y$ \ $\boldsymbol{m}_{ef}$	1,00	1,18	1,25
1,00	0,653		0,600
1,39	0,606	0,573	0,559
1,50	0,593		0,548

$\varphi_e = 0.573$

Stability check for eccentric compression

$\sigma = \boldsymbol{F}/(\varphi_e\cdot \boldsymbol{A}) = 1080/(0.573\cdot 98.8) = 19.1\ \text{kN/cm}^2 < 21.4\ \text{kN/cm}^2$

Slenderness: $\qquad \bar{\lambda}_z = \lambda_z\cdot (\boldsymbol{R}_y/\boldsymbol{E})^{1/2} = 49\cdot (214/206000)^{1/2} = 1.58$

Buckling factor: $\qquad \varphi_z = 1-(0.073 - 5.53\cdot \boldsymbol{R}_y/\boldsymbol{E})\cdot \bar{\lambda}_z^{3/2} =$

$= 1-(0.073 - 5.53\cdot 214/206000)\cdot 1.58^{3/2} = 0.866$

$\boldsymbol{c} = \beta/(1+\alpha\cdot \boldsymbol{m}_x) = 1/(1+0.7\cdot 0.732) = 0.661 \qquad \varphi_z\cdot \boldsymbol{c} = 0.866\cdot 0.661 = 0.572$

Buckling check for eccentric compression out of plane of bending

$\sigma = \boldsymbol{F}/(\varphi_z\cdot \boldsymbol{c}\cdot \boldsymbol{A}) = 1080/(0.572\cdot 98.8) = 19.1\ \text{kN/cm}^2 < 21.4\ \text{kN/cm}^2$

This example is checked with Steel Expert. Since there is no option to define tapered flanges, section is defined approximately with parallel flanges with mean thickness.

Proektsoft - Steel Expert EC 2.0/2010

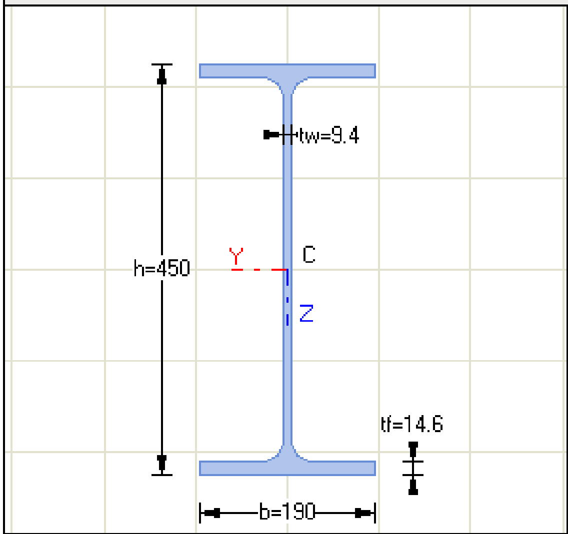
Steel Element Design To NPSK'87

Project:	Element:
Building:	Author/Date:
Client:	Checked By:

Input Data

Steel S235 t < 40 - Ry = 215 MPa γ_c = 1.00

Section Dimensions And Properties - IPE 450 - I-SECTION



h [mm]	t_w [mm]	b [mm]	t_f [mm]	b_2 [mm]	t_{f2} [mm]
450.0	9.4	190.0	14.6	190.0	14.6
r_i [mm]	r_o [mm]	A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
21.0		98.8	40.9	46.2	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
33742.9	1675.9	1499.7	176.4	1701.8	276.4
i_y [cm]	i_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
18.5	4.1	22.5	9.5	66.5	45.0

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 800.0cm

About axis "z" - $L_{eff,z}$ = 200.0cm

LT buckling - $L_{eff,b}$ = 800.0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - End moments

Web stiffeners at 0.0cm

Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	-1080.0	120.0	0.0	0.0	0.0	0.0

Design Results

Stress Checks

Case	σ_x	τ_{xy}	τ_{xz}	τ_{max}	σ_{red}
1	189.3	0.0	0.0	0.0	181.9

Case	$\sigma_x / \gamma_c R_y$	$\tau_{xy} / 0.58\gamma_c R_y$	$\tau_{xz} / 0.58\gamma_c R_y$	$\tau_{max} / 0.58\gamma_c R_y$	$\sigma_{red} / 1.15\gamma_c R_y$
1	0.88	0.00	0.00	0.00	0.74

Element Buckling - $\{\sigma\} < \gamma_c R_y$

λ_y	λ_z	φ	φ_{ey}	$\varphi_z c$	$\varphi_{e,z}$	$\varphi_{e,yz}$	φ_b
43.3	48.6	0.87	0.57	0.57	0.00	0.00	0.53

Case	$\{\sigma\}$	$\frac{N}{\varphi A}$	$\frac{N}{\varphi_{ey} A}$	$\frac{N}{\varphi_z c A}$	$\frac{N}{\varphi_{e,z} A}$	$\frac{N}{\varphi_{e,yz} A}$	$\frac{M}{\varphi_b W_c}$
1	[MPa]	125.9	192.3	190.5	0.0	0.0	160.4
	Ratio	0.59	0.89	0.89	0.00	0.00	0.75

Local Buckling

max b/t_f = 16.9

$t_{f,min}$ = 4.1 mm $< t_f$

max h/t_f = 76.2

$t_{w,min}$ = 5.0 mm $< t_w$

Design checks are satisfied: K = 0.89

Difference from manual calculation is (191−190.5)/191·100 = 0.31%

Design to EN 1993 1-1:2005

Classification of cross sections

Section class is based on the assumption that section is loaded either with uniform compression or pure bending. Class is determined for each part of the section, using Table 5.1 for the respective stress diagram. In case of bending of non-symmetrical sections, equations for bending and compression are used for stress calculation, and the c/t factor is determined assuming that neutral axis passes through the centre of area.

Sections with class 1 and 2 are designed for plastic resistance and sections class 3 – for elastic resistance. Sections of class 4 are not designed in current version. In these sections local buckling occurs before steel yielding.

The most conservative result from classification of separate parts is relevant for the whole section. If no compression is defined, classification for compression is not taken into account. If no bending moments are defined, classification for bending is not taken into account, respectively.

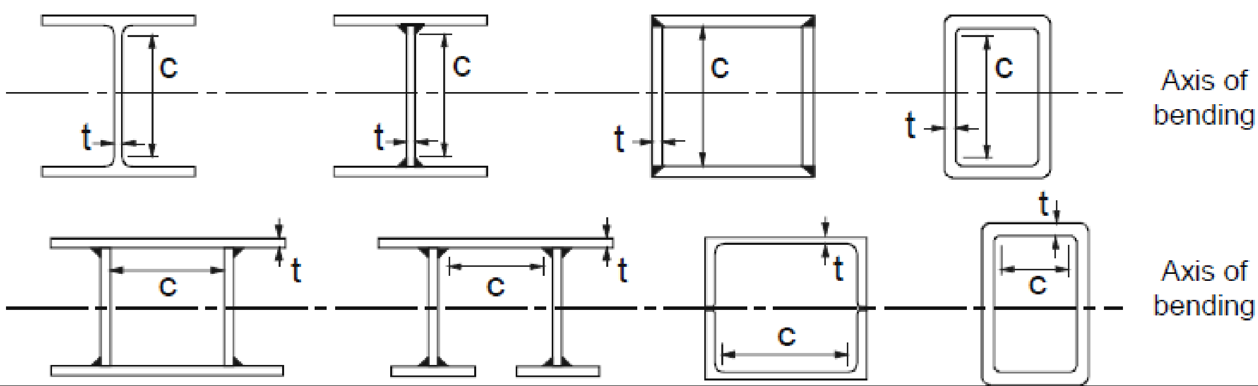
In case of combined loading (compression and bending) this approach gives conservative results. Take for instance beam with IPE 400 section, loaded with bending 300 KN.m and compression -0,10 KN. Web shall be classified as Class 1 for bending and Class 4 for compression. But final class for the section is Class 4, due to the presence of compression. In such cases when the effects of compressive forces upon final stresses are negligible, it is better not to consider them.

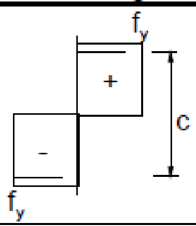
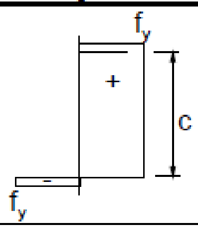
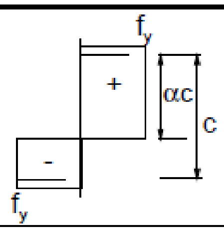
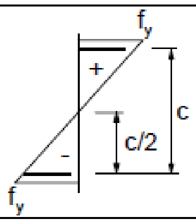
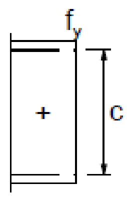
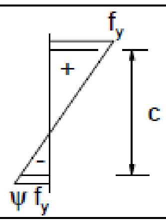
Results of section classification are provided in tabular form:

	Compression	**Bending**
Web	Class 4	Class 1
Flanges	Class 1	Class 1

Table 5.2. Maximum width-to-thickness ratios for compression parts
Sheet 1 of 3. Internal compression parts

Internal compression parts

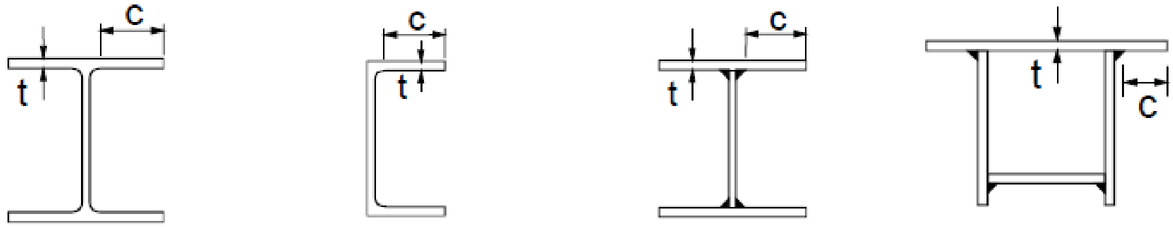


Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t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

