Czech Technical University in Prague
Faculty of Civil Engineering

Master's Thesis

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Statutory declaration

I hereby declare that the master thesis entitled "Škoda Showroom" submitted to CTU in Prague was written by myself under the guidance of doc. Ing. Michal Jandera, Ph.D. I have stated all the resources used to elaborate this project in conformity with the Methodical guide for ethical development of university final thesis.

Oğuz Demirbaş

……………………………….. 01th of Jan 2021

Annotation

The project is focused on a structural solution and design of the connections of a single hall including a mezzanine which is used for administrative purposes. Column bases and the beams on the mezzanine are considered hinged, while rest of the structure designed according to moment resistances. Static analysis contains the design of the two frames mezanine, in addition to that column bases and bracings. Calculations correspond to ČSN EN.

Key words: Exposition building
Steel frame
Shear connections
Bolt connections

Thanks

Throughout the writing diploma work, I have received great support and assistance from Allcons Industry by using licenced applications, strong devices and most importantly from friendly people who is willing to share their institutional and intellectual knowledge. In this case I especially want to thank Ing. David Šedlbauer, Ph.D. and Ing. Václav Duda.

Abstract

The aim of the study is to make a basic design and detail design of the some of the connections of an exposition building according to Eurocode. Stiffness of a moment connection is especially assessed in details. Column bases are considered pinned, connections on beams in the mezzanine are considered as hinged, while connections on the gable wall and the main frame are considered fixed. Throughout the writing diploma work, Scia Engineering 19.1, IDEA Statica 9 and LTBeams applications used to make the calculations easier. The study recognises what is to be done to presume a connection rigid, and what is to be done if connection is not rigid, while it was presumed rigid previously.

TECHNICAL REPORT

CONTENT

1. Description of the structure

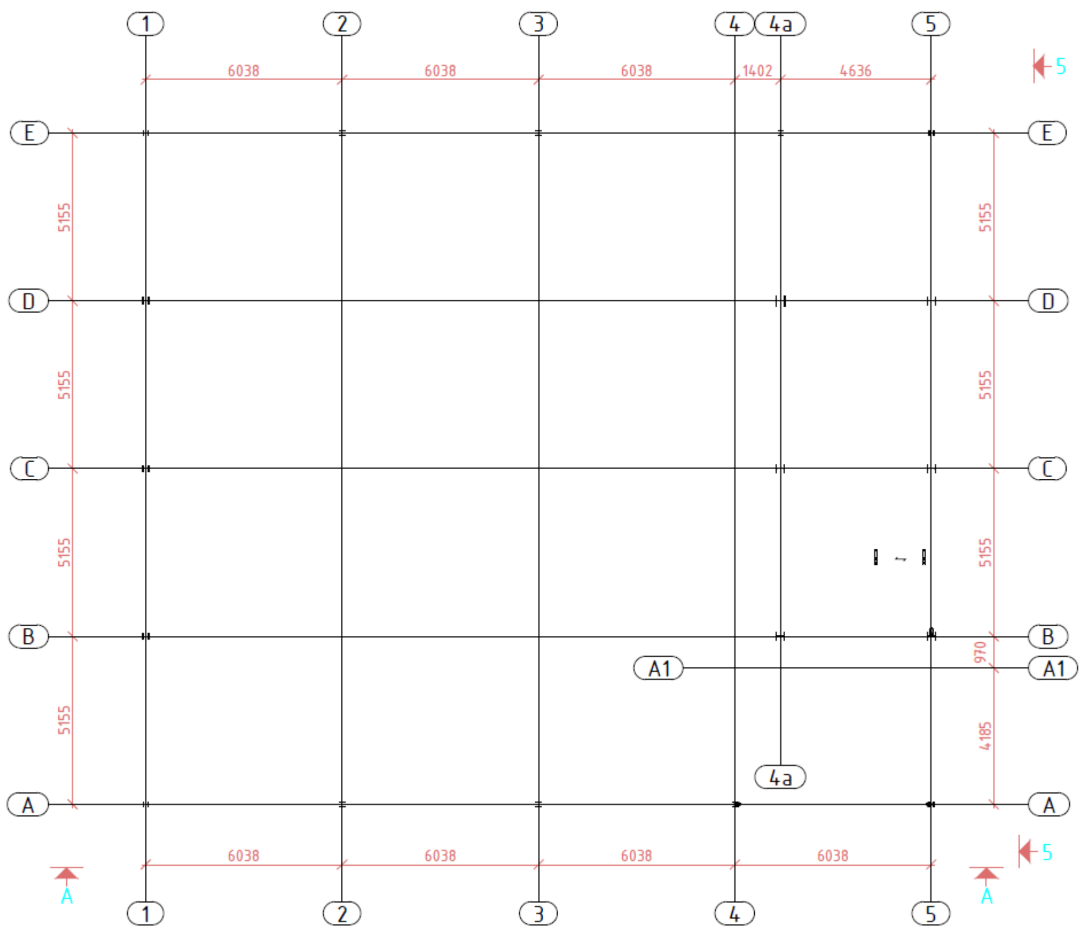
Subject of the work is the preliminary basic and detail design and calculation of a hall which its purpose is to serve as a showroom with a span of 24 m long, to 20 m wide and also a mezanine for administrative purposes inside the building which extends on one edge of the building. Transverse connections are designed with pinned HEA columns and fixed IPE beams which are 6,038 m apart while in longitudal direction columns are 5,155 m apart. Height of the structure is 8 meters. Parapets are installed on the roof of the structure causes drifted snow. The hall will be surrounded by glass facade which provides insulation. One couple of bracing are designed both in the transverse and longitudal direction. Roof will be consist of trapozedial sheets.

The structure is located in Prague area. Terrain category is III. Altitude of the building is inbetween 0-500 m.

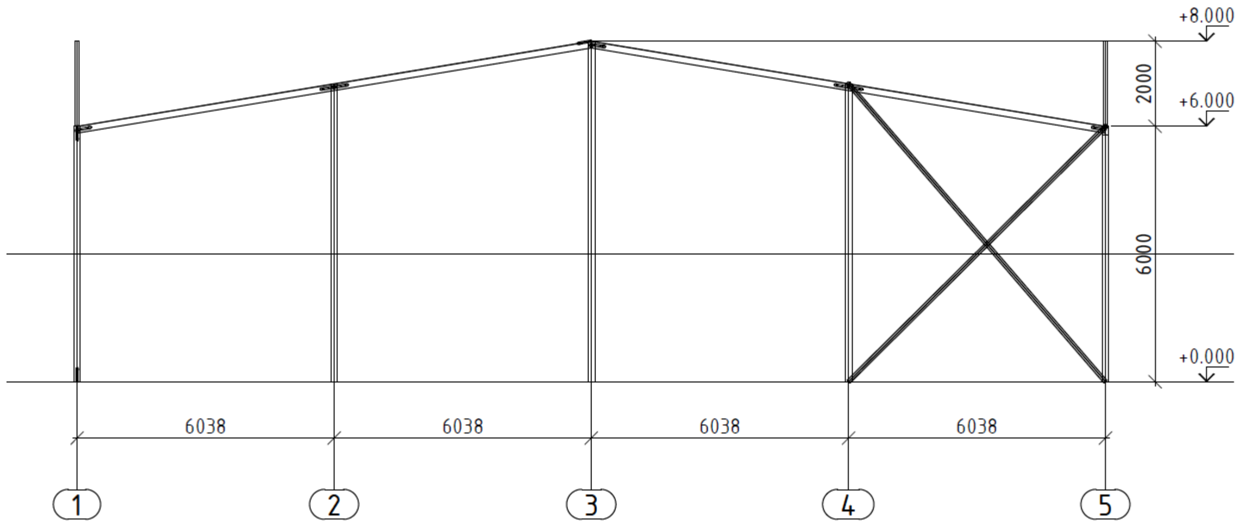
Calculations and load combinations are performed according to ČSN EN 1990, ČSN EN 1991.

Layouts, sections and detail of its connections are given in the appendixes in more details.

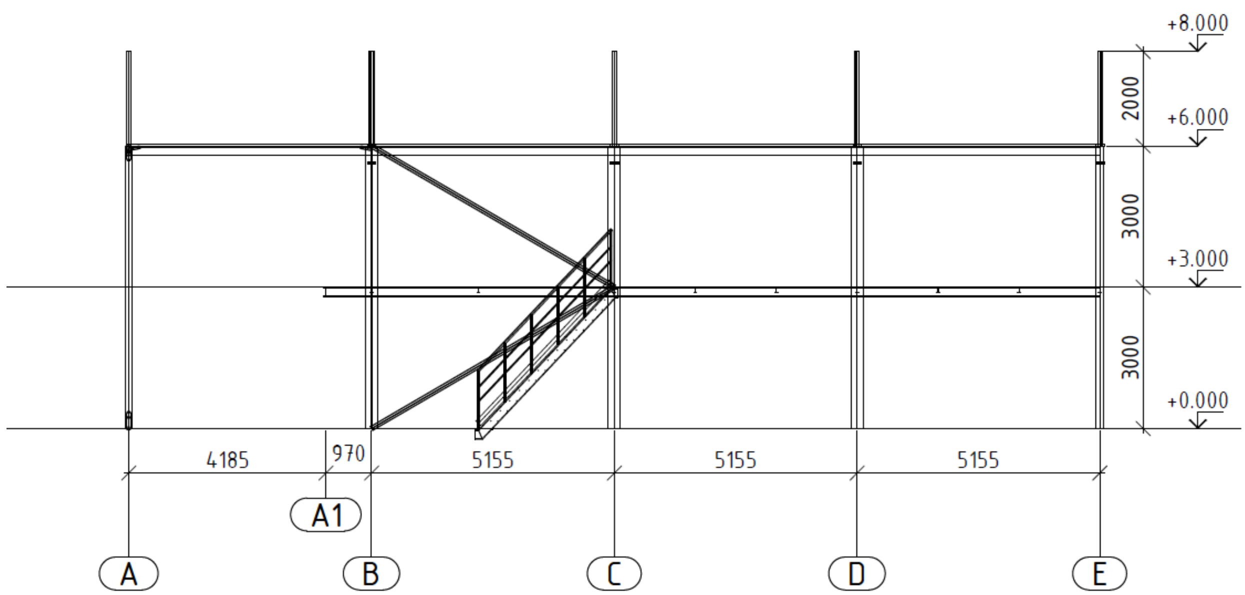
Layout



Section A – A



Section 5 - 5



2. Loading

The structure is located in Prague area and its altitude is inbetween 0-500 m. According to EC.EN 1993-1-1-3 and EN 1991-1-1-4, the site is considered inside the "snow area I" and "terrain category III" Safety factors according to EC.EN.1990 are $\gamma G = 1{,}35$, $\gamma Q = 1{,}5$.

2.1. Snow load

- snow area I, $s_k = 0,7\ kN/m^2$
- $\alpha = \arctan(2/12) = 9,5°$ $\mu_I = 0,8 \quad 0° < \alpha < 30°$
- accumulation of snow $\mu_2 = 2,0 \quad 0° < \alpha < 30°$
- exposure coefficient $C_e = 1,0$ (there is no significant displacement of snow caused by the wind)
- heat coefficient $C_t = 1,0$

2.2. Wind load

air density p = 1,25 kg/m^3

$V_b = c_{dir} \cdot c_{season} \cdot V_{b,0} = 1 \cdot 1 \cdot 22,5 = 22,5$ m/s

where,

terrain category III $V_{b,0} = 22,5$ m/s

season factor $c_{season} = 1$

direction factor $c_{dir} = 1$

3. Material

Based on ČSN EN 1991-1-10, steel grade is S355 J2 for all the structure including profiles and plates. The trapozedial metal sheet is made of steel grade S320 GD. The bolts are used in connections are steel grade 8.8.

4. Assembly

All welds and cuts are completed in workshop. Scaffolding and formwork is needed during the construction of primary, secondary beams and the slab of the mezzanine. For the rest of the structure, one truck-mounted with 10 ton capacity crane and a basket crane for the installations of the the connections is enough.

5. Corrosion protection

The class of corrosive environment is C1. The anti-corrosion coating will be designed for this class according to Eurocode (it is not a part of this project). The bolts are galvanized in the workshop.

6. Fire safety

Fire safety has to be verified by calculations. It's not part of this project.

STATIC ANALYSIS

CONTENT

REFERENCES

[1] EN 1991-1-1 General actions – Densities, self-weight and imposed loads for buildings

[2] EN 1991-1-3 General actions – Snow loads (including the amendments)

[3] EN 1991-1-4 General actions – Wind actions

[4] EN 1993-1-1 Design of steel structures – General rules and rules for buildings

[5] EN 1993-1-8 Design of steel structures – Design of joints

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[9] Wald F., Sabatka L., Bajer M., Jehlicka P., Kabelac J., Kozich M., Kurikova M., Vild M.: Benchmark Cases for Advanced Structural Steel Connections, ČVUT v Praze, BUT, 2016

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[11] *Ing. Michal Jandera Ph.D.: personal webpages http://people.fsv.cvut.cz/~xjanderm/*

[12] *Trapozeidal sheet for roofing https://balex.eu/en/products?format=raw&task=download&fid=283*

[13] *Software Scia Engineer 2019.1, Student version*

[14] *Idea Statica, Licenced by Allcons Industry*

[15] *Software LTBeam 1.0.11, Free software*

1. LOADING

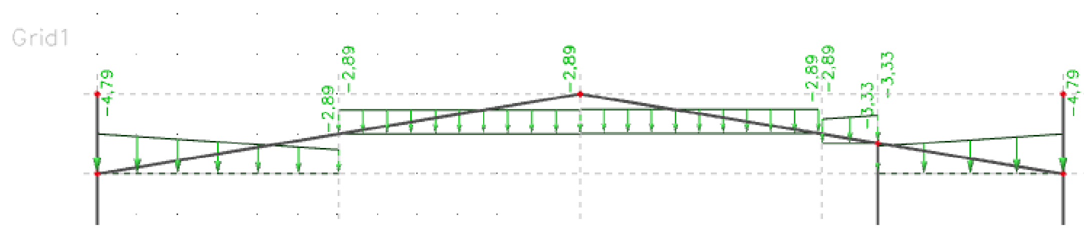
1.1. Snow load

$s = \mu_I \, C_e \, C_t \, s_k$

$s_k = 0{,}7 \text{ kN/m}^2$

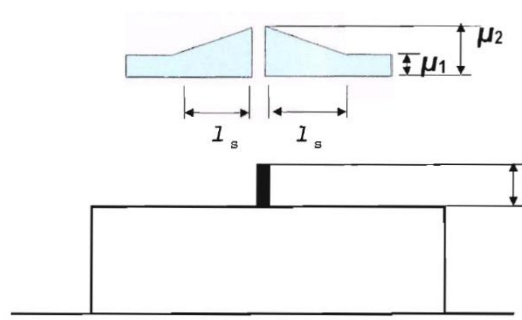
$s_{k1} = 0{,}8 \cdot 1 \cdot 1 \cdot 0{,}7 = 0{,}56 \text{ kN/m}^2$

$s_{k2} = 2{,}0 \cdot 1 \cdot 1 \cdot 0{,}7 = 1{,}4 \text{ kN/m}^2$



An example of how load is applied shown above.

where,



- snow area I, $s_k = 0{,}7 \text{ kN/m}^2$

- $\alpha = \arctan(2/12) = 9{,}5°$ $\mu_I = 0{,}8$ $0° < \alpha < 30°$

- accumulation of snow $\mu_2 = 2{,}0$ $0° < \alpha < 30°$

- exposure coefficient $C_e = 1{,}0$ (there is no significant displacement of snow caused by the wind)

- heat coefficient $C_t = 1{,}0$

Because of the parapet, it is necessary to calculate the accumulation of snow.

Regulation suggest the l_s value to be in-between 5 and 15 meters. In this case, 6,038 meter to be used.

1.2. Wind load

$w_e = q_b \; c_e(z_e) \, c_{pe} = 0{,}316 \cdot 1{,}6 \; c_{pe} = 0{,}51 \; c_{pe}$

where,

terrain category III $V_{b,0} = 22{,}5$ m/s

$V_b = c_{dir} \cdot c_{season} \cdot V_{b,0} = 1 \cdot 1 \cdot 22{,}5 = 22{,}5$ m/s

$q_b = \frac{p}{2} \cdot V_{b,0}{}^2 = \frac{1{,}25}{2} \cdot 22{,}5 = 0{,}316 \text{ kN/m}^2$

b = 20,62 m

2h = 16 m

e = min(b, 2h) = 16 m

d = 24,15 m; d > e

e/5 = 3,2 m; 4e/5 = 12,8 m

d-e = 8,15 m

For the walls, wind blowing parallel to the ridge,

b = 24,15 m

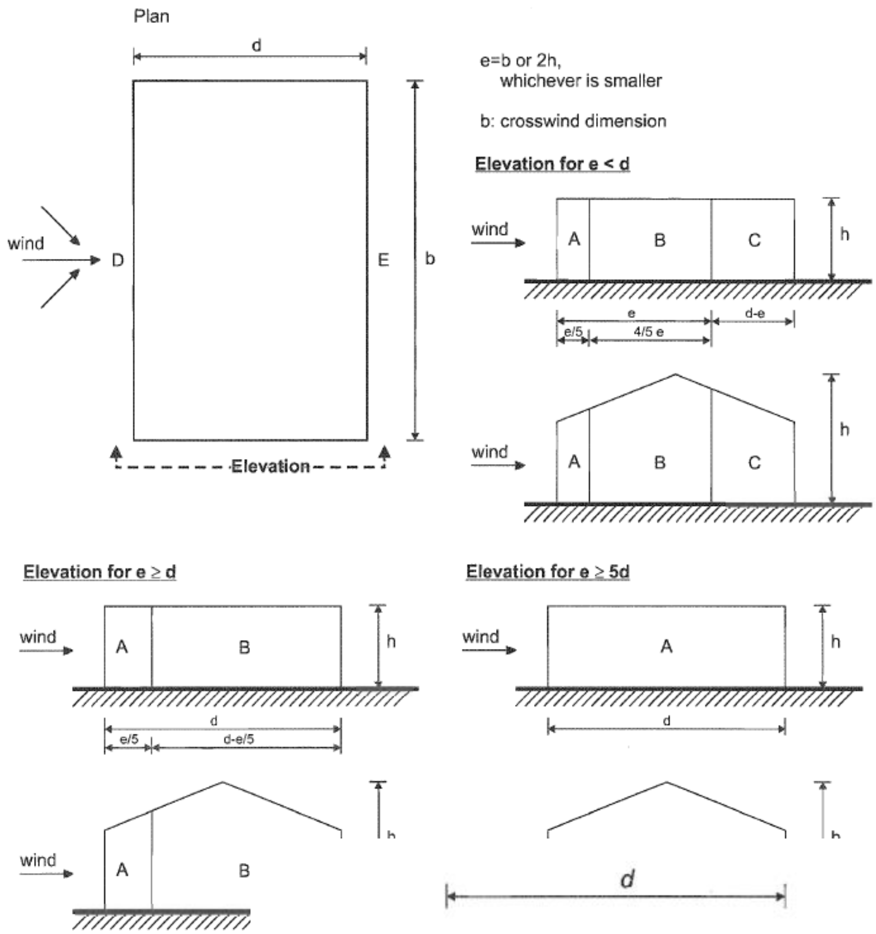
2h = 16 m

d = 20,62 m; d > e

e/5 = 3,2 m; 4e/5 = 12,8 m

2h = 16 m

d-e = 4,62 m



For the roof, wind blowing perpendicular to the ridge,

b= 20,6 m

2h = 16 m

e = min(b, 2h) = 16 m

e/10 = 1,6 m

e/4 = 4 m

For the roof, wind blowing parallel to the ridge,

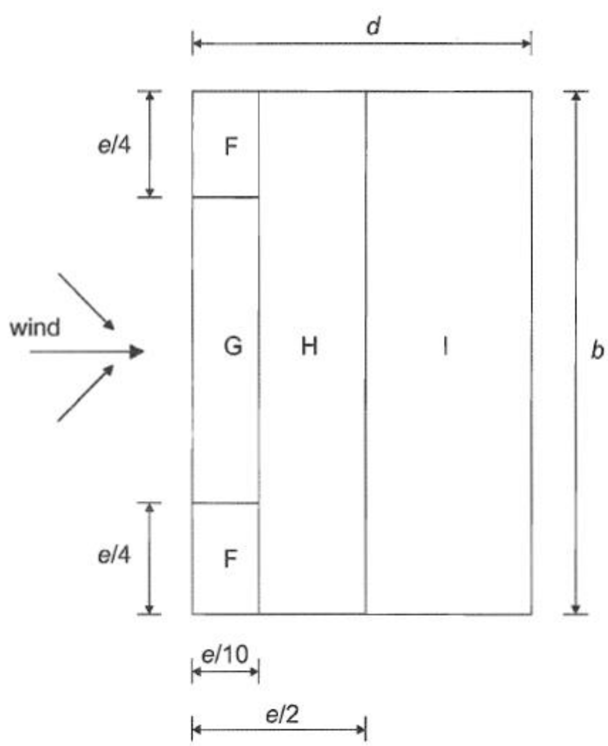
b= 24,15 m

2h = 16 m

e = min(b,2h)

e/10 = 1,6 m

e/4 = 4 m



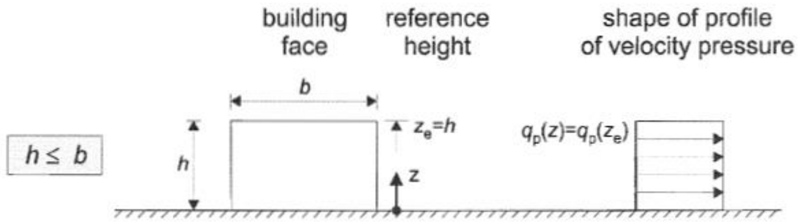
e = *b* or 2*h*
whichever is smaller

b : crosswind dimension

$c_e(z_e)$ = 1,60 in which z = h = 8 m

The building will be considered as a one piece by applying the wind load. Because;

h = 8 m < b = 24,152 m



Wind load values in transverse and longitudinal directions are shown in the table below.

