Worked examples

Idea Steel 4.0

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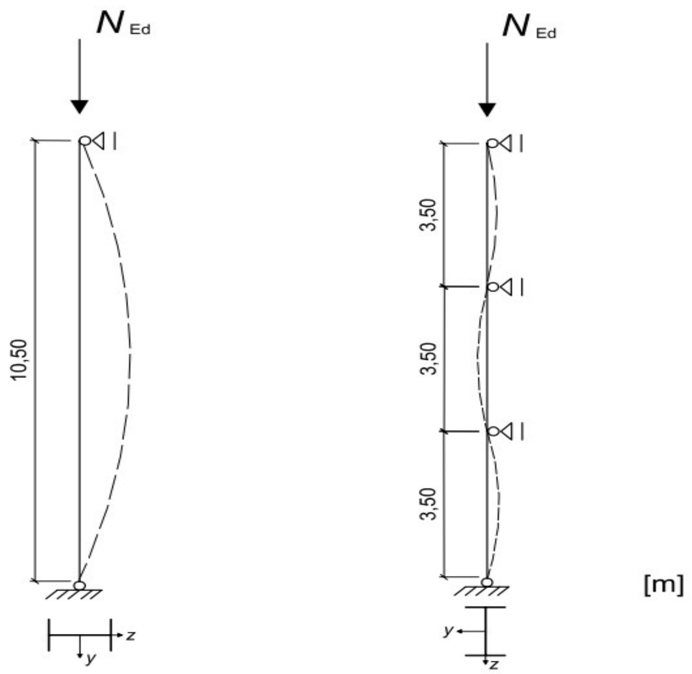
Obsah

1. Buckling resistance of hinged column with intermediate supports

Example was prepared according to the document SX002a-EN-EU STEEL Access published on [2].

Input



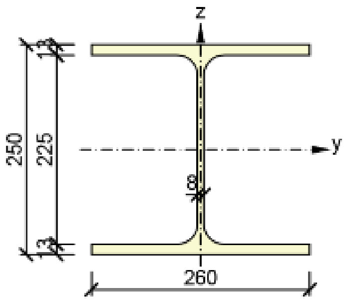
Member is made of hot-rolled profile HE 260 A, with lateral supports over 3.5 m.

$N_{Ed} = 1000\,kN$

Material: steel grade S275

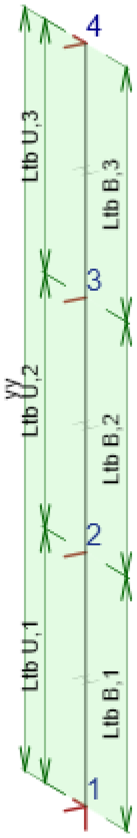
7.2. Cross-section: HE 260 A

Cross-section characteristics



Symbol	Value	Unit
A	8682	mm2
Iu	104500000	mm4
Iv	36680000	mm4
It	523700	mm4
Iw	516000000000	mm6
Wel,u	836000	mm3
Wel,v	282154	mm3
Wpl,u	919800	mm3
Wpl,v	430200	mm3

Design buckling resistance of compressed member



yy: ky = 1,00, Ly = 10,50
Ltb U,1: kz = 1,00, kw = 1,00, Ly = 3,50
Ltb U,2: kz = 1,00, kw = 1,00, Ly = 3,50
Ltb U,3: kz = 1,00, kw = 1,00, Ly = 3,50
Ltb B,1: kz = 1,00, kw = 1,00, Lz = 3,50
Ltb B,2: kz = 1,00, kw = 1,00, Lz = 3,50
Ltb B,3: kz = 1,00, kw = 1,00, Lz = 3,50

$E = 210000\, N/mm^2$

$L_{cr,y} = 10{,}5\, m$

$L_{cr,z} = 3{,}5\, m$

Flexible critical force related to appropriate buckling shape:

$$N_{cr,y}=\frac{\pi^2 EI_y}{L_{cr,y}^2}=\frac{\pi^2\cdot 21000\cdot 10450}{1050^2}=1964{,}5\,kN$$

$$N_{cr,z}=\frac{\pi^2 EI_z}{L_{cr,z}^2}=\frac{\pi^2\cdot 21000\cdot 3668}{350^2}=6206\,kN$$

relative slenderness 6.3.1.2(1):

$$\bar{\lambda}_y=\sqrt{\frac{Af_y}{N_{cr,y}}}=\sqrt{\frac{8682\cdot 235}{1964520}}=1{,}019$$

$$\bar{\lambda}_y=\sqrt{\frac{Af_y}{N_{cr,y}}}=\sqrt{\frac{8682\cdot 235}{6206000}}=0{,}573$$

Buckling coefficient:

Value χ, corresponding to the relative slenderness $\bar{\lambda}$, to be determined according to equation (6.49).

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

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

α is the imperfection factor, to be determined by tables 6.1 and 6.2

In Table 6.2, the corresponding line is selected for I section according to parameters: h/b = 250/260 = 0,96 < 1,2 a $t_f=12{,}5<100\,mm$

- buckling to axis y-y:

Buckling curve **b ->** imperfection coefficient $\alpha=0{,}34$

$$\varphi_y=0{,}5\left[1+0{,}34\cdot\left(1{,}019-0{,}2\right)+1{,}019^2\right]=1{,}158$$

$$\chi_y=\frac{1}{1{,}158+\sqrt{1{,}158^2-1{,}019^2}}=0{,}585$$

$$N_{b,Rd}=\chi_y\frac{Af_y}{\gamma_{M1}}=\frac{0{,}585\cdot 8682\cdot 235}{1{,}0}=1193{,}6\,kN>1000\text{kN} \quad \text{pass}$$

- buckling to axis z-z:

Buckling curve **c ->** imperfection coefficient $\alpha=0{,}49$

$$\varphi_y=0{,}5\left[1+0{,}49\cdot\left(0{,}573-0{,}2\right)+0{,}573^2\right]=0{,}756$$

$$\chi_z=\frac{1}{0{,}576+\sqrt{0{,}576^2-0{,}573^2}}=0{,}801$$

$$N_{b,Rd}=\chi_z\frac{Af_y}{\gamma_{M1}}=\frac{0{,}801\cdot 8682\cdot 235}{1{,}0}=1634{,}3\,kN>1000\text{kN} \quad \text{vyhoví}$$

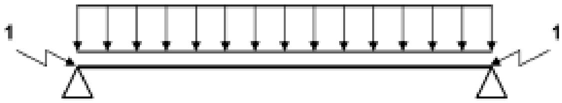
Flexural buckling check

Item name	Symbol	Value Y-Y	Value Z-Z	Unit	Reference
Reduction factor	χ	0,58	0,80	-	6.3.1.2 (1)
Slenderness	λ	1,02	0,57	-	6.3.1.2 (1)
Buckling curve		b	c		Tab. 6.2
Imperfection factor	α	0,34	0,49	-	6.3.1.2 (1)
Buckling factor	k	1,00	1,00	-	
Critical length	L_{cr}	10,50	3,50	m	6.3.1.3 (1)
Critical force	N_{cr}	1964,52	6206,01	kN	6.3.1.2 (1)
Resistance force	$N_{b,Rd}$	1193,46	1634,36	kN	6.3.1.1 (3)
Utilisation	UC	83,79	61,19	%	6.3.1.1 (1)

2. Stability of simple supported beam

Example was prepared according to the document SX001a-EN-EU STEEL Access published on [3]

Input



1 : Lateral restraint

Load:

Uniform load includes:

- self-weight of a beam
- concrete slab
- live load

Partial safety factors:

- $\gamma_G = 1{,}35$ (permanent load)
- $\gamma_Q = 1{,}5$ (variable load)
- $\gamma_{M0} = 1{,}0$
- $\gamma_{M1} = 1{,}0$

Basic data:

It is a design of beams in multi-storey building for the following input data. It is assumed that the beam is laterally supported only at the ends.

- Span: 5,70m
- Pitch: 2,50 m
- Thickness of concrete slab: 12 cm
- Crossbars: 0,75 kN/m2
- Live load: 2,5 kN/m2
- Steel grade: S235

Self weight of a slab: $0{,}12 \cdot 24\, kN/m^3 = 2{,}88\, kN/m^2$

Self weight of a beam: $(49{,}1 \cdot 9{,}81) \cdot 10^{-3} = 0{,}482\, kN/m$

Permanent load:

$$G = 0{,}482 + (2{,}88 + 0{,}75) \cdot 2{,}50 = 9{,}56\, kN/m$$

Variable load (live load)

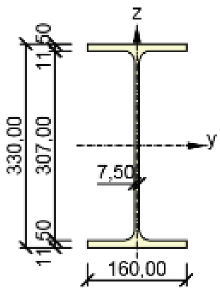
$$Q = 2{,}5 \cdot 2{,}5 = 6{,}25\, kN/m$$

The combination of ULS:

$$\gamma_G G + \gamma_Q Q = 1{,}35 \cdot 9{,}56 + 1{,}5 \cdot 6{,}25 = 22{,}28\, kN/m$$

7.2. Cross-section: IPE330

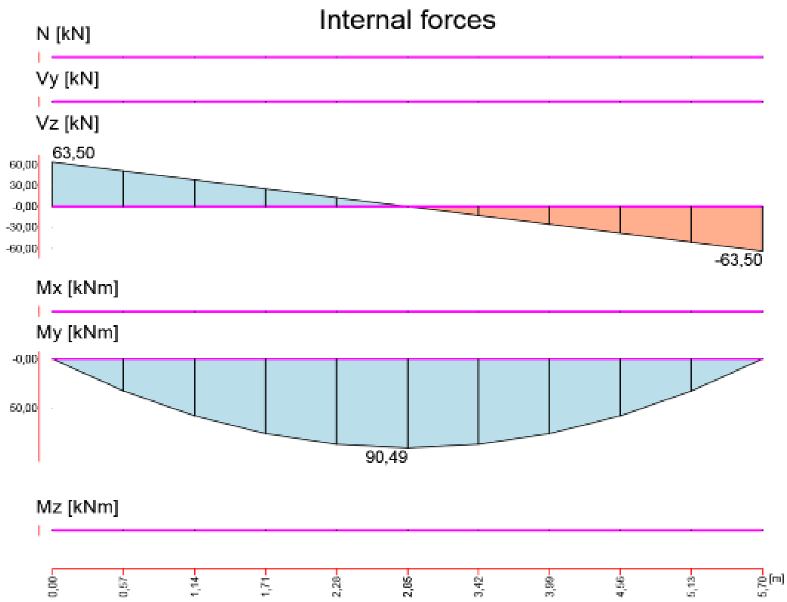
Cross-section characteristics



Symbol	Value	Unit
A	6261,00	mm2
Iu	117700000,00	mm4
Iv	7881000,00	mm4
It	281500,00	mm4
Iw	201128928000,00	mm6
Wel,u	713333,33	mm3
Wel,v	98512,50	mm3
Wpl,u	804000,00	mm3
Wpl,v	153800,00	mm3

Material: S235 - for thickness of 11,5mm → $f_y = 235\, N/mm^2$

Internal forces



Cross-section classification

Article 5.5.2

$$\varepsilon=\sqrt{235/fy}=\sqrt{235/235}=1{,}0$$

The upper flange is under compression

$$c=(b-t_w-2r)/2=(160-7{,}5-2\cdot 18)/2=58{,}25$$

$$c/t_f=58{,}25/11{,}5=5{,}07$$

The limit value for class 1

$$9\,\varepsilon=9$$

Classification of flange: 9> 5,07 → **Class 1**

Web subject to bending

$$c=h-2t_f-2r=330-2\cdot 11{,}5-2\cdot 18=271$$

$$c/t_f=271/7{,}5=36{,}1$$

The limit value for class 1

$$72\,\varepsilon=72$$

Classification of web: 72> 36,1 → **Class 1**

Cross-section is a Class 1

Shear resistance

Shear check at support:

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

For shear area of standard rolled I-section the code quotes the formula in 6.2.6 (3)

$$A-2bt_f+(t_w+2r)t_f=6260-2\cdot 160\cdot 11{,}5+(7{,}5+2\cdot 18)\cdot 11{,}5=3080$$

$$\eta h_w t_w=1{,}2\cdot 307\cdot 7{,}5=2763$$

$$A_v=max(3080\,;2763)$$

$$V_{Pl,Rd}=\frac{3080\cdot(235/\sqrt{3})}{1{,}0}=417{,}9\,kN$$

The limit for the assessment of web buckling

$$72\frac{\varepsilon}{\eta}=\frac{72\cdot 1{,}0}{1{,}2}=60$$

web slenderness: $h_w/t_w=307/7{,}5=40{,}93<60$ - buckling need not be considered.