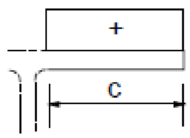
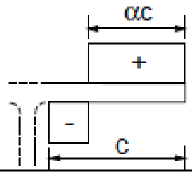
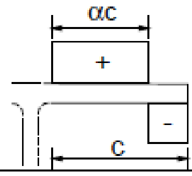
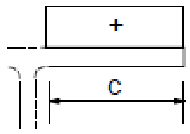
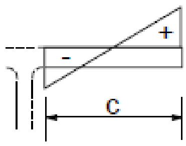
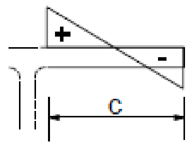
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Sheet 2 of 3. Outstand flanges

Outstand flanges

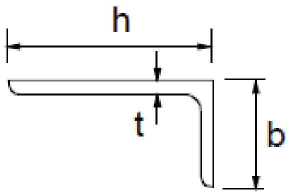


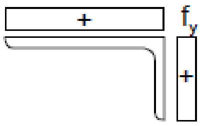
Rolled sections | Welded sections

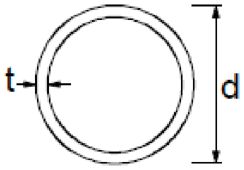
Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \dfrac{9\varepsilon}{\alpha}$	$c/t \le \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
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

Sheet 3 of 3. Angles

	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

Resistance of cross sections

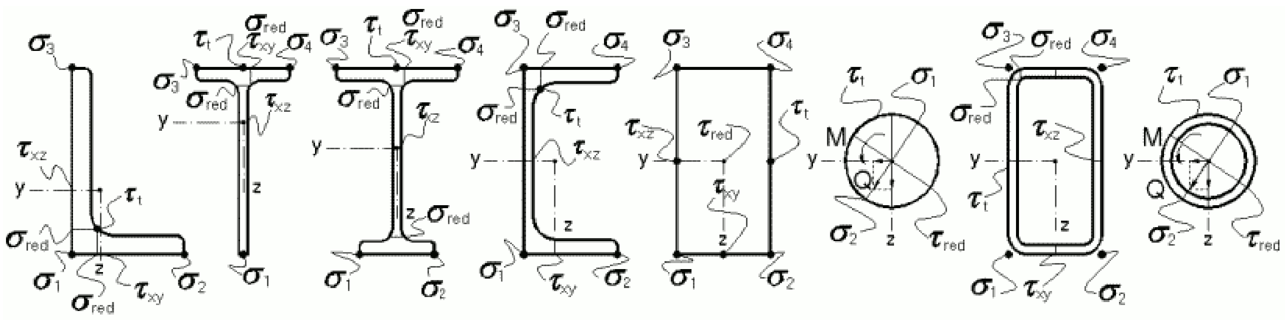
Elastic design

Sections of Class 3 are checked using the following equation:

$$\sqrt{\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2+\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)^2-\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)+3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right)^2} \le 1 \; (6.1)^*$$

*In Eurocode this equation is presented without square root operation. However, the original form of the equation is used in the program in order to obtain true factor of safety (FOS) and true safety margin for the section. Further the original notations of equations are presented according to EN 1993-1-1.

This equation is applied to different points of cross section and stresses are calculated according to principles of science for strength of materials.



Design checks for points with maximum values for normal and shear stresses and their combined action are relevant. Results are presented in tabular form.

$\dfrac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}$ (6.42)	$\dfrac{\tau_{xy,Ed}}{f_y/\sqrt{3}\gamma_{M0}}$ (6.19)	$\dfrac{\tau_{xz,Ed}}{f_y/\sqrt{3}\gamma_{M0}}$ (6.19)	$\dfrac{\tau_{max,Ed}}{f_y/\sqrt{3}\gamma_{M0}}$	$\dfrac{\sqrt{\sigma_{x,Ed}{}^2+3\tau_{Ed}{}^2}}{f_y/\gamma_{M0}}$

Equations (6.19) and (6.42) may be considered as a partial cases of (6.1).

$$1/\sqrt{3} \approx 0.58$$

$$\sigma_{x,Ed}(y,z) = \frac{N_{Ed}}{A} \pm \frac{M_{y,Ed}}{I_y} \cdot z \pm \frac{M_{z,Ed}}{I_z} \cdot y$$

$$\tau_{xz,Ed}(y) = \frac{V_{z,Ed} \cdot S_z(y)}{b(y) \cdot I_y} \quad (6.20)$$

$$\tau_{xy,Ed}(z) = \frac{V_{y,Ed} \cdot S_y(z)}{b(z) \cdot I_z} \quad (6.20)$$

$$\tau_{T,Ed} = \frac{T_{Ed}}{W_t}$$

$$\tau_{red,Ed} = \sqrt{\tau_{xy,Ed}{}^2 + \tau_{xz,Ed}{}^2}$$

$$\tau_{Max,Ed} = \max\left(\tau_{xy,Ed} | \tau_{xz,Ed} | \tau_{T,Ed} | \tau_{red,Ed}\right)$$

$$\sigma_{x,Ed} = \max\left(\sigma_1 | \sigma_2 | \sigma_3 | \sigma_4\right)$$

Normal stress $\sigma_{z,Ed}$, due to local transverse load, is not considered in this version of the software. Additional design checks should be performed in zones with local effects from significant transverse load (for instance, under supports of secondary beams).

If holes are specified, they are always considered in calculation of effective section properties.

For unsymmetrical I sections, shear force $V_{y,Ed}$ is distributed between top (1) and bottom (2) flange according to the equations:

$$V_{y,1} = V_{y,Ed} \frac{I_{f1}}{I_{f1} + I_{f2}}$$

$$V_{y,2} = V_{y,Ed} \frac{I_{f2}}{I_{f1} + I_{f2}}$$

Plastic design

Plastic design is performed for sections Class 1 and 2. Elastic design is also performed and the results are presented for information only. They are not relevant for the final bearing capacity of the section.

Design checks for uniform tension:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1 \;(6.5)$$

Tension capacity of the section is determined according to the equation:

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \;(6.6)$$

When holes are specified check is performed using the effective section properties.

$$N_{u,Rd} = \frac{0.9A_{net} \cdot f_u}{\gamma_{M2}} (6.7)$$

Design check for uniform compression:

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1 \;(6.9)$$

Compression capacity of the section is determined according to the equation:

$$N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \;(6.10)$$

Reduced bearing capacity for axial force $N_{pl,V,Rd}$ is calculated instead of $N_{pl,Rd}$ and $N_{c,Rd}$ in case of design shear force $V_{Ed} \geq 0.5V_{pl,Rd}$ and reduced yield strength is used for shear area instead of f_y.

$$(1-\rho)f_y \;(6.29) \text{ , where } \rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$$

When torsion is present, factor ρ is computed with $V_{pl,T,Rd}$ instead of $V_{pl,Rd}$.

Bending check

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1 \;(6.12)$$

Bending capacity of the section is determined according to the equations:

$$M_{y,Rd} = M_{pl,y,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} \;(6.13)$$

$$M_{z,Rd} = M_{pl,z,Rd} = \frac{W_{pl,z} \cdot f_y}{\gamma_{M0}} \;(6.13)$$

When bending moment and axial force are present, design check is performed according to equation:

$$\frac{M_{Ed}}{M_{N,Rd}} \leq 1 \;(6.31)$$

Bearing capacity of the section for bending with axial force is determined according to the equations:

For rectangular section

$$M_{N,Rd} = M_{pl,Rd}\left[1 - \left(\frac{N_{Ed}}{N_{pl,Rd}}\right)^2\right] \; (6.32)$$

For I section

$$M_{N,y,Rd} = M_{pl,y,Rd}\frac{1-n}{1-0.5a} \; (6.36)$$

For $n \leq a$: $M_{N,z,Rd} = M_{pl,z,Rd}\,(6.37)$

For $n > a$: $M_{N,z,Rd} = M_{pl,z,Rd}\left[1 - \left(\frac{n-a}{1-a}\right)^2\right]\,(6.38)$

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

$$a = \frac{A - 2bt_f}{A}$$

For hollow sections

$$M_{N,y,Rd} = M_{pl,y,Rd}\cdot\frac{1-n}{1-0.5a_w} \; (6.39)$$

$$M_{N,z,Rd} = M_{pl,z,Rd}\cdot\frac{1-n}{1-0.5a_f} \; (6.40)$$

$$a_w = \frac{A - 2bt_f}{A}$$

$$a_f = \frac{A - 2bt_w}{A}$$

<u>Design check for biaxial bending</u>

$$\left(\frac{M_{y,Ed}}{M_{y,Rd}}\right)^{\alpha} + \left(\frac{M_{z,Ed}}{M_{z,Rd}}\right)^{\beta} \leq 1 \; (6.41)$$

For I sections	α = 2, β = 5n ≥ 1
For circular hollow sections	α = 2, β = 2
For rectangular hollow sections	$\alpha = \beta = 1.66/(1-1.13n^2)$

For biaxial bending with axial force $M_{y,Rd}$ and $M_{z,Rd}$ in equation 6.41 are replaced by $M_{N,y,Rd}$ and $M_{N,z,Rd}$, respectively.

Where V_{Ed} exceeds 50% of $V_{pl,Rd}$ the design resistance for combined bending with axial force should be calculated using reduced yield strength

$$(1-\rho)f_y \; (6.29) \; \text{for the shear area,}$$

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

If torsion moment is present, the ρ factor is calculated with $V_{pl,T,Rd}$ instead of $V_{pl,Rd}$.

<u>Shear force check</u>

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1 \; (6.17)$$

Bearing capacity of the section for shear is determined by the following formulas:

$$V_{y,Rd} = V_{pl,y,Rd} = \frac{A_{vy} \cdot f_y}{\sqrt{3}\,\gamma_{M0}} \quad (6.18)$$

$$V_{z,Rd} = V_{pl,z,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}\,\gamma_{M0}} \quad (6.18)$$

Shear area A_v, is calculated according to 6.2.6 (3).