	Transverse		Longitudinal	
Area	c_{pe}	w_{ek} [kN/m²]	c_{pe}	w_{ek} [kN/m²]
A	-1,2	-0,61	-1,2	-0,61
B	-0,8	-0,41	-0,8	-0,41
C	-0,5	-0,26	-0,5	-0,26
D	0,7	0,36	0,7	0,36
E	-0,3	-0,15	-0,3	-0,15
F	-1,3	-0,66	-1,3	-0,66
G	-1	-0,51	-1	-0,51
H	0,45	0,23	0,45	0,23
I	-0,5	-0,26	-0,5	-0,26
J	-0,8	-0,41	-0,8	-0,41

1.3. Mezanine loads

To assess the trapozedial sheets and beams, it is necessary to determine both assemby stage and operational stage individually.

a) Assembly stage

Dead load

	g_k [kN/m²]	γ_g	gd [kN/m²]
- Wet concrete (weight of fresh concrete is 26 kN/m³) Thickness of the plate: $t$plate $= 70 + (\frac{54+30{,}5}{250})$ = 87 mm 0,087m · 26 kN/m³ = 2,26 kN/m²	2,26	1,35	3,05
- Trapozedial sheet (estimate)	0,10	1,35	0,14
total:	2,36	1,35	3,19

Live load

	g_k [kN/m²]	γ_g	gd [kN/m²]

- regular live load	0,75	1,50	1,125
- as concrete poured, the worst case on 3x3 m^2 area	1,50	1,50	2,25
total:	2,25	1,50	3,375

b) Operational Stage

Dead load

	*g*k [kN/m^2]	γ_g	*g*d [kN/m^2]
- thread layer thickness 60 mm	1,20	1,35	1,62
- concrete thickness 0,87 ⍰⍰25 kN/m^3 = 2,18 kN/m^2	2,18	1,35	2,94
- trapozedial sheet	0,10	1,35	0,14
- ceiling	0,15	1,35	0,20
total:	3,63	1,35	4,90

Live load

	*g*k [kN/m^2]	γ_g	*g*d [kN/m^2]
- live load	2,50	1,50	3,375
- partition wall	0,80	1,50	1,20
total:	3,30	1,50	4,95

1.4. Roof cladding loads

	*g*k [kN/m^2]	γ_g	*g*d [kN/m^2]
- waterproof foil based on PVC	0,03	1,35	0,04
- mineral wool thickness 200 mm	0,12	1,35	0,16
- inner TR sheet	0,08	1,35	0,10
total:	0,23	1,35	0,31

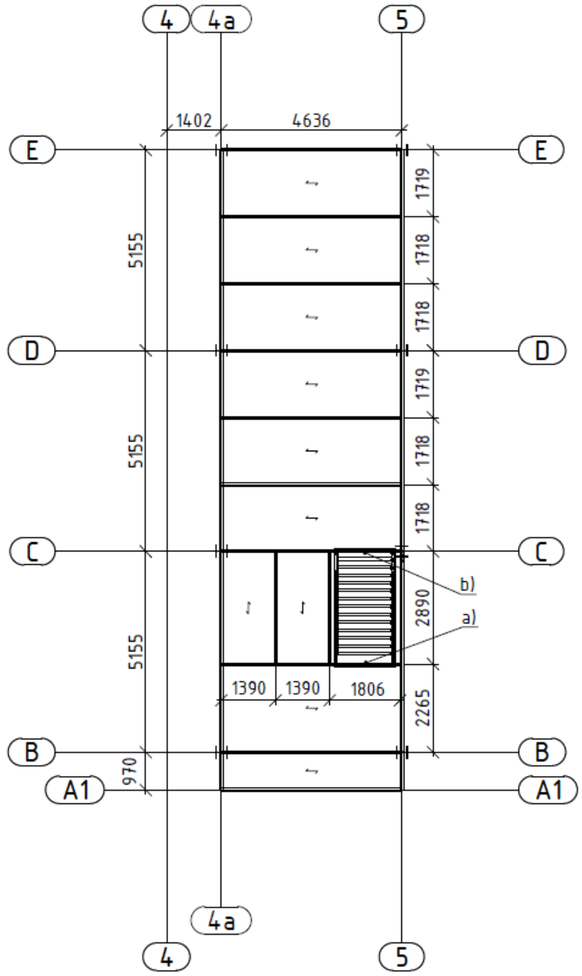
1.5. Wall cladding loads

	*g*k [kN/m^2]	γ_g	*g*d [kN/m^2]
- 2 x 6 mm glass 15 kg/m^2 · 2 · 10 = 0,3 kN/m^2 SHS 150x150x5 profile longitudal on every 4 m, G = 22,3 kg/m Area of the frontal section 24 · 7 = 168 m^2	0,30	1,35	0,4

Weight of the profiles 7 · 7 · 22,3 = 1092 kg $= \frac{1092}{168}$ = 6,5 kg/m^2 = 0,06 kN/m^2 SHS 100x100x5 profile on horizontal direction G = 14,4 kg/m	0,06 0,04	1,35 1,35	0,08 0,06
total:	0,40	1,35	0,54

2. DESIGN OF MEZZANINE AND ROOF

2.1. Trepozeidal Sheet



So

Structure is presumed supported during construction. In operational stage load is already transmitted by the reinforced slab. calculation related to assembly stage is ignored. Only assembly stage is assessed.

Slab on the B axis is assessed because of the length of the span.

$g_k + q_k = 2,36 + 1,50 = 3,86$ kN/m^2

$g_d + q_d = 3,19 + 2,25 = 5,44$ kN/m^2

TR 50/250/1,00 Steel S320GD

M = 10,0 kg/m^2

$W_{eff,min}$ = 12 400 mm^3/m

$I_{eff,min}$ = 311 000 mm^4/m

$M_{Ed} = \frac{1}{8} \cdot 5,44 \cdot 2,265^2 = 3,48$ kNm/m

Relation shown above applies for a simply supported beam.

Required cross-sectional modulus for TR sheet steel S320:

$f_{yd} = fy/\gamma M1 = 320/1,0 = 320$ MPa

$g_d + q_d = 3,19 + 2,25 = 5,44$ kN/m^2 $< g_{Rd} = 5,41 + \frac{0,265}{0,5} \cdot (7,94-5,41) = 6,75$kN/m^2

SLS Check

Deflection is determined by the dead load. Moment on the supports presumed not exist.

$$\delta = \frac{1}{E \cdot Ieff} \cdot (\frac{5}{384} \cdot g_k \cdot l^4) = \frac{1}{210 \cdot 10^3 \cdot 311000} \cdot (\frac{5}{384} \cdot 2,36 \cdot g_k \cdot 2,265^4) = 12,38 \text{ mm}$$

$$\delta_{max} = \frac{L}{180} = \frac{2265}{180} = 12,58 \text{ mm}$$

$\delta_{max} = 12,58$ mm $> \delta = 12,38$ mm

Values of the trapozeidal sheet are shown in Appendix D.

TR plate barely satisfies the SLS check.

2.2. Secondary Beam

Ceiling is presumed supported during casting concrete. So calculations on assembly stage is ignored. Only operational stage is assessed.

2.2.1. Estimating beam which is the most stressed:

To assess the primary beams, it is needed to estimate which one takes the highest load.

Stairway with the width of 1400 mm between railings is estimated to weight 380 kg. It is considered 400 kg. Stairway is connected to the beam on the C axis and to the bottom floor on the opposite side.

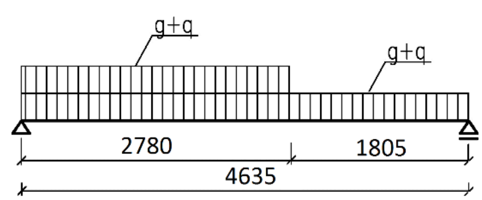
Dead load of the stairway

	*g*k [kN/m²]	γG	gd [kN/m²]
Weight of the stairway, the height of 3 m and 1,4 m between two railings (estimate) 400 kg - F_k = 400 · 10 = 4,0 kN - To convert a distributed load g_k = 4,0/1,8 =2,22 kN/m	2,22	1,35	3
total:	2,22	1,35	3

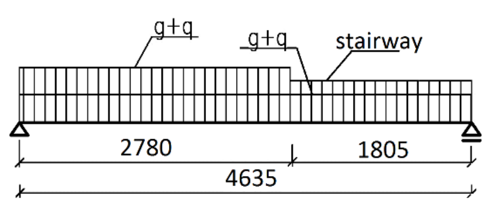
Live load of the stairway

	*g*k [kN/m²]	γG	*g*d [kN/m²]
- live load	2,50	1,50	3,375
total:	2,5	1,50	3,375

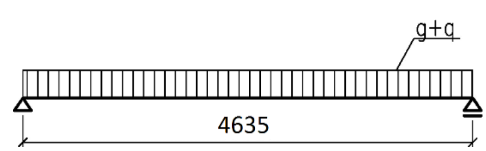
a) Selection a) shown on the image on the previous page which carries slab loads on both ways.



b) Selection b) carries slabs on both ways and half weight of the stairway.



c) Any other beam between C and D axis.



a) $B_{eff} = B/8$ =>

$g_k = (3{,}63 + 3{,}3) \cdot \frac{2{,}265}{2} + 0{,}25 + (3{,}63 + 3{,}3) \cdot \frac{2{,}78}{4{,}635} \cdot \frac{2{,}89}{2} = 14{,}10$ kN/m

$g_d = (4{,}9 + 4{,}95) \cdot \frac{2{,}265}{2} + 0{,}34 + (4{,}9 + 4{,}95) \cdot \frac{2{,}78}{4{,}635} \cdot \frac{2{,}89}{2} = 20{,}03$ kN/m

0,25 kN/m is the estimation of the weight of the beam.

b)

$g_k = (3{,}63 + 3{,}3) \cdot \frac{1{,}718}{2} + 0{,}25 + \frac{2{,}22}{2} + 2{,}5 \cdot \frac{1{,}8}{4{,}635} \cdot \frac{2{,}89}{2} + (3{,}63 + 3{,}3) \cdot \frac{2{,}78}{4{,}635} \cdot \frac{2{,}89}{2} = 14{,}72$ kN/m

$g_d = (4{,}9 + 4{,}95) \cdot \frac{1{,}718}{2} + 0{,}34 + \frac{2{,}22}{2} + 3{,}375 \cdot \frac{1{,}8}{4{,}635} \cdot \frac{2{,}89}{2} + (4{,}9 + 4{,}95) \cdot \frac{2{,}78}{4{,}635} \cdot \frac{2{,}89}{2} = 20{,}34$ kN/m

0,25 kN/m is the estimation of the weight of the beam.

c)

$g_k = (3{,}63 + 3{,}3) \cdot 1{,}857 + 0{,}25 = 13{,}12$ kN/m

$g_d = (4{,}9 + 4{,}95) \cdot 1{,}857 + 0{,}25 = 18{,}63$ kN/m

b) option would be calculated. Because load is higher and B_{eff} is smaller, which means less $M_{Pl,Rd}$ value.

2.2.2. Design of the secondary beam

<u>IPE120</u>

m = 10,4 kg/m

A = 1321 mm^2

$W_{Pl,Rd} = 60730\ mm^3$

$I_y = 3{,}178 \cdot 10^3\ mm^4$

Class 1 for bending to the y-axis S355

m<25 kN/m, no need to recalculate

<u>ULS Check</u>

$b_{eff} = b_{e1} = L/8 = 5155/8 = 644$ mm

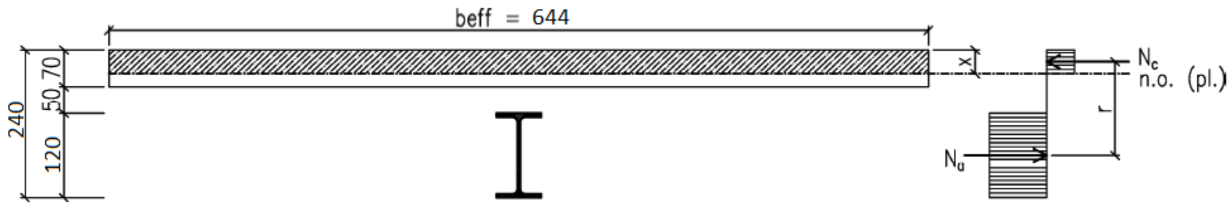
$b_{eff} = 644$ mm < B = 1,857/2+ … ≤ 928 mm

Concrete C25/30 selected.

$f_{ck} = 25$ MPa

$f_{yd} = 355$ MPa

$f_{cd} = 0,85 \cdot \frac{fck}{2\gamma c} = 0,85 \cdot 85 \cdot \frac{2,5}{1,5} = 14,2$ MPa



$N_a = N_c$

$A_a \cdot f_{yd} = x \cdot b_{eff} \cdot f_{cd}$

$1321 \cdot 355 = x \cdot 644 \cdot 14,2$ =>

x = 57 mm

$r = \frac{120}{2} + 50 + 70 - \frac{57}{2} = 151$ mm

$M_{Pl.Rd} = N_a \cdot r = N_c \cdot r = 1321 \cdot 355 \cdot 151 = 71,01$ kNm

$M_{Pl.Rd} = 71,01$ kNm > $M_{Ed} = 61,5$ kNm

$V_{Pl.Rd} = A_{vz} \cdot f_{yd}/\sqrt{3} = 966*355/\sqrt{3} = 73,38$ kN

$V_{Pl.Rd} = 73,38$ kN > $V_{Ed} = 53,07$ kNm

IPE120 is suitable for load bearing capacity.

SLS Check

For concrete

$E_c' = \frac{E}{2} = \frac{31000}{2} = 15500$ MPa

$n = E_a / E_c = 210000/15500 = 13,55$

$A_i = 1321 + 70 \cdot 644/13,55 = 4314$ mm^2

$e = \frac{1321 \cdot \frac{120}{2} + 70 \cdot 644/13,55(120 + 50 + \frac{70}{2})}{7996} = 161$ mm

$I_i = 4314 + 1321 \cdot (161 - \frac{120}{2}) + (\frac{1}{13,55} \cdot (\frac{1159 \cdot 70^3}{12} + 644 \cdot 70 \cdot (161-120-50 - \frac{70}{2})^2 = 23,67 \cdot 10^6$ mm^4

$\sigma_{a,max} = M_{Ek} \cdot z_d / I_i = 44,74 \cdot 10^6 \cdot 161 / (23,67 \cdot 10^6) = 303,55$ MPa < f_{yd} =355 MPa

$\sigma_{c,max} = M_{Ek} \cdot z_h / (n \cdot I_i) = (44,74 \cdot 10^6 \cdot 161 / (13,55 \cdot 23,67 \cdot 10^6)) \cdot (240-161) = 11,08$ MPa

11,08 MPa > $0,85 \cdot f_{ck} = 0,85 \cdot 25 = 21,25$ MPa

$\delta_2 = \frac{5}{384} \cdot \frac{q_p \cdot L^4}{E \cdot I_i} = \frac{5}{384} \cdot \frac{(3,3 \cdot 3,53 \cdot 10^3) \cdot 5155^4}{210 \cdot 10^3 \cdot 23,67 \cdot 10^6} = 12,76$ mm

$\delta_2 = 12{,}76 < \frac{L}{250} = 18{,}54$

Beam satisfies the ULS check.

Mandrel check

Mandrel 19/100

d = 19 mm, h_{sc} = 100 mm, S235, f_u = 360 MPa

$$P_{Rd,1} = 0{,}8\, f_u \frac{\pi d^2}{4} \frac{1}{\gamma_V} = 0{,}8 \cdot 360 \cdot \frac{\pi\, 19^2}{4} \cdot \frac{1}{1{,}25} = 65\,325 \text{ N}$$

$$P_{Rd,2} = 0{,}29\, \alpha\, d^2 \sqrt{f_{ck}\, E_{cm}}\, \frac{1}{\gamma_V} = 0{,}29 \cdot 1 \cdot 19^2 \cdot \sqrt{25 \cdot 31000} \cdot \frac{1}{1{,}25} = 73\,730 \text{ N}$$

where $\alpha = 0{,}2\left(\frac{h_{sc}}{d} + 1\right)$ for $3 \le \frac{h_{sc}}{d} \le 4$

$\alpha = 1$ for $h_{sc} > 4\, d$

E_{cm} is the section modulus of the concrete

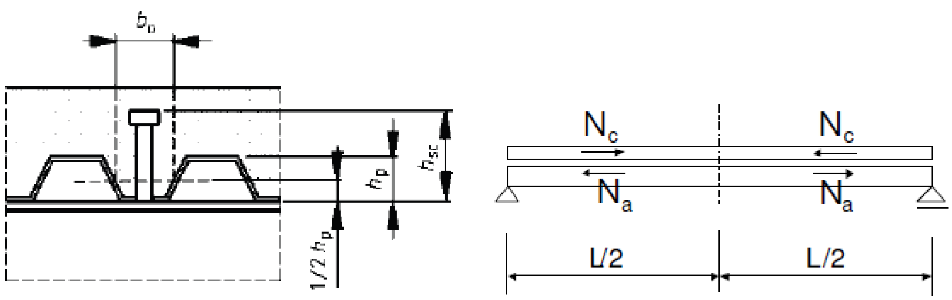
$\frac{h_{sc}}{d} = \frac{100}{19} = 5{,}3 > 4$, proto $\alpha = 1$

For trapozedial sheets, load capacity is recuded by k_t factor.

$$k_t = \frac{0{,}7}{\sqrt{n_r}} \frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right) = \frac{0{,}7}{\sqrt{1}} \frac{84{,}5}{50}\left(\frac{100}{50} - 1\right) = 1{,}18$$

where n_r is the number of mandrels in the rib

h_{sc} is the mandrel height



P_{Rd} = 0,85 · 65 = 55,5 kN

$F_{cf} = N_c = N_a$ = 1321 355 = 468,96 kN

$n_f = \frac{468{,}96}{55{,}5} = 8{,}45$ $\quad n_f = 9$

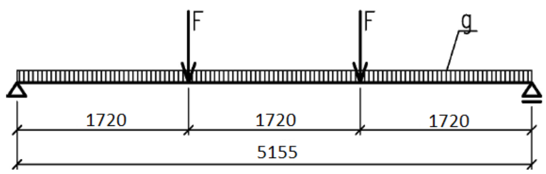
Mandrels can only be placed in the ribs of the TR sheet. Rib of the sheet is 250 mm. So mandrels can only be placed on one flange of the beam.

4635/250 = 18,54 > n_f = 9

2.3. Primary Beam

Slab is presumed supported during casting concrete. Calculations related assembly stage is ignored. Only the operational stage is assessed.

Beam along D-E axis is considered the most stressed.



$F_{Ek} = ((3{,}63 + 2{,}5 \cdot 1 + 0{,}8) \cdot 1{,}72 + 0{,}16)\,\frac{4{,}635}{2} = 28$ kN

$F_{Ed} = ((4{,}90 + 3{,}75 \cdot 1 + 1{,}2) \cdot 1{,}72 + 0{,}22)\,\frac{4{,}635}{2} = 39{,}77$ kN

$R_{Ed} = V_{Ed} = 39{,}77 + 0{,}54 \cdot \frac{4{,}635}{2} = 41{,}02$

$M_{Ek} = 28 \cdot 1{,}72 + \frac{1}{8} \cdot 0{,}4 \cdot 4{,}635^2 = 48{,}17$ kNm

$M_{Ed} = 39{,}77 \cdot 1{,}72 + \frac{1}{8} \cdot 0{,}54 \cdot 4{,}635^2 = 69{,}85$ kNm

<u>IPE200</u>

$m = 22{,}4$ kg/m

$A = 2848$ mm^2

$W_{pl,y} = 220600$ mm^3

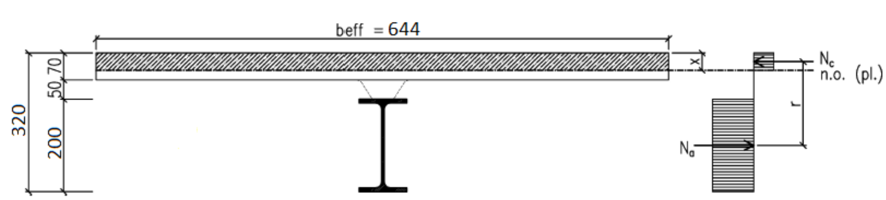
$I_y = 1943 \cdot 10^3$ mm^4

$A_{vz} = 1400$ mm^2

$h = 200$ mm

$b_f = 100$ mm

$t_f = 8{,}5$ mm



$N_a = N_c$

$A_a \cdot f_{yd} = x \cdot b_{eff} \cdot f_{cd}$

$2848 \cdot 355 = x \cdot 644 \cdot 14{,}17$

$x = 110{,}75$ mm > 70 mm

Condition is NOT satisfied.