Utilization: 63,5 / 417,9 = 15,2% Pass -> utilization is less than 50%. Interaction of M + V is not to be assessed. Moreover, in this case the maximal shear force occurs in a different section as the maximal bending moment.

Shear force check Vz

Item name	Symbol	Value	Unit	Reference
Design plastic shear resistance	$V_{pl,Rd}$	418,16	kN	6.2.6 (2)
Design plastic shear resistance reduced by Torsion	$V_{pl,T,Rd}$	418,16	kN	6.2.7 (9)
Design plastic shear resistance	$V_{c,Rd}$	418,16	kN	6.2.6 (1)
Utilisation	UC	15,19	%	6.2.6 (1)

Item name	Symbol	Value	Unit	Reference
Shear reduction	ρ	0,00	-	6.2.8 (3),(4)

Shear buckling need not be considered.

Bending

according to 6.2.5

$$M_{c,y,Rd}=M_{pl,y,Rd}=\frac{W_{pl,y}f_y}{\gamma_{M0}}=\frac{804{,}000\cdot 235}{1{,}0}=188{,}94\,kNm$$

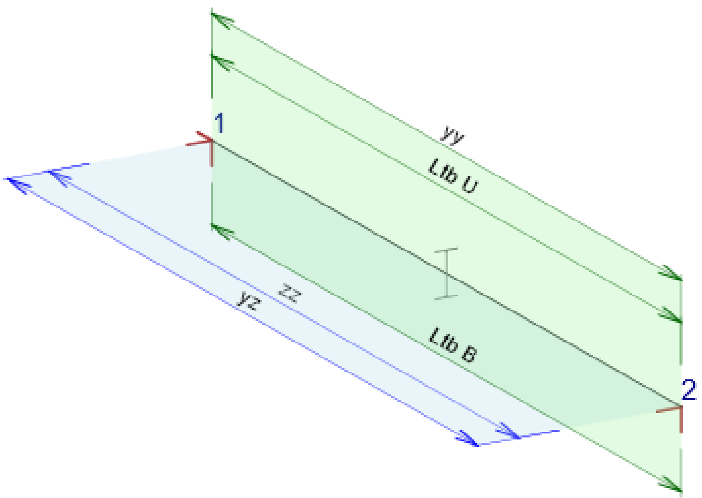
$$M_{y,Ed}/M_{c,y,Rd}=\frac{90{,}49}{188{,}94}=0{,}479<1 \quad \text{pass}$$

Bending moment check My

Item name	Symbol	Value	Unit	Reference
Section modulus	$W_{pl,min}$	804000	mm3	(6.13)
Design resistance moment	$M_{c,Rd}$	188,94	kNm	6.2.5 (2)
Utilisation	UC	47,89	%	6.2.5 (1)

LT buckling

according to 6.3.2.3 and 6.3.2.1



yy: ky = 1,00, Ly = 5,70
zz: kz = 1,00, Lz = 5,70
yz: kyz = 1,00, kw = 1,00, Ly = 5,70
Ltb U: kz = 1,00, kw = 1,00, Ly = 5,70
Ltb B: kz = 1,00, kw = 1,00, Lz = 5,70

Calculation of the critical moment Mcr:

(According to EN1999-1-1, Annex I)

kz = 1,0

kw = 1,0

L = 5,7m

The critical moment by I.1.2 (I.2)

$$M_{cr}=\mu_{cr}\frac{\pi\sqrt{EI_z GI_t}}{L}\quad \text{kde}$$

$$\mu_{cr}=\frac{C_1}{k_z}\left[\sqrt{1+\kappa_{wt}^2+(C_2\zeta_g-C_3\zeta_j)^2}-(C_2\zeta_g-C_3\zeta_j)\right]$$

$$\kappa_{wt}=\frac{\pi}{k_w L}\sqrt{\frac{EI_w}{GI_t}}=\frac{\pi}{1{,}0\cdot 5{,}7}\sqrt{\frac{21\cdot 10^{10}\cdot 20{,}11\cdot 10^{-8}}{8{,}077\cdot 10^{10}\cdot 28{,}15\cdot 10^{-8}}}=0{,}751$$

$$\zeta_g=\frac{\pi z_g}{k_z L}\sqrt{\frac{EI_z}{GI_t}}=\frac{\pi\cdot 0{,}165}{1{,}0\cdot 5{,}7}\sqrt{\frac{21\cdot 10^{10}\cdot 788{,}1\cdot 10^{-8}}{8{,}077\cdot 10^{10}\cdot 28{,}15\cdot 10^{-8}}}=0{,}776$$

$$\zeta_j=\frac{\pi z_j}{k_z L}\sqrt{\frac{EI_z}{GI_t}}=\frac{\pi\cdot 0{,}0}{1{,}0\cdot 5{,}7}\sqrt{\frac{21\cdot 10^{10}\cdot 788{,}1\cdot 10^{-8}}{8{,}077\cdot 10^{10}\cdot 28{,}15\cdot 10^{-8}}}=0{,}0$$

For the parabolic shape of the bending moment diagram, we determine the parameters from Table I.2

$$C_1=C_{1,0}+(C_{1,1}-C_{1,0})\kappa_{wt}=1{,}127+(1{,}132-1{,}127)\cdot 0{,}751=1{,}131$$

$$C_2=0{,}459$$

$$C_3=0{,}525$$

$$\mu_{cr}=\frac{1{,}131}{1{,}0}\left[\sqrt{1+0{,}751^2+(0{,}459\cdot 0{,}776-0{,}525\cdot 0{,}0)^2}-(0{,}459\cdot 0{,}776-0{,}525\cdot 0{,}0)\right]=1{,}068$$

$$M_{cr}=\frac{1{,}068\cdot\pi\cdot\sqrt{21\cdot 10^{10}\cdot 788{,}1\cdot 10^{-8}\cdot 8{,}077\cdot 10^{10}\cdot 28{,}15\cdot 10^{-8}}}{5{,}7}=114{,}185\ kNm$$

Relative slenderness:

$$\bar{\lambda}_{LT}=\sqrt{\frac{W_{pl,y}f_y}{M_{cr}}}=\sqrt{\frac{804000\cdot 235\cdot 10^{-6}}{114{,}185}}=1{,}286$$

$\bar{\lambda}_{LT0}=0{,}4$ for rolled sections

$\bar{\lambda}_{LT}=1{,}286>\bar{\lambda}_{LT0}=0{,}4$ LT buckling can not be ignored

LT buckling coefficient by 6.3.2.3:

$$\chi_{LT}=\frac{1}{\varphi_{LT}+\sqrt{\varphi_{LT}^{2}-\beta\bar{\lambda}_{LT}^{2}}}\quad \text{ale}\quad \begin{array}{l}\chi_{LT}\le 1{,}0\\ \chi_{LT}\le\dfrac{1}{\bar{\lambda}_{LT}^{2}}\end{array}$$

$$\varphi_{LT}=0{,}5\cdot\left[1+\alpha_{LT}(\bar{\lambda}_{LT}-\bar{\lambda}_{LT0})+\beta\bar{\lambda}_{LT}^{2}\right]$$

$$\beta=0{,}75$$

imperfection factor in Table 6.5

h/b = 330/160 = 2,06 > 2

Buckling curve c - imperfection factor $\alpha_{LT}=0{,}49$

$$\varphi_{LT}=0{,}5\left[1+0{,}49\cdot(1{,}286-0{,}4)+0{,}75\cdot1{,}286^{2}\right]=1{,}337$$

$$\chi_{LT}=\frac{1}{1{,}337+\sqrt{1{,}337^{2}-0{,}75\cdot1{,}286^{2}}}=0{,}482$$

$$0{,}482<1\quad \text{ok}$$

$$0{,}482<1/1{,}286^{2}=0{,}604\quad \text{ok}$$

modification parameter f:

$$f=1-0{,}5(1-k_c)\left[1-2(\bar{\lambda}_{LT}-0{,}8)^{2}\right]\quad \text{, ale}\quad f\le1{,}0$$

for the parabolic shape of the moment diagram the Table 6.6 quotes

$$k_c=0{,}94$$

$$f=1-0{,}5(1-0{,}95)\left[1-2(1{,}286-0{,}8)^{2}\right]=0{,}987$$

the resulting reduction factor according to (6.58)

$$\chi_{LT,mod}=\frac{\chi_{LT}}{f}=\frac{0{,}482}{0{,}987}=0{,}488$$

0,488 < 1 ok

0,488 < 0,604 ok

The design moment resistance according to (6.55)

$$M_{b,Rd}=\chi_{LT,mod}W_{pl,y}\frac{f_y}{\gamma_{M1}}=0{,}488*804{,}0*235/1{,}0=92{,}202\,kNm$$

Check:

90,49/92,202 = 98,1% pass

Lateral torsional buckling check - rolled section or equivalent welded section

Item name	Symbol	Value	Unit	Reference
Reduction factor	$\chi_{LT,mod}$	0,49	-	(6.58)
Slenderness	λ_{LT}	1,29	-	6.3.2.2 (1)
Correction factor	k_c	0,94	-	Table 6.6
	f	0,98	-	6.3.2.3 (2)
	$\lambda_{LT,0}$	0,40	-	6.3.2.3 (1)
	β	0,75	-	6.3.2.3 (1)
Buckling curve for LTB		c		Table 6.5
	α_{LT}	0,49	-	Table 6.3
Buckling factor	k_w	1,00	-	EN1999-1-1:I.1.2 (1)
Buckling factor	k_z	1,00	-	EN1999-1-1:I.1.2 (1)
Length between lateral suports	L	5,70	m	
Considered moment diagram type				
C1		1,13	-	
C2		0,46	-	
C3		0,53	-	
Parameter of symmetry	z_j	0	mm	EN1999-1-1:I.1.2 (1)
Load position related to shear centre	z_g	165	mm	EN1999-1-1:I.1.2 (1)
Critical moment	M_{cr}	114,16	kNm	6.3.2.2 (2)
Resistance moment	$M_{b,Rd}$	92,36	kNm	6.3.2.1 (3)
Utilisation	UC	97,97	%	6.3.2.1 (1)

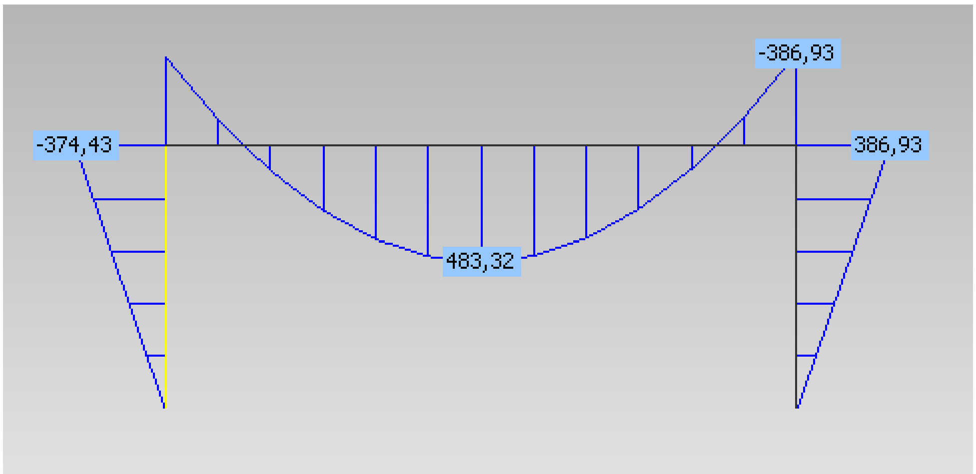
3. Stability of simple frame

Example was prepared according to [4]. Enclosed calculations focus only on the part of the stability check. Calculation of input data is given in [4].