When shear force and torsion moment are present check is performed according to the following formulas:

$$\frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1 \ (6.25)$$

For I section

$$V_{pl,T,Rd} = \sqrt{1 - \frac{\tau_{t,\mathrm{Ed}}}{1.25\left(f_y/\sqrt{3}\right)/\,\gamma_{M0}}}\; V_{pl,Rd} \ (6.26)$$

For U section

$$V_{pl,T,Rd} = \left(\sqrt{1 - \frac{\tau_{t,\mathrm{Ed}}}{1.25\left(f_y/\sqrt{3}\right)/\,\gamma_{M0}}} - \frac{\tau_{w,\mathrm{Ed}}}{\left(f_y/\sqrt{3}\right)/\,\gamma_{M0}}\right) V_{pl,Rd} \ (6.27)$$

For hollow sections

$$V_{pl,T,Rd} = \left(1 - \frac{\tau_{t,\mathrm{Ed}}}{1.25\left(f_y/\sqrt{3}\right)/\,\gamma_{M0}}\right) V_{pl,Rd} \ (6.28)$$

<u>Torsion check</u>

$$\frac{T_{Ed}}{T_{Rd}} \leq 1 \ (6.23)$$

Torsion capacity of the section is determined by the following formula:

$$T_{Rd} = \frac{W_t \cdot f_y}{\gamma_{M0}}$$

Results from design checks are presented in tabular form.

$\frac{N_{Ed}}{N_{Rd}}$ (6.5) (6.9)	$\frac{N_{Ed}}{N_{u,Rd}}$ (6.7) -	$\frac{M_{y,Ed}}{M_{y,Rd}}$ (6.12) (6.31)	$\frac{M_{z,Ed}}{M_{z,Rd}}$ (6.12) (6.31)	$\frac{V_{z,Ed}}{V_{z,Rd}}$ (6.17) (6.25)	$\frac{V_{y,Ed}}{V_{y,Rd}}$ (6.17) (6.25)	$\frac{T_{Ed}}{T_{Rd}}$ (6.23) -	$\left(\frac{M_{y,Ed}}{M_{y,Rd}}\right)^{\alpha} + \left(\frac{M_{z,Ed}}{M_{z,Rd}}\right)^{\beta}$ (6.12) (6.31)

Buckling design of members

Uniform members in compression

Compression members are designed for buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \text{ (6.47)}$$

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \text{ (6.48)}$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \leq 1 \text{ (6.49)}$$

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

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}};\ N_{cr} = \frac{\pi^2 E A}{\lambda^2};\ \lambda = \frac{l_{eff}}{r};\ r = \sqrt{\frac{I}{A}}$$

Factor α considers initial imperfections. It is provided in Table 6.1 for the respective buckling curves depending on cross section type

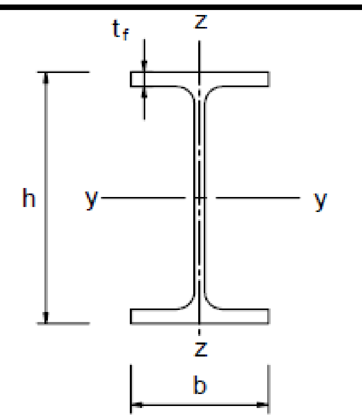
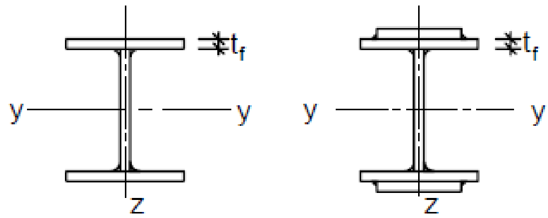
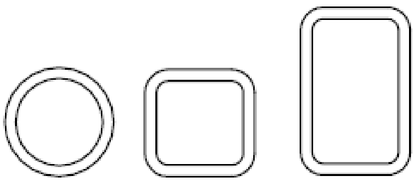
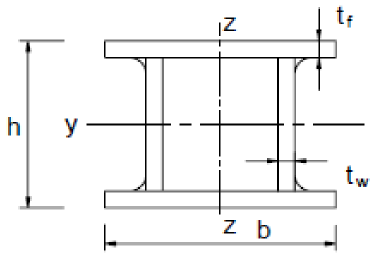
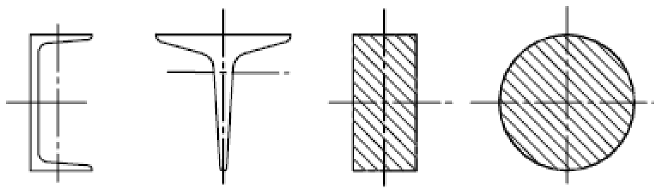
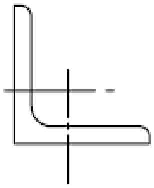
Buckling curve	a_0	a	b	c	d
Imperfection factor	0.13	0.21	0.34	0.49	0.76

Verification is performed for both main axis with relevant section properties and effective lengths for the respective support conditions. Effective length l_{eff} in Steel Expert is defined either as $L*\mu_y$ or as L_y (depending on user selection) for buckling about "y" axis and either as $L*\mu_z$ or as L_z for buckling about "z" axis (L_y and L_z are distances between lateral restraints).

In current version of the program no check for torsional-flexural buckling under uniform compression is performed.

Buckling curve is chosen from Table 6.2 depending on section type and steel class.

Table 6.2 Selection of buckling curve for a cross-section

<table>
<tr><th colspan="2" rowspan="2">Cross section</th><th colspan="2" rowspan="2">Limits</th><th rowspan="2">Buckling about axis</th><th colspan="2">Buckling curve</th></tr>
<tr><th>S 235
S 275
S 355
S 420</th><th>S 460</th></tr>
<tr><td rowspan="4">Rolled sections</td><td rowspan="4"></td><td rowspan="2">$h/b > 1,2$</td><td>$t_f \le 40$ mm</td><td>y – y
z – z</td><td>a
b</td><td>a_0
a_0</td></tr>
<tr><td>40 mm < $t_f \le 100$</td><td>y – y
z – z</td><td>b
c</td><td>a
a</td></tr>
<tr><td rowspan="2">$h/b \le 1,2$</td><td>$t_f \le 100$ mm</td><td>y – y
z – z</td><td>b
c</td><td>a
a</td></tr>
<tr><td>$t_f > 100$ mm</td><td>y – y
z – z</td><td>d
d</td><td>c
c</td></tr>
<tr><td rowspan="2">Welded I-sections</td><td rowspan="2"></td><td colspan="2">$t_f \le 40$ mm</td><td>y – y
z – z</td><td>b
c</td><td>b
c</td></tr>
<tr><td colspan="2">$t_f > 40$ mm</td><td>y – y
z – z</td><td>c
d</td><td>c
d</td></tr>
<tr><td rowspan="2">Hollow sections</td><td rowspan="2"></td><td colspan="2">hot finished</td><td>any</td><td>a</td><td>a_0</td></tr>
<tr><td colspan="2">cold formed</td><td>any</td><td>c</td><td>c</td></tr>
<tr><td rowspan="2">Welded box sections</td><td rowspan="2"></td><td colspan="2">generally (except as below)</td><td>any</td><td>b</td><td>b</td></tr>
<tr><td colspan="2">thick welds: $a > 0,5t_f$
$b/t_f < 30$
$h/t_w < 30$</td><td>any</td><td>c</td><td>c</td></tr>
<tr><td>U-, T- and solid sections</td><td colspan="3"></td><td>any</td><td>c</td><td>c</td></tr>
<tr><td>L-sections</td><td colspan="3"></td><td>any</td><td>b</td><td>b</td></tr>
</table>

Uniform members in bending

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

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

$W_y = W_{pl,y}$ for sections class 1 and 2 and $W_y = W_{el,y}$ for sections class 3.

$$\chi_{LT} = \frac{1}{\Phi_{\mathrm{LT}} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \leq 1 \ (6.56)$$

$$\Phi_{\mathrm{LT}} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right]$$

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

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(kL)^2}\left[\sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 G I_t}{\pi^2 E I_z} + \left(C_2 z_g - C_3 z_j\right)^2} - \left(C_2 z_g - C_3 z_j\right)\right]$$

Factor k depends on support conditions against member rotation at supports around vertical axis and is selected to be 0.5 for both ends fixed, 0.7 for one fixed and one hinged and 1.0 for both ends hinged.

Length L is defined in Steel Expert as "Lateral restraints spacing" Lb.

Factors C_1, C_2 and C_3 are given in a tabular form depending on the type of transverse load (shape of M diagram) and the k factor.

Factor k_w accounts for possibility for rotation at member ends. It is accepted to be equal to 1.0 conservatively.

$z_g = z_a - z_s$ is the height between loading point and shear center. Loading point can be selected to be: bottom flange (favorable), top flange (unfavorable) and neutral ($z_g = 0$).

$$z_j = z_s - \frac{\int_A (y^2 + z^2) z \, \mathrm{d}A}{2 I_y}$$

The α_{LT} factor accounts for initial imperfections. It is defined in Table 6.3 for the respective lateral-torsional buckling curves, depending on the type of cross section.

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0.21	0.34	0.49	0.76

Members in bending with axial compression

Members which are subjected to combined bending and axial compression should satisfy:

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

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

$$N_{Rk} = Af_y$$

$M_{y,Rk} = W_{pl,y} f_y$ – for sections of class 1 and 2

$M_{y,Rk} = W_{el,y} f_y$ – for sections of class 3

$M_{z,Rk} = W_{pl,z} f_y$ – for sections of class 1 and 2

$M_{z,Rk} = W_{el,z} f_y$ – for sections of class 3

Interaction factors k_{yy}, k_{yz}, k_{zy} и k_{zz} are defined in Table B1 and table B2 in Annex B of EN 1993-1-1.

Table B1. Interaction factors for members, not susceptible to torsional deformations

Interaction factors	Type of sections	Design assumption: elastic cross-sectional properties class 3, class 4	Design assumption: plastic cross-sectional properties class 1, class 2
k_{yy}	I-sections RHS-sections	$C_{my}\left(1+0{,}6\bar{\lambda}_y \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{my}\left(1+0{,}6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$	$C_{my}\left(1+\left(\bar{\lambda}_y-0{,}2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{my}\left(1+0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
k_{yz}	I-sections RHS-sections	k_{zz}	$0{,}6\, k_{zz}$
k_{zy}	I-sections RHS-sections	$0{,}8\, k_{yy}$	$0{,}6\, k_{yy}$
k_{zz}	I-sections	$C_{mz}\left(1+0{,}6\bar{\lambda}_z \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+0{,}6\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$	$C_{mz}\left(1+\left(2\bar{\lambda}_z-0{,}6\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+1{,}4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
	RHS-sections		$C_{mz}\left(1+\left(\bar{\lambda}_z-0{,}2\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+0{,}8\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
For I- and H-sections and rectangular hollow sections under axial compression and uniaxial bending $M_{y,Ed}$ the coefficient k_{zy} may be $k_{zy} = 0$.			