$N_a = \frac{70 \cdot 644 \cdot 0{,}85 \cdot 25}{1{,}5} = 639$ kN   where f_{ck} =25 MPa and γ_c =1,5

$N_c = A \cdot f_{yd} = 2848 \cdot 355 = 1011$ kN

$x' = \frac{(N_c - N_a)}{2\, t_f \cdot f_{yd}} = \frac{1011 - 639}{2 \cdot 8{,}5 \cdot 355} = 5{,}24$ mm < t_f = 8,5 mm

$N_{a1} = b_f \cdot f_{yd} \cdot x' = 8{,}5 \cdot 355 \cdot 5{,}24 = 186{,}01$ kN

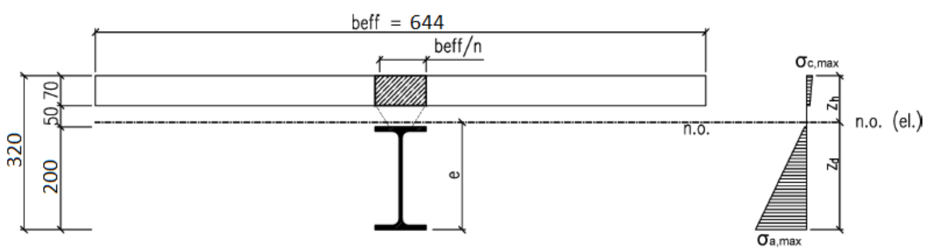
$r_1 = \frac{200+70+50}{2} = 185$ mm

$r_2 = \frac{70+50}{2} = 85$ mm

$M_{pl,Rd} = (N_c + 2 \cdot N_{a1})\, r_1 – 2 \cdot N_{a1} \cdot r_2 = (1011 + 2 \cdot 639)\, 185 - 2 \cdot 186 \cdot 85 = 155$ kNm > M_{Ed} = 69,85 kNm

$V_{pl,Rd} = \frac{A_{vz} \cdot f_{yd}}{\sqrt{3}} = 1400 \cdot 355/\sqrt{3} = 286{,}94$ kN > V_{Ed} = 41,02 kN

IPE200 satisfies the ULS check.



$E_c' = \frac{E_{cm}}{2} = \frac{31000}{2} = 15500$ MPa

$n = \frac{E_a}{E_c'} = \frac{210000}{15500} = 13{,}55$

$A_i = 2848 + 70 \cdot 679/13{,}55 = 6355\ mm^2$

L= 5155 mm

$B_{eff} = \frac{L}{8} = \frac{5155}{8} = 644$ mm

$e = \frac{2848 \cdot 100 + 70 \cdot \frac{644}{13{,}55}\ (200 + 50 + \frac{70}{2})}{6355} = 199{,}71$ mm

$I_i = (19{,}43 \cdot 10^6 + (199{,}71\text{-}200/2)^2 + \frac{1}{13{,}55} \cdot (\frac{679 \cdot 70^3}{13{,}55} + 1500 \cdot 70 \cdot (199{,}71 – 200 - 50\ 70/2)^2)$

$= 55{,}83 \cdot 10^6\ mm^4$

$$\sigma_{a,max} = \frac{M_{Ek}}{I_i} z_d = \frac{48{,}17 \cdot 10^6}{57{,}16 \cdot 10^6} 199{,}71 = 170{,}29\ MPa < 355\ MPa$$

$$\sigma_{c,max} = \frac{M_{Ek}}{n \cdot I_i} z_h = \frac{48{,}17 \cdot 10^6}{13{,}55 \cdot 57{,}16 \cdot 10^6} (360 - 199{,}71) = 7{,}66\ MPa < 0{,}85 \cdot f_{ck} = 21{,}3\ MPa$$

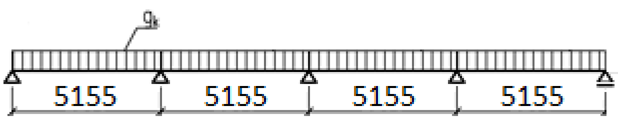
The beam behave flexibly under operating loads.

SLS Check

$$\delta_2 = \frac{23}{648} \frac{F_k \cdot L^3}{E \cdot I_i} = \frac{23}{648} \frac{26{,}04 \cdot 644^3}{210000 \cdot 55{,}83} = 10{,}80 < \frac{L}{400} = \frac{5155}{400} = 12{,}89\ mm$$

Deformation satisfies.

2.4. Trapezoidal Sheet on the Roof



Loads

Waterproof foil based on PVC 0,03 kN/m²

Mineral wool thick 200 mm 0,12 kN/m²

Inner TR sheet 0,13 kN/m²

$g_k = 0{,}28\ kN/m^2$

$g_{Ed} = y_f \cdot g_k = 0{,}28 \cdot 1{,}35 = 0{,}38\ kN/m^2$

Dead load and snow load;

$q_k = 0{,}28 + 0{,}56 = 0{,}85\ kN/m^2$

$q_{Ed} = 0{,}38 + 0{,}56 \cdot 1{,}5 = 1{,}22\ kN/m^2$

TR 85.280.1120 1.25 mm

$f_y = 320\ MPa$

$m = 13{,}14\ kg/m^2$

$q_{Rd} = 1{,}82\ kN/m^2$

$I = 1{,}619\ 10^6\ mm^4$

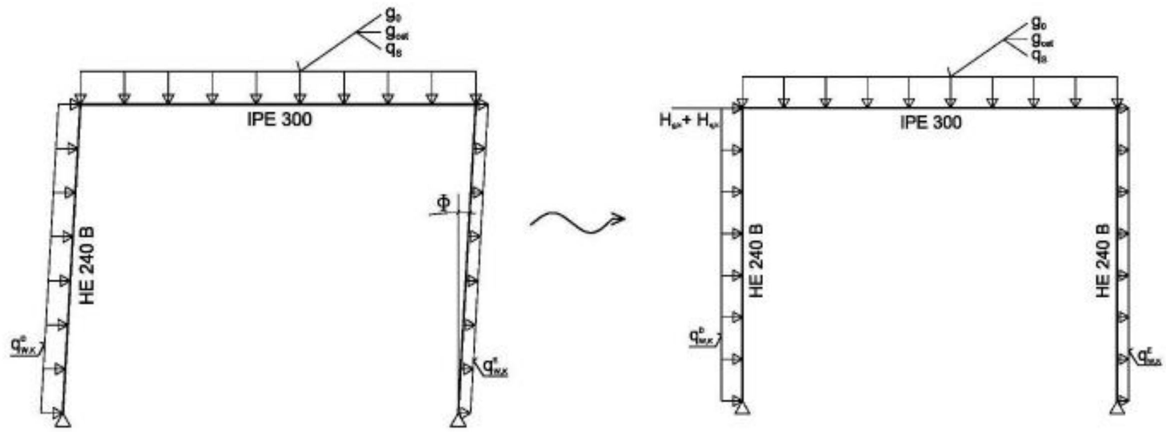
Values of the trapozeidal sheet are shown in Appendix D.

Sum effect of the moment is ignored. Moment on the supports are considered zero.

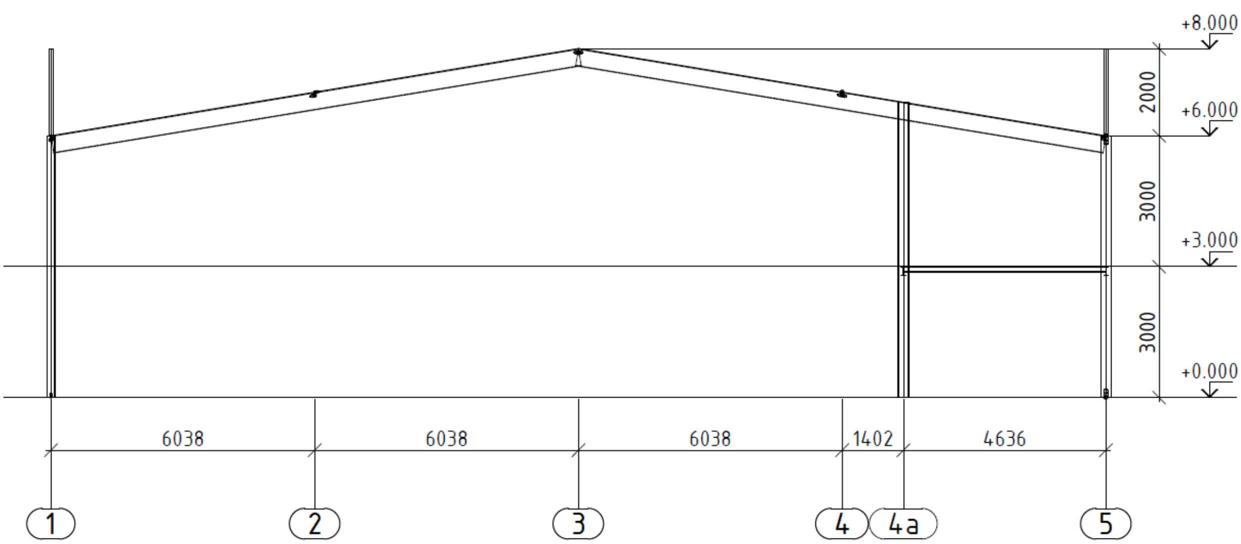
$\delta_2 = \frac{5}{348}\frac{q_{Ed} \cdot L^4}{E \cdot I} = \frac{5}{384}\frac{1{,}22 \cdot 5155^4}{210000 \cdot 1{,}619\ 10^6}$ = 22,98 mm < $\frac{L}{200} = \frac{5155}{200}$ = 25,77 mm

q_{Ed} = 1,22 kN/m^2 < q_k = 1,82 kN/m^2

3. IMPERFECTIONS



In the sketch above, as lateral forces are applied, structure tend to deform laterally. It is perfectly OK, if there are no any gravity loads, but gravity loads applies additional moment to the structure, by the amount of lateral deformation.



B-C-D axis will be assessed. Effect of the deformations should be affected to the structure if they change the behaviour of the structure significantly. IPE400 profile will be used for assessment. The condition is;

$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \leq 10$ for plastic analysis

$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \leq 15$ for elastic analysis

Combinations

These combinations is intended to find the value of deformation on assessed member.

n:1 combination

$(g_0 + g_{ost})_k \cdot \gamma_G + (q_{wk}^D + q_{wk}^E) \cdot \gamma_Q + q_s \cdot \gamma_Q \cdot \Psi_0$

n:2 combination

$(g_0 + g_{ost})_k \cdot \gamma_G + (q_{wk}^D + q_{wk}^E) \cdot \gamma_Q \cdot \Psi_0 + q_s \cdot \gamma_Q$

For snow $\Psi_0 = 0,5$, for wind $\Psi_0 = 0,6$

$\gamma_G = 1,35$; $\gamma_Q = 1,50$ (safety coefficients)

$\alpha_h = \frac{2}{\sqrt{h}} = \frac{2}{\sqrt{8}} = 0,71$ for $2/3 < \alpha_h < 1$ $\alpha_h = 0,71$

$\alpha_m = \sqrt{0,5 \cdot (1 + \frac{1}{m})}$ where m is the number of columns on assessed axis. In this case m = 3

$\alpha_m = \sqrt{0,5 \cdot (1 + \frac{1}{3})} = 0,82$

$\phi = \phi_0 \cdot \alpha_h \cdot \alpha_m = 2,911 \cdot 10^{-3}$

Calculation of equivalent forces:

$H_{gk} = g_r \cdot L \cdot \phi = 1,19 \cdot 24 \cdot 2,911 \cdot 10^{-3} = 0,08$ kN

$H_{qk} = q_s \cdot L \cdot \phi = 3,37 \cdot 24 \cdot 2,911 \cdot 10^{-3} = 0,24$ kN

Simplified procedure for portal frame

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

where;

$\sum H_{Ed}$ is the total horizontal load which also includes equivalent horizontal forces;

$\sum V_{Ed}$ is the total designed vertical load;

h is the height of the frame

δ_{Ed} is horizontal deformation obtained by a BIM software

Two combinations are considered to calculate the deformation.

Combination N:1

$H_{Ed}^I = (q_{wk}^D + q_{wk}^E) \cdot h \cdot 0,5 \cdot \gamma_Q + H_{gk} \cdot \gamma_G + H_{qk} \cdot \gamma_Q \cdot \Psi_0$

$= (1,86 + 0,77) \cdot 8 \cdot 0,5 \cdot 1,5 + 0,08 \cdot 1,5 \cdot 0,5 + 0,24 \cdot 1,5 \cdot 0,5 = 16,02$ kN

$V_{Ed}^{I} = g_r \cdot L \cdot \gamma_G + q_s \cdot L \cdot \gamma_Q \cdot \Psi_0$

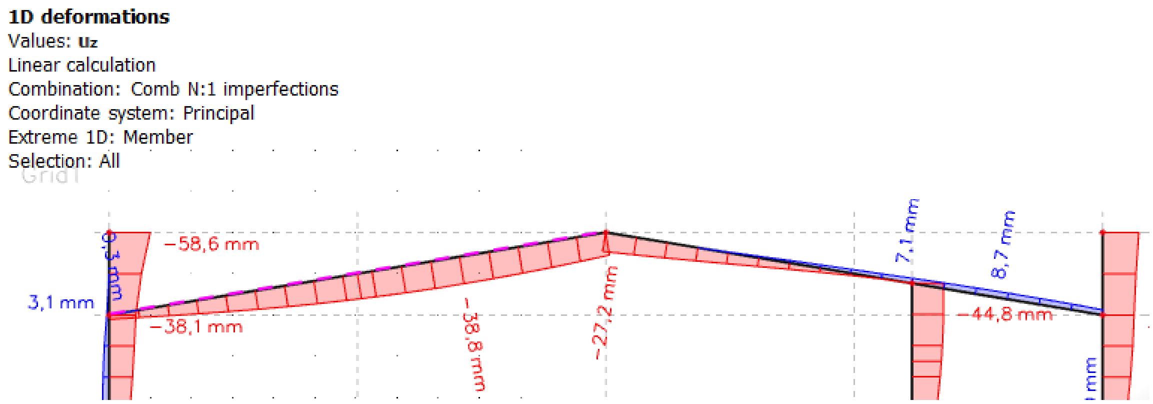
$= 1{,}19 \cdot 24 \cdot 1{,}35 + 3{,}37 \cdot 24 \cdot 1{,}5 \cdot 0{,}5 = 99{,}22$ kN

$\Delta_{Ed}^{I} = 38{,}8$ mm

where;

$q_{s(ave)} = (\frac{4{,}79+2{,}89}{2} + 2{,}89)/2 = 3{,}37$ kN/m [$q_{s(ave)}$ means average snow load on the beam]

$\alpha^{I}_{cr} = (\frac{16{,}02}{99{,}22}) \cdot (\frac{8000}{38{,}8}) = 33{,}29$



Combination N:2

$H_{Ed}^{II} = (q_{wk}^{D} + q_{wk}^{E}) \cdot h \cdot 0{,}5 \cdot \gamma_Q \cdot \Psi_0 + H_{gk} \cdot \gamma_G + H_{qk} \cdot \gamma_Q$

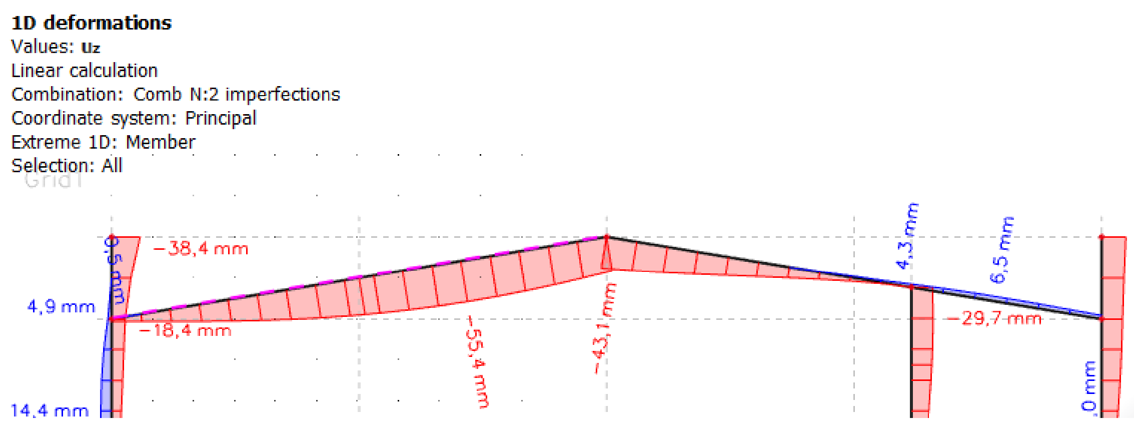
$= (1{,}86 + 0{,}77) \cdot 8 \cdot 0{,}5 \cdot 1{,}5 \cdot 0{,}6 + 0{,}08 \cdot 1{,}35 + 0{,}42 \cdot 1{,}5$

$= 10{,}21$ kN

$V_{Ed}^{II} = g_r \cdot L \cdot \gamma_G + q_s \cdot L \cdot \gamma_Q$ [g_r is the dead load of the roof]

$=1{,}19 \cdot 24 \cdot 1{,}35 + 3{,}37 \cdot 24 \cdot 1{,}5 = 159{,}88$ kN

$\Delta_{Ed}^{II} = 55{,}4$ mm



$\alpha^{II}_{cr} = (\frac{10,21}{159,88}) \cdot (\frac{8000}{55,4}) = 8,75$

coefficient for the second order effect is;

$(\frac{1}{1-\frac{1}{\alpha_{cr}}}) = (\frac{1}{1-\frac{1}{8,75}}) = 1,13$

Instead of using the result of handmade simplifications, ϕ value will be input to the Scia Engineering to the combinations.

4. SLS CHECK

For the columns in A axis

δ_{A1} = 13,5 mm (Case 2.2) L/250 = 6000/150 = 40 mm > 13,5 mm

δ_{A2} = 36,5 mm (Case 2.7) L/250 = 7000/150 = 46,6 mm > 36,5 mm

δ_{A3} = 38 mm (Case 2.6) L/250 = 8000/150 = 53,3 mm > 38 mm

δ_{A4} = 36,5 mm (Case 2.6) L/250 = 7000/150 = 46,6 mm > 36,5 mm

δ_{A5} = 12 mm (Case2.5) L/250 = 6000/150 = 40 mm > 12 mm

For the columns on B Axis

δ_{B1} = 30,2 mm (Case 2.6) L/250 = 6000/150 = 40 mm > 30,2 mm

$\delta_{B4bottom}$ = 17,6 mm (Case 2.1) L/250 = 3000/150 = 20 mm > 17,6 mm

δ_{B4top} = 26,9 mm (Case 2.1) L/250 = 6768/150 = 45 mm > 26,9 mm

$\delta_{B5bottom}$ = 17,5 mm (Case 2.1) L/250 = 3000/150 = 20 mm > 17,5 mm

δ_{B5top} = 26,4 mm (Case 2.1) L/250 = 6000/150 = 40 mm > 26,4 mm

Beam on the gable wall

δ_{A} = 23,1 mm (Case 2.2) L/250 = 6120/250 = 24,5 mm > 23,1 mm

δ_{B} = 65,1 mm (Case 1.1) L/250 = 19400/250 = 77,6 mm > 65,1 mm

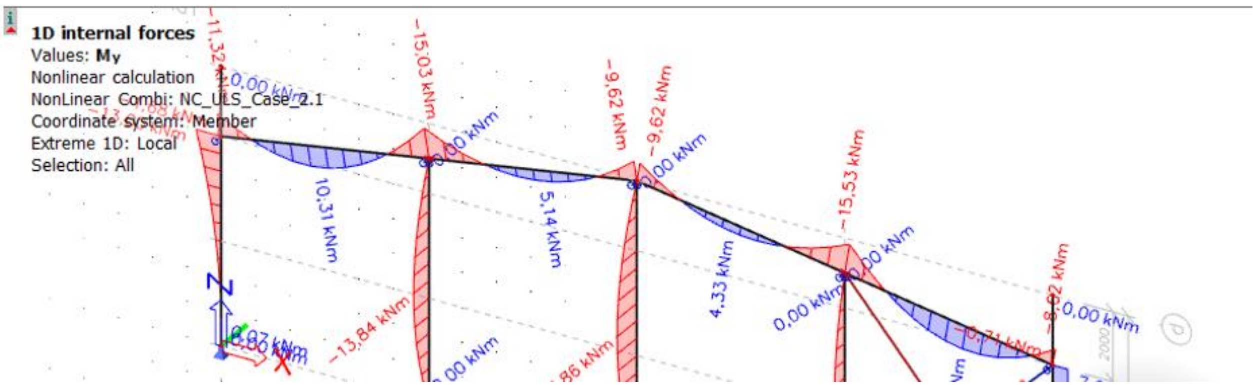
Diagrams can be found in Appendixes.