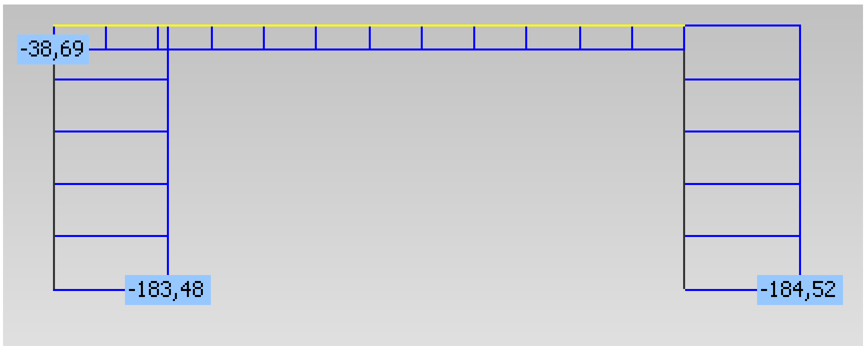
Input

Hinged frame with the course of internal forces:

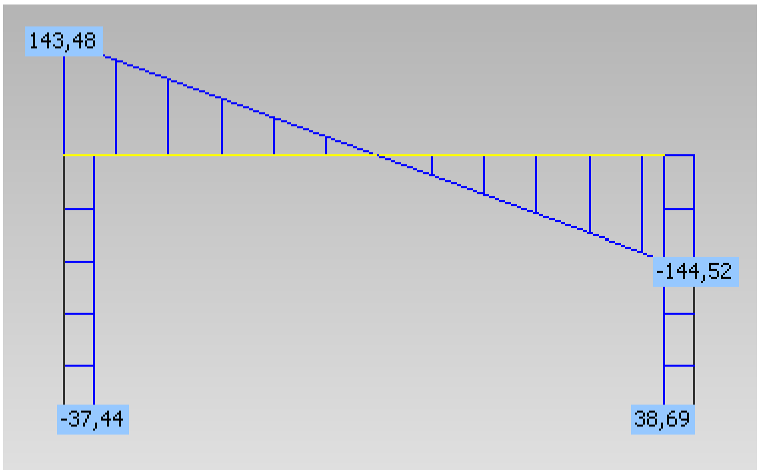
My



N



Vz



Columns: HEB section 340, steel grade S235

Girder: IPE section 550, steel grade S235

Span: 24 m

Height: 10 m

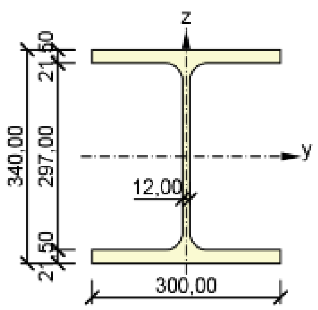
Check of column

Internal Forces

Position [m]	Combination	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
10,00	CO1 /ULS	-184,52	0,00	38,69	0,00	386,93	0,00

7.2. Cross-section: HEB340

Cross-section characteristics



Symbol	Value	Unit
A	17090,00	mm2
Iu	366600000,00	mm4
Iv	96900000,00	mm4
It	2572000,00	mm4
Iw	2462153985839,84	mm6
Wel,u	2156470,59	mm3
Wel,v	646000,00	mm3
Wpl,u	2400000,00	mm3
Wpl,v	986000,00	mm3

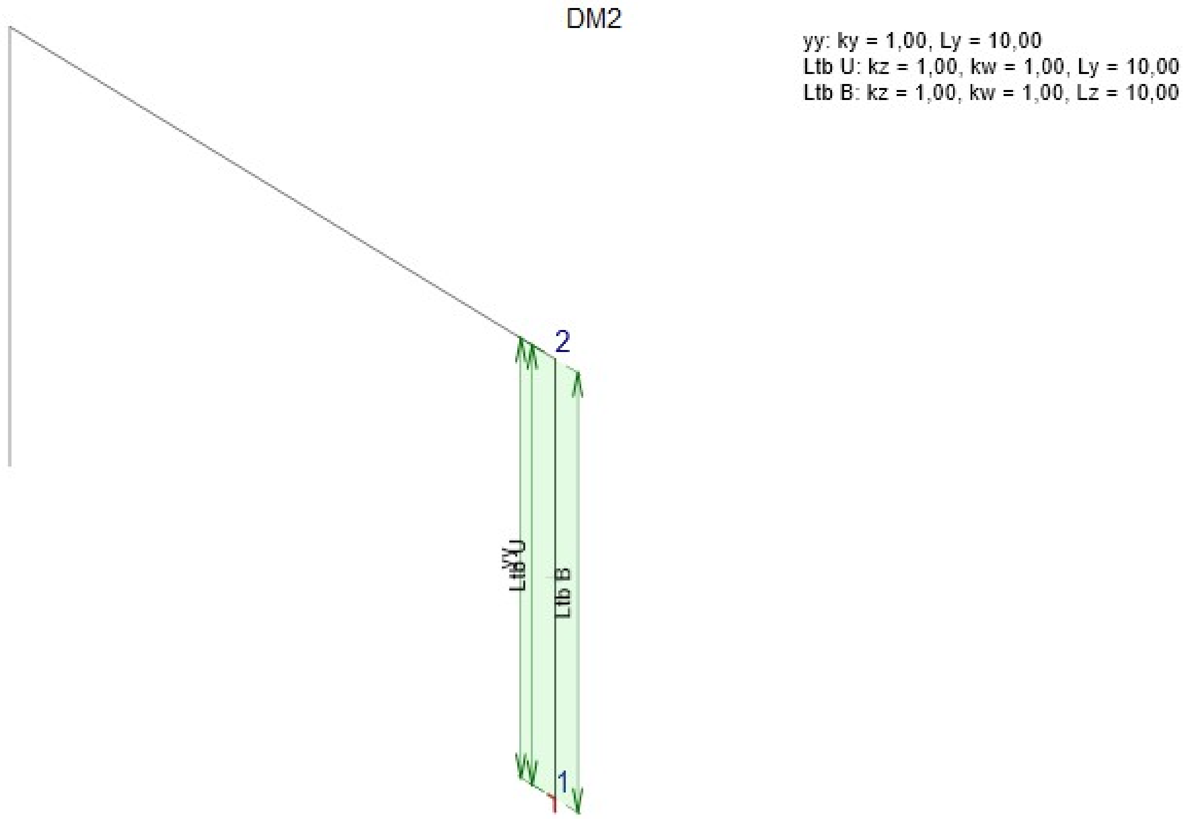
Cross-section class 1 (classification procedure is described in the previous examples)

Cross-section classification

	σ1 [MPa]	σ2 [MPa]	ψ [-]	α [-]	c/t [-]	CL1 [-]	CL2 [-]	CL3 [-]	Class
Web	-235,00	208,32	-0,89	0,63	20,25	54,62	62,89	111,27	1
Left top flange	-235,00	-235,00	1,00	1,00	5,44	9,00	10,00	14,00	1
Right top flange	-235,00	-235,00	1,00	1,00	5,44	9,00	10,00	14,00	1
Left bottom flange	208,32	208,32	0,00	0,00	5,44	0,00	0,00	0,00	1
Right bottom flange	208,32	208,32	0,00	0,00	5,44	0,00	0,00	0,00	1

Check of the column - flexural buckling

$E = 210000\, N/mm^2$



$L_{cr,y} = 10\text{m}$

$L_{cr,z} = 10\text{m}$

Flexible critical force related to appropriate buckling shape:

$$N_{cr,y} = \frac{\pi^2 EI_y}{L_{cr,y}^2} = \frac{\pi^2 \cdot 21 \cdot 10^{10} \cdot 366{,}6 \cdot 10^{-6}}{10^2} = 7598{,}21\,kN$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{L_{cr,z}^2} = \frac{\pi^2 \cdot 21 \cdot 10^{10} \cdot 96{,}6 \cdot 10^{-6}}{10^2} = 2008{,}37\,kN$$

relative slenderness 6.3.1.2(1):

$$\bar{\lambda}_y = \sqrt{\frac{Af_y}{N_{cr,y}}} = \sqrt{\frac{17090 \cdot 235}{7598210}} = 0{,}727$$

$$\bar{\lambda}_z = \sqrt{\frac{Af_y}{N_{cr,z}}} = \sqrt{\frac{17090 \cdot 235}{2008370}} = 1{,}414$$

Buckling coefficient:

Value χ corresponding to the relative slenderness $\bar{\lambda}$, to be determined according to equation (6.49).

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

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

α is the imperfection factor, to be determined by tables 6.1 and 6.2

In Table 6.2, the corresponding line for I-section is selected according to parameters:
$h/b=340/300=1{,}133<1{,}2$ a $t_f=21{,}5<100\,mm$

- buckling to axis y-y:

Buckling curve **b** -> imperfection factor $\alpha=0{,}34$

$$\varphi_y=0{,}5\left[1+0{,}34\cdot(0{,}727-0{,}2)+0{,}727^2\right]=0{,}854$$

$$\chi_y=\frac{1}{0{,}854+\sqrt{0{,}854^2-0{,}727^2}}=0{,}768$$

$$N_{b,Rd}=\chi_y\frac{Af_y}{\gamma_{M1}}=\frac{0{,}768\cdot 17090\cdot 235}{1{,}0}=3085{,}3\,kN>184{,}5\,kN \quad \text{vyhoví}$$

- buckling to axis z-z:

Buckling curve **c** -> imperfection coefficient $\alpha=0{,}49$

$$\varphi_y=0{,}5\left[1+0{,}49\cdot(1{,}414-0{,}2)+1{,}414^2\right]=1{,}797$$

$$\chi_z=\frac{1}{1{,}797+\sqrt{1{,}797^2-1{,}414^2}}=0{,}344$$

$$N_{b,Rd}=\chi_z\frac{Af_y}{\gamma_{M1}}=\frac{0{,}344\cdot 17090\cdot 235}{1{,}0}=1381{,}7\,kN>184{,}5\,kN \quad \text{vyhoví}$$

Flexural buckling check

Item name	Symbol	Value Y-Y	Value Z-Z	Unit	Reference
Reduction factor	χ	0,77	0,34	-	6.3.1.2 (1)
Slenderness	λ	0,73	1,41	-	6.3.1.2 (1)
Buckling curve		b	c		Tab. 6.2
Imperfection factor	α	0,34	0,49	-	6.3.1.2 (1)
Buckling factor	k	1,00	1,00	-	
Critical length	L_{cr}	10,00	10,00	m	6.3.1.3 (1)
Critical force	N_{cr}	7598,21	2008,37	kN	6.3.1.2 (1)
Resistance force	$N_{b,Rd}$	3085,33	1381,71	kN	6.3.1.1 (3)
Utilisation	UC	5,98	13,35	%	6.3.1.1 (1)

Check of the column - LT buckling

In the example [4] a calculation of a general reduction factor was performed according to 6.3.2.2.

Code settings was therefore changed so that the Section 6.3.2.2 was used in calculation.

3.7 If possible, determine LTB curve acc. to eq. (6.57)

Calculation of the critical moment Mcr:

(According to EN1999-1-1, Annex I)

kz = 1,0

kw = 1,0

L = 10m

The critical moment by I.1.2 (I.2)

$$M_{cr}=\mu_{cr}\frac{\pi\sqrt{EI_z GI_t}}{L}\quad \text{kde}$$

$$\mu_{cr}=\frac{C_1}{k_z}\left[\sqrt{1+\kappa_{wt}^2+(C_2\zeta_g-C_3\zeta_j)^2}-(C_2\zeta_g-C_3\zeta_j)\right]$$

$$\kappa_{wt}=\frac{\pi}{k_w L}\sqrt{\frac{EI_w}{GI_t}}=\frac{\pi}{1{,}0\cdot 10{,}0}\sqrt{\frac{21\cdot 10^{10}\cdot 246{,}22\cdot 10^{-8}}{8{,}077\cdot 10^{10}\cdot 257{,}2\cdot 10^{-8}}}=0{,}496$$

parameters ζ_g and ζ_j can be set equal to 0. They does not take effect in calculation.