Table B2. Interaction factors for elements, susceptible to torsion deformations

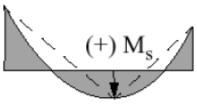
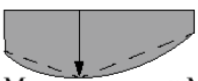
<table>
<tr><th rowspan="2">Interaction factors</th><th colspan="2">Design assumptions</th></tr>
<tr><th>elastic cross-sectional properties
class 3, class 4</th><th>plastic cross-sectional properties
class 1, class 2</th></tr>
<tr><td>k_{yy}</td><td>k_{yy} from Table B.1</td><td>k_{yy} from Table B.1</td></tr>
<tr><td>k_{yz}</td><td>k_{yz} from Table B.1</td><td>k_{yz} from Table B.1</td></tr>
<tr><td>k_{zy}</td><td>$\left[1-\frac{0{,}05\bar{\lambda}_z}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$
$\geq\left[1-\frac{0{,}05}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$</td><td>$\left[1-\frac{0{,}1\bar{\lambda}_z}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$
$\geq\left[1-\frac{0{,}1}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$
for $\bar{\lambda}_z<0{,}4$:
$k_{zy}=0{,}6+\bar{\lambda}_z\leq 1-\frac{0{,}1\bar{\lambda}_z}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}$</td></tr>
<tr><td>k_{zz}</td><td>k_{zz} from Table B.1</td><td>k_{zz} from Table B.1</td></tr>
</table>

For members not susceptible to torsional deformations flexural form of buckling will occur. For those susceptible to torsional deformations torsional-flexural buckling will occur. Members not susceptible to torsional deformations are assumed to be those with $\chi_{LT}=1$.

Equivalent uniform moment factors C_{my}, C_{mz} и C_{mLT} are defined in Table B3 depending on the diagram type between points of lateral restraints:

Factor	Bending axis	Points braced in direction	Steel Expert Notations
C_{my}	y-y	z-z	Distance between restraints: Ly
C_{mz}	z-z	y-y	Distance between restraints: Lz
C_{mLT}	y-y	y-y	Distance between restraints: Lb

Table B3. Equivalent uniform moment factors C_{my}, C_{mz} и C_{mLT}

<table>
<tr><th rowspan="2">Moment Diagram</th><th rowspan="2" colspan="2">Range</th><th colspan="2">C_{my}, C_{mz} и C_{mLT} under loading</th></tr>
<tr><th>Distributed</th><th>Concentrated</th></tr>
<tr><td>M  ψM</td><td colspan="2">$-1\leq\psi\leq1$</td><td colspan="2">$0{,}6+0.4\,\psi\geq0{,}4$</td></tr>
<tr><td rowspan="6">(–) M_h ψM_h

(+) M_s
$\alpha_s = M_s/M_h$
(+) M_s

(+) M_h ψM_h
$\alpha_h = M_h/M_s$</td><td>$0\leq\alpha_s\leq1$</td><td>$-1\leq\psi\leq1$</td><td>$0{,}2+0.8\,\alpha_s\geq0{,}4$</td><td>$0{,}2+0.8\alpha_s\geq0{,}4$</td></tr>
<tr><td rowspan="2">$-1\leq\alpha_s<0$</td><td>$0\leq\psi\leq1$</td><td>$0{,}1-0.8\,\alpha_s\geq0{,}4$</td><td>$-0.8\alpha_s\geq0{,}4$</td></tr>
<tr><td>$-1\leq\psi<0$</td><td>$0{,}1(1-\psi)-0.8\,\alpha_s\geq0{,}4$</td><td>$0{,}2(-\psi)-0.8\,\alpha_s\geq0{,}4$</td></tr>
<tr><td>$0\leq\alpha_h\leq1$</td><td>$-1\leq\psi\leq1$</td><td>$0{,}95+0{,}05\,\alpha_h$</td><td>$0{,}90+0{,}10\,\alpha_h$</td></tr>
<tr><td rowspan="2">$-1\leq\alpha_h<0$</td><td>$0\leq\psi\leq1$</td><td>$0{,}95+0{,}05\,\alpha_h$</td><td>$0{,}90+0{,}10\,\alpha_h$</td></tr>
<tr><td>$-1\leq\psi<0$</td><td>$0{,}95+0{,}05\,\alpha_h(1+2\psi)$</td><td>$0{,}90-0{,}10\,\alpha_h(1+2\psi)$</td></tr>
</table>

Local buckling resistance of beam webs

Check is preformed for I and C sections according to EN 1993-1-5 using the following formula:

$$\frac{V_{Ed}}{V_{b,Rd}} \leq 1 \; (5.10)$$

$$V_{b,Rd} = V_{bw,Rd} = \frac{\chi_w h_w s f_y}{\sqrt{3}\gamma_{M1}} \leq \frac{\eta h_w s f_y}{\sqrt{3}\gamma_{M1}} \; (5.1), (5.2)$$

Webs can be stiffened with transverse ribs or not stiffened. Contribution of flanges is neglected conservatively.

Factor χ_w is defined in Table 5.1 for rigid end posts.

$$\chi_w = \eta - \text{за } \bar{\lambda}_w < \frac{0{,}83}{\eta}$$

$$\chi_w = \frac{0{,}83}{\bar{\lambda}_w} - \text{за } \frac{0{,}83}{\eta} \leq \bar{\lambda}_w < 1{,}08$$

$$\chi_w = \frac{1{,}37}{0{,}7 + \bar{\lambda}_w} - \text{за } \bar{\lambda}_w \geq 1{,}08$$

Slenderness parameter $\bar{\lambda}_w$ is defined by the following formulas:

- When the web is not stiffened

$$\bar{\lambda}_w = \frac{h_w}{86{,}4 s\varepsilon} \; (5.5)$$

$$\varepsilon = \sqrt{\frac{235}{f_y}}$$

- When the web is stiffened with transverse ribs

$$\bar{\lambda}_w = \frac{h_w}{37.4 s\varepsilon\sqrt{k_\tau}} \; (5.6)$$

Shear buckling factor is defined according to Annex A.3.

$$k_\tau = \begin{cases} 5{,}34 + \dfrac{4{,}00}{\alpha^2} \text{ за } \alpha \geq 1 \\ 4{,}00 + \dfrac{5{,}34}{\alpha^2} \text{ за } \alpha < 1 \end{cases} \; (A.5)$$

$\alpha = a/h_w$; $h_w = h - t - t'$ – height of the web; a – spacing between ribs

s – Web thickness

Results are displayed in tabular form

$\frac{N_{Ed}}{N_{by,Rd}}$ (6.46)	$\frac{N_{Ed}}{N_{bz,Rd}}$ (6.46)	$\frac{M_{y,Ed}}{M_{b,Rd}}$ (6.54)	$\frac{V_{z,Ed}}{V_{bw,Rd}}$ (5.10)*	по (6.61)	по (6.62)

Examples to EN 1993 1-1:2005

Example 1.

Find the cross section capacity of hot rolled IPE 300 section loaded with design axial force N_{Ed} = 300 kN and design bending moment $M_{y,Ed}$ = 120 kN.m

Design checks using Steel Expert

Proektsoft - Steel Expert EC 2.0/2010

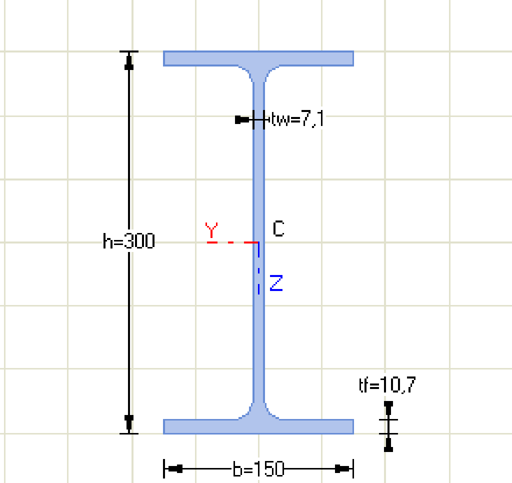
Steel Element Design To Eurocode 3

Project:	Example 5.1	Element:	
Building:		Author/Date:	
Client:		Checked By:	

Input Data

Steel S235 - t < 40 - fy = 235 MPa γ_{M0} = 1,05 γ_{M1} = 1,05 γ_{M2} = 1,25

Section Dimensions And Properties - IPE 300 - I-SECTION



h [mm]	t_w [mm]	b [mm]	t_f [mm]	b_2 [mm]	t_{f2} [mm]
300,0	7,1	150,0	10,7	150,0	10,7
r_i [mm]	r_o [mm]	A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
15,0		53,8	25,7	32,1	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
8356,1	603,8	557,1	80,5	628,4	125,2
r_y [cm]	r_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
12,5	3,3	15,0	7,5	19,9	18,4

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 0,0cm

About axis "z" - $L_{eff,z}$ = 0,0cm

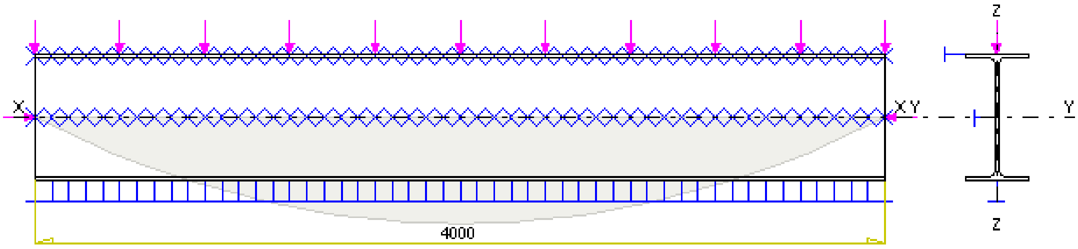
LT buckling - $L_{eff,b}$ = 0,0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - Uniformly distributed

Web stiffeners at 0,0cm



Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	350,0	120,0	0,0	0,0	0,0	0,0

Design Results

Section Classification

	Compression	Bending
Web	Class 2	Class 1
Flange	Class 1	Class 1

Section Elastic Design To Eq (6.1.)