5. ULS CHECK

The members are exposed to bending and axial compression. Results of buckling resistance in most unfavorable combinations are presented for each member.

Internal forces and deformation diagrams calculated by Scia Software and critical moment resistance of lateral torsional buckling of the the main beam calculated by LTBeam. Diagrams can be found in Appendixes.

5.1. Beam A



IPE160

Profile Properties

$A = 2009\ mm^2$ $I_y = 8{,}693 \cdot 10^6\ mm^4$ $i_y = 65{,}8\ mm$

$A_{vz} = 966 \cdot 10^3\ mm^2$ $I_z = 683 \cdot 10^3\ mm^4$ $i_z = 18{,}4\ mm$

$W_{pl,y} = 123{,}9 \cdot 10^3\ mm^3$ $I_t = 36 \cdot 10^3\ mm^4$

$W_y = 109 \cdot 10^3\ mm^3$ $I_w = 3{,}96 \cdot 10^9\ mm^6$

Internal forces

$M_{Ed} = 15{,}53$ kNm (Co 2.1)

$N_{Ed} = 9{,}92$ kNm (pressure) (Co 2.4)

$V_{Ed} = 15{,}22$ kN (Co 2.1)

Critical buckling lengths

$L_{cry} = L_{crz} = 6120$ mm

Reduction factors due to flexural buckling

$$\lambda_y = \frac{L_{cry}}{i_y} = \frac{6120}{65{,}8} = 93{,}01 \qquad \bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{93{,}01}{76{,}4} = 1{,}22$$

$$\lambda_z = \frac{L_{crz}}{i_z} = \frac{6120}{18{,}4} = 332{,}61 \qquad \bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{322{,}61}{76{,}4} = 4{,}35$$

where; $\lambda_1 = 93{,}9 \cdot \sqrt{235/f_y} = 93{,}9 \cdot \sqrt{235/355} = 76{,}4$

$\chi_y = 0{,}38$ from buckling curve b

$\chi_z = 0{,}05$ from buckling curve c

Determination of critical moment

$k_w = 1$

$k_z = 1$

$$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 6120}\sqrt{\frac{210000 \cdot 3{,}96 \cdot 10^9}{81000 \cdot 36 \cdot 10^3}} = 0{,}27$$

$C_1 = 1{,}13$

$$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1 + \kappa_{wt}^2} = \frac{1{,}13}{1{,}0}\sqrt{1 + 0{,}27^2} = 1{,}17$$

$$M_{cr} = \mu_{cr}\frac{\pi\sqrt{E \cdot I_z \cdot G \cdot I_t}}{L} = 1{,}17 \cdot \frac{\pi\sqrt{210000 \cdot 683 \cdot 10^3 \cdot 81000 \cdot 36 \cdot 10^3}}{6120} = 389{,}01 \text{ kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl.y} f_y}{M_{cr}}} = \sqrt{\frac{123 \cdot 10^3 \cdot 355}{389{,}01 \cdot 10^6}} = 0{,}34$$

$\chi_{LT} = 0{,}92$ (from buckling curve a)

$$C_{mLT} = 0{,}1 - 0{,}8 \cdot \frac{10{,}26}{-11{,}58} = 0{,}81 \qquad C_{my} = 0{,}81$$

$$k_{yy1} = C_{my}\left(1 + (\bar{\lambda}_y - 0{,}2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}81\ (1 + (1{,}22 - 0{,}2)\frac{9920}{0{,}38 \cdot 2009 \cdot 355/1{,}0} = 0{,}839$$

$$k_{yy2} = C_{my}\left(1 + 0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}81\ (1 + 0{,}8\ \frac{9920}{0{,}38 \cdot 2009 \cdot 355/1{,}0} = 0{,}832$$

$k_{yy} = \min(k_{yy1}, k_{yy2}) = 0{,}832$

$$k_{zy1} = 1 - \frac{0{,}1\bar{\lambda}_z}{(C_{mLT} - 0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1 \cdot 4{,}35}{0{,}61 - 0{,}25} \cdot \frac{9920}{0{,}05 \cdot 2009 \cdot 355/1{,}0} = 0{,}783$$

$$k_{zy2} = 1 - \frac{0{,}1}{(C_{mLT} - 0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1}{0{,}61 - 0{,}25} \cdot \frac{9920}{0{,}05 \cdot 2009 \cdot 355/1{,}0} = 0{,}950$$

$k_{zy} = \max(k_{yy1}, k_{yy2}) = 0{,}950$

Conditions to be satisfied;

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{16410}{0{,}38 \cdot 2009 \cdot 355/1{,}0} + 0{,}832\ \frac{15{,}53 \cdot 10^6}{0{,}92 \cdot 43{,}98 \cdot 10^6/1{,}0} = 0{,}356 < 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{9920}{0{,}05 \cdot 2009 \cdot 355/1{,}0} + 0{,}918 \frac{15{,}53 \cdot 10^6}{0{,}92 \cdot 43{,}98 \cdot 10^6/1{,}0} = 0{,}598 < 1$$

where $M_{y,Rk} = f_{yd} \cdot W_{pl,y} = 355 \cdot 123{,}9 \cdot 10^3 = 43{,}98$ kNm

Shear force check

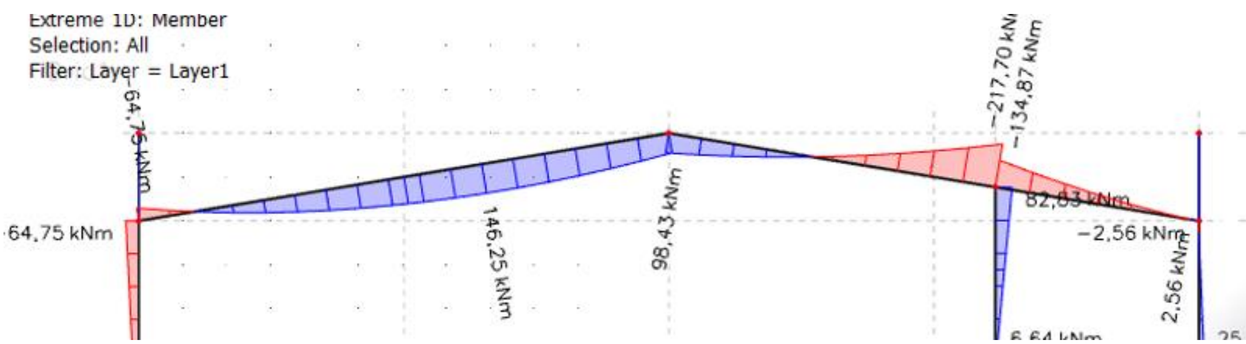
$$V_{pl,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}} = \frac{966 \cdot 355}{\sqrt{3}} = 198 \text{ kN} > V_{Ed} = 15{,}22 \text{ kN}$$

Slenderness check

$$\lambda = \frac{L_{crz}}{i_z} = \frac{6120}{18{,}4} = 332{,}61 < 400 \text{ is suggested.}$$

IPE160 satisfies the requirements.

5.2. Beam B



Influence of lateral torsion buckling

LTBeam used to determine the critical bending moment.

$M_{cr} = 271{,}09$ kNm > $M_{Ed} = 217{,}7$ kNm [See Appendix C]

IPE400

Profile Properties

$A = 8446$ mm^2	$I_y = 231{,}3 \cdot 10^6$ mm^4	$i_y = 165{,}5$ mm
$A_{vz} = 4269$ mm^2	$I_z = 13180 \cdot 10^3$ mm^4	$i_z = 39{,}5$ mm
$W_{pl,y} = 1307 \cdot 10^3$ mm^3	$I_t = 510{,}8 \cdot 10^3$ mm^4	
$W_y = 1157 \cdot 10^3$ mm^3	$I_w = 490 \cdot 10^9$ mm^6	

Internal forces

$M_{Ed} = 217{,}7$ kNm (Co 1.1)

N_{Ed} = 11 kNm (Co 2.1)

V_{Ed} = 63,88 kN (Co 1.1)

Critical buckling lengths

L_{cry} = 19600 mm

L_{crz} = 6038 mm

Reduction factors due to flexural buckling

$\lambda_y = \frac{L_{cry}}{i_y} = \frac{19600}{165,5} = 118,43$ $\bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{118,43}{76,4} = 1,55$

$\lambda_z = \frac{L_{crz}}{i_z} = \frac{6038}{39,5} = 152,86$ $\bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{152,86}{76,4} = 2,00$

where; $\lambda_1 = 93,9 \cdot \sqrt{235/f_y} = 93,9 \cdot \sqrt{235/355} = 76,4$

χ_y = 0,33 from buckling curve b

χ_z = 0,20 from buckling curve c

Determination of critical moment

$k_w = 1$

$k_z = 1$

$$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 6120}\sqrt{\frac{210000 \cdot 490 \cdot 10^9}{81000 \cdot 510,8 \cdot 10^3}} = 0,81$$

$C_1 = 0,31^{-0,5} = 1,796$

$$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1+\kappa_{wt}^2} = \frac{1,796}{1,0}\sqrt{1+0,81^2} = 2,32$$

$$M_{cr} = \mu_{cr}\frac{\pi\sqrt{E \cdot I_z \cdot G \cdot I_t}}{L} = 2,32 \cdot \frac{\pi\sqrt{210000 \cdot 13,180 \cdot 10^3 \cdot 81000 \cdot 510,8 \cdot 10^3}}{6038} = 12935,85 \text{ kNm}$$

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl.y} f_y}{M_{cr}}} = \sqrt{\frac{1307 \cdot 10^3 \cdot 355}{12935,85 \cdot 10^6}} = 0,19$ χ_{LT} = 0,91 (from buckling curve a)

$C_{my} = 0,1-0,8 \cdot \frac{146,25}{-217,7} = 0,64$ $C_{mLT} = 0,1$

$$k_{yy1} = C_{my}\left(1+(\bar{\lambda}_y - 0,2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0,81\left(1+(1,22-0,2)\frac{12370}{0,33 \cdot 8446 \cdot 355/1,0}\right) = 0,651$$

$$k_{yy2} = C_{my}\left(1+0,8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0,81\left(1+0,8\frac{12370}{0,33 \cdot 8446 \cdot 355/1,0}\right) = 0,646$$

$k_{yy} = \min(k_{yy1}, k_{yy2}) = 0,646$

$$k_{zy1} = 1 - \frac{0,1\bar{\lambda}_z}{(C_{mLT} - 0,25)} \frac{N_{Ed}}{\chi_z N_{Rk} / \gamma_{M1}} = 1 - \frac{0,1 \cdot 2}{1 - 0,25} \cdot \frac{16410}{0,2 \cdot 8446 \cdot 355/1,0} = 0,994$$

$$k_{zy2} = 1 - \frac{0,1}{(C_{mLT} - 0,25)} \frac{N_{Ed}}{\chi_z N_{Rk} / \gamma_{M1}} = 1 - \frac{0,1}{1 - 0,25} \cdot \frac{16410}{0,2 \cdot 8446 \cdot 355/1,0} = 0,997$$

$k_{zy} = \max(k_{yy1}, k_{yy2}) = 0,997$

Conditions to be satisfied;

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{12370}{0,33 \cdot 8446 \cdot 355/1,0} + 0,646 \frac{217,7 \cdot 10^6}{0,91 \cdot 463,99 \cdot 10^6/1,0} = 0,346 < 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{12370}{0,20 \cdot 8446 \cdot 355/1,0} + 0,997 \frac{217,7 \cdot 10^6}{0,91 \cdot 463,99 \cdot 10^6/1,0} = 0,354 < 1$$

where $M_{y,Rk} = f_{yd} \cdot W_{pl,y} = 355 \cdot 1307 \cdot 10^3 = 463,99$ kNm

Shear force check

$$V_{pl,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}} = \frac{4269 \cdot 355}{\sqrt{3}} = 874,97 \text{ kN} > V_{Ed} = 63,88 \text{ kN}$$

IPE400 satisfies the requirements.

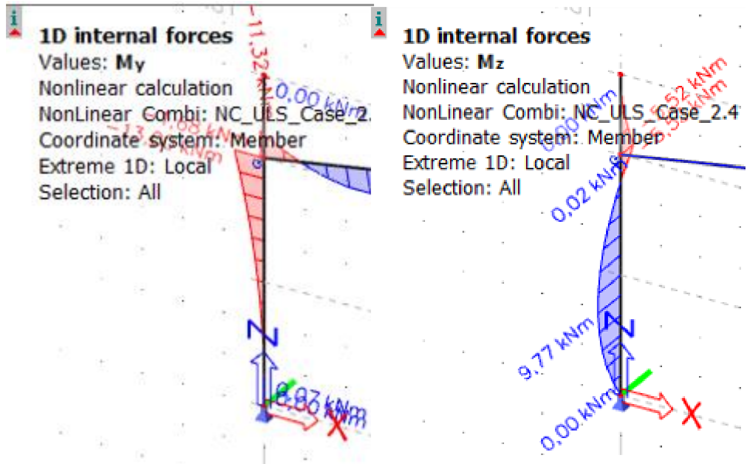
5.3. Column A1

HEA140

Profile Properties

$A = 3140$ mm^2	$I_y = 10,33 \cdot 10^6$ mm^4	$i_y = 57,3$ mm
$A_{vz} = 1010$ mm^2	$I_z = 3,893 \cdot 10^3$ mm^4	$i_z = 35,2$ mm
$W_{pl,y} = 173,5 \cdot 10^3$ mm^3	$I_t = 81,3 \cdot 10^3$ mm^4	
$W_y = 155,3 \cdot 10^3$ mm^3	$I_w = 15,06 \cdot 10^9$ mm^6	
$W_z = 55,61 \cdot 10^3$ mm^3		

Internal forces



$M_{y,Ed}$ = 13,82 kNm (Co 2.1)

$M_{Z,Ed}$ = 9,77 kNm (Co 2.7)

N_{Ed} = 19,31 kNm (Co 1.1)

V_{Ed} = 16,29 kN (Co 2.3)

For z direction;

Critical buckling lengths

L_{cry} = 6000 mm

L_{crz} = 6000 mm

Recution factors due to flexural buckling

$\lambda_y = \frac{L_{cry}}{i_y} = \frac{6000}{57,3} = 104,71$ $\qquad$ $\bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{104,71}{76,4} = 1,37$

$\lambda_z = \frac{L_{crz}}{i_z} = \frac{6000}{35,2} = 170,45$ $\qquad$ $\bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{170,45}{76,4} = 2,23$

where; $\lambda_1 = 93,9 \cdot \sqrt{235/f_y} = 93,9 \cdot \sqrt{235/355} = 76,4$

χ_y = 0,37 from buckling curve b

χ_z = 0,18 from buckling curve c

Determination of critical moment

k_w = 1

k_z = 1

$$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 7000}\sqrt{\frac{210000 \cdot 15,06 \cdot 10^9}{81000 \cdot 81,3 \cdot 10^3}} = 0,36$$

C_1 = 1,13

$$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1+\kappa_{wt}^2} = \frac{1,13}{1,0}\sqrt{1+0,36^2} = 1,20$$

$$M_{cr} = \mu_{cr}\frac{\pi\sqrt{E \cdot I_z \cdot G \cdot I_t}}{L} = 1,20 \cdot \frac{\pi\sqrt{210000 \cdot 10,33 \cdot 10^6 \cdot 81000 \cdot 81,3 \cdot 10^3}}{6000} = 74017 \text{ kNm}$$

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl.y} f_y}{M_{cr}}} = \sqrt{\frac{173,5 \cdot 10^3 \cdot 355}{38,97 \cdot 10^6}} = 1,15$ $\qquad$ χ_{LT} = 0,60 (from buckling curve a)

C_{my} = 0,95 $\qquad$ C_{mLT} = 0,95

$k_{yy1} = C_{my}\left(1+(\bar{\lambda}_y - 0{,}2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}95\ (1 + (1{,}37 - 0{,}2)\ \frac{19{,}31 \cdot 10^3}{0{,}38 \cdot 1114{,}7 \cdot 10^3 \cdot 355/1{,}0} = 1{,}002$

$k_{yy2} = C_{my}\left(1+0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}95\ (1 + 0{,}8\ \frac{19{,}31 \cdot 10^3}{0{,}20 \cdot 1114{,}7 \cdot 10^3 \cdot 355/1{,}0} = 0{,}986$

$k_{yy} = \min(k_{yy1}, k_{yy2}) = 0{,}986$

$k_{zy1} = 1 - \frac{0{,}1\bar{\lambda}_z}{(C_{mLT} - 0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1 \cdot 2{,}6}{1 - 0{,}25} \cdot \frac{19{,}31 \cdot 10^3}{0{,}2 \cdot 1114{,}7 \cdot 10^3 \cdot 355/1{,}0} = 0{,}969$

$k_{zy2} = 1 - \frac{0{,}1}{(C_{mLT} - 0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1}{1 - 0{,}25} \cdot \frac{19{,}31 \cdot 10^3}{0{,}2 \cdot 1114{,}7 \cdot 10^3 \cdot 355/1{,}0} = 0{,}986$

$k_{zy} = \max(k_{yy1}, k_{yy2}) = 0{,}986$

where;

$N_{Rk} = A \cdot f_y = 3140 \cdot 355 = 1114{,}7$ kN

For y direction;

Reduction factors due to flexural buckling

$\lambda_y = \frac{L_{cry}}{i_y} = \frac{6000}{35{,}2} = 170{,}45$

$\bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{170{,}45}{76{,}4} = 2{,}23$

$\lambda_z = \frac{L_{crz}}{i_z} = \frac{6000}{57{,}3} = 104{,}71$

$\bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{104{,}71}{76{,}4} = 1{,}37$

where; $\lambda_1 = 93{,}9 \cdot \sqrt{235/f_y} = 93{,}9 \cdot \sqrt{235/355} = 76{,}4$

$\chi_y = 0{,}18$ from buckling curve b

$\chi_z = 0{,}37$ from buckling curve c

Determination of critical moment

$k_w = 1$

$k_z = 1$

$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 6000}\sqrt{\frac{210000 \cdot 15{,}06 \cdot 10^9}{81000 \cdot 81{,}3 \cdot 10^3}} = 0{,}36$

$C_1 = = 1{,}13$

$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1 + \kappa_{wt}^2} = \frac{1{,}13}{1{,}0}\sqrt{1 + 0{,}36^2} = 1{,}20$

$M_{cr} = \mu_{cr}\frac{\pi\sqrt{E \cdot I_z \cdot G \cdot I_t}}{L} = 1{,}17 \cdot \frac{\pi\sqrt{210000 \cdot 3{,}893 \cdot 10^3 \cdot 81000 \cdot 81{,}3 \cdot 10^3}}{6000} = 46{,}18$ kNm

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl.y} f_y}{M_{cr}}} = \sqrt{\frac{84{,}84 \cdot 10^3 \cdot 355}{46{,}18 \cdot 10^6}} = 0{,}80$

$\chi_{LT} = 0{,}60$ (from buckling curve a)

C_{my} = 0,95 C_{mLT} = 0,95

$$k_{yy1} = C_{my}\left(1+(\bar{\lambda}_y-0{,}2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}95\left(1+(2{,}33-0{,}2)\frac{19{,}31\cdot 10^3}{0{,}18\cdot 1114{,}7\cdot 10^3\cdot 355/1{,}0}\right) = 1{,}136$$

$$k_{yy2} = C_{my}\left(1+0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}95\left(1+0{,}8\frac{19{,}31\cdot 10^3}{0{,}18\cdot 1114{,}7\cdot 10^3\cdot 355/1{,}0}\right) = 1{,}023$$

k_{yy} = min(k_{yy1}, k_{yy2}) = 1,023

$$k_{zy1} = 1-\frac{0{,}1\bar{\lambda}_z}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1\cdot 2{,}6}{1-0{,}25}\cdot\frac{19{,}31\cdot 10^3}{0{,}37\cdot 1114{,}7\cdot 10^3\cdot 355/1{,}0} = 0{,}991$$

$$k_{zy2} = 1-\frac{0{,}1}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1}{1-0{,}25}\cdot\frac{19{,}31\cdot 10^3}{0{,}37\cdot 1114{,}7\cdot 10^3\cdot 355/1{,}0} = 0{,}993$$

k_{zy} = max(k_{yy1}, k_{yy2}) = 0,993

where;

$N_{Rk} = A \cdot f_y$ = 3140 · 355 = 1114,7 kN

Conditions to be satisfied;

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

$$= \frac{19{,}39\cdot 10^3}{0{,}37\cdot 1114{,}7\cdot 10^3\cdot 355/1{,}0} + 0{,}986\frac{13{,}82\cdot 10^6}{0{,}60\cdot 61{,}59\cdot 10^6/1{,}0} + 1{,}024\frac{9{,}77\cdot 10^6}{61{,}59\cdot 10^6/1{,}0} = 0{,}747 < 1$$

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

$$= \frac{19{,}49\cdot 10^3}{0{,}18\cdot 3140\cdot 355/1{,}0} + 0{,}986\frac{23{,}05\cdot 10^6}{0{,}60\cdot 61{,}59\cdot 10^6/1{,}0} + 0{,}993\frac{23{,}05\cdot 10^6}{61{,}59\cdot 10^6/1{,}0} = 0{,}679 < 1$$

where

$M_{y,Rk} = M_{z,Rk}\ f_{yd} \cdot W_{pl,y} = 355 \cdot 173{,}5 \cdot 10^3$ = 61,59 kNm

Shear force check

$V_{Pl,Rd} = A_{vz} \cdot f_y / \sqrt{3} = 3140 \cdot 355 / \sqrt{3}$ = 207,01 kN > V_{Ed} = 16,93 kN

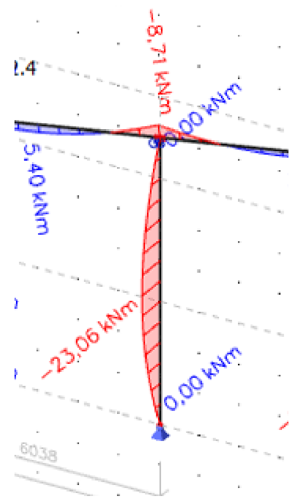
HEA140 satisfies the requirements.