For linear shape of the bending moment My diagram, we determine the parameters from Table I.1

$$C_1=C_{1,0}+(C_{1,1}-C_{1,0})\kappa_{wt}=1{,}77+(1{,}85-1{,}77)\cdot 0{,}496=1{,}81$$

$$C_2=0$$

$$C_3=1{,}0$$

$$\mu_{cr}=\frac{1{,}81}{1{,}0}\left[\sqrt{1+0{,}496^2}\right]=2{,}02$$

$$M_{cr}=\frac{2{,}02\cdot\pi\cdot\sqrt{21\cdot 10^{10}\cdot 9690\cdot 10^{-8}\cdot 8{,}077\cdot 10^{10}\cdot 257{,}2\cdot 10^{-8}}}{10{,}0}=1304{,}768\,kNm$$

Relative slenderness:

$$\bar{\lambda}_{LT}=\sqrt{\frac{W_{pl,y}f_y}{M_{cr}}}=\sqrt{\frac{0{,}0024\cdot 235\cdot 10^6}{1304768}}=0{,}657$$

$$\bar{\lambda}_{LT0}=0{,}4\quad \text{for rolled sections}$$

$$\bar{\lambda}_{LT}=0{,}657>\bar{\lambda}_{LT0}=0{,}4\quad \text{LT buckling can not be ignored}$$

Buckling coefficient according to 6.3.2.2:

$$\chi_{LT}=\frac{1}{\varphi_{LT}+\sqrt{\varphi_{LT}^2-\bar{\lambda}_{LT}^2}}\quad \text{ale}\quad \chi_{LT}\le 1{,}0$$

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

imperfection factor in Table 6.3

h/b = 360/300 = 1,2 < 2

Buckling curve **a** -> imperfection factor $\alpha_{LT} = 0{,}21$

$$\varphi_{LT} = 0{,}5\left[1 + 0{,}21 \cdot (0{,}657 - 0{,}2) + 0{,}657^2\right] = 0{,}764$$

$$\chi_{LT} = \frac{1}{0{,}764 + \sqrt{0{,}764^2 - 0{,}657^2}} = 0{,}867$$

0,867< 1 ok

The design moment resistance according to (6.55)

$$M_{b,Rd} = \chi_{LT} W_{pl,y} \frac{f_y}{\gamma_{M1}} = 0{,}867 \cdot 0{,}0024 \cdot 235 \cdot 10^6 / 1{,}0 = 488{,}988\, kNm$$

Check:

386,93 / 488,988 = 79,1% pass

Lateral torsional buckling check - general case

Item name	Symbol	Value	Unit	Reference
Reduction factor	χ_{LT}	0,87	-	6.3.2.2 (1)
Slenderness	λ_{LT}	0,66	-	6.3.2.2 (1)
Buckling curve for LTB		a		Table 6.4
	α_{LT}	0,21	-	Table 6.3
	$\lambda_{LT,0}$	0,40	-	6.3.2.3 (1)
Buckling factor	k_w	1,00	-	EN1999-1-1:I.1.2 (1)
Buckling factor	k_z	1,00	-	EN1999-1-1:I.1.2 (1)
Length between lateral suports	L	10,00	m	
Considered moment diagram type				
C1		1,81	-	
C2		0,00	-	
C3		1,00	-	
Parameter of symmetry	z_j	0	mm	EN1999-1-1:I.1.2 (1)
Load position related to shear centre	z_g	170	mm	EN1999-1-1:I.1.2 (1)
Critical moment	M_{cr}	1303,51	kNm	6.3.2.2 (2)
Resistance moment	$M_{b,Rd}$	488,81	kNm	6.3.2.1 (3)
Utilisation	UC	79,08	%	6.3.2.1 (1)

Check of the column - interaction by the N + V according to 6.3.3

Interaction of axial force and bending under Article 6.3.3. Interaction coefficients are determined by the method 2 of Annex B

Members stressed by a combination of bending and axial pressure must satisfy the condition (6.61) and (6.62)

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

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

The column is not bended around an axis zz. Interaction coefficients kyz and kzz need not be determined.

Column is prone to torsion.

Coefficients kyy and kzy are determined from Table B.2:

$$k_{yy}=C_{my}\left(1+(\bar{\lambda}_y-0{,}2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\leq C_{my}\left(1+0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$$

$$k_{zy}=\left[1-\frac{0{,}1\bar{\lambda}_z}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]\geq\left[1-\frac{0{,}1}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$$

$C_{my}=0{,}9$ sway

$C_{mLT}=0{,}6+0{,}4\psi=0{,}6>0{,}4$ for linear moment diagram My ($\psi=0$)

$$k_{yy}=0{,}9\cdot\left(1+(0{,}727-0{,}2)\frac{184{,}5}{0{,}768\cdot 4016{,}15/1{,}0}\right)\leq 0{,}9\cdot\left(1+0{,}8\frac{184{,}5}{0{,}768\cdot 4016{,}15/1{,}0}\right)$$

$$k_{yy}=0{,}928\leq 0{,}943\rightarrow k_{yy}=0{,}928$$

$$k_{zy}=\left[1-\frac{0{,}1\cdot 1{,}414}{(0{,}6-0{,}25)}\frac{184{,}5}{0{,}344\cdot 4016{,}15/1{,}0}\right]\geq\left[1-\frac{0{,}1}{(0{,}6-0{,}25)}\frac{184{,}5}{0{,}344\cdot 4016{,}15/1{,}0}\right]$$

$$k_{zy}=0{,}946\ \textit{but should be}\geq 0{,}962\rightarrow k_{zy}=0{,}962$$

after substituting into equations (6.61) and (6.62)

$$\frac{184{,}5}{\frac{0{,}768\cdot 4016{,}15}{1{,}0}}+0{,}928\cdot\frac{386{,}93}{\frac{488{,}988}{1{,}0}}=0{,}06+0{,}734=0{,}794<1\quad \text{pass}$$

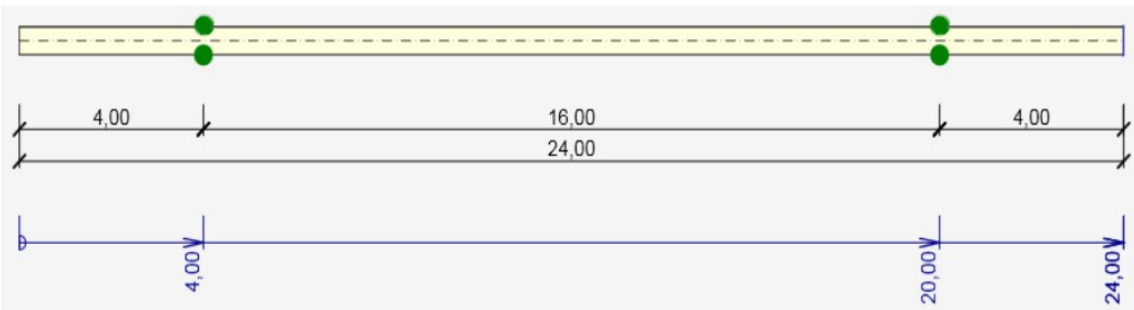
$$\frac{184,5}{\frac{0,344\cdot 4016,15}{1,0}}+0,962\cdot\frac{386,93}{\frac{488,988}{1,0}}=0,134+0,761=0,895<1 \quad \text{pass}$$

Combined stability check in case of bending and axial compression - alternative method 2

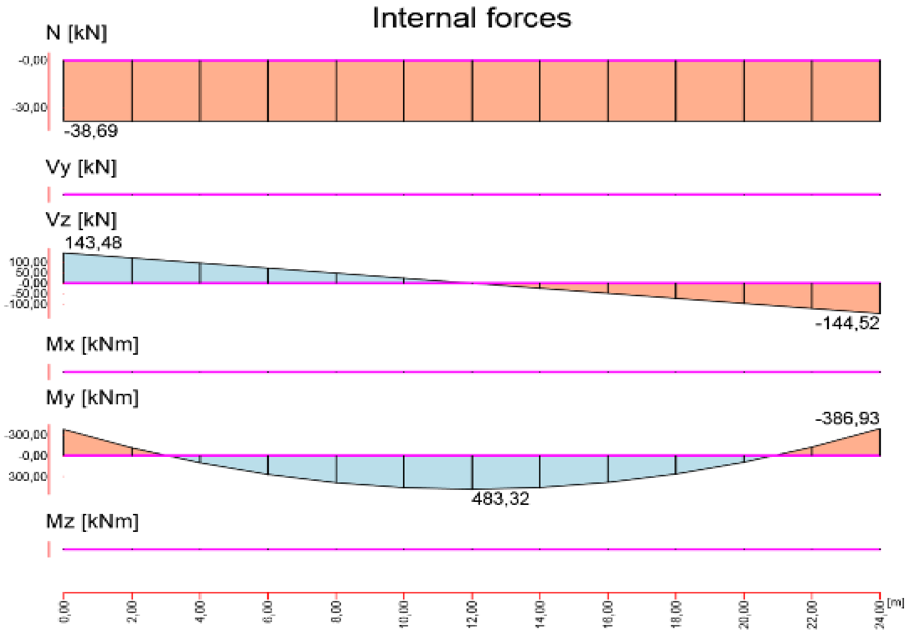
Item name	Symbol	Value	Unit	Reference
	C_{my}	0,90	-	
	$M_{h,z}$	0,00	kNm	Table B.3
	$M_{s,z}$	0,00	kNm	Table B.3
	Ψ_z	1,00	-	Table B.3
	$\alpha_{s,z}$	0,00	-	Table B.3
	C_{mz}	1,00	-	
	$M_{h,LT}$	0,00	kNm	Table B.3
	$M_{s,LT}$	309,54	kNm	Table B.3
	Ψ_{LT}	0,00	-	Table B.3
	$\alpha_{s,LT}$	0,00	-	Table B.3
	C_{mLT}	0,60	-	
	k_{yy}	0,93	-	
	k_{yz}	0,71	-	
	k_{zy}	0,96	-	
	k_{zz}	1,19	-	
	N_{Ed}	-184,52	kN	6.3.3 (4)
	$M_{y,Ed}$	386,93	kNm	6.3.3 (4)
	$M_{z,Ed}$	0,00	kNm	6.3.3 (4)
	N_{Rk}	4016,15	kN	6.3.3 (4)
	$M_{y,Rk}$	564,00	kNm	6.3.3 (4)
	$M_{z,Rk}$	231,71	kNm	6.3.3 (4)
Utilisation	UC	79,47	%	6.3.3 (4) (6.61)
Utilisation	UC	89,49	%	6.3.3 (4) (6.62)

Check of frame girder

Frame girder is laterally supported at the ends and the bottom flange at a distance of 4 meters from the end. (See picture)



The check is performed in frame corner, where is the maximum negative moment. (Position 24m)



Check of frame girder - classification

Procedure for the classification of the flange has been demonstrated in previous examples.