Case	$\sigma_{x,Ed}$	$\tau_{xy,Ed}$	$\tau_{xz,Ed}$	$\tau_{max,Ed}$	$(\sigma_{x,Ed}^2 + 3\tau_{Ed}^2)^{1/2}$
1	280,5	0,0	0,0	0,0	258,6

Case	$\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}$	$\frac{\tau_{xy,Ed}}{0.58f_y/\gamma_{M0}}$	$\frac{\tau_{xz,Ed}}{0.58f_y/\gamma_{M0}}$	$\frac{\tau_{max,Ed}}{0.58f_y/\gamma_{M0}}$	$\frac{(\sigma_{x,Ed}^2 + 3\tau_{Ed}^2)^{1/2}}{f_y/\gamma_{M0}}$
1	1,25	0,00	0,00	0,00	1,16

Section Plastic Design For Class 1 Or 2

Case	N_{Rd} (2) (6.6)(6.10)	$N_{u,Rd}$ (6.7)	$M_{y,Rd}$ (2)(3) (6.13)	$M_{z,Rd}$ (2)(3) (6.13)	$V_{z,Rd}$ (1) (6.18)	$V_{y,Rd}$ (1) (6.18)	T_{Rd}
1	1204,4	1394,8	125,0	28,0	331,9	414,8	2,4

Case	$\frac{N_{Ed}}{N_{Rd}}$ (6.5) (6.9)	$\frac{N_{Ed}}{N_{u,Rd}}$ (6.7) -	$\frac{M_{y,Ed}}{M_{y,Rd}}$ (6.12) (6.31)	$\frac{M_{z,Ed}}{M_{z,Rd}}$ (6.12) (6.31)	$\frac{V_{z,Ed}}{V_{z,Rd}}$ (6.17) (6.25)	$\frac{V_{y,Ed}}{V_{y,Rd}}$ (6.17) (6.25)	$\frac{T_{Ed}}{T_{Rd}}$ (6.23) -	$\left[\frac{M_{y,Ed}}{M_{Ny,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{Nz,Rd}}\right]^{\beta}$ (6.12) (6.31)
1	0,29	0,25	0,96	0,00	0,00	0,00	0,00	0,92

(1) Reduced values $V_{T,Rd}$ are calculated to eq. (6.26) - (6.28) in case of shear combined with torsion T_{Ed}

(2) Reduced values $N_{V,Rd}$ and $M_{V,Rd}$ are calculated to eq. (6.29) in case of axial load or bending combined with shear V_{Ed}

(3) Reduced values $M_{N(V),Rd}$ are calculated to eq. (6.32) - (6.40) in case of bending combined with axial load N_{Ed}

Manual checks

$$M_{pl,y,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = \frac{628 \cdot 23,5}{1,05} = 14055\ kNcm = 140,55\ kNm$$

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{53,8 \cdot 23,5}{1,05} = 1204\ kN$$

$$n = \frac{N_{Ed}}{N_{pl,Rd}} = \frac{350}{1204} = 0,291$$

$$a = \frac{A - 2bt_f}{A} = \frac{53,8 - 2 \cdot 15 \cdot 1,07}{53,8} = 0,403$$

$$M_{N,y,Rd} = M_{pl,y,Rd} \frac{1-n}{1-0.5a} = 140,55 \frac{1-0,291}{1-0.5 \cdot 0,403} = 124.8\ kNm$$

$$\frac{M_{Ed}}{M_{N,Rd}} = \frac{120}{124.8} = 0.96 < 1$$

Example 2.

Verify bearing capacity of an IPE 400 section. Steel is S235 with $f_y = 23{,}5\ \mathrm{kN/cm^2}$, $\gamma_{M0} = 1{,}05$. Section is class 1. Design forces: $M_{y,Ed}$ = 240 kNm; N_{Ed} = 96 kN; V_{Ed} = 315 kN

Design checks using Steel Expert

Proektsoft - Steel Expert EC 2.0/2010

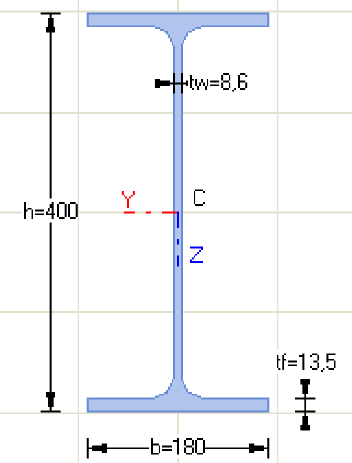
Steel Element Design To Eurocode 3

Project:	Example 5.2	Element:	
Building:		Author/Date:	
Client:		Checked By:	

Input Data

Steel S235 ëè.è ïð. t < 40 - fy = 235 MPa γ_{M0} = 1,05 γ_{M1} = 1,05 γ_{M2} = 1,25

Section Dimensions And Properties - 00 - I-SECTION



h [mm]	t_w [mm]	b [mm]	t_f [mm]	b_2 [mm]	t_{f2} [mm]
400,0	8,6	180,0	13,5	180,0	13,5
r_i [mm]	r_o [mm]	A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
21,0		84,5	42,7	48,6	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
23128,4	1317,8	1156,4	146,4	1307,1	229,0
r_y [cm]	r_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
16,5	3,9	20,0	9,0	50,5	37,3

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 0,0cm

About axis "z" - $L_{eff,z}$ = 0,0cm

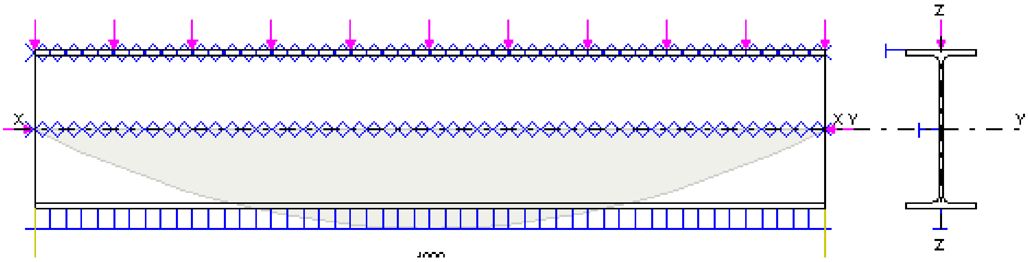
LT buckling - $L_{eff,b}$ = 0,0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - Uniformly distributed

Web stiffeners at 0,0cm



Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	96,0	240,0	0,0	315,0	0,0	0,0

Design Results

Section Classification

	Compression	Bending
Web	Class 3	Class 1
Flange	Class 1	Class 1

Section Elastic Design To Eq (6.1.)

Case	$\sigma_{x,Ed}$	$\tau_{xy,Ed}$	$\tau_{xz,Ed}$	$\tau_{max,Ed}$	$(\sigma_{x,Ed}^2 + 3\tau_{Ed}^2)^{1/2}$
1	218,9	0,0	98,1	98,1	205,8

Case	$\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}$	$\frac{\tau_{xy,Ed}}{0.58f_y/\gamma_{M0}}$	$\frac{\tau_{xz,Ed}}{0.58f_y/\gamma_{M0}}$	$\frac{\tau_{max,Ed}}{0.58f_y/\gamma_{M0}}$	$\frac{(\sigma_{x,Ed}^2 + 3\tau_{Ed}^2)^{1/2}}{f_y/\gamma_{M0}}$
1	0,98	0,00	0,76	0,76	0,92

Section Plastic Design For Class 1 Or 2

Case	N_{Rd} (2) (6.6)(6.10)	$N_{u,Rd}$ (6.7)	$M_{y,Rd}$ (2)(3) (6.13)	$M_{z,Rd}$ (2)(3) (6.13)	$V_{z,Rd}$ (1) (6.18)	$V_{y,Rd}$ (1) (6.18)	T_{Rd}
1	1871,1	2189,3	291,2	51,2	551,7	628,0	4,8

Case	$\frac{N_{Ed}}{N_{Rd}}$ (6.5) (6.9)	$\frac{N_{Ed}}{N_{u,Rd}}$ (6.7) -	$\frac{M_{y,Ed}}{M_{y,Rd}}$ (6.12) (6.31)	$\frac{M_{z,Ed}}{M_{z,Rd}}$ (6.12) (6.31)	$\frac{V_{z,Ed}}{V_{z,Rd}}$ (6.17) (6.25)	$\frac{V_{y,Ed}}{V_{y,Rd}}$ (6.17) (6.25)	$\frac{T_{Ed}}{T_{Rd}}$ (6.23) -	$\left[\frac{M_{y,Ed}}{M_{Ny,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{Nz,Rd}}\right]^{\beta}$ (6.12) (6.31)
1	0,05	0,04	0,82	0,00	0,57	0,00	0,00	0,68

(1) Reduced values $V_{T,Rd}$ are calculated to eq. (6.26) - (6.28) in case of shear combined with torsion T_{Ed}

(2) Reduced values $N_{V,Rd}$ and $M_{V,Rd}$ are calculated to eq. (6.29) in case of axial load or bending combined with shear V_{Ed}

(3) Reduced values $M_{N(V),Rd}$ are calculated to eq. (6.32) - (6.40) in case of bending combined with axial load N_{Ed}

Manual checks

$$V_{pl,z,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}\,\gamma_{M0}} = \frac{42,7 \cdot 23,5}{\sqrt{3} \cdot 1,05} = 551,75\ kN; \qquad V_{Ed} = 315kN \geq 0.5V_{pl,Rd} = 275,8\ kN$$

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2 = \left(\frac{2 \cdot 315}{551,75} - 1\right)^2 = 0.02$$

$$N_{pl,V,Rd} = \frac{A - \rho A_{vz}}{\gamma_{M0}} f_y = \frac{84,5 - 0,02 \cdot 42,7}{1,05} 23,5 = 1872\ kN$$

$$M_{pl,V,y,Rd} = \frac{W_{pl,y} - \rho s h_w{}^2/4}{\gamma_{M0}} f_y = \frac{1307,1 - 0,02 \cdot 0,86 \cdot 37,3^2/4}{1,05} 23,5 = 29120\ kNcm = 291,2\ kNm$$

$$n_v = \frac{N_{Ed}}{N_{pl,V,Rd}} = \frac{96}{1872} = 0,0513; \qquad a_v = \frac{A - 2bt_f}{A}(1 - \rho) = \frac{84,5 - 2 \cdot 18 \cdot 1,35}{84,5}(1 - 0,02) = 0,416$$

$$M_{NV,y,Rd} = M_{pl,V,y,Rd} \frac{1 - n_v}{1 - 0.5a_v} = 291,2 \frac{1 - 0,0513}{1 - 0.5 \cdot 0,416} = 348,8\ kNm > M_{pl,V,y,Rd} = 291,2\ kNm$$

$$\frac{M_{Ed}}{M_{pl,V,y,Rd}} = \frac{240}{291,2} = 0.82 < 1$$

Example 3.