5.4. Column A2

HEA140

Profile Properties

A = 3140 mm^2	I_y = 10,33 · 10^6 mm^4	i_y = 57,3 mm
A_{vz} = 1010 mm^2	I_z = 3,893 · 10^3 mm^4	i_z = 35,2 mm
$W_{pl,y}$ = 173,5 · 10^3 mm^3	I_t = 81,3 · 10^3 mm^4	
W_y = 155,3 · 10^3 mm^3	I_w = 15,06 · 10^9 mm^6	



Internal forces

M_{Ed} = 23,05 kNm (Co 2.4)

N_{Ed} = 31,18 kNm

V_{Ed} = 13,18 kN (Co 2.7)

Critical buckling lengths

L_{cry} = 7000 mm

L_{crz} = 7000 mm

Reduction factors due to flexural buckling

$\lambda_y = \frac{L_{cry}}{i_y} = \frac{7000}{57,3} = 122,16$ $\qquad \bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{122,16}{76,4} = 1,60$

$\lambda_z = \frac{L_{crz}}{i_z} = \frac{7000}{35,2} = 198,86$ $\qquad \bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{198,86}{76,4} = 2,60$

where; $\lambda_1 = 93,9 \cdot \sqrt{235/f_y} = 93,9 \cdot \sqrt{235/355} = 76,4$

χ_y = 0,38 from buckling curve b

χ_z = 0,15 from buckling curve c

Determination of critical moment

k_w = 1

k_z = 1

$$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 7000}\sqrt{\frac{210000 \cdot 15,06 \cdot 10^9}{81000 \cdot 81,3 \cdot 10^3}} = 0,31$$

C_1 = = 1,13

$$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1 + \kappa_{wt}^2} = \frac{1,13}{1,0}\sqrt{1 + 0,31^2} = 1,18$$

$M_{cr} = \mu_{cr} \frac{\pi\sqrt{E \cdot I_z \cdot G \cdot I_t}}{L} = 1{,}17 \cdot \frac{\pi\sqrt{210000 \cdot 13{,}180 \cdot 10^3 \cdot 81000 \cdot 510{,}8 \cdot 10^3}}{7000} = 38{,}97$ kNm

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{173{,}5 \cdot 10^3 \cdot 355}{38{,}97 \cdot 10^6}} = 1{,}26$ $\chi_{LT} = 0{,}53$ (from buckling curve a)

$C_{my} = 0{,}90$ $C_{mLT} = 0{,}95$

$k_{yy1} = C_{my}\left(1 + (\bar{\lambda}_y - 0{,}2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}90\ (1 + (1{,}60 - 0{,}2)\frac{31180}{0{,}38 \cdot 3140 \cdot 355/1{,}0} = 0{,}993$

$k_{yy2} = C_{my}\left(1 + 0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}90\ (1 + 0{,}8\ \frac{31180}{0{,}38 \cdot 3140 \cdot 355/1{,}0} = 0{,}953$

$k_{yy} = \min(k_{yy1}, k_{yy2}) = 0{,}953$

$k_{zy1} = 1 - \frac{0{,}1\bar{\lambda}_z}{(C_{mLT} - 0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1 \cdot 2{,}6}{1 - 0{,}25} \cdot \frac{31180}{0{,}2 \cdot 3140 \cdot 355/1{,}0} = 0{,}931$

$k_{zy2} = 1 - \frac{0{,}1}{(C_{mLT} - 0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1}{1 - 0{,}25} \cdot \frac{31180}{0{,}2 \cdot 3140 \cdot 355/1{,}0} = 0{,}973$

$k_{zy} = \max(k_{yy1}, k_{yy2}) = 0{,}973$

Conditions to be satisfied;

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{31180}{0{,}38 \cdot 3140 \cdot 355/1{,}0} + 0{,}953\ \frac{23{,}05 \cdot 10^6}{0{,}53 \cdot 61{,}59 \cdot 10^6/1{,}0} = 0{,}747 < 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{31180}{0{,}15 \cdot 3140 \cdot 355/1{,}0} + 0{,}973\ \frac{23{,}05 \cdot 10^6}{0{,}53 \cdot 61{,}59 \cdot 10^6/1{,}0} = 0{,}859 < 1$$

where $M_{y,Rk} = f_{yd} \cdot W_{pl,y} = 355 \cdot 173{,}5 \cdot 10^3 = 61{,}59$ kNm

Shear force check

$V_{pl,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}} = \frac{1010 \cdot 355}{\sqrt{3}} = 207$ kN > $V_{Ed} = 13{,}18$ kN

HEA140 satisfies the requirements.

5.5. Column A3

HEA160

Profile Properties

$A = 3880\ mm^2$ | $I_y = 16{,}73 \cdot 10^6\ mm^4$ | $i_y = 65{,}7\ mm$

$A_{vz} = 1320\ mm^2$ | $I_z = 6{,}156 \cdot 10^3\ mm^4$ | $i_z = 39{,}8\ mm$

$W_{pl,y} = 245{,}1 \cdot 10^3\ mm^3$ | $I_t = 121{,}9 \cdot 10^3\ mm^4$

$W_y = 220{,}1 \cdot 10^3\ mm^3$ | $I_w = 31{,}41 \cdot 10^9\ mm^6$

Internal forces

$M_{Ed} = 29{,}76$ kNm (Co 2.4)

$N_{Ed} = 23{,}72$ kNm (Co 1.1)

$V_{Ed} = 14{,}88$ kN (Co 2.7)

Critical buckling lengths

$L_{cry} = 8000$ mm

$L_{crz} = 8000$ mm

Reduction factors due to flexural buckling

$\lambda_y = \frac{L_{cry}}{i_y} = \frac{8000}{65{,}7} = 121{,}77$ $\quad\quad \overline{\lambda_y} = \frac{\lambda_y}{\lambda_1} = \frac{122{,}16}{76{,}4} = 1{,}59$

$\lambda_z = \frac{L_{crz}}{i_z} = \frac{8000}{39{,}8} = 201{,}01$ $\quad\quad \overline{\lambda_z} = \frac{\lambda_z}{\lambda_1} = \frac{198{,}86}{76{,}4} = 2{,}63$

where; $\lambda_1 = 93{,}9 \cdot \sqrt{235/f_y} = 93{,}9 \cdot \sqrt{235/355} = 76{,}4$

$\chi_y = 0{,}31$ from buckling curve b

$\chi_z = 0{,}11$ from buckling curve c

Determination of critical moment

$k_w = 1$

$k_z = 1$

$$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 7000}\sqrt{\frac{210000 \cdot 31{,}41 \cdot 10^9}{81000 \cdot 121{,}9 \cdot 10^3}} = 0{,}32$$

$C_1 = = 1{,}13$

$$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1 + \kappa_{wt}^2} = \frac{1{,}13}{1{,}0}\sqrt{1 + 0{,}32^2} = 1{,}19$$

$M_{cr} = \mu_{cr} \frac{\pi\sqrt{E \cdot I_z \cdot G \cdot I_t}}{L} = 1{,}19 \cdot \frac{\pi\sqrt{210000 \cdot 6{,}156 \cdot 10^3 \cdot 81000 \cdot 121{,}9 \cdot 10^3}}{8000} = 52{,}65$ kNm

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{245{,}1 \cdot 10^3 \cdot 355}{52{,}65 \cdot 10^6}} = 0{,}19$ $\chi_{LT} = 0{,}91$ (from buckling curve a)

$C_{my} = 0{,}90$ $C_{mLT} = 0{,}95$

$k_{yy1} = C_{my}\left(1 + (\bar{\lambda}_y - 0{,}2)\frac{N_{Ed}}{\chi_y N_{Rk} / \gamma_{M1}}\right) = 0{,}90\ (1 + (1{,}59 - 0{,}2)\frac{23720}{0{,}31 \cdot 3880 \cdot 355/1{,}0} = 0{,}970$

$k_{yy2} = C_{my}\left(1 + 0{,}8\frac{N_{Ed}}{\chi_y N_{Rk} / \gamma_{M1}}\right) = 0{,}90\ (1 + 0{,}8\ \frac{23720}{0{,}31 \cdot 3880 \cdot 355/1{,}0} = 0{,}940$

$k_{yy} = \min(k_{yy1}, k_{yy2}) = 0{,}940$

$k_{zy1} = 1 - \frac{0{,}1\bar{\lambda}_z}{(C_{mLT} - 0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk} / \gamma_{M1}} = 1 - \frac{0{,}1 \cdot 2{,}63}{1 - 0{,}25} \cdot \frac{23720}{0{,}2 \cdot 3880 \cdot 355/1{,}0} = 0{,}941$

$k_{zy2} = 1 - \frac{0{,}1}{(C_{mLT} - 0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk} / \gamma_{M1}} = 1 - \frac{0{,}1}{1 - 0{,}25} \cdot \frac{23720}{0{,}2 \cdot 3880 \cdot 355/1{,}0} = 0{,}978$

$k_{zy} = \max(k_{yy1}, k_{yy2}) = 0{,}978$

Conditions to be satisfied;

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{23720}{0{,}31 \cdot 3880 \cdot 355/1{,}0} + 0{,}953\ \frac{29{,}76 \cdot 10^6}{0{,}47 \cdot 87{,}01 \cdot 10^6/1{,}0} = 0{,}740 < 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{23720}{0{,}11 \cdot 3880 \cdot 355/1{,}0} + 0{,}973\ \frac{29{,}76 \cdot 10^6}{0{,}47 \cdot 87{,}01 \cdot 10^6/1{,}0} = 0{,}978 < 1$$

where $M_{y,Rk} = f_{yd} \cdot W_{pl,y} = 355 \cdot 245{,}1 \cdot 10^3 = 87{,}01$ kNm

Shear force check

$V_{pl,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}} = \frac{1320 \cdot 355}{\sqrt{3}} = 270{,}55$ kN > $V_{Ed} = 13{,}18$ kN

HEA160 satisfies the requirements.

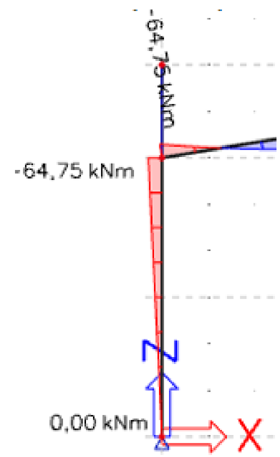
Columns on A4 axis is considered the same as S2 column similarly A5 is considered as the same as A1 column.

5.6. B1 column

HEA200

Profile Properties

$A = 5380\ mm^2$	$I_y = 36{,}9 \cdot 10^6\ mm^4$	$i_y = 82{,}8\ mm$
$A_{vz} = 1810\ mm^2$	$I_z = 13{,}36 \cdot 10^3\ mm^4$	$i_z = 49{,}8\ mm$
$W_{pl,y} = 429{,}5 \cdot 10^3\ mm^3$	$I_t = 209{,}8 \cdot 10^3\ mm^4$	
$W_y = 389 \cdot 10^3\ mm^3$	$I_w = 108 \cdot 10^9\ mm^6$	



Internal forces

$M_{Ed} = 64{,}75$ kNm (Co 1.1)

$N_{Ed} = 64{,}04$ kNm (Co 1.1)

$V_{Ed} = 14{,}10$ kN (Co 2.2)

Critical buckling lengths

$L_{cry} = 6000$ mm

$L_{crz} = 6000$ mm

Reduction factors due to flexural buckling

$\lambda_y = \frac{L_{cry}}{i_y} = \frac{6000}{82{,}8} = 72{,}46$ $\qquad \bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{72{,}46}{76{,}4} = 0{,}95$

$\lambda_z = \frac{L_{crz}}{i_z} = \frac{6000}{49{,}8} = 120{,}48$ $\qquad \bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{120{,}48}{76{,}4} = 1{,}58$

where; $\lambda_1 = 93{,}9 \cdot \sqrt{235/f_y} = 93{,}9 \cdot \sqrt{235/355} = 76{,}4$

$\chi_y = 0{,}63$ from buckling curve b

$\chi_z = 0{,}28$ from buckling curve c

Determination of critical moment

$k_w = 1$

$k_z = 1$

$$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 7000}\sqrt{\frac{210000 \cdot 108 \cdot 10^9}{81000 \cdot 209{,}8 \cdot 10^3}} = 0{,}60$$

$C_1 = 0{,}31^{-0{,}5} = 1{,}796$ (from the ampirical formula)

$$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1+\kappa_{wt}^2} = \frac{1{,}796}{1{,}0}\sqrt{1+0{,}60^2} = 2{,}10$$

$M_{cr} = \mu_{cr} \frac{\pi\sqrt{E \cdot I_z \cdot G \cdot I_t}}{L} = 2{,}10 \cdot \frac{\pi\sqrt{210000 \cdot 13{,}36 \cdot 10^3 \cdot 81000 \cdot 209{,}8 \cdot 10^3}}{6000} = 239{,}98$ kNm

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{429{,}5 \cdot 10^3 \cdot 355}{239{,}98 \cdot 10^6}} = 0{,}80$ $\chi_{LT} = 0{,}85$ (from buckling curve a)

$C_{my} = 0{,}90$ $C_{mLT} = 0{,}60$

$k_{yy1} = C_{my}\left(1 + (\bar{\lambda}_y - 0{,}2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}90\ (1 + (0{,}95 - 0{,}2)\frac{64040}{0{,}63 \cdot 5380 \cdot 355/1{,}0} = 0{,}936$

$k_{yy2} = C_{my}\left(1 + 0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}90\ (1 + 0{,}8\ \frac{64040}{0{,}63 \cdot 5380 \cdot 355/1{,}0} = 0{,}938$

$k_{yy} = \min(k_{yy1}, k_{yy2}) = 0{,}936$

$k_{zy1} = 1 - \frac{0{,}1\bar{\lambda}_z}{(C_{mLT} - 0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1 \cdot 1{,}58}{1 - 0{,}25} \cdot \frac{64040}{0{,}28 \cdot 5380 \cdot 355/1{,}0} = 0{,}946$

$k_{zy2} = 1 - \frac{0{,}1}{(C_{mLT} - 0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1}{1 - 0{,}25} \cdot \frac{64040}{0{,}28 \cdot 5380 \cdot 355/1{,}0} = 0{,}966$

$k_{zy} = \max(k_{yy1}, k_{yy2}) = 0{,}966$

Conditions to be satisfied;

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{64040}{0{,}63 \cdot 5380 \cdot 355/1{,}0} + 0{,}936\ \frac{64{,}75 \cdot 10^6}{0{,}85 \cdot 152{,}47 \cdot 10^6/1{,}0} = 0{,}521 < 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{61040}{0{,}28 \cdot 5380 \cdot 355/1{,}0} + 0{,}966\ \frac{64{,}75 \cdot 10^6}{0{,}85 \cdot 152{,}47 \cdot 10^6/1{,}0} = 0{,}587 < 1$$

where $M_{y,Rk} = f_{yd} \cdot W_{pl,y} = 355 \cdot 429{,}5 \cdot 10^3 = 152{,}47$ kNm

Shear force check

$V_{pl,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}} = \frac{1810 \cdot 355}{\sqrt{3}} = 370{,}98$ kN $> V_{Ed} = 14{,}10$ kN

HEA200 satisfies the requirements.

5.7. B4 column bottom part

B4 column will be assessed into two parts, because mezzanine divides it into two and both acts differently.

HEA260

Profile Properties

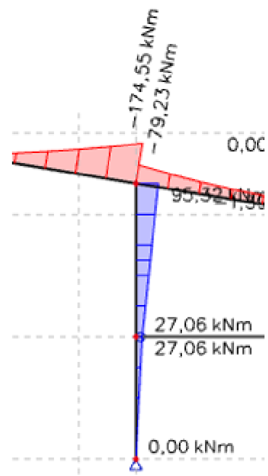
$A = 8680\ mm^2$	$I_y = 104{,}5 \cdot 10^6\ mm^4$	$i_y = 109{,}7\ mm$
$A_{vz} = 2876\ mm^2$	$I_z = 36{,}68 \cdot 10^3\ mm^4$	$i_z = 65\ mm$
$W_{pl,y} = 919{,}8 \cdot 10^3\ mm^3$	$I_t = 523{,}7 \cdot 10^3\ mm^4$	
$W_y = 836 \cdot 10^3\ mm^3$	$I_w = 516{,}4 \cdot 10^9\ mm^6$	

Internal forces

$M_{Ed} = 27{,}06$ kNm (Co 2.1)

$N_{Ed} = 209{,}59$ kNm (Co 2.3)

$V_{Ed} = 12{,}42$ kN (Co 2.9)



Critical buckling lengths

$L_{cry} = 3000$ mm

$L_{crz} = 3000$ mm

Reduction factors due to flexural buckling

$$\lambda_y = \frac{L_{cry}}{i_y} = \frac{3000}{109{,}7} = 27{,}35 \qquad \bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{27{,}35}{76{,}4} = 0{,}36$$

$$\lambda_z = \frac{L_{crz}}{i_z} = \frac{3000}{65} = 46{,}15 \qquad \bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{46{,}15}{76{,}4} = 0{,}60$$

where; $\lambda_1 = 93{,}9 \cdot \sqrt{235/f_y} = 93{,}9 \cdot \sqrt{235/355} = 76{,}4$

$\chi_y = 0{,}92$ from buckling curve b

$\chi_z = 0{,}90$ from buckling curve c

Determination of critical moment

$k_w = 1$

$k_z = 1$

$$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 3000}\sqrt{\frac{210000 \cdot 516{,}4 \cdot 10^9}{81000 \cdot 523{,}7 \cdot 10^3}} = 1{,}33$$

$C_1 = 0{,}31^{-0{,}5} = 1{,}796$ (from the ampirical formula)

$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1+\kappa_{wt}^2} = \frac{1,796}{1,0}\sqrt{1+1,33^2} = 3,50$

$M_{cr} = \mu_{cr}\frac{\pi\sqrt{E \cdot I_z \cdot G \cdot I_t}}{L} = 3,50 \cdot \frac{\pi\sqrt{210000 \cdot 36,68 \cdot 10^3 \cdot 81000 \cdot 523,7 \cdot 10^3}}{3000} = 2096,74$ kNm

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl.y} f_y}{M_{cr}}} = \sqrt{\frac{919,8 \cdot 10^3 \cdot 355}{2096,74 \cdot 10^6}} = 0,80$ $\chi_{LT} = 0,94$ (from buckling curve a)

$C_{my} = 0,90$ $C_{mLT} = 0,60$

$k_{yy1} = C_{my}\left(1+(\bar{\lambda}_y - 0,2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0,90\ (1 + (0,36 - 0,2)\frac{209,59 \cdot 10^3}{0,92 \cdot 8680 \cdot 355/1,0} = 0,911$

$k_{yy2} = C_{my}\left(1+0,8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0,90\ (1 + 0,8\ \frac{209,59 \cdot 10^3}{0,92 \cdot 8680 \cdot 355/1,0} = 0,953$

$k_{yy} = \min(k_{yy1}, k_{yy2}) = 0,911$

$k_{zy1} = 1 - \frac{0,1\bar{\lambda}_z}{(C_{mLT} - 0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0,1 \cdot 1,58}{1 - 0,25} \cdot \frac{209,59 \cdot 10^3}{0,90 \cdot 8680 \cdot 355/1,0} = 0,987$

$k_{zy2} = 1 - \frac{0,1}{(C_{mLT} - 0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0,1}{1 - 0,25} \cdot \frac{209,59 \cdot 10^3}{0,90 \cdot 8680 \cdot 355/1,0} = 0,978$

$k_{zy} = \max(k_{yy1}, k_{yy2}) = 0,987$

Conditions to be satisfied;

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{209590}{0,92 \cdot 8680 \cdot 355/1,0} + 0,936\ \frac{27,06 \cdot 10^6}{0,94 \cdot 326,53 \cdot 10^6/1,0} = 0,154 < 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{209590}{0,90 \cdot 8680 \cdot 355/1,0} + 0,966\ \frac{27,06 \cdot 10^6}{0,94 \cdot 326,53 \cdot 10^6/1,0} = 0,156 < 1$$

where $M_{y,Rk} = f_{yd} \cdot W_{pl,y} = 355 \cdot 919,8 \cdot 10^3 = 326,53$ kNm

Shear force check

$V_{pl,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}} = \frac{2876 \cdot 355}{\sqrt{3}} = 589,46\ kN > V_{Ed} = 12,42$ kN

HEA260 satisfies the requirements.