Classification of web

$$c = H - 2t_f - 2r = 550 - 2 \cdot 17{,}2 - 2 \cdot 24 = 467{,}6\,mm$$

part of the web is plastized by axial force

$$z = \frac{N_{Ed}}{t_w f_{yd}} = \frac{38690}{11{,}1 \cdot 235} = 14{,}94$$

$$\alpha = \frac{0{,}5(c + z)}{c} = \frac{0{,}5 \cdot (467{,}6 + 14{,}94)}{467{,}6} = 0{,}516$$

$$c/t_w = 467{,}6/11{,}1 = 42{,}1$$

The limit value for class 1 (tabulka 5.2) pro $\alpha > 0{,}5$

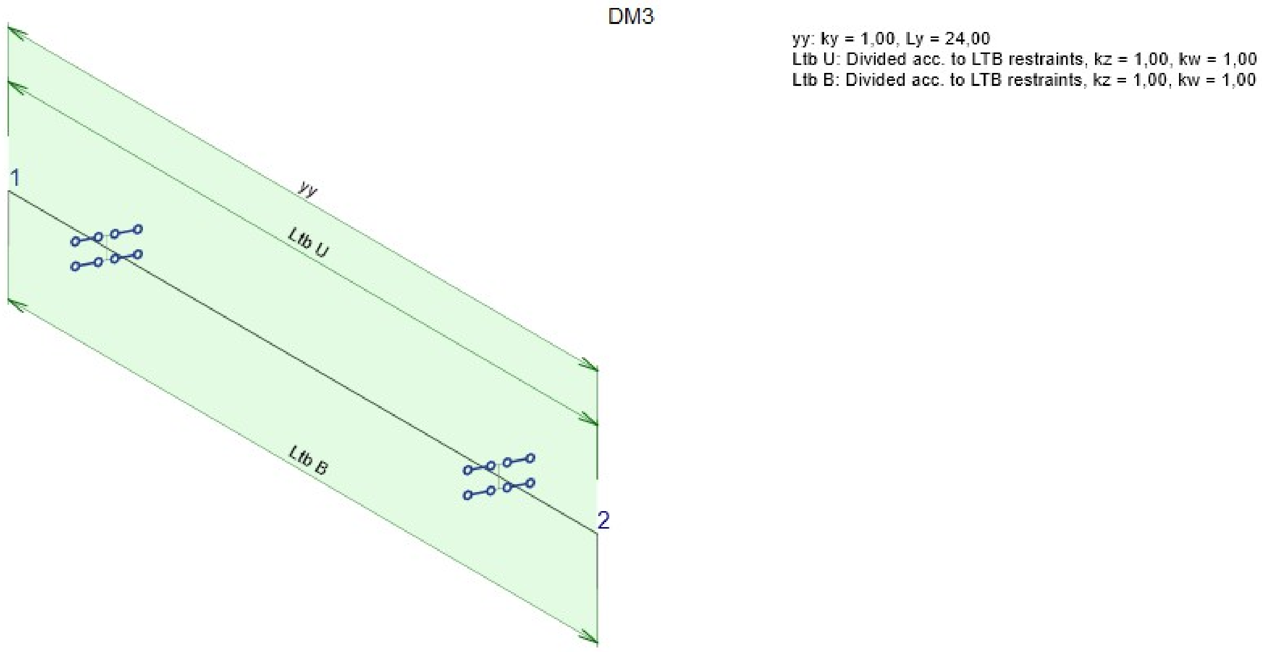
$$\frac{396\,\varepsilon}{13\,\alpha - 1} = \frac{396 \cdot 1{,}0}{13 \cdot 0{,}516 - 1} = 69{,}38$$

Classification of web: $69{,}38 > 42{,}1 \rightarrow$ **Class 1**

Cross-section classification

	σ1 [MPa]	σ2 [MPa]	ψ [-]	α [-]	c/t [-]	CL1 [-]	CL2 [-]	CL3 [-]	Class
Web	226,62	-235,00	-0,96	0,52	42,13	69,40	79,91	119,39	1
Left top flange	226,62	226,62	0,00	0,00	4,39	0,00	0,00	0,00	1
Right top flange	226,62	226,62	0,00	0,00	4,39	0,00	0,00	0,00	1
Left bottom flange	-235,00	-235,00	1,00	1,00	4,39	9,00	10,00	14,00	1
Right bottom flange	-235,00	-235,00	1,00	1,00	4,39	9,00	10,00	14,00	1

Check of frame girder - flexural buckling



Buckling lengths

$$L_{cr,y}=24\text{m}$$

To determine the buckling length about zz axis we take into account in the case of LT restraints the sign of bending moment. Buckling length will be determined from the distance of lateral support of the compressed flange.

Moment at the frame corner has a negative sign - the bottom flange is in compression. The bottom flange is laterally supported at 4m.

$$L_{cr,z}=4\text{m}$$

The calculation procedure is the same as for the column.

Flexural buckling check

Item name	Symbol	Value Y-Y	Value Z-Z	Unit	Reference
Reduction factor	χ	0,57	0,63	-	6.3.1.2 (1)
Slenderness	λ	1,14	0,96	-	6.3.1.2 (1)
Buckling curve		a	b		Tab. 6.2
Imperfection factor	α	0,21	0,34	-	6.3.1.2 (1)
Buckling factor	k	1,00	1,00	-	
Critical length	L_{cr}	24,00	4,00	m	6.3.1.3 (1)
Critical force	N_{cr}	2415,17	3456,09	kN	6.3.1.2 (1)
Resistance force	$N_{b,Rd}$	1789,54	1974,52	kN	6.3.1.1 (3)
Utilisation	UC	2,16	1,96	%	6.3.1.1 (1)

Check of frame girder - LT buckling

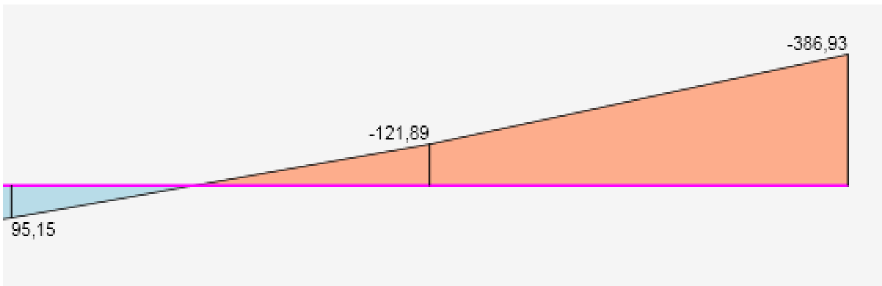
To determine the critical moment Mcr on a beam with LT restraints, the buckling length for each checked section is set according to the sign of bending moment in the section.

L = 4m

kw = 1

kz = 1

course of moment My in the span of buckling



The parameters C1, C2: for the parabolic course of moments with different moments at the ends

the Belgian national annex NBN EN 1993-1-1 ANB recomendes diagrams D.4 D.6

$$\mu=\frac{qL^2}{8M}=\frac{12\cdot 4^2}{8\cdot -386{,}9}=-0{,}062$$

$$\psi=95/-386{,}9=-0{,}246$$

C1 = 2,24

C2 = 0,05

Lateral torsional buckling check - general case

Item name	Symbol	Value	Unit	Reference
Reduction factor	χ_{LT}	0,87	-	6.3.2.2 (1)
Slenderness	λ_{LT}	0,53	-	6.3.2.2 (1)
Buckling curve for LTB		b		Table 6.4
	α_{LT}	0,34	-	Table 6.3
	$\lambda_{LT,0}$	0,40	-	6.3.2.3 (1)
Buckling factor	k_w	1,00	-	EN1999-1-1:I.1.2 (1)
Buckling factor	k_z	1,00	-	EN1999-1-1:I.1.2 (1)
Length between lateral suports	L	4,00	m	
Considered moment diagram type				
C1		2,24	-	
C2		0,05	-	
C3		0,00	-	
Parameter of symmetry	z_j	0	mm	EN1999-1-1:I.1.2 (1)
Load position related to shear centre	z_g	275	mm	EN1999-1-1:I.1.2 (1)
Critical moment	M_{cr}	2353,87	kNm	6.3.2.2 (2)
Resistance moment	$M_{b,Rd}$	569,77	kNm	6.3.2.1 (3)
Utilisation	UC	67,44	%	6.3.2.1 (1)

Check of frame girder - interaction by 6.3.3 - method 2

For parameter C_{my} we consider the span between LT restraints preventing displacement in the z direction Such restraints are at the ends of the girder. We take into account the entire girder.

$$M_h = -386{,}93\,kNm$$

$$M_s = 483{,}32\,kNm$$

$$\psi = 0{,}97$$

$$\alpha_h = M_h / M_s = -386{,}93/483{,}32 = -0{,}8$$

from Table B.3 for uniform load

$$C_{my} = 0{,}95 + 0{,}05\,\alpha_h = 0{,}95 + 0{,}05 \cdot (-0{,}8) = 0{,}91$$

For parameter C_{mLT} we consider the span between LT restraints preventing displacement in the y direction. Such restraints are at the ends of the girder and 4m from the end. We take into account the same span as for LT buckling.

$$M_h = -386{,}93\,kNm$$

$$M_s = -121{,}89\,kNm$$

$$\psi = -0{,}25$$

$$\alpha_s = M_s / M_h = -121{,}89/-386{,}93 = 0{,}315$$

from Table B.3 for uniform load

$$C_{mLT}=0,2+0,8\alpha_h=0,2+0,8\cdot(0,315)=0,452$$

Frame girder is not bended around an axis zz. Interaction coefficients kyz and kzz need not be determined.

Frame girder is prone to torsion.

Coefficients kyy and kzy are determined from Table B.2:

$$k_{yy}=C_{my}\left(1+(\bar{\lambda}_y-0,2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\leq C_{my}\left(1+0,8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$$

$$k_{yy}=0,91\cdot\left(1+(1,14-0,2)\frac{38,69}{0,57\cdot 3158,4/1,0}\right)\leq 0,91\cdot\left(1+0,8\frac{38,69}{0,57\cdot 3158,4/1,0}\right)$$

$$k_{yy}=0,928\leq 0,925 \rightarrow k_{yy}=0,925$$

$$k_{zy}=\left[1-\frac{0,1\bar{\lambda}_z}{(C_{mLT}-0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]\geq\left[1-\frac{0,1}{(C_{mLT}-0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$$

$$k_{zy}=\left[1-\frac{0,1\cdot 0,96}{(0,452-0,25)}\frac{38,69}{0,63\cdot 3158,4/1,0}\right]\geq\left[1-\frac{0,1}{(0,452-0,25)}\frac{38,69}{0,63\cdot 3158,4/1,0}\right]$$

$$k_{zy}=0,991\geq 0,990 \rightarrow k_{zy}=0,991$$

after substituting into equations (6.61) and (6.62)

$$\frac{38,69}{\frac{0,57\cdot 3158,4}{1,0}}+0,925\cdot\frac{386,93}{\frac{569,77}{1,0}}=0,022+0,63=0,65<1,0 \quad \text{pass}$$

$$\frac{38,69}{\frac{0,63\cdot 3158,4}{1,0}}+0,991\cdot\frac{386,93}{\frac{569,77}{1,0}}=0,019+0,673=0,692<1 \quad \text{pass}$$

Combined stability check in case of bending and axial compression - alternative method 2

Item name	Symbol	Value	Unit	Reference
	$M_{h,y}$	-386,93	kNm	Table B.3
	$M_{s,y}$	483,32	kNm	Table B.3
	Ψ_{y}	0,97	-	Table B.3
	$\alpha_{s,y}$	-0,80	-	Table B.3
	C_{my}	0,91	-	
	$M_{h,z}$	-386,93	kNm	Table B.3
	$M_{s,z}$	0,00	kNm	Table B.3
	Ψ_{z}	1,00	-	Table B.3
	$\alpha_{s,z}$	0,00	-	Table B.3
	C_{mz}	1,00	-	
	$M_{h,LT}$	-386,93	kNm	Table B.3
	$M_{s,LT}$	-121,89	kNm	Table B.3
	Ψ_{LT}	-0,25	-	Table B.3
	$\alpha_{s,LT}$	0,32	-	Table B.3
	C_{mLT}	0,45	-	
	k_{yy}	0,93	-	
	k_{yz}	0,62	-	
	k_{zy}	0,99	-	
	k_{zz}	1,03	-	
	N_{Ed}	-38,69	kN	6.3.3 (4)
	$M_{y,Ed}$	386,93	kNm	6.3.3 (4)
	$M_{z,Ed}$	0,00	kNm	6.3.3 (4)
	N_{Rk}	3158,40	kN	6.3.3 (4)
	$M_{y,Rk}$	653,30	kNm	6.3.3 (4)
	$M_{z,Rk}$	94,00	kNm	6.3.3 (4)
Utilisation	UC	65,03	%	6.3.3 (4) (6.61)
Utilisation	UC	69,24	%	6.3.3 (4) (6.62)

4. Fire resistance of welded box section

Example was prepared according to [5].