Find the bearing capacity for the welded I section, presented on the figure, steel class S235.
Design forces: $M_{y,Ed}$ = 405 kN.m; V_{Ed} = 338 kN

Design checks using Steel Expert

Proektsoft - Steel Expert EC 2.0/2010

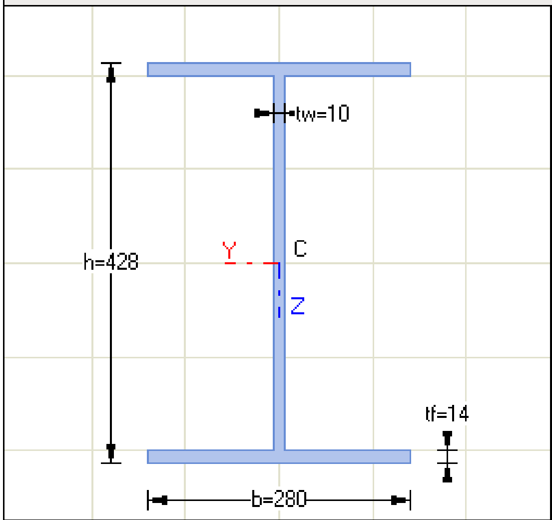
Steel Element Design To Eurocode 3

Project:	Example 5.3	Element:	
Building:		Author/Date:	
Client:		Checked By:	

Input Data

Steel S235 ëè.è ïð. t < 40 - fy = 235 MPa γ_{M0} = 1,05 γ_{M1} = 1,05 γ_{M2} = 1,25

Section Dimensions And Properties - 28 - I-SECTION



h [mm]	t_w [mm]	b [mm]	t_f [mm]	b_2 [mm]	t_{f2} [mm]
428,0	10,0	280,0	14,0	280,0	14,0
r_i [mm]	r_o [mm]	A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
		118,4	48,0	78,4	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
38939,8	5125,5	1819,6	366,1	2022,9	558,8
r_y [cm]	r_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
18,1	6,6	21,4	14,0	64,1	42,6

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 0,0cm

About axis "z" - $L_{eff,z}$ = 0,0cm

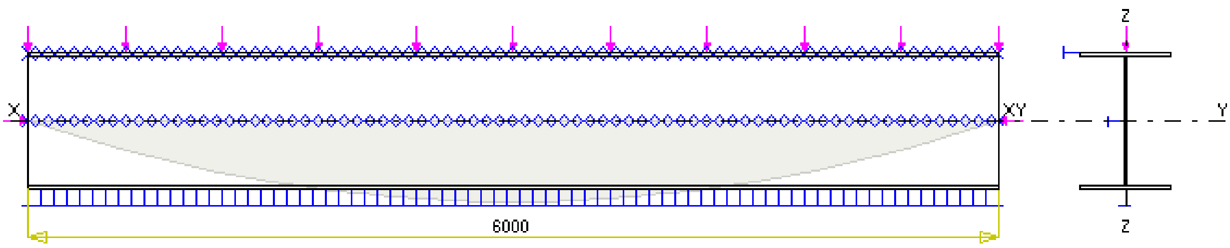
LT buckling - $L_{eff,b}$ = 0,0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - Uniformly distributed

Web stiffeners at 0,0cm



Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	0,0	405,0	0,0	338,0	0,0	0,0

Design Results

Section Classification

	Compression	Bending
Web	Class 3	Class 1
Flange	Class 2	Class 2

Section Elastic Design To Eq (6.1.)

Case	$\sigma_{x,Ed}$	$\tau_{xy,Ed}$	$\tau_{xz,Ed}$	$\tau_{max,Ed}$	$(\sigma_{x,Ed}{}^2 + 3\tau_{Ed}{}^2)^{1/2}$
1	222,6	0,0	87,8	87,8	241,1

Case	$\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}$	$\frac{\tau_{xy,Ed}}{0.58f_y/\gamma_{M0}}$	$\frac{\tau_{xz,Ed}}{0.58f_y/\gamma_{M0}}$	$\frac{\tau_{max,Ed}}{0.58f_y/\gamma_{M0}}$	$\frac{(\sigma_{x,Ed}{}^2 + 3\tau_{Ed}{}^2)^{1/2}}{f_y/\gamma_{M0}}$
1	0,99	0,00	0,68	0,68	1,08

Section Plastic Design For Class 1 Or 2

Case	$N_{Rd}{}^{(2)}$ (6.6)(6.10)	$N_{u,Rd}$ (6.7)	$M_{y,Rd}{}^{(2)(3)}$ (6.13)	$M_{z,Rd}{}^{(2)(3)}$ (6.13)	$V_{z,Rd}{}^{(1)}$ (6.18)	$V_{y,Rd}{}^{(1)}$ (6.18)	T_{Rd}
1	2641,2	0,0	452,0	125,0	620,2	1013,1	5,5

Case	$\frac{N_{Ed}}{N_{Rd}}$ (6.5) (6.9)	$\frac{N_{Ed}}{N_{u,Rd}}$ (6.7) -	$\frac{M_{y,Ed}}{M_{y,Rd}}$ (6.12) (6.31)	$\frac{M_{z,Ed}}{M_{z,Rd}}$ (6.12) (6.31)	$\frac{V_{z,Ed}}{V_{z,Rd}}$ (6.17) (6.25)	$\frac{V_{y,Ed}}{V_{y,Rd}}$ (6.17) (6.25)	$\frac{T_{Ed}}{T_{Rd}}$ (6.23) -	$\left(\frac{M_{y,Ed}}{M_{Ny,Rd}}\right)^{\alpha} + \left(\frac{M_{z,Ed}}{M_{Nz,Rd}}\right)^{\beta}$ (6.12) (6.31)
1	0,00	0,00	0,90	0,00	0,54	0,00	0,00	0,80

(1) Reduced values $V_{T,Rd}$ are calculated to eq. (6.26) - (6.28) in case of shear combined with torsion T_{Ed}

(2) Reduced values $N_{V,Rd}$ and $M_{V,Rd}$ are calculated to eq. (6.29) in case of axial load or bending combined with shear V_{Ed}

(3) Reduced values $M_{N(V),Rd}$ are calculated to eq. (6.32) - (6.40) in case of bending combined with axial load N_{Ed}

Manual checks

$$A_{vz} = \eta \cdot h_w \cdot s = 1.2 \cdot 40 \cdot 10 = 48cm^2$$

$$M_{pl,y,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = \frac{2022{,}9 \cdot 23{,}5}{1{,}05} = 45274\ kNcm = 452{,}74\ kNm$$

$$V_{pl,z,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}\ \gamma_{M0}} = \frac{48 \cdot 23{,}5}{\sqrt{3} \cdot 1{,}05} = 620{,}2\ kN$$

$$V_{Ed} = 338\ kN \geq 0.5V_{pl,Rd} = 310{,}1\ kN; \quad \rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2 = \left(\frac{2 \cdot 338}{620{,}2} - 1\right)^2 = 0.00809$$

$$M_{pl,V,y,Rd} = \frac{W_{pl,y} - \rho s h_w{}^2/4}{\gamma_{M0}} f_y = \frac{2022{,}9 - 0.00809 \cdot 1{,}0 \cdot 40^2/4}{1{,}05} 23{,}5 = 45202\ kNcm = 452\ kNm$$

$$\frac{M_{Ed}}{M_{pl,V,y,Rd}} = \frac{405}{452} = 0{,}90 > 1$$

Example 4.

Design an axially loaded column, with 6m length, loaded with force N_{Ed} = 840 kN. Cross section is hot rolled circular hollow section with D = 219 mm and t = 7 mm, steel S235JR.

The column is fixed at bottom and hinged at top, L_{eff} = 0,7*600 = 420 cm.