5.8. B4 column top part

HEA260

Profile Properties

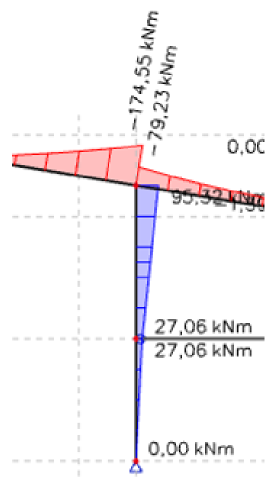
A = 8680 mm^2	I_y = 104,5 · 10^6 mm^4	i_y = 109,7 mm
A_{vz} = 2876 mm^2	I_z = 36,68 · 10^3 mm^4	i_z = 65 mm
$W_{pl,y}$ = 919,8 · 10^3 mm^3	I_t = 523,7 · 10^3 mm^4	
W_y = 836 · 10^3 mm^3	I_w = 516,4 · 10^9 mm^6	

Internal forces

M_{Ed} = 95,32 kNm (Co 2.1)

N_{Ed} = 106,31 kNm (Co 2.3)

V_{Ed} = 20,22 kN (Co 1.1)



Critical buckling lengths

L_{cry} = 3768 mm

L_{crz} = 3768 mm

Reduction factors due to flexural buckling

$\lambda_y = \frac{L_{cry}}{i_y} = \frac{3768}{109,7} = 34,35$ $\overline{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{34,35}{76,4} = 0,45$

$\lambda_z = \frac{L_{crz}}{i_z} = \frac{3768}{65} = 57,97$ $\overline{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{57,97}{76,4} = 0,76$

where; $\lambda_1 = 93,9 \cdot \sqrt{235/f_y} = 93,9 \cdot \sqrt{235/355} = 76,4$

χ_y = 0,91 from buckling curve b

χ_z = 0,65 from buckling curve c

Determination of critical moment

k_w = 1

k_z = 1

$$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 3768}\sqrt{\frac{210000 \cdot 516,4 \cdot 10^9}{81000 \cdot 523,7 \cdot 10^3}} = 1,33$$

$C_1 = (0,31+0,428\,\frac{27,06}{95,32} + 0,262\left(\frac{27,06}{95,32}\right)^2)^{-0,5}$ =1,486 (from ampirical formula)

$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1+\kappa_{wt}^2} = \frac{1,486}{1,0}\sqrt{1+1,67^2} = 2,48$

$M_{cr} = \mu_{cr}\frac{\pi\sqrt{E \cdot I_z \cdot G \cdot I_t}}{L} = 2,48 \cdot \frac{\pi\sqrt{210000 \cdot 36,68 \cdot 10^3 \cdot 81000 \cdot 523,7 \cdot 10^3}}{3768} = 1184,66$ kNm

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{919,8 \cdot 10^3 \cdot 355}{1184,66 \cdot 10^6}} = 0,80$ $\chi_{LT} = 0,85$ (from buckling curve a)

$C_{my} = 0,90$ $C_{mLT} = 0,6 + 0,4\frac{27,06}{95,32} = 0,71$

$k_{yy1} = C_{my}\left(1+(\bar{\lambda}_y - 0,2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0,90\ (1 + (0,45 - 0,2)\frac{106310}{0,91 \cdot 8680 \cdot 355/1,0} = 0,909$

$k_{yy2} = C_{my}\left(1+0,8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0,71\ (1 + 0,8\ \frac{106310}{0,91 \cdot 8680 \cdot 355/1,0} = 0,927$

$k_{yy} = \min(k_{yy1}, k_{yy2}) = 0,911$

$k_{zy1} = 1 - \frac{0,1\bar{\lambda}_z}{(C_{mLT} - 0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0,1 \cdot 0,76}{1 - 0,25} \cdot \frac{106310}{0,65 \cdot 8680 \cdot 355/1,0} = 0,991$

$k_{zy2} = 1 - \frac{0,1}{(C_{mLT} - 0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0,1}{1 - 0,25} \cdot \frac{106310}{0,65 \cdot 8680 \cdot 355/1,0} = 0,988$

$k_{zy} = \max(k_{yy1}, k_{yy2}) = 0,991$

Conditions to be satisfied;

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{106310}{0,91 \cdot 8680 \cdot 355/1,0} + 0,936\ \frac{95,32 \cdot 10^6}{0,85 \cdot 326,53 \cdot 10^6/1,0} = 0,350 < 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{106310}{0,65 \cdot 8680 \cdot 355/1,0} + 0,966\ \frac{95,32 \cdot 10^6}{0,85 \cdot 326,53 \cdot 10^6/1,0} = 0,365 < 1$$

where $M_{y,Rk} = f_{yd} \cdot W_{pl,y} = 355 \cdot 919,8 \cdot 10^3 = 326,53$ kNm

Shear force check

$V_{pl,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}} = \frac{2876 \cdot 355}{\sqrt{3}} = 589,46\ kN > V_{Ed} = 20,22$ kN

HEA260 satisfies the requirements.

5.9. B5 Bottom part

HEA260

Profile Properties

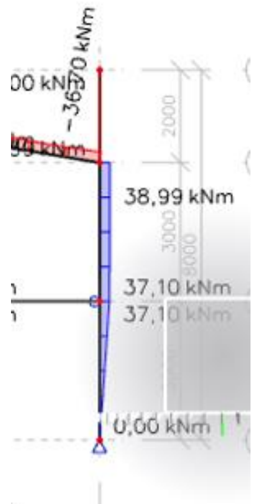
$A = 8680\ mm^2$	$I_y = 104{,}5 \cdot 10^6\ mm^4$	$i_y = 109{,}7\ mm$
$A_{vz} = 2876\ mm^2$	$I_z = 36{,}68 \cdot 10^3\ mm^4$	$i_z = 65\ mm$
$W_{pl,y} = 919{,}8 \cdot 10^3\ mm^3$	$I_t = 523{,}7 \cdot 10^3\ mm^4$	
$W_y = 836 \cdot 10^3\ mm^3$	$I_w = 516{,}4 \cdot 10^9\ mm^6$	

Internal forces

$M_{Ed} = 37{,}10$ kNm (Co 2.1)

$N_{Ed} = 135{,}19$ kNm (Co 2.4)

$V_{Ed} = 13{,}41$ kN (Co 2.1)



Critical buckling lengths

$L_{cry} = 3000$ mm

$L_{crz} = 3000$ mm

Recution factors due to flexural buckling

$$\lambda_y = \frac{L_{cry}}{i_y} = \frac{3000}{109{,}7} = 27{,}35 \qquad \bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{34{,}35}{76{,}4} = 0{,}36$$

$$\lambda_z = \frac{L_{crz}}{i_z} = \frac{3000}{65} = 46{,}15 \qquad \bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{57{,}97}{76{,}4} = 0{,}60$$

where; $\lambda_1 = 93{,}9 \cdot \sqrt{235/f_y} = 93{,}9 \cdot \sqrt{235/355} = 76{,}4$

$\chi_y = 0{,}93$ from buckling curve b

$\chi_z = 0{,}78$ from buckling curve c

Determination of critical moment

$k_w = 1$

$k_z = 1$

$$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 3000}\sqrt{\frac{210000 \cdot 516{,}4 \cdot 10^9}{81000 \cdot 523{,}7 \cdot 10^3}} = 1{,}67$$

$C_1 = 0{,}31^{-0{,}5} = 1{,}796$

$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1+\kappa_{wt}^2} = \frac{1{,}796}{1{,}0}\sqrt{1+1{,}67^2} = 3{,}50$

$M_{cr} = \mu_{cr}\frac{\pi\sqrt{E \cdot I_z \cdot G \cdot I_t}}{L} = 3{,}50 \cdot \frac{\pi\sqrt{210000 \cdot 36{,}68 \cdot 10^3 \cdot 81000 \cdot 523{,}7 \cdot 10^3}}{3000} = 2096{,}74\text{ kNm}$

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{919{,}8 \cdot 10^3 \cdot 355}{2096{,}74 \cdot 10^6}} = 0{,}39$ $\chi_{LT} = 0{,}95$ (from buckling curve a)

$C_{my} = 0{,}90$ $C_{mLT} = 0{,}60$

$k_{yy1} = C_{my}\left(1+(\bar{\lambda}_y - 0{,}2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}90\,(1+(0{,}36-0{,}2)\frac{135190}{0{,}93 \cdot 8680 \cdot 355/1{,}0} = 0{,}907$

$k_{yy2} = C_{my}\left(1+0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0{,}90\,(1+0{,}8\frac{135190}{0{,}93 \cdot 8680 \cdot 355/1{,}0} = 0{,}934$

$k_{yy} = \min(k_{yy1}, k_{yy2}) = 0{,}907$

$k_{zy1} = 1-\frac{0{,}1\bar{\lambda}_z}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1 \cdot 0{,}60}{1-0{,}25} \cdot \frac{135190}{0{,}78 \cdot 8680 \cdot 355/1{,}0} = 0{,}990$

$k_{zy2} = 1-\frac{0{,}1}{(C_{mLT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0{,}1}{1-0{,}25} \cdot \frac{135190}{0{,}78 \cdot 8680 \cdot 355/1{,}0} = 0{,}984$

$k_{zy} = \max(k_{yy1}, k_{yy2}) = 0{,}990$

Conditions to be satisfied;

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{135190}{0{,}93 \cdot 8680 \cdot 355/1{,}0} + 0{,}936\,\frac{135{,}19 \cdot 10^6}{0{,}95 \cdot 326{,}53 \cdot 10^6/1{,}0} = 0{,}156 < 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{135190}{0{,}78 \cdot 8680 \cdot 355/1{,}0} + 0{,}966\,\frac{135{,}19 \cdot 10^6}{0{,}95 \cdot 326{,}53 \cdot 10^6/1{,}0} = 0{,}165 < 1$$

where $M_{y,Rk} = f_{yd} \cdot W_{pl,y} = 355 \cdot 919{,}8 \cdot 10^3 = 326{,}53$ kNm

Shear force check

$V_{pl,Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3}} = \frac{2876 \cdot 355}{\sqrt{3}} = 589{,}46\text{ kN} > V_{Ed} = 17{,}30\text{ kN}$

HEA260 satisfies the requirements.

5.10. B5 top part

HEA260

Profile Properties

$A = 8680\ mm^2$ | $I_y = 104{,}5 \cdot 10^6\ mm^4$ | $i_y = 109{,}7\ mm$

$A_{vz} = 2876\ mm^2$ | $I_z = 36{,}68 \cdot 10^3\ mm^4$ | $i_z = 65\ mm$

$W_{pl,y} = 919{,}8 \cdot 10^3\ mm^3$ | $I_t = 523{,}7 \cdot 10^3\ mm^4$

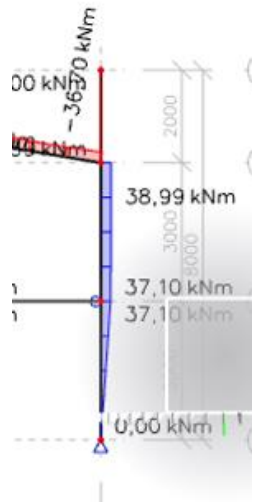
$W_y = 836 \cdot 10^3\ mm^3$ | $I_w = 516{,}4 \cdot 10^9\ mm^6$

Internal forces

$M_{Ed} = 39$ kNm (Co 2.1)

$N_{Ed} = 28{,}89$ kNm (Co 2.4)

$V_{Ed} = 17{,}30$ kN (Co 2.6)



Critical buckling lengths

$L_{cry} = 3000$ mm

$L_{crz} = 3000$ mm

Recution factors due to flexural buckling

$\lambda_y = \frac{L_{cry}}{i_y} = \frac{3000}{109{,}7} = 27{,}35$ $\qquad \bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{34{,}35}{76{,}4} = 0{,}36$

$\lambda_z = \frac{L_{crz}}{i_z} = \frac{3000}{65} = 46{,}15$ $\qquad \bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{57{,}97}{76{,}4} = 0{,}60$

where; $\lambda_1 = 93{,}9 \cdot \sqrt{235/f_y} = 93{,}9 \cdot \sqrt{235/355} = 76{,}4$

$\chi_y = 0{,}93$ from buckling curve b

$\chi_z = 0{,}78$ from buckling curve c

Determination of critical moment

$k_w = 1$

$k_z = 1$

$$\kappa_{wt} = \frac{\pi}{k_w \cdot L}\sqrt{\frac{E \cdot I_w}{G \cdot I_t}} = \frac{\pi}{1 \cdot 3000}\sqrt{\frac{210000 \cdot 516{,}4 \cdot 10^9}{81000 \cdot 523{,}7 \cdot 10^3}} = 1{,}67$$

$C_1 = (0{,}31+0{,}428\,\frac{37{,}10}{38{,}99} + 0{,}262\left(\frac{37{,}10}{38{,}99}\right)^2)^{-0,5} = 1{,}02$ (from ampirical formula)

$C_1 = 1{,}0$ is assumed

$\mu_{cr} = \frac{C_1}{k_z}\sqrt{1+\kappa_{wt}^2} = \frac{1,0}{1,0}\sqrt{1+1,67^2} = 1,95$

$M_{cr} = \mu_{cr}\frac{\pi\sqrt{E\cdot I_z\cdot G\cdot I_t}}{L} = 2,10 \cdot \frac{\pi\sqrt{210000\cdot 36,68\cdot 10^3\cdot 81000\cdot 523,7\cdot 10^3}}{3000} = 1167,42$ kNm

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl.y} f_y}{M_{cr}}} = \sqrt{\frac{919,8\cdot 10^3\cdot 355}{1167,42\cdot 10^6}} = 0,53$ $\chi_{LT} = 0,85$ (from buckling curve a)

$C_{my} = 1 \quad C_{mLT} = 1$

$k_{yy1} = C_{my}\left(1+(\bar{\lambda}_y - 0,2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 1\ (1+(0,36-0,2)\frac{28890}{0,93\cdot 8680\cdot 355/1,0} = 1,002$

$k_{yy2} = C_{my}\left(1+0,8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0,90\ (1+0,8\ \frac{28890}{0,93\cdot 8680\cdot 355/1,0} = 1,008$

$k_{yy} = \min(k_{yy1}, k_{yy2}) = 1,002$

$k_{zy1} = 1 - \frac{0,1\bar{\lambda}_z}{(C_{mLT}-0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0,1\cdot 0,60}{1-0,25}\cdot\frac{28890}{0,78\cdot 8680\cdot 355/1,0} = 0,999$

$k_{zy2} = 1 - \frac{0,1}{(C_{mLT}-0,25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} = 1 - \frac{0,1}{1-0,25}\cdot\frac{28890}{0,78\cdot 8680\cdot 355/1,0} = 0,998$

$k_{zy} = \max(k_{yy1}, k_{yy2}) = 0,999$

Conditions to be satisfied;

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{28890}{0,93\cdot 8680\cdot 355/1,0} + 0,936\ \frac{39\cdot 10^6}{0,95\cdot 326,53\cdot 10^6/1,0} = 0,151 < 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{28890}{0,78\cdot 8680\cdot 355/1,0} + 0,966\ \frac{39\cdot 10^6}{0,95\cdot 326,53\cdot 10^6/1,0} = 0,153 < 1$$

where $M_{y,Rk} = f_{yd} \cdot W_{pl,y} = 355 \cdot 919,8 \cdot 10^3 = 326,53$ kNm

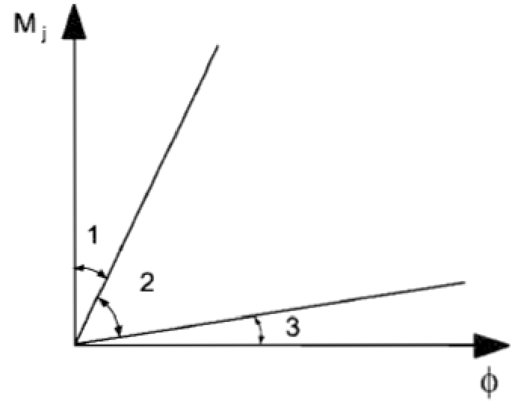
Shear force check

$V_{pl,Rd} = \frac{A_{vz}\cdot f_y}{\sqrt{3}} = \frac{2876\cdot 355}{\sqrt{3}} = 589,46\ kN > V_{Ed} = 17,30$ kN

HEA260 satisfies the requirements.

6. MOMENT CONNECTIONS

6.1 Introduction

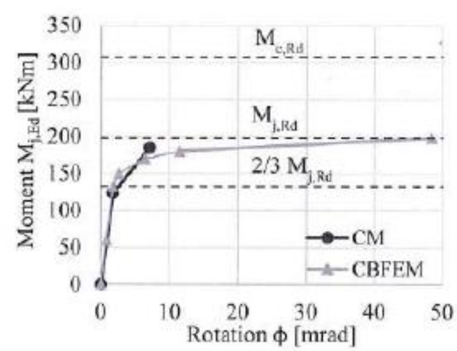


Connections are classified into three categories. Rigid connections are assumed to have enough rotational stiffness to be presumed on full continuity. In the sketch on the left, they are in Zone 1. Semi-rigid connections provide some degree of interaction between members based on the moment-angular rotation. They are in Zone 2. Nominally pinned connections transmit internal forces, developing almost no moment. It should be designed to satisfy the rotation under design loads. They are positioned in the Zone 3. For hinged connection rotational stiffness value in BIM softwares are considered infinite. But majority of the connections are semi-rigid in structures.

Table on the left shows an example of numerical calculation of rotational stiffness in any stages of loading. Connection is made up with HEB300 and IPE400. The beam web is connected with 5 mm thickness of weld while beam flange is connected with 9 mm weld. No head plate is used. Material is S235. Plasticity is applied in welds.

	$M_{j,Ed}$	S_j	φ
	[kNm]	[MNm/rad]	[mrad]
CBFEM	0	0	0,0
	60	80,5	0,7
	132	77,6	1,7
	150	60,0	2,5
	170	26,6	6,4
	180	15,8	11,4
	198	4,1	48,4

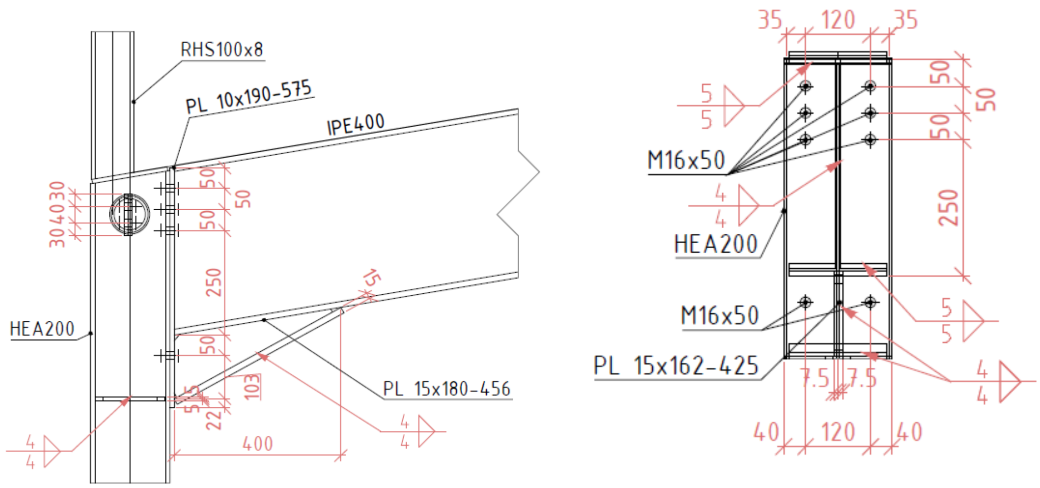
Initial stiffness is calculated by 2/3 $M_{j,Rd}$, where $M_{j,Rd}$ is the design moment resistance of the joint. $M_{c,Rd}$ is the design moment resistance of the beam. CBFEM stands for Component Based Finite Element Model. CM means Component Model.



As a result, rigid connections just doesn't satisfy the moment resistance, they have to satisfy much higher moment values. In this case some more bolts which has no work against moment resistance could be employed in order to increase the connections's rigidity. Table below shows the S_j and $S_{j,ini}$ values based on the CM and CBFEM methods.

CM				CBFEM			
$M_{j,Ed}$		$S_{j,ini}$	S_j	$M_{j,Ed}$		$S_{j,ini}$	S_j
[kNm]		[MNm/rad]	[MNm/rad]	[kNm]		[MNm/rad]	[MNm/rad]
2/3 $M_{j,Rd}$	123	74,3	-	2/3 $M_{j,Rd}$	132	77,6	-
$M_{j,Rd}$	185	-	26,0	$M_{j,Rd}$	198	-	4,1

6.2. C1 Connection



Frame Corner

Scheme of the connection is shown on the figure. All grades of the steel are S355. Bolts are not prestressed and with diameter M16 and stiffness 8.8. Frame corner is loaded by bending moment M_{Ed} = 51,3 kNm, and shear force V_{Ed} = 55,1 kN.