Introduction

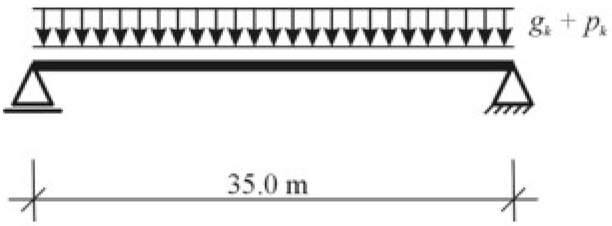


Figure 1: Static system

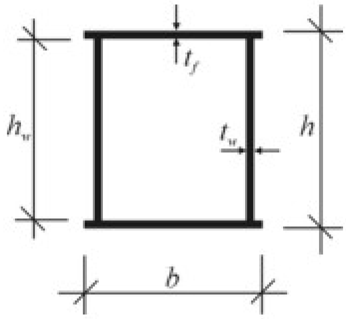


Figure 2: Cross-section

Basic data

Cross- section:

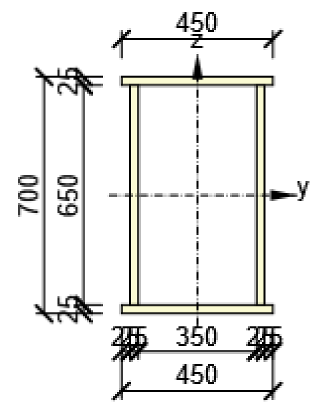
Steel grade:	S 355
Yield stress:	fy = 355 N/mm²
Height:	h = 700 mm
Height of web	hw = 650 mm
Width:	b = 450 mm
Thickness of flange:	tf = 25 mm
Thickness of web:	tw = 25 mm
Cross-sectional area of the flange:	Af = 11250 mm²
Cross-sectional area of the web:	Aw = 16250 mm²
Specific heat:	ca = 600 J/(kg·K)
Density:	ρa = 7850 kg/m³
Emissivity of the beam:	εm = 0,7
Emissivity of the fire:	εr = 1,0
Configuration factor	Φ = 1,0

Coefficient of heat transfer: αc = 25,0 W/m²K

Stephan Boltzmann constant: σ = 5,67 × 10−8 W/m²K

BoxFl700x(450/450)

Symbol	Value	Unit
Material 1	S 355	
Material 2	S 355	
Material 3	S 355	
Material 4	S 355	
A	55000	[mm²]
I_u	3708333333	[mm⁴]
I_v	1523958333	[mm⁴]
I_t	3051060268	[mm⁴]
I_w	0	[mm⁶]
$W_{el,u}$	10595238	[mm³]
$W_{el,v}$	6773148	[mm³]
$W_{pl,u}$	12875000	[mm³]
$W_{pl,v}$	8625000	[mm³]



Loads:

Permanent actions:

Beam: g a,k = 4,32 kN/m

Roof: g r,k = 5,0 kN/m

Variable actions:

Snow: p s,k = 11,25 kN/m

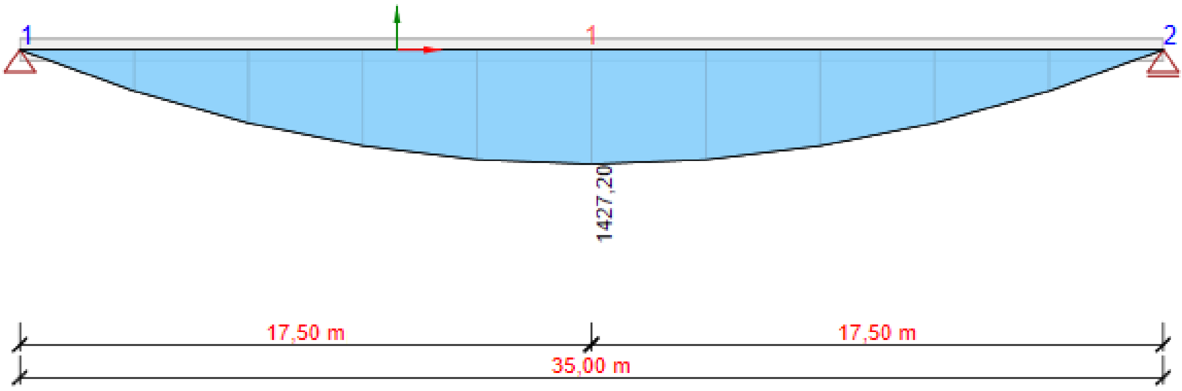
Combinations:

$$E_{dA}=E\left(\sum G_k+A_d+\sum \psi_{2,i}\cdot Q_{k,i}\right)$$

The combination factor for snow loads is $\psi_{2,i}=0,0$.

	Name	Type	Evaluation		Self Weight	střecha	sníh
	ULS fire	ULS Fundam	Linear		1	1	0
>	SLSC	SLS Char	Code		1	1	1

Internal forces



Bending moment My, Combination ULS fire

Calculation of the steel temperature

The temperature increase of steel section is calculated to:

$$\Delta\theta_a,t = k_{sh} \cdot \frac{A_m/V}{c_a \cdot \rho_a} \cdot h_{net,d} \cdot \Delta t$$

where:

k_{sh} is the correction factor for shadow effect (k_{sh} = 1,0),

Δt is the time interval (5 seconds)

c_a is the specific heat (J/kgK), dependant on steel temperature (EN1993-1-2 §3.4.1.2)

A_m/V is section factor for unprotected member

In our case:

$$A_m/V = \frac{(0{,}45\,.\,2+0{,}025\,.\,8+0{,}65\,.\,2)}{0{,}055} = 43{,}64$$

(Note: there is used simplified formula 1/t in example [5], which results to little difference in calculated cross-section temperature)

The net heat flux is calculated according to EN 1991 Part 1-2:

$$h_{net} = \alpha_c\,.\,(\theta_g - \theta_m) + \Phi\,.\,\varepsilon_m\,.\,\varepsilon_r\,.\,\sigma \cdot \left((\theta_g+273)^4 - (\theta_m+273)^4\right)$$

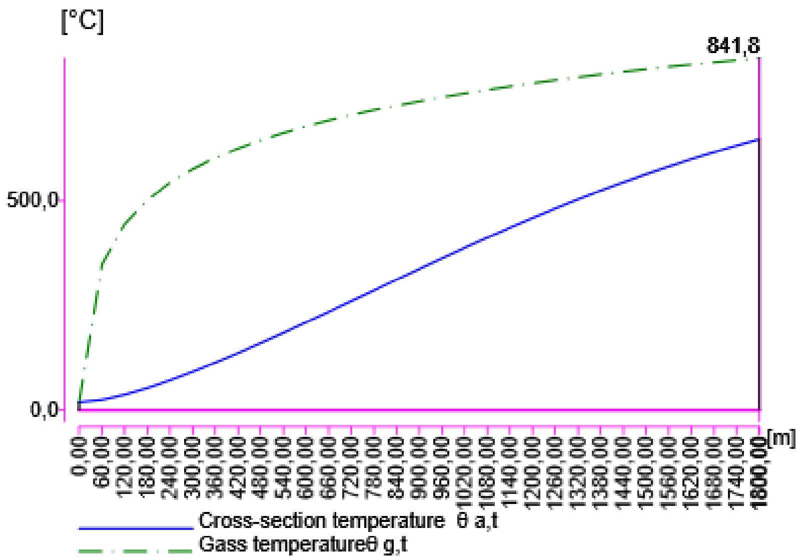
$$h_{net} = 25\,.\,(\theta_g - \theta_m) + 1{,}0\,.\,0{,}7\,.\,1{,}0\,.\,5{,}67\,.\,10^{-8} \cdot \left((\theta_g+273)^4 - (\theta_m+273)^4\right)$$

The standard temperature-time curve

$$\theta_g = 20 + 345\,.\,\log_{10}(8.t+1)$$

Calculation of cross-section temperature

min	sec	t [s]	θ_g [°C]	$h_{net,r}$ [W/m^2]	$h_{net,c}$ [W/m^2]	$h_{net,d}$ [W/m^2]	c_a [kJ/(kg.K)]	θ_a [°C]
0	0	0	20,00	0,00	0,00	0,00	0,4398	20,00
0	5	5	96,54	447,63	1913,45	2361,07	0,4398	20,00
0	10	10	146,95	941,38	3170,22	4111,60	0,4399	20,14
0	15	15	184,61	1446,32	4105,37	5551,69	0,4401	20,39
0	20	20	214,67	1949,47	4848,64	6798,11	0,4403	20,73
...	...	...	...	...	...	...	...	...
29	45	1785	840,55	32930,85	4907,42	37838,27	0,8047	644,25
29	50	1790	840,96	32868,63	4886,54	37755,16	0,8066	645,50
29	55	1795	841,38	32806,32	4865,77	37672,09	0,8085	646,75
30	0	1800	841,80	32743,94	4845,12	37589,06	0,8105	647,99
30	5	1805	842,21	32681,50	4824,58	37506,09	0,8125	649,23



Calculation of cross-section temperature

Item name	Symbol	Value	Unit	Reference
Protection type		Not protected		
Cross-section temperature	$\theta_{a,t}$	647,99	°C	4.2.5
Reduction factor for effective yield strength	$k_{y,\theta}$	0,35	-	3.2.1
Reduction factor for proportional limit	$k_{p,\theta}$	0,13	-	3.2.1
Reduction factor for the slope of the linear elastic range	$k_{E,\theta}$	0,22	-	3.2.1

Cross-section classification

Article 4.2.2

$$\varepsilon = 0{,}85\sqrt{235/\,fy} = \sqrt{235/355} = 0{,}6916$$

The upper flange is under compression - internal part

$$c = b - 2.\mathrm{t}_w - 2.\mathrm{b}_{tip} = 450 - 2.25 - 2.25 = 350$$

$c/t_f = 350/25 = 14$

The limit value for class 1

$33\varepsilon = 33 . 0{,}6916 = 22{,}8228$

Classification of flange: $22{,}8228 > 14 \rightarrow$ **Class 1**

Web subject to bending

$c = h_w = 650$

$c/t_f = 650/25 = 26$

The limit value for class 1

$72\varepsilon = 72 . 0{,}6916 = 49{,}795$

Classification of web: $49{,}795 > 26 \rightarrow$ **Class 1**

Cross-section is a Class 1

Verification in the strength domain

To calculate the cross-section resistance, the reduction factor $k_{y,\theta}$ has to be determined acc. to exression give in EN 1993-1-2 §4.2.3.3

$\theta_{a,30} = 648\,°C \rightarrow k_{y,\theta} = 0{,}3548$

Bending resistance

Adaptation factors are set to:

$\kappa_1 = 1{,}0$

$\kappa_2 = 1{,}0$

Design moment resistance is calculated to:

$$M_{fi,t,Rd} = M_{pl,Rd,20°C} . k_{y,\theta} . \frac{\gamma_{M1}}{\gamma_{M,fi}} . \frac{1}{\kappa_1 . \kappa_2}$$

$$M_{fi,t,Rd} = (0{,}012875 . 355 . 10^6 / 1.0) . 0{,}3548 . \frac{1{,}0}{1{,}0} . \frac{1}{1{,}0 . 1{,}0} = 1621{,}7\,kNm$$

$UC = \dfrac{1427{,}1}{1621{,}7} = 0{,}88$ **pass**

Bending moment and Shear check My + V acc. 6.2.8

Item name	Symbol	Value	Unit	Reference
Section modulus	$W_{pl,min}$	12875000	mm3	EN1993-1-1: (6.13)
Design resistance moment	$M_{c,Rd}$	4570,63	kNm	EN1993-1-1: 6.2.5 (2)
Design resistance monent of cross-section with uniform temperature	$M_{fi,\theta,Rd}$	1621,76	kNm	4.2.3.3 (4.8)
Utilisation	UC	88,00	%	6.2.5 (1)

Verification in the temperature domain

The design moment resistance during fire exposure at the time t=0 is needed to get the utilization factor.

$$M_{fi,Rd,0} = M_{c,Rd} = W_{pl} . f_y / \gamma_{M,fi} = 0{,}012875 . 355.10^6 / 1.0 = 4570{,}6\,kNm$$

The utilization factor is calculated using:

$$\mu_0 = E_{fi,d} / R_{fi,d,0} = M_{fi,d} / M_{fi,Rd,0} = 1427{,}2 / 4570{,}6 = 0{,}312$$

The critical temperature is given in Table 4.1 of EN 1993-1-2 as

$$\theta_{a,cr} = 657{,}7\,°C$$

$$UC = \frac{648}{657{,}7} = 0{,}985 \quad \textbf{pass}$$

Note: There have to be set the Calculation model to 'Temperature time domain'

EN1993-1-2 Structural fire design	
γ Mθ	1
Calculation model	Temperature time domain

Bending moment and Shear check My + V at time t = 0

Item name	Symbol	Value	Unit	Reference
Section modulus	$W_{pl,min}$	12875000	mm3	EN1993-1-1: (6.13)
Design resistance moment	$M_{c,Rd}$	4570,63	kNm	EN1993-1-1: 6.2.5 (2)
Design resistance monent of cross-section with uniform temperature	$M_{fi,\theta,Rd}$	4570,63	kNm	4.2.3.3 (4.8)
Utilisation	UC	31,23	%	6.2.5 (1)

Calculation of cross-section temperature

Item name	Symbol	Value	Unit	Reference
Cross-section temperature	$\theta_{a,t}$	647,99	°C	4.2.5
	μ_θ	31,23	%	4.2.4 (4.23)
	$\theta_{a,cr}$	657,70	°C	4.2.4 (4.22)
Utilisation	UC	98,52	%	4.2.4

5. Fire design of an unprotected IPE section beamexposed to the standard time temperature curve

The example was prepared according to [6].