Design checks using Steel Expert

Proektsoft - Steel Expert EC 2.0/2010

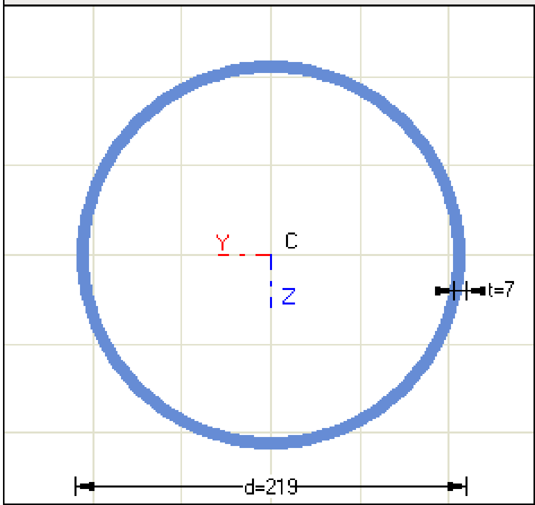
Steel Element Design To Eurocode 3

Project:	Example 6.1	Element:	
Building:		Author/Date:	
Client:		Checked By:	

Input Data

Steel S235 ëè.è ïð. t < 40 - fy = 235 MPa γ_{M0} = 1,05 γ_{M1} = 1,05 γ_{M2} = 1,25

Section Dimensions And Properties - 219x7 - CIRCULAR TUBE



		d [mm]	t [mm]		
		219,0	7,0		
		A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
		46,6	29,7	29,7	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
2622,0	2622,0	239,5	239,5	2517,8	2517,8
r_y [cm]	r_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
7,5	7,5	11,0	11,0	5244,1	478,9

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 420,0cm

About axis "z" - $L_{eff,z}$ = 420,0cm

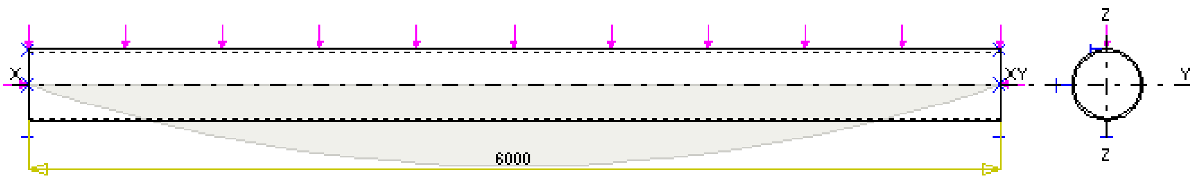
LT buckling - $L_{eff,b}$ = 600,0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - Uniformly distributed

Web stiffeners at 0,0cm



Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	-840,0	0,0	0,0	0,0	0,0	0,0

Design Results

Section Classification

	Compression	Bending
Web	Class 1	Class 1
Flange	Class 1	Class 1

Element Design

λ_y	λ_z	λ_{LT}	λ_w	χ_y	χ_z	χ_{LT}	χ_w	k_{yy}	k_{yz}	k_{zy}	k_{zz}
0,60	0,60	0,00	0,00	0,89	0,89	1,00	1,20	1,29	0,88	0,77	1,46

$N_{by,Rd}$ (6.47)	$N_{bz,Rd}$ (6.47)	$M_{b,Rd}$ (6.55)	$V_{bw,Rd}$ (5.2)*
930,1	930,1	563,5	383,5

Case	$\frac{N_{Ed}}{N_{by,Rd}}$ (6.46)	$\frac{N_{Ed}}{N_{bz,Rd}}$ (6.46)	$\frac{M_{y,Ed}}{M_{b,Rd}}$ (6.54)	$\frac{V_{z,Ed}}{V_{bw,Rd}}$ (5.10)*	To Eq. (6.61)	To Eq. (6.62)
1	0,90	0,90	0,00	0,00	0,90	0,90

*according to EN1993-1-5

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

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

Design checks are satisfied: K = 0.90

Manual checks

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2622}{46,6}} = 7,5\ cm; \quad \lambda = \frac{l_{eff}}{r} = \frac{420}{7,5} = 56; \quad N_{cr} = \frac{\pi^2 EA}{\lambda^2} = \frac{\pi^2 \cdot 21000 \cdot 46,6}{56^2} = 3021\ kN$$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{46,4 \cdot 23,5}{3021}} = 0,60; \quad buckling\ curve: a \ \rightarrow \ \alpha = 0,21$$

$$\Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0,2\right) + \bar{\lambda}^2\right] = 0.5 \cdot [1 + 0,21 \cdot (0,60 - 0,2) + 0,60^2] = 0,722$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0,722 + \sqrt{0,722^2 - 0,60^2}} = 0,89 < 1$$

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0,89 \cdot 46,6 \cdot 23,5}{1,05} = 928\ kN$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{840}{928} = 0,90 < 1$$

Example 5.

Check the resistance of an 8m long column, loaded with axial force N_{Ed} = 3025 kN, cross section is hot-rolled HE360B, steel is S235JR.

Buckling lengths are $L_{eff,y}$ = 0,7·800 = 560 cm, $L_{eff,y}$ = 0,5·800 = 400 cm.

Design checks using Steel Expert

Proektsoft - Steel Expert EC 2.0/2010

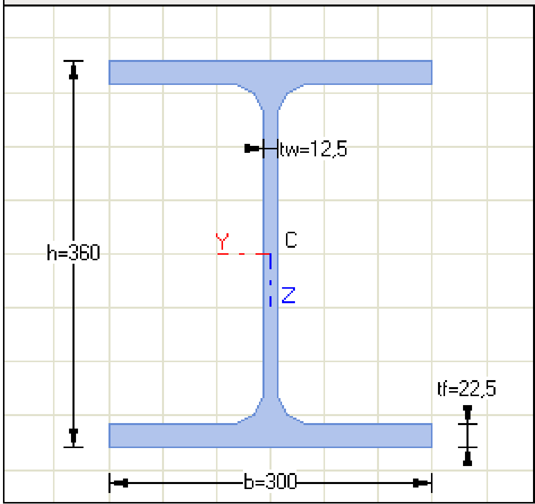
Steel Element Design To Eurocode 3

Project:	Example 6.2	Element:	
Building:		Author/Date:	
Client:		Checked By:	

Input Data

Steel S235 ëè.è ïð. t < 40 - fy = 235 MPa γ_{M0} = 1,05 γ_{M1} = 1,05 γ_{M2} = 1,25

Section Dimensions And Properties - HE 360 B - I-SECTION



h [mm]	t_w [mm]	b [mm]	t_f [mm]	b_2 [mm]	t_{f2} [mm]
360,0	12,5	300,0	22,5	300,0	22,5
r_i [mm]	r_o [mm]	A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
27,0		180,6	60,6	135,0	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
43193,5	10141,2	2399,6	676,1	2683,0	1032,5
r_y [cm]	r_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
15,5	7,5	18,0	15,0	300,5	127,9

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 560,0cm

About axis "z" - $L_{eff,z}$ = 400,0cm

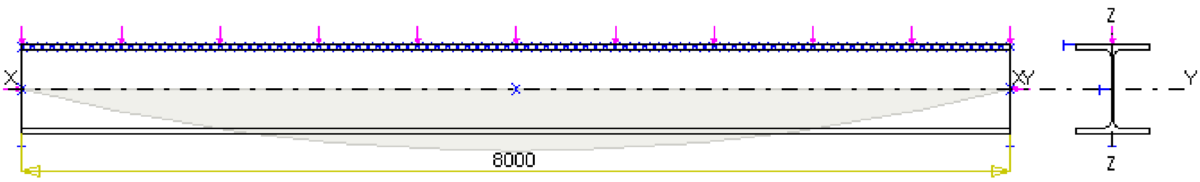
LT buckling - $L_{eff,b}$ = 0,0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - Uniformly distributed

Web stiffeners at 0,0cm



Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	-3025,0	0,0	0,0	0,0	0,0	0,0

Design Results

Section Classification

	Compression	Bending
Web	Class 1	Class 1
Flange	Class 1	Class 1

Element Design

λ_y	λ_z	λ_{LT}	λ_w	χ_y	χ_z	χ_{LT}	χ_w	k_{yy}	k_{yz}	k_{zy}	k_{zz}
0,39	0,57	0,00	0,29	0,93	0,80	1,00	1,20	1,09	0,72	0,65	1,20

$N_{by,Rd}$ (6.47)	$N_{bz,Rd}$ (6.47)	$M_{b,Rd}$ (6.55)	$V_{bw,Rd}$ (5.2)*
3766,8	3250,1	600,5	610,5

Case	$\frac{N_{Ed}}{N_{by,Rd}}$ (6.46)	$\frac{N_{Ed}}{N_{bz,Rd}}$ (6.46)	$\frac{M_{y,Ed}}{M_{b,Rd}}$ (6.54)	$\frac{V_{z,Ed}}{V_{bw,Rd}}$ (5.10)*	To Eq. (6.61)	To Eq. (6.62)
1	0,80	0,93	0,00	0,00	0,80	0,93

*according to EN1993-1-5

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

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

Design checks are satisfied: K = 0,93

Manual checks

$$\lambda_z = \frac{l_{eff,z}}{r} = \frac{400}{7,5} = 53,33; \quad N_{cr} = \frac{\pi^2 EA}{\lambda^2} = \frac{\pi^2 \cdot 21000 \cdot 180,6}{53,33^2} = 13161\ kN$$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{180,6 \cdot 23,5}{13161}} = 0,568; \quad \frac{h}{b} = 1,2\ buckling\ curve: c\ \rightarrow\ \alpha = 0,49$$

$$\Phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2\right] = 0.5 \cdot [1 + 0,49 \cdot (0,568 - 0,2) + 0,568^2] = 0,753$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0,753 + \sqrt{0,753^2 - 0,568^2}} = 0,802 < 1$$

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0,802 \cdot 180,6 \cdot 23,5}{1,05} = 3240\ kN$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{3025}{3240} = 0,93 < 1$$

Example 6.

Find the bearing capacity of the column from Example 5, in case it is made from the given built-up section and steel S275.

Design checks using Steel Expert

Proektsoft - Steel Expert EC 2.0/2010

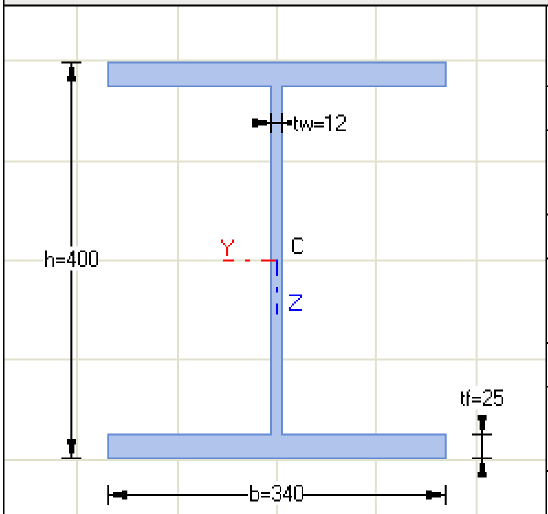
Steel Element Design To Eurocode 3

Project:	Example 6.3	Element:	
Building:		Author/Date:	
Client:		Checked By:	

Input Data

Steel S275 ëè.è ïð. t < 40 - fy = 275 MPa γ_{M0} = 1,05 γ_{M1} = 1,05 γ_{M2} = 1,25