Design of the welds

The weld between the front plate and the tensioned flange of the beam;

$$a_1 \geq \frac{t_f}{2} \cdot \left(\frac{f_y}{\gamma_{m0}} \right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}} \right)$$

$$= \frac{10,7}{2} \cdot \frac{235}{1,0} \cdot \frac{0,8 \cdot 1,25}{355/\sqrt{2}} = 5,00 \text{ mm}$$

a_1 = 5 mm

where;

t_f is the flange thickness of the rung

The weld between the front plate and the compressed flange of the beam;

Weld on both sides of the construction are considered to be the same.

a_2 = 5 mm

$$a_3 \geq \frac{t_w}{2} \cdot \left(\frac{f_y}{\gamma_{m0}} \right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}} \right)$$

$$= \frac{6,5}{2} \cdot \frac{235}{1,0} \cdot \frac{0,8 \cdot 1,25}{355/\sqrt{2}} = 3,04 \text{ mm}$$

a_3 = 4 mm

where;

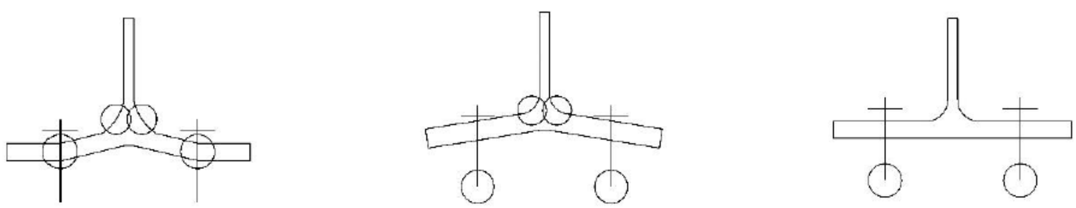
t_w is the thickness of the web of the beam

Stiffener for web of the column

t_v = 15 mm

Modes		Design resistance
Mode 1	Plastic mechanism (4 plastic joints)	$F_{t,a,Rd} = \frac{4 \cdot M_{pl,1,Rd}}{m}$
Mode 2	Plastic mechanism of levering (2 plsatic joints + bolts in tension)	$F_{t,b,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \sum F_{t,Rd}}{m+n}$
Mode 3	Bolts in tension	$F_{t,c,Rd} = \sum F_{t,Rd}$

Graphical representation of failure modes



Mode 1 *Mode 2* *Mode 3*

Thickness of the front plate

t_d = 10 mm

First row

$A_s \gamma_{M1}$

where,

$$F_{t,Rd} = \frac{0{,}9 \cdot A_s \cdot f_{ub}}{\gamma_{m,2}}$$

$$= \frac{0{,}9 \cdot 157 \cdot 800}{1{,}25} = 90{,}43 \text{ kN}$$ [A_s =157 mm^2 for M16 8.8]

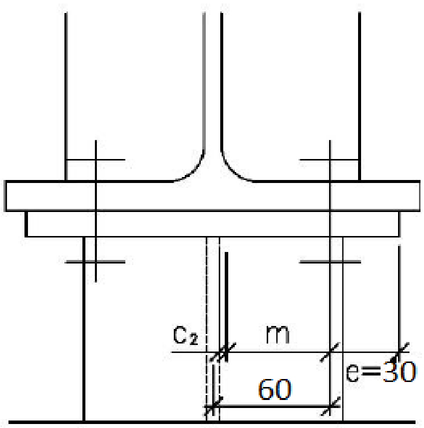
$$M_{pl,1,rd} = \frac{1}{4} l_{eff,1} \cdot t_{fc}^2 \cdot \frac{J_y}{\gamma_{m0}}$$ - t_{fc} flange thickness of the column

$$M_{pl,2,rd} = \frac{1}{4} l_{eff,2} \cdot t_{fc}^2 \cdot \frac{f_y}{\gamma_{m0}}$$ - t_{fc} flange thickness of the column

$l_{eff,1}$ is the effective width of the T-section for circular damage

$l_{eff,2}$ je is the effective width of the T-section for noncircular damage.

First row



$c_1 = 0,8 \cdot a_1 \cdot \sqrt{2} = 0,8 \cdot 10 \cdot \sqrt{2} = 11,31$ mm

$m_2 = 36,5 – c_1 = 36,5 – 11,31 = 15,19$ mm

$c_2 = 0,8 \cdot a_3 \cdot \sqrt{2} = 0,8 \cdot 5 \cdot \sqrt{2} = 5,66$ mm

m = 60 – 5,66 = 54,34 mm

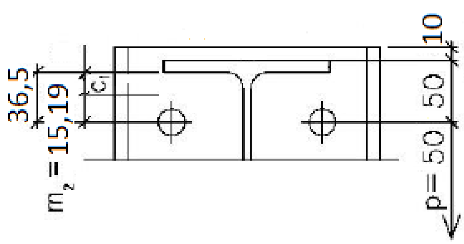
e = 30 mm

$\lambda_1 = \frac{m}{m+e} = \frac{54,34}{54,34+30} = 0,64$

$\lambda_2 = \frac{m_2}{m_2+e} = \frac{15,19}{15,19+30} = 0,34$

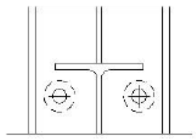
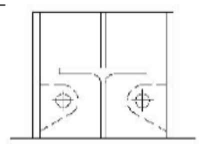
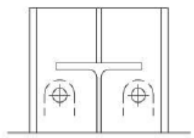
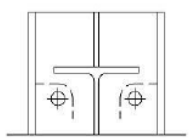
α = 5,5 (from table)

n = min(e; 1,25·m) = (30; 1,25·54,34) = 30 mm



*Figures are for demonstration. Plates doesn’t extend over the beam. But figure shows otherwise.

Effective width calculation for substituted T-section for bolts under tensioned flange

Series bolts acts individually		*Circular damage*	$l_{eff,cp}$ = 2 · π · m = 2 · π · 54,34 = 341,42 mm
		*Noncircular damage*	$l_{eff,nc}$ = α · m = 5,5 · 54,34 = 298,87 mm
Series bolts act as part of a group		*Circular damage*	$l_{eff,cp}$ = π · m + p = π · 54,34 + 50 = 220,71 mm
		*Noncircular damage*	$l_{eff,nc}$ = 0,5 · p + α · m – (2 · m + 0,625 · e) = 0,5 · 50 + 5,5 · 54,34 – (2 · 54,34 + 0,625 · 30) = 196,44 mm

$l_{eff,1}$ = min($l_{eff,nc}$; $l_{eff,cp}$) =196,44 mm

$$M_{pl,1,rd} = \frac{1}{4} l_{eff,1} \cdot t_{fc}^2 \cdot \frac{f_y}{\gamma_{m0}}$$

$= 0,25 \cdot 196,44 \cdot 10^2 \cdot 355/1 = 1,74 \cdot 10^6$ Nmm

$$M_{pl,2,rd} = \frac{1}{4} l_{eff,2} \cdot t_{fc}^2 \cdot \frac{f_y}{\gamma_{m0}}$$

$= 1,74 \cdot 10^6$ Nmm

Design capacity of bolts under tensioned flange

Mode 1

$$F_{t,a,Rd} = \frac{4 \cdot M_{pl,1,Rd}}{m}$$

$$= \frac{4 \cdot 1,74 \cdot 10^6}{54,34} = 128,08 \text{ kN}$$

Mode 2

$$F_{t,b,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \sum F_{t,Rd}}{m + n}$$

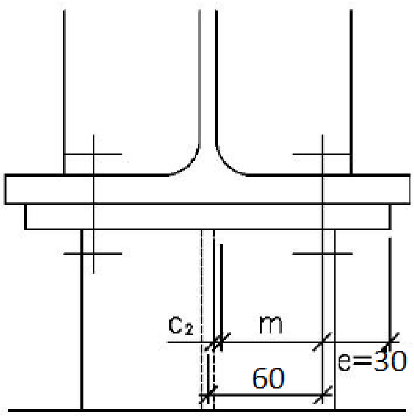
$$= \frac{2 \cdot 1,74 \cdot 10^6 + 30 \cdot 2 \cdot 90,43 \cdot 10^3}{54,34 + 30} = 105,60 \text{ kN}$$

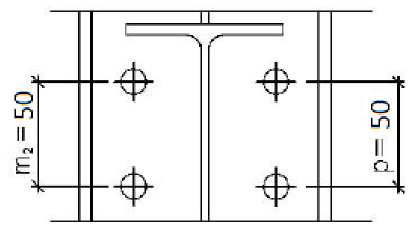
Mode 3

$F_{t,c,Rd} = \sum F_{t,Rd}$ = 2 · 90,43 = 180,46 kN

$F_{t,1,Rd} = \min(F_{t,a,Rd};\ F_{t,b,Rd};\ F_{t,c,Rd})$ = 105,60 kN

Second row

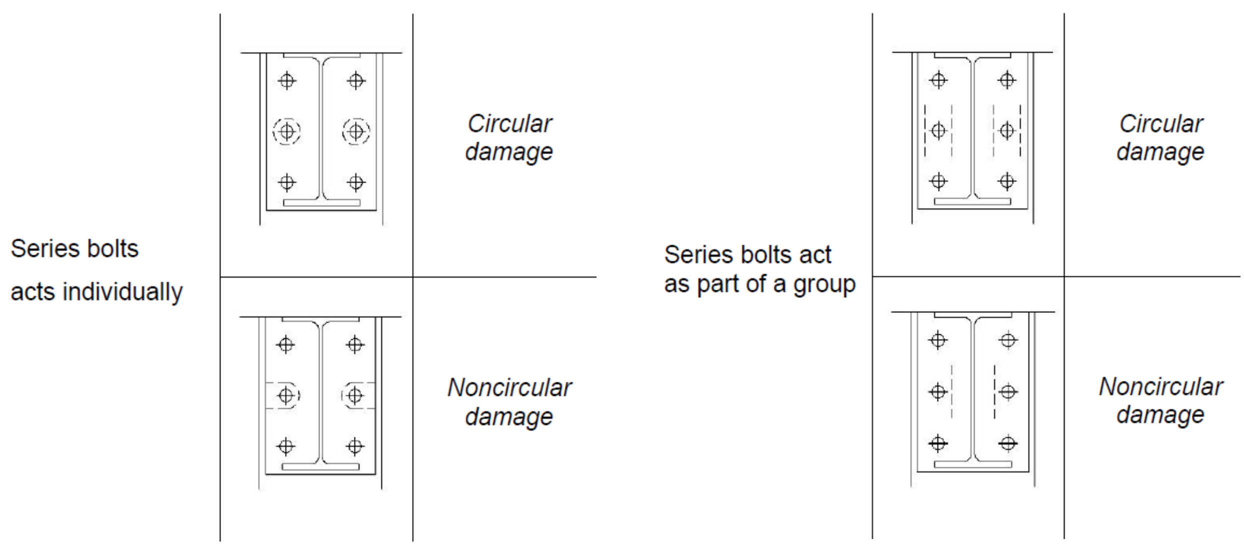




$c_2 = 0{,}8 \cdot a_3 \cdot \sqrt{2}$ = 5,66 mm

m = 60 – 5,66 = 54,34 mm

e = 30 mm



$l_{eff,cp} = 2 \cdot \pi \cdot m$ = 341,42 mm

$l_{eff,op} = 4 \cdot m + 1{,}25 \cdot e$ = 254,7 mm

$l_{eff,2} = \min(l_{eff,cp};\ l_{eff,op})$ = 50 mm

$l_{eff,cp} = 2 \cdot p$ = 100 mm

$l_{eff,op} = p$ = 50 mm

$$M_{pl,1,rd} = \frac{1}{4} l_{eff,1} \cdot t_{fc}^2 \cdot \frac{f_y}{\gamma_{m0}}$$

$= 0{,}25 \cdot 50 \cdot 10^2 \cdot 355/1 = 0{,}443 \cdot 10^6$ Nmm

$$M_{pl,2,rd} = \frac{1}{4} l_{eff,2} \cdot t_{fc}^2 \cdot \frac{f_y}{\gamma_{m0}}$$

$= 0{,}443 \cdot 10^6$ Nmm

Design capacity of bolts under tensioned flange

Mode 1

$$F_{t,a,Rd} = \frac{4 \cdot M_{pl,1,Rd}}{m}$$

$$= \frac{4 \cdot 0{,}443 \cdot 10^6}{54{,}34} = 32{,}61 \text{ kN}$$

Mode 2

$$F_{t,b,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \sum F_{t,Rd}}{m + n}$$

$$= \frac{2 \cdot 0{,}443 \cdot 10^6 + 30 \cdot 2 \cdot 90{,}43 \cdot 10^3}{54{,}34+30} = 74{,}83 \text{ kN}$$

Mode 3

$F_{t,c,Rd} = \sum F_{t,Rd}$ $= 2 \cdot 90{,}43 = 180{,}46$ kN

$F_{t,2,Rd} = \min(F_{t,a,Rd}; F_{t,b,Rd}; F_{t,c,Rd}) = 32{,}61$ kN

Third row is considered the same. p value is much higher in reality. But to be on the safe side, it is considered 50 mm as on the second row;

$F_{t,3,Rd} = 32{,}61$ kN

Calculation of the 4th bolt is ignored. It will have close to zero effect on moment resistance.

Shear Resistance

$$F_{v,Rd} = \frac{\alpha_v \cdot A_s \cdot f_{ub}}{\gamma_{m2}}$$

$$= \frac{0{,}6 \cdot 157 \cdot 800}{1{,}25} = 60{,}28 \text{ kN}$$ [$\alpha_v = 0{,}6$ for bolt class 4.6, 5.6, 8.6]

$t_f = 10$ mm

$t_d = 15$ mm (end plate thickness)

$F_{b,Rd} = 104{,}2$ kN (from table recommended section)

$V_{Rd} = 2 \cdot (F_{b,Rd}, F_{v,Rd}) = 2 \cdot 60{,}28 = 120{,}56 \text{ kN} > V_{Ed} = 55$ kN

Even though we haven't include the haunch in our calculations, the connection satisfies.

Capacity of the rungs flange under compression

$$F_{c,fb,Rd} = \frac{W_{pl,y} \cdot f_{yk}}{(h - t_f) \cdot \gamma_{m0}}$$

$$= \frac{628{,}4 \cdot 10^3 \cdot 355}{(400 - 13{,}5) \cdot 1} = 1200 \text{ N}$$

$$F_{c,fb,Rd} \geq \sum_{i=1}^{n} F_{t,i,Rd}$$

$$\sum_{i=1}^{i} (F_{t,i,Rd}) = 102{,}75 + 32{,}61 + 32{,}61 = 167{,}97 \text{ kN}$$

Condition is satisfied. No need to check stiffnesses.

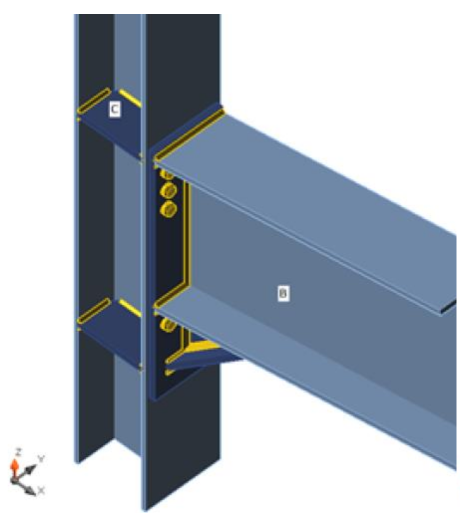
Row	Capasity	h_i
1	102,75 kN	510 mm
2	32,61 kN	460 mm
3	32,61 kN	410 mm

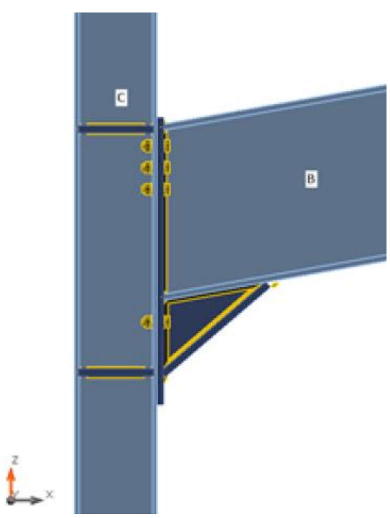
$M_{Rd} = \sum_{i=1}^{i} (F_{t,i,Rd}) \cdot h_i$

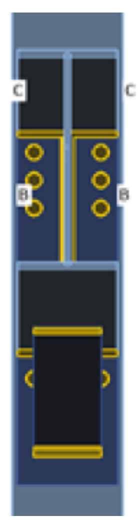
M_{Rd} = 102,75 · 0,51 + 32,61 · 0,46 · 2 = 80,77 kNm > M_{Ed} = 51,3 kNm

Even without considering the effect of the haunch the connection satisfies the moment resistance.

Calculation by IDEA Statica 9







Rotational stiffness

Name	Comp.	Loads	Mj,Rd [kNm]	Sj,ini [MNm/rad]	Φc [mrad]	L [m]	Sj,R [MNm/rad]	Sj,P [MNm/rad]	Class.
B	My	LE1	171,7	80,7	11,6	19,40	62,6	1,3	Rigid

Secant rotational stiffness

Name	Comp.	Loads	M [kNm]	Sjs [MNm/rad]	Φ [mrad]
B	My	LE1	51,3	120,2	0,4

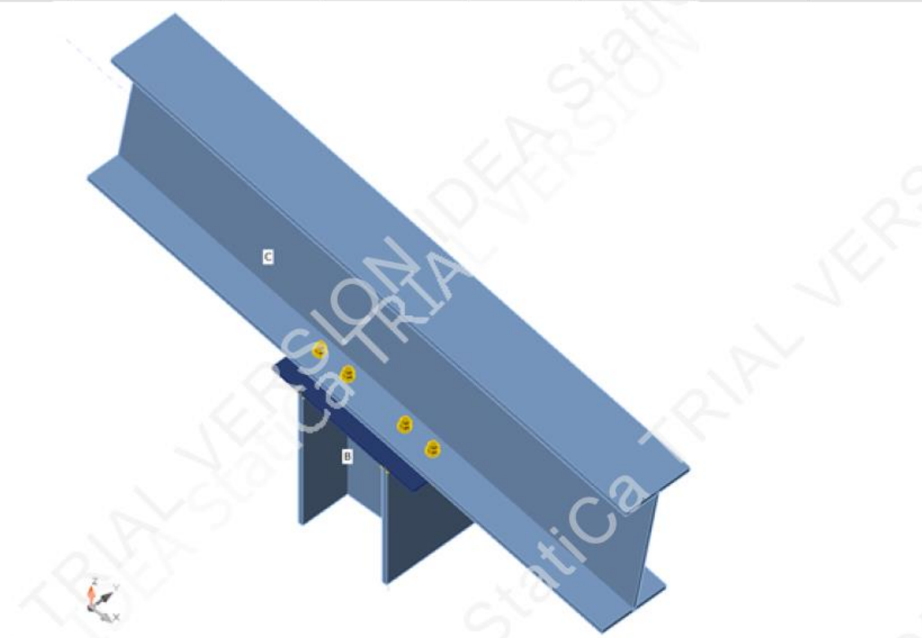
M_{Rd} = 171,7 kNm which is way above M_{Ed} value. As shown on the report of the application, connection is considered as “rigid”. Otherwise we have to set the rotational stiffness value and recalculate the moment resistance for the beam. It may lead us to change the profile for a larger one.

Details are in Appendix E.

6.3. B4 Moment Connection

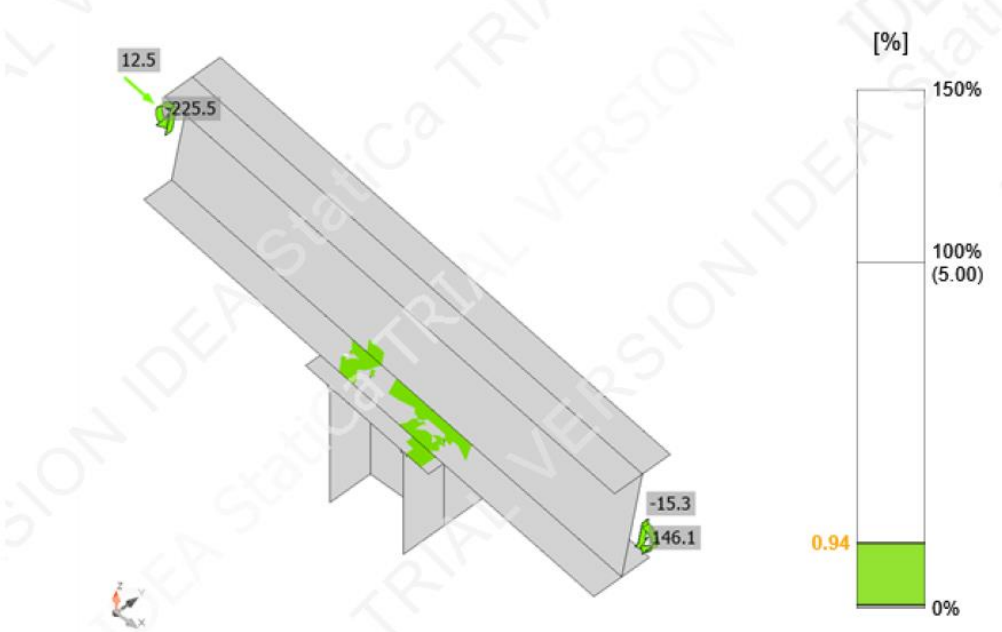
Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]	Forces in
C	3 - IPE400	0.0	10.0	0.0	0	0	0	Node
B	5 - HEA260	0.0	90.0	0.0	0	0	0	Node



B4 moment connection is calculated by IDEA Statica 9. It satisfies.