Introduction

A beam made of hot-rolled IPE section is a part of the floor structure of an office building. The beam is loaded uniformly and restrained against lateral torsional buckling by a concrete slab. The beam is design to achieve a fire resistance rating of R15.

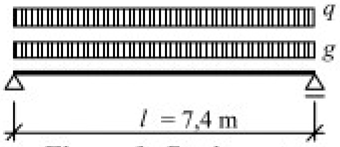


Figure 1: Static system

Basic data

Material properties

Steel grade: S 275

Yield stress: f_y = 275 N/mm²

Density: ρ_a = 7850 kg/m³

Loads

Permanent action:

g_k = 4,8 kN/m

Variable action:

q_k = 7,8 kN/m

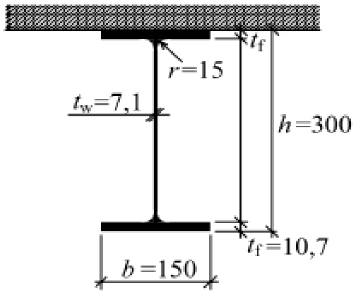
Partial safety factors

$\gamma_G = 1,35$

$\gamma_Q = 1,50$

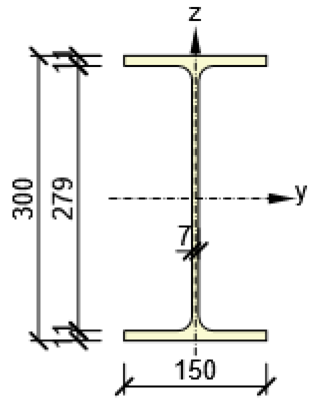
$\gamma_{M0} = 1,00$

$\gamma_{M,fi} = 1,00$



IPE300

Symbol	Value	Unit
Material	S 275	
A	5381	[mm^2]
I_u	83560000	[mm^4]
I_v	6038000	[mm^4]
I_t	201200	[mm^4]
I_w	127219092178	[mm^6]
$W_{el,u}$	557100	[mm^3]
$W_{el,v}$	80500	[mm^3]
$W_{pl,u}$	628000	[mm^3]
$W_{pl,v}$	125200	[mm^3]



Internal forces

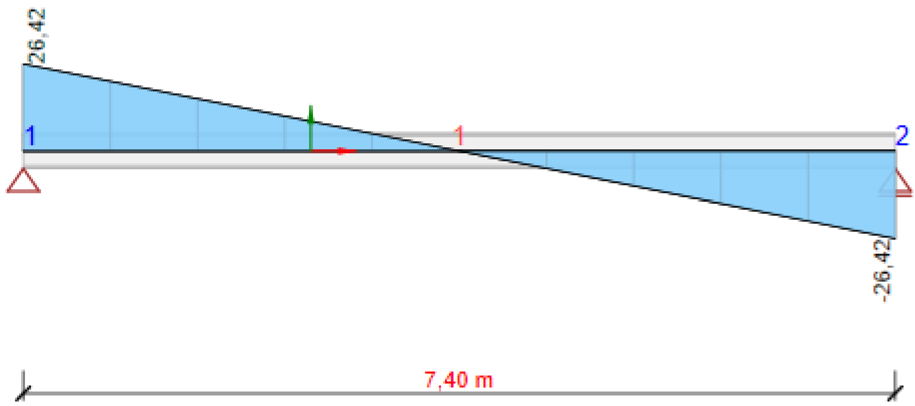
Internal forces of combination for the fire situation

$$E_{dA}=E\left(\sum G_k + A_d + \sum \psi_{2,i} \cdot Q_{k,i}\right)$$

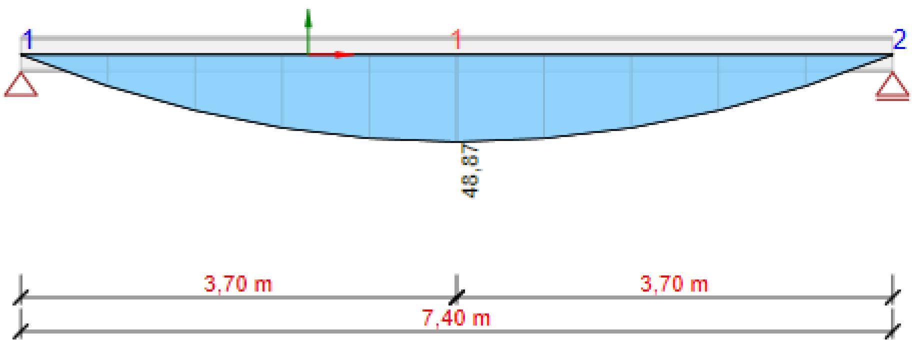
The combination factor $\psi_{2,i}=0{,}3$ .for office buildings

Combinations Delete all

	Name	Type	Evaluation		Self Weig	G	Q
	ULS Fundame	ULS Fundam	Code (6.10)		0	1	1
	SLSC	SLS Char	Code		0	1	1
>	fire	ULS Acciden	Linear		0	1	0,3



Shear force Vz diagram



Bending moment My diagram

Cross-section temperature calculation

The standard temperature-time curve is used for the gas temperature

$$\theta_g = 20 + 345.\log_{10}(8.t+1)$$

Section factor

$$\frac{A_m}{V} = \frac{3.b + 2(h - tw - 4r) + 2\pi r}{A} = \frac{3.150 + 2(300 - 7{,}1 - 4.15) + 2\pi.15}{5381} = 188\mathrm{m}^{-1}$$

Correction factor for shadow effect

$$k_{sh} = 0{,}9\frac{(A_m/V)_b}{(A_m/V)} = 0{,}9\frac{\frac{b+2\mathrm{h}}{A}}{0{,}188} = 0{,}9\frac{\frac{150+2.300}{5381}}{0{,}188} = 0{,}9.0{,}741 = 0{,}667$$

The increase of temperature of steel section is calculated using an incremental calculation procedure to determine the increase in steel temperature given in EN1993-1-2 by the following equation:

$$\Delta\theta_a, t = k_{sh}\cdot\frac{A_m/V}{c_a\cdot\rho_a}\cdot h_{net,d}\cdot\Delta t$$

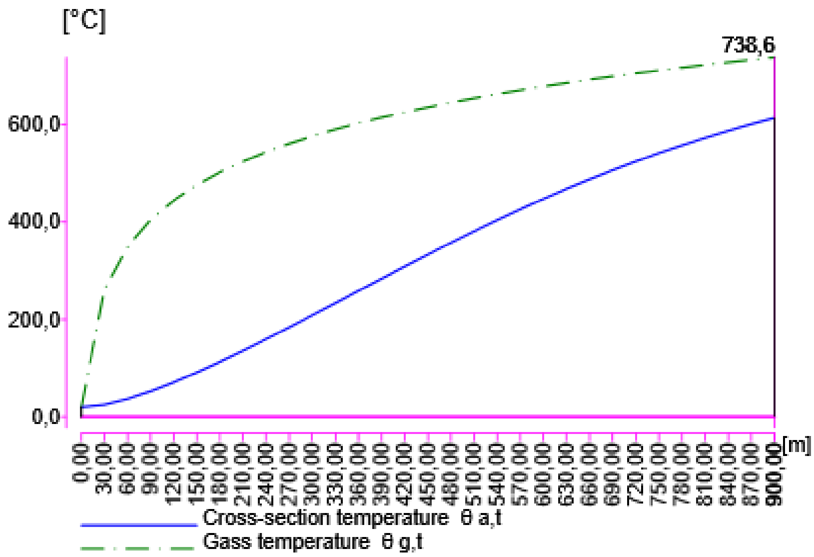
Time interval $\Delta t = 5\,sec$ is used in the tempearture calculation.

Code settings:

Fire exposion	3 sides
Type of protection	None
Temperature curve	Standard curve

Calculation of cross-section temperature

min	sec	t [s]	θ_g [°C]	$h_{net,r}$ [W/m^2]	$h_{net,c}$ [W/m^2]	$h_{net,d}$ [W/m^2]	c_a [kJ/(kg.K)]	θ_a [°C]
0	0	0	20,00	0,00	0,00	0,00	0,4398	20,00
0	5	5	96,54	447,63	1913,45	2361,07	0,4398	20,00
0	10	10	146,95	940,23	3163,08	4103,31	0,4401	20,43
0	15	15	184,61	1443,17	4085,82	5529,00	0,4406	21,17
0	20	20	214,67	1943,60	4812,43	6756,03	0,4413	22,18
...	...	...	...	...	...	...	...	...
14	45	885	736,06	17300,52	3215,92	20516,44	0,7656	607,43
14	50	890	736,90	17204,39	3183,32	20387,71	0,7672	609,57
14	55	895	737,73	17108,11	3151,05	20259,16	0,7689	611,69
15	0	900	738,56	17011,74	3119,12	20130,86	0,7707	613,80
15	5	905	739,38	16915,30	3087,52	20002,83	0,7725	615,88



The reduction factor $k_{y,\theta}$ for the steel temperature $\theta_a = 614\,°C$ is:

$k_{y,\theta} = 0{,}436$

Calculation of cross-section temperature

Item name	Symbol	Value	Unit	Reference
Protection type		Not protected		
Cross-section temperature	$\theta_{a,t}$	613,80	°C	4.2.5
Reduction factor for effective yield strength	$k_{y,\theta}$	0,44	-	3.2.1
Reduction factor for proportional limit	$k_{p,\theta}$	0,17	-	3.2.1
Reduction factor for the slope of the linear elastic range	$k_{E,\theta}$	0,29	-	3.2.1

Clasification

Article 4.2.2

$\varepsilon = 0{,}85\sqrt{235/fy} = 0{,}85.\sqrt{235/275} = 0{,}786$

The upper flange is under compression - internal part

$c = (b - t_w - 2r)/2 = (150 - 7{,}1 - 2.15)/2 = 56{,}45$

$c/t_f = 56{,}45/10{,}7 = 5{,}275$

The limit value for class 1

$9\varepsilon = 9.0{,}786 = 7{,}072$

Classification of flange: $7{,}07 > 5{,}28 \rightarrow$ **Class 1**

Web subject to bending

$c = h - 2t_f - 2r = 300 - 2\cdot 10{,}7 - 2\cdot 15 = 248{,}6\,mm$

$c/t_f = 248{,}6/7{,}1 = 35{,}01$

The limit value for class 1

$72\varepsilon = 72.0{,}786 = 56{,}6$

Classification of web: $56{,}6 > 35{,}01 \rightarrow$ **Class 1**

Cross-section is a Class 1

Cross-section classification acc. to 4.2.2

	σ1 [MPa]	σ2 [MPa]	ψ [-]	α [-]	c/t [-]	CL1 [-]	CL2 [-]	CL3 [-]	Class
Web	-275,00	275,00	-1,00	0,50	35,01	56,57	65,22	97,06	1
Left top flange	-275,00	-275,00	1,00	1,00	5,28	7,07	7,86	11,00	1
Right top flange	-275,00	-275,00	1,00	1,00	5,28	7,07	7,86	11,00	1
Left bottom flange	275,00	275,00	0,00	0,00	5,28	0,00	0,00	0,00	1
Right bottom flange	275,00	275,00	0,00	0,00	5,28	0,00	0,00	0,00	1

Verification in the strength domain

To calculate the cross-section resistance, the reduction factor $k_{y,\theta}$ has to be determined acc. to exression give in EN 1993-1-2 §4.2.3.3

$\theta_{a,30} = 613{,}8\,°C \rightarrow k_{y,\theta} = 0{,}436$

Bending resistance

Adaptation factor

$\kappa_1 = 0{,}7$

is used for an unprotected beam exposed to fire on three sides and the adaptation factor

$\kappa_2 = 1{,}0$

is used for a simply supported beam.