Section Dimensions And Properties - 00 - I-SECTION



h [mm]	t_w [mm]	b [mm]	t_f [mm]	b_2 [mm]	t_{f2} [mm]
400,0	12,0	340,0	25,0	340,0	25,0
r_i [mm]	r_o [mm]	A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
		212,0	50,4	170,0	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
64141,7	16381,7	3207,1	963,6	3555,0	1457,6
r_y [cm]	r_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
17,4	8,8	20,0	17,0	363,0	140,3

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 560,0cm

About axis "z" - $L_{eff,z}$ = 400,0cm

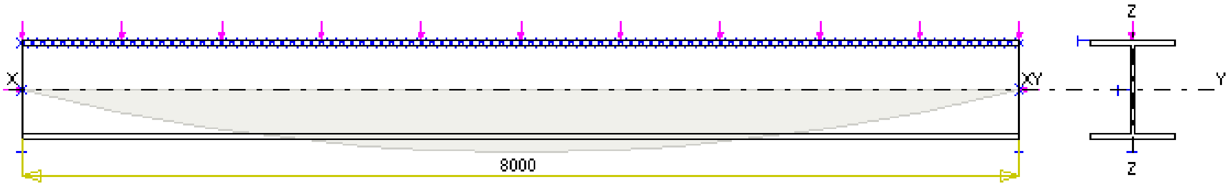
LT buckling - $L_{eff,b}$ = 0,0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - Uniformly distributed

Web stiffeners at 0,0cm



Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	-3025,0	0,0	0,0	0,0	0,0	0,0

Design Results

Section Classification

	Compression	Bending
Web	Class 1	Class 1
Flange	Class 1	Class 1

Element Design

λ_y	λ_z	λ_{LT}	λ_w	χ_y	χ_z	χ_{LT}	χ_w	k_{yy}	k_{yz}	k_{zy}	k_{zz}
0,37	0,52	0,00	0,37	0,94	0,83	1,00	1,20	1,04	0,74	0,63	1,23

$N_{by,Rd}$ (6.47)	$N_{bz,Rd}$ (6.47)	$M_{b,Rd}$ (6.55)	$V_{bw,Rd}$ (5.2)*
5205,3	4605,1	931,1	762,1

Case	$\frac{N_{Ed}}{N_{by,Rd}}$ (6.46)	$\frac{N_{Ed}}{N_{bz,Rd}}$ (6.46)	$\frac{M_{y,Ed}}{M_{b,Rd}}$ (6.54)	$\frac{V_{z,Ed}}{V_{bw,Rd}}$ (5.10)*	To Eq. (6.61)	To Eq. (6.62)
1	0,58	0,66	0,00	0,00	0,58	0,66

*according to EN1993-1-5

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

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

Design checks are satisfied: K = 0,66

Manual checks

$$\lambda_z = \frac{l_{eff,z}}{r} = \frac{400}{8{,}8} = 45{,}45; \quad N_{cr} = \frac{\pi^2 EA}{\lambda^2} = \frac{\pi^2 \cdot 21000 \cdot 212}{45{,}45^2} = 21271\ kN$$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{212 \cdot 27{,}5}{21271}} = 0{,}524; \quad buckling\ curve: c \ \rightarrow \ \alpha = 0{,}49$$

$$\Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] = 0.5 \cdot [1 + 0{,}49 \cdot (0{,}524 - 0{,}2) + 0{,}524^2] = 0{,}717$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0{,}717 + \sqrt{0{,}717^2 - 0{,}524^2}} = 0{,}829 < 1$$

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0{,}829 \cdot 212 \cdot 27{,}5}{1{,}05} = 4602\ kN$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{3025}{4602} = 0{,}66 < 1$$

Example 7.

Design an 1.9 m long diagonal lattice column bracing, loaded with axial force 210 KN, with single L100x10 angle section to EN10056-1, steel S235JR.

Calculations using Steel Expert

Proektsoft - Steel Expert EC 2.0/2010

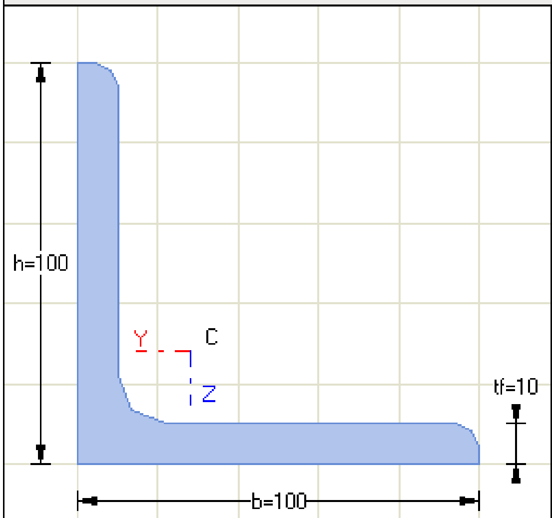
Steel Element Design To Eurocode 3

Project:	Example 6.4	Element:	
Building:		Author/Date:	
Client:		Checked By:	

Input Data

Steel S235 ëè.è ïð. t < 40 - fy = 235 MPa γ_{M0} = 1,05 γ_{M1} = 1,05 γ_{M2} = 1,25

Section Dimensions And Properties - L100x100x10 - ANGLE



h [mm]	t_w [mm]	b [mm]	t_f [mm]		
100,0	10,0	100,0	10,0		
r_i [mm]	r_o [mm]	A [cm^2]	A_{vz} [cm^2]	A_{vy} [cm^2]	
12,0	6,0	19,2	10,0	10,0	
I_y [cm^4]	I_z [cm^4]	$W_{el,y}$ [cm^3]	$W_{el,z}$ [cm^3]	$W_{pl,y}$ [cm^3]	$W_{pl,z}$ [cm^3]
176,7	176,7	24,6	24,6	50,4	50,4
r_y [cm]	r_z [cm]	C_z [cm]	C_y [cm]	I_t [cm^4]	W_t [cm^3]
3,0	3,0	2,8	2,8	7,2	4,3

Buckling Lengths

About axis "y" - $L_{eff,y}$ = 190,0cm

About axis "z" - $L_{eff,z}$ = 190,0cm

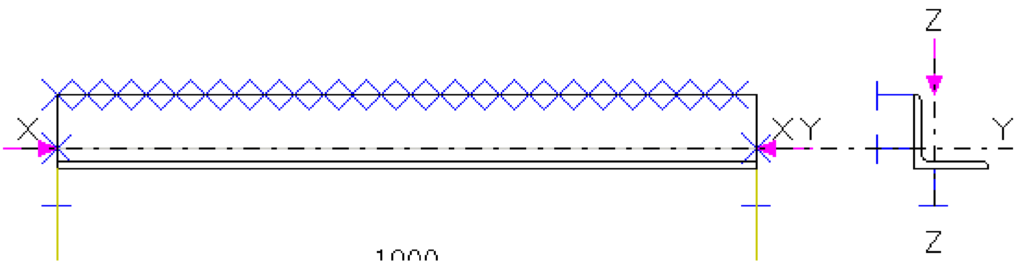
LT buckling - $L_{eff,b}$ = 0,0cm

Lateral-Torsional Buckling

Load position - Top flange

Load Type - End moments

Web stiffeners at 0,0cm



Internal Forces

Case	N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kN]	$V_{z,Ed}$ [kN]	$V_{y,Ed}$ [kN]	T_{Ed} [kNm]
1	-210,0	0,0	0,0	0,0	0,0	0,0

Design Results

Section Classification

	Compression	Bending
Web	Class 1	Class 1
Flange	Class 1	Class 1

Element Design

λ_y	λ_z	λ_{LT}	λ_w	χ_y	χ_z	χ_{LT}	χ_w	k_{yy}	k_{yz}	k_{zy}	k_{zz}
1,05	0,00	0,00	0,00	0,57	1,00	1,00	1,20	1,69	0,42	1,01	0,71

$N_{by,Rd}$ (6.47)	$N_{bz,Rd}$ (6.47)	$M_{b,Rd}$ (6.55)	$V_{bw,Rd}$ (5.2)*
243,5	428,7	11,3	129,2

Case	$\frac{N_{Ed}}{N_{by,Rd}}$ (6.46)	$\frac{N_{Ed}}{N_{bz,Rd}}$ (6.46)	$\frac{M_{y,Ed}}{M_{b,Rd}}$ (6.54)	$\frac{V_{z,Ed}}{V_{bw,Rd}}$ (5.10)*	To Eq. (6.61)	To Eq. (6.62)
1	0,86	0,49	0,00	0,00	0,86	0,00

*according to EN1993-1-5

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

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

Design checks are satisfied: K = 0,86

Manual checks

$$r_{min} = 1{,}95\ cm;\quad \lambda = \frac{l_{eff}}{r} = \frac{190}{1{,}95} = 97{,}44;\quad N_{cr} = \frac{\pi^2 EA}{\lambda^2} = \frac{\pi^2 \cdot 21000 \cdot 19{,}2}{97{,}44^2} = 419{,}1\ kN$$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{19{,}2 \cdot 23{,}5}{419{,}1}} = 1{,}038;\ \text{крива на изкълчване}: b\ \rightarrow\ \alpha = 0{,}34$$

$$\Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] = 0.5 \cdot [1 + 0{,}34 \cdot (1{,}038 - 0{,}2) + 1{,}038^2] = 1{,}181$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1{,}181 + \sqrt{1{,}181^2 - 1{,}038^2}} = 0{,}573 < 1$$

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0{,}573 \cdot 19{,}2 \cdot 23{,}5}{1{,}05} = 246\ kN$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{210}{246} = 0{,}85 < 1$$

All examples were developed using

„Manual for design of steel structures to Eurocode 3" 2009.

Prof. Ph.D. Eng. Ljubcho Venkov

Assoc. Prof. Ph.D. Eng. Borislav Belev

Eng. Chavdar Penelov

A lot of additional sources have been used for verification including older manuals, foreign books, etc. The book "Design of steel members to Eurocode 3", 2006 by Prof. Nicola Draganov was very helpful and also had a lot of examples.

Calculation Report

Calculation report in **html** format is generated for each problem by selecting the "**Results**" button. Report is viewed in **Internet Explorer**, but other web programs may be also used. Most text editors like e.g. MS Word, can also open **html** files. Report file is named **name_of_ data _file.html**.

A directory named **name_of_ data _file.html_files** is created with each file. It should always be kept together with the html file, otherwise pictures and formats will be lost.