Design moment resistance at temperature 614°C is given by:

$$M_{fi,t,Rd} = M_{pl,Rd,20°C} \cdot k_{y,\theta} \cdot \frac{\gamma_{M1}}{\gamma_{M,fi}} \cdot \frac{1}{\kappa_1 \cdot \kappa_2}$$

$$M_{fi,t,Rd} = (0{,}000628 \cdot 275.10^6 / 1.0) \cdot 0{,}436 \cdot \frac{1{,}0}{1{,}0} \cdot \frac{1}{0{,}7 \cdot 1{,}0} = 107{,}57\, kNm$$

$$UC = \frac{48{,}87}{107{,}57} = 0{,}454 \quad \textbf{pass}$$

Bending moment and Shear check My + V acc. 6.2.8

Item name	Symbol	Value	Unit	Reference
Section modulus	$W_{pl,min}$	628000	mm3	EN1993-1-1: (6.13)
Design resistance moment	$M_{c,Rd}$	172,70	kNm	EN1993-1-1: 6.2.5 (2)
Design resistance monent of cross-section with uniform temperature	$M_{fi,\theta,Rd}$	75,45	kNm	4.2.3.3 (4.8)
Adaptation factor for non-uniform temperature across the cross-section	κ_1	0,70	-	4.2.3.3 (7)
Adaptation factor for non-uniform temperature along the member	κ_2	1,00	-	4.2.3.3 (8)
Design resistance moment of cross-section with non-uniform tempearture	$M_{fi,t,Rd}$	107,79	kNm	4.2.3.3 (4.10)
Utilisation	UC	45,34	%	6.2.5 (1)

Shear resistance

The design shear resistance is given by:

$$V_{fi,t,Rd} = k_{y,\theta} \frac{A_{V,z} f_y}{\sqrt{3}\gamma_{M,fi}} = 0{,}436 \cdot \frac{2569 * 275}{\sqrt{3} \cdot 1{,}0} = 177{,}84\, kN$$

$$UC = \frac{26{,}42}{177{,}84} = 0{,}1485 \quad \textbf{pass}$$

Shear force check Vz

Item name	Symbol	Value	Unit	Reference
Design plastic shear resistance	$V_{pl,Rd}$	407,88	kN	6.2.6 (2)
Design shear resistance at maximum steel temperature θa	$V_{fi,t,Rd}$	178,20	kN	4.2.3.3 (4.16)
Design plastic shear resistance reduced by Torsion	$V_{fi,t,T,Rd}$	178,20	kN	6.2.7 (9)
Utilisation	UC	14,82	%	6.2.6 (1)

Item name	Symbol	Value	Unit	Reference
Shear reduction	ρ	0,00	-	6.2.8 (3),(4)

6. Fire design of protected HEB section column

The example was prepared acc. to [7].

Introduction

The column, fabricated from a hot-rolled HEB section, supports two floors and is fire protected with sprayed vermiculite cement. The required period of fire resistance is R 90.

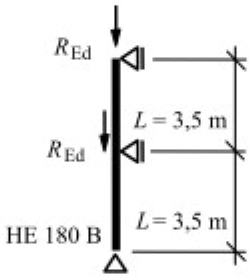


Figure 1: Static system

Basic data

Material properties

Steel grade: S 355

Yield stress: f_y = 355 N/mm²

Density: ρ_a = 7850 kg/m³

Loads

Reaction at each floor level due to permanent actions:

$R_{G,k}$ = 185 kN

Reaction at each floor level due to variable actions:

$R_{Q,k}$ = 175 kN

Partial safety factors

$\gamma_G = 1,35$

$\gamma_Q = 1,50$

$\gamma_{M1} = 1,00$

Data for fire calculation

Material properties of fire protection– sprayed vermiculite cement

thickness $d_p = 20$ mm

density $\rho_p = 550$ kgm^{-3}

specific heat $c_p = 1100$ Jkg^{-1}K^{-1}

thermal conductivity $\lambda_p = 0,12$ Wm^{-1}K^{-1}

Mechanical actions for fire design situation

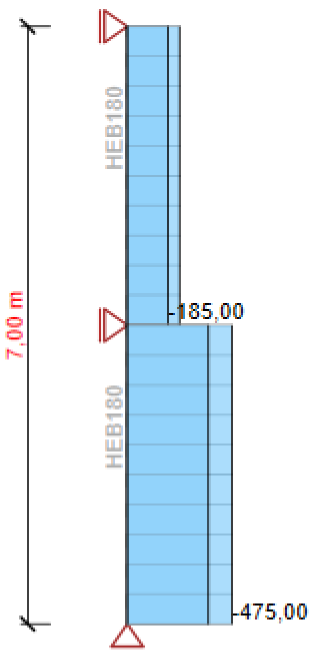
Internal forces of combination for the fire situation

$$E_{dA}=E\left(\sum G_k+A_d+\sum \psi_{2,i}\cdot Q_{k,i}\right)$$

The combination factor $\psi_{2,i}=0,3$ .for office buildings

Combinations | + | Delete all

	Name	Type	Evaluation		Self Weig	LC1	LC2
>	Fire	ULS Acciden	Envelope	✖	0	1	0,3



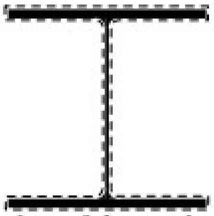
Normal forces N [kN]

Cross-section temperature

The section factor of protected member is calculated as follows:

$$\frac{A_p}{V}=\frac{2b+2h-4r+2(b-tw-2r)+2\pi r}{A}=\frac{4.180-4.15+2(180-8{,}5-2.15)+2.\pi.15}{6525}$$

$$\frac{A_p}{V}=0{,}159\,mm^{-1}=159\,m^{-1}$$



Evaluation of the section parameter A_p/V

The increase of temperature of the steel section is calculated by step-by-step procedure using:

$$\Delta\theta_{a,t}=\frac{\lambda_p\ A_p/V}{d_p\ c_a\ \rho_a}\ \frac{\theta_{g,t}-\theta_{a,t}}{1+\frac{\phi}{3}}\ \Delta t-(e^{\phi/10}-1)\ \Delta\theta_{g,t}\qquad \text{but}\quad \Delta\theta_{a,t}\geq 0$$

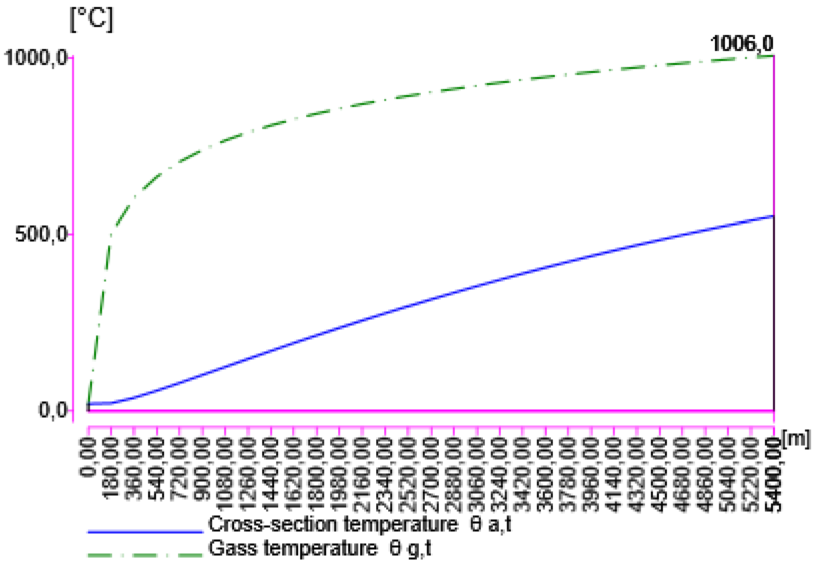
where

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

Time interval Δt = 30 sec is used in the temperature calculation.

Calculation of cross-section temperature

min	sec	t [s]	θ_g [°C]	c_a [kJ/(kg.K)]	ø	θ_a [°C]
0	0	0	20,00	0,4398	0,557	20,00
0	30	30	261,14	0,4398	0,557	20,00
1	0	60	349,21	0,4398	0,557	20,00
1	30	90	404,31	0,4398	0,557	20,00
2	0	120	444,50	0,4398	0,557	20,00
...	...	...	...	...	...	...
88	30	5310	1003,47	0,704	0,348	545,39
89	0	5340	1004,32	0,706	0,347	547,48
89	30	5370	1005,15	0,7079	0,346	549,56
90	0	5400	1005,99	0,7098	0,345	551,64
90	30	5430	1006,82	0,7117	0,344	553,70



Calculation of cross-section temperature

Item name	Symbol	Value	Unit	Reference
Protection type		Spray		
Cross-section temperature	$\theta_{a,t}$	551,64	°C	4.2.5
Reduction factor for effective yield strength	$k_{y,\theta}$	0,62	-	3.2.1
Reduction factor for proportional limit	$k_{p,\theta}$	0,27	-	3.2.1
Reduction factor for the slope of the linear elastic range	$k_{E,\theta}$	0,45	-	3.2.1

Flexural buckling

Provided that the column forms part of a braced frame and the fire resistance of the concrete slab separating the floors is not less than the fire resistance of the column the buckling length is reduced to

$$L_{cr,y,fi} = L_{cr,z,fi} = 0{,}7 \cdot L = 0{,}7 \cdot 3{,}5 = 2{,}45 \text{ m}$$

The critical buckling load at normal temperature

$$N_{cr} = \frac{\pi E I_z}{L_{cr,z}^2} = \frac{\pi . 210000 . 13630000}{2450^2} = 4706{,}3\, kN$$

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} = \sqrt{\frac{6525.355}{4706{,}3 . 10^3}} = 0{,}7016$$

The non-dimensional slenderness at temperature θ_a is

$$\bar{\lambda}_\theta = \bar{\lambda}\sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}} = 0{,}7016 . \sqrt{\frac{0{,}62}{0{,}45}} = 0{,}824$$

$$\alpha = 0{,}65\sqrt{235/f_y} = 0{,}65 . \sqrt{235/355} = 0{,}53$$

$$\phi_{z,\theta} = 0{,}5(1 + \alpha \bar{\lambda}_{z,\theta} + \bar{\lambda}_{z,\theta}^2) = 0{,}5 . (1 + 0{,}53 . 0{,}824 + 0{,}824^2) = 1{,}058$$

$$\chi_{z,fi} = \frac{1}{\phi_{z,\theta} + \sqrt{\phi_{z,\theta}^2 - \bar{\lambda}_{z,\theta}^2}} = \frac{1}{1{,}058 + \sqrt{1{,}058^2 - 0{,}824^2}} = 0{,}581$$

The design resistance at temperature θ a = 551,64 °C is given by:

$$N_{b,fi,\theta,Rd} = \chi_{z,fi} A K_{y,\theta} f_y / \gamma_{M,fi} = 0{,}581 . 6525 . 0{,}62 . 355 / 1{,}0 = 834{,}4\, kN$$

Utilisation:

$$UC = \frac{475}{834{,}4} = 0{,}569$$

Flexural buckling check

Item name	Symbol	Value Y-Y	Value Z-Z	Unit	Reference
Reduction factor	χ_{fi}	0,76	0,58	-	4.2.3.2 (2)
Slenderness	λ_θ	0,49	0,82	-	4.2.3.2 (2)
Slenderness	λ	0,42	0,70	-	EN1993-1-1: 6.3.1.2 (1)
Imperfection factor	α	0,53	0,53	-	4.2.3.2 (2)
Buckling factor	k	0,70	0,70	-	
Critical length	L_{cr}	2,45	2,45	m	EN1993-1-1: 6.3.1.3 (1)
Critical force	N_{cr}	13228,15	4706,33	kN	EN1993-1-1: 6.3.1.2 (1)
Resistance force	$N_{b,fi,t,Rd}$	1089,66	835,48	kN	4.2.3.2 (1)
Utilisation	UC	43,59	56,85	%	4.2.3.2

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