Overall check, LE1



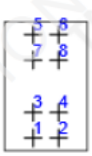
Summary

Name	Value	Status
Analysis	100.0%	OK
Plates	0.9 < 5.0%	OK
Bolts	98.7 < 100%	OK
Welds	98.8 < 100%	OK
Buckling	Not calculated	

Plates

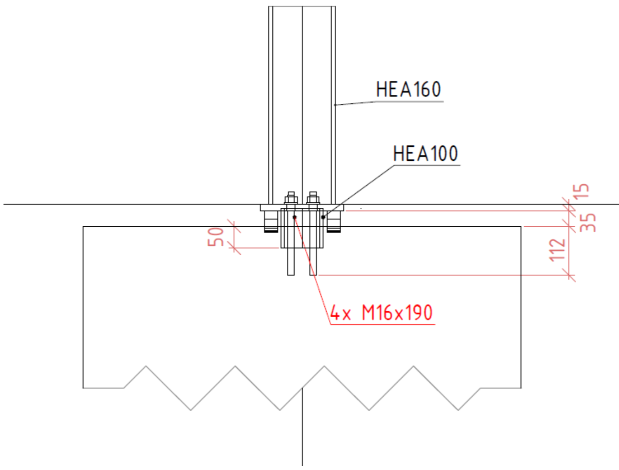
Name	Thickness [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	σc_{Ed} [MPa]	Status
C-bfl 1	13.5	LE1	355.8	0.4	85.6	OK
C-tfl 1	13.5	LE1	243.0	0.0	0.0	OK
C-w 1	8.6	LE1	328.7	0.0	0.0	OK
B-bfl 1	12.5	LE1	329.2	0.0	0.0	OK
B-tfl 1	12.5	LE1	284.5	0.0	0.0	OK
B-w 1	7.5	LE1	175.1	0.0	0.0	OK
EP1	10.0	LE1	357.0	0.9	130.6	OK

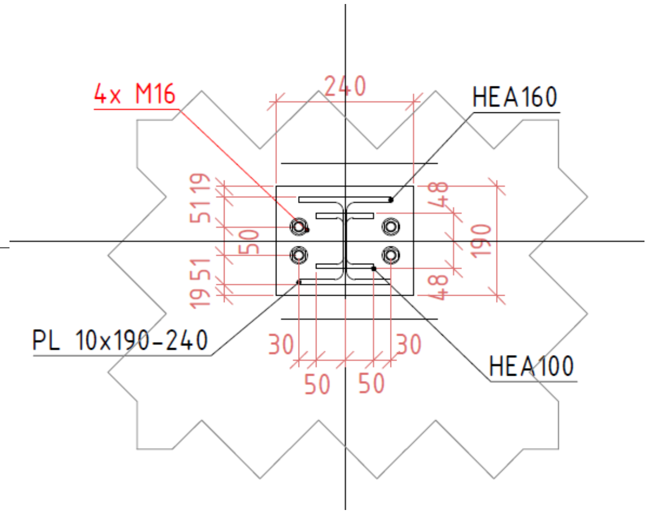
Bolts

	Name	Loads	$F_{t,Ed}$ [kN]	V [kN]	Ut_t [%]	$F_{b,Rd}$ [kN]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	LE1	85.3	2.8	94.3	130.3	4.7	72.1	OK
	B2	LE1	85.5	2.8	94.5	130.9	4.7	72.2	OK
	B3	LE1	89.3	4.0	98.7	156.8	6.6	77.1	OK
	B4	LE1	89.3	4.0	98.7	156.8	6.6	77.1	OK
	B5	LE1	0.0	10.3	0.0	156.8	17.0	17.0	OK
	B6	LE1	0.0	10.3	0.0	156.8	17.0	17.0	OK
	B7	LE1	27.1	2.9	30.0	156.8	4.7	26.1	OK
	B8	LE1	27.1	2.8	30.0	156.8	4.7	26.1	OK

7. OTHER CONNECTIONS

7.1. A3 Column Base





N_{Ed} = 31 kN (Co 1.1, highest normal force on A2 column)

a_c = 1,0 m b_c = 1,0 m   h = 0,8 m

a_0 = 270 mm b_0 = 270 mm

$a_1 = \min(3 \cdot a_0, a_0 + h, a_c)$

$= \min(3 \cdot 270, 280{+}800, 1000)$

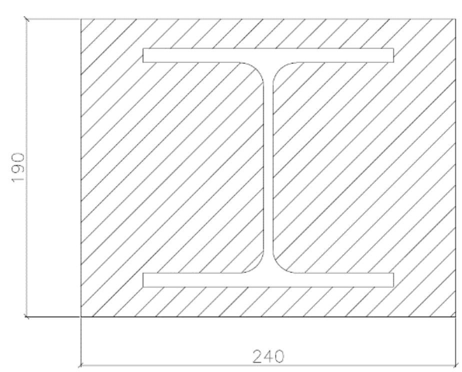
$= \min(840, 1080, 1000) = 810$ mm

$b_1 = \min(3 \cdot b_0, b_0 + h, a_c) = 810$ mm

$$k_j = \sqrt{\frac{a_1 \cdot b_1}{a_0 \cdot b_0}} = \sqrt{\frac{810 \cdot 810}{270 \cdot 270}} = 3$$

$$f_{jd} = \frac{B_j \cdot k_j \cdot f_{ck}}{\gamma_c} = \frac{2}{3}\,\frac{3 \cdot 16}{1,5} = 21,33 \text{ MPa}$$

$$c = t_p \cdot \sqrt{\frac{f_{yd}}{3 \cdot f_{jd}}} = 30\,\sqrt{\frac{355}{3 \cdot 21,33}} = 70,66 \text{ mm}$$

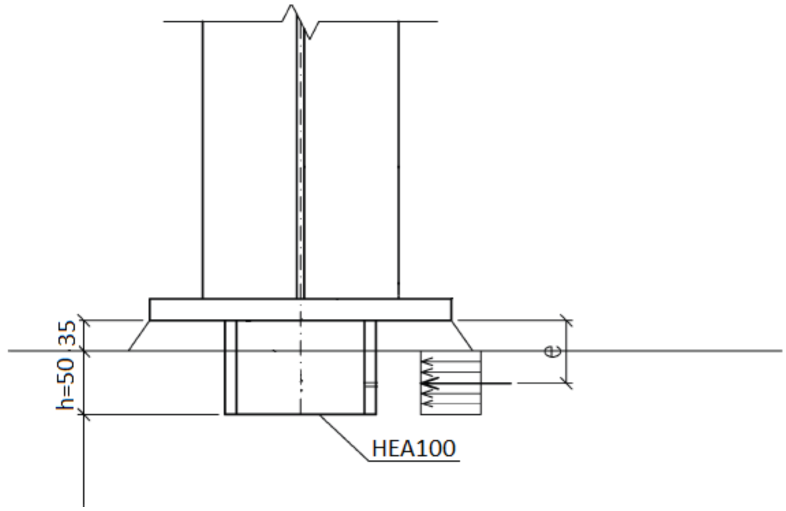


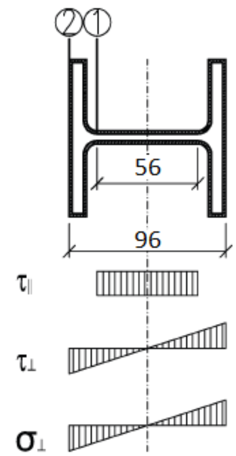
$A_{eff} = 240 \cdot 190 = 45600$ mm

$N_{Rd} = A_{eff} \cdot f_{jd} = 45600 \cdot 21,33 = 972$ kN

$N_{Rd} = 972 \text{ kN} >> N_{Ed} = 31$ kN

HEA100 for shear lug





V_{Ed} = 14,87 kN = $F_{v,Ed}$ (Co 2.9 Reactions)

h > $\frac{F_{v,Ed}}{b\,\frac{f_{ck}}{y_c}} = \frac{14{,}87}{200\,\frac{16}{1{,}5}}$ = 6,97 mm

h= 90 mm (50 mm below concrete)

$V_{Rd} = \frac{A_{vz} \cdot f_y}{\sqrt{3} \cdot y_{mo}} = \frac{0{,}76 \cdot 10^3 \cdot 355}{\sqrt{3} \cdot 1{,}0}$ = 155,27 kN

0,5 · V_{Rd} = 0,5 · 155,27 = 77,89 kN >> V_{Ed} = 14,87 kN

M_{Ed} = 0

$M_{Pl,Rd} = W_{Pl,y} \cdot f_y = 83{,}01 \cdot 10^3 \cdot 355$ = 29,47 kNm

Assessment in point 1

$\tau_{II} = \frac{F_{v,Ed}}{2 \cdot a \cdot l} = \frac{10 \cdot 10^3 \cdot 355}{2 \cdot 3 \cdot 40}$ = 41,67 MPa

$\tau_{\perp} = \sigma_{\perp} = \frac{1}{\sqrt{2}} \frac{F_{v,Ed} \cdot e}{\frac{I_w}{z}} = \frac{1}{\sqrt{2}} \frac{10 \cdot 10^3 \cdot (40+\frac{90}{2})}{\frac{10 \cdot 10^6}{56/2}}$ = 1,68 MPa

$\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{II}^2) = \sqrt{1{,}68^2 + 3(1{,}68^2 + 41{,}67^2)}$ = 72,63 MPa

$\frac{f_u}{\beta_w\ \gamma_{M2}} = \frac{490}{0{,}9 \cdot 1{,}25}$ = 435,6 MPa

72 MPa < 435,6 MPa

$\sigma_{\perp} = 4{,}8 < \frac{0{,}9\ f_u}{\gamma_{M2}} = \frac{0{,}9 \cdot 490}{1{,}25} = 352{,}8$ MPa

Assessment in point 2

τ_{II} = 0 MPa

$$\tau_{\perp} = \sigma_{\perp} = \frac{1}{\sqrt{2}} \frac{F_{v,Ed} \cdot e}{\frac{I_W}{z}} = \frac{1}{\sqrt{2}} \frac{10 \cdot 10^3 \cdot (40+\frac{90}{2})}{\frac{10 \cdot 10^6}{96/2}} = 2{,}88 \text{ MPa}$$

$$\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{II}^2) = \sqrt{2{,}88^2 + 3(2{,}88^2 + 0^2)} = 5{,}76 \text{ MPa}$$

$$\frac{f_u}{\beta_w \ \gamma_{M2}} = \frac{490}{0{,}9 \cdot 1{,}25} = 435{,}6 \text{ MPa}$$

5,76 MPa < 435,6 MPa

$$\sigma_{\perp} = 2{,}88 < \frac{0{,}9\, f_u}{\gamma_{M2}} = \frac{0{,}9 \cdot 490}{1{,}25} = 352{,}8 \text{ MPa}$$

Design of welds

The weld between the flange of the shear lug and the plate

$$a \geq \frac{t_f}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{8}{2} \cdot \frac{235}{1{,}0} \cdot \frac{0{,}8 \cdot 1{,}25}{355/\sqrt{2}} = 3{,}75 \text{ mm} \qquad a = 4 \text{ mm}$$

$$F_{w,Rd} = \frac{f_u \cdot 4 \cdot a \cdot l}{\sqrt{3} \cdot 0{,}8 \cdot \gamma_{M0}} = \frac{360 \cdot 4 \cdot 4 \cdot 50}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} = 166{,}28 \text{ kN} >> V_{Ed} = 14{,}87 \text{ kN}$$

The weld between the web of the shear lug and the plate

$$a \geq \frac{t_w}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{5}{2} \cdot \frac{235}{1{,}0} \cdot \frac{0{,}8 \cdot 1{,}25}{355/\sqrt{2}} = 2{,}34 \text{ mm} \qquad a = 3 \text{ mm}$$

$$F_{w,Rd} = \frac{f_u \cdot 4 \cdot a \cdot l}{\sqrt{3} \cdot 0{,}8 \cdot \gamma_{M0}} = \frac{360 \cdot 4 \cdot 3 \cdot 50}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} = 124{,}7 \text{ kN} >> V_{Ed} = 14{,}87 \text{ kN}$$

The weld between the flange of the column and the plate

$$a \geq \frac{t_f}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{9}{2} \cdot \frac{235}{1{,}0} \cdot \frac{0{,}8 \cdot 1{,}25}{355/\sqrt{2}} = 3{,}75 \text{ mm} \qquad a = 4 \text{ mm}$$

The weld between the web of the column and the plate

$$a \geq \frac{t_w}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{6}{2} \cdot \frac{235}{1{,}0} \cdot \frac{0{,}8 \cdot 1{,}25}{355/\sqrt{2}} = 2{,}34 \text{ mm} \qquad a = 3 \text{ mm}$$

Base plate highly satisfies.

7.2. B4a Column Base

Cross-sections

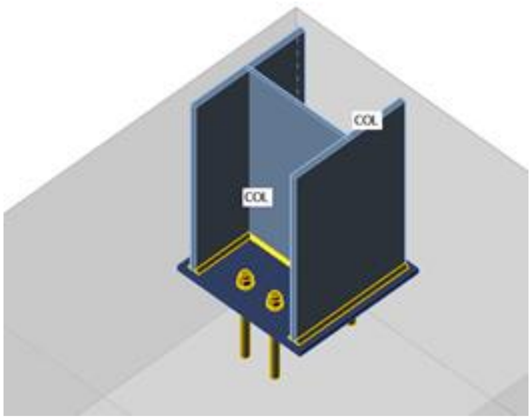
Name	Material
3 - HEA260	S 355

Anchors

Name	Bolt assembly	Diameter [mm]	fu [MPa]	Gross area [mm²]
M16 8.8	M16 8.8	16	800,0	201

Load effects (equilibrium not required)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	COL	-209,6	0,0	15,0	0,0	0,0	0,0



This base plate is designed by IDEA Statica 9. This time no shear lug is used. It satisfies.

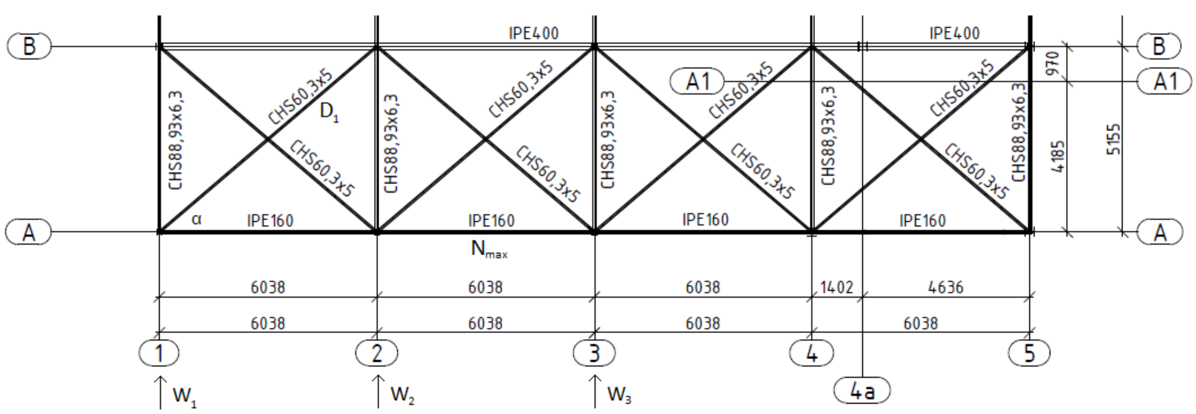
Summary

Name	Value	Status
Analysis	100,0%	OK
Plates	0,0 < 5%	OK
Anchors	7,8 < 100%	OK
Welds	18,2 < 100%	OK
Concrete block	17,6 < 100%	OK
Shear	27,8 < 100%	OK
Buckling	Not calculated	

Column base satisfies.

7.3. Design of the roof

Horizontal Bracing



W_1 = 14,74 kN

W_2 = 13,22 kN

W_3 = 14,89 kN

α = arctan(5155/6038) = 40,50°

R_{Ed} = 14,74 + 13,22 + 14,89/2 = 35,41 kN (pressure)

$D_1 = \frac{(35{,}41-14{,}74)}{2 \cdot \sin(40{,}5)}$ = 15,91 kN

$N_{max} = \frac{(35{,}41-14{,}74) \cdot 13{,}22 \cdot 6}{5{,}155}$ = 32,73 kN

Bracing

<u>CHS60,3x5</u>

A = 869 mm^2

i = 19,6 mm

$L = \sqrt{6038^2 + 5155^2}$ = 7940 mm

$L_{cry} = \frac{L}{2} = \frac{7940}{2}$ = 3970 mm

$\lambda = \frac{L_{cry}}{i} = \frac{3970}{19{,}6}$ = 202,55 $\overline{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{202{,}55}{76{,}4}$ = 2,65 χ = 0,15 (a line)

$N_{b,Rd} = \chi \cdot A \cdot f_y$ = 0,15 · 869 · 355 = 42,27 kN > D_1 = 15,91 kN

M16 8.8

d_0 = 18 mm

p_1 = 55 mm

e_1 = 40 mm

$F_{b,Rd}$ = 104,2 kN (t=10 mm S355)

$n = \frac{15{,}91}{104{,}2}$ = 0,15 Design: 4 bolts

$N_{u,Rd} = \frac{0{,}9 \cdot (100-2 \cdot 18) \cdot 360}{1{,}25}$ = 16,59 kN > D_1 = 15,91 kN

Weld design

$a \geq \frac{t}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{5}{2} \frac{355}{1{,}0} \frac{0{,}8 \cdot 1{,}25}{\frac{355}{\sqrt{2}}}$ = 2,34 mm a = 3 mm

$F_{w,Rd} = \frac{f_u \cdot 4 \cdot a \cdot l}{\sqrt{3} \cdot 0{,}8 \cdot y_{M0}} = \frac{360 \cdot 4 \cdot 3 \cdot 50}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25}$ = 124,7 kN >> V_{Ed} = 15,91 kN

Bracing satisfies.

Longitudal profile on the roof

CHS89x6,3

A= 1637 mm^2

i = 29,3 mm

L_{cry} = L = 5155 mm

$\lambda = \frac{L_{cry}}{i} = \frac{5155}{29,3} = 175,94$ $\quad \bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{175,94}{76,4} = 2,30$ $\quad \chi = 0,18$ (a line)

$N_{b,Rd} = \chi \cdot A \cdot f_y = 0,18 \cdot 1637 \cdot 355 = 104,60$ kN > R_{Ed} = 35,41 kN

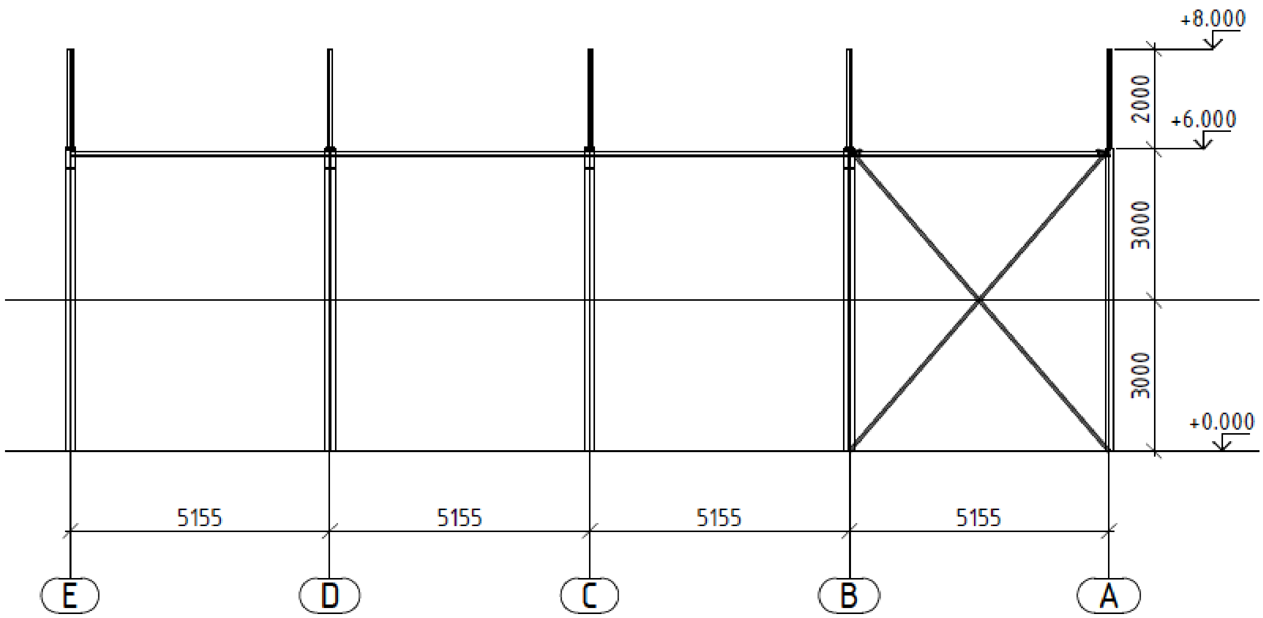
Weld design

$$a \geq \frac{t}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{6,3}{2} \frac{355}{1,0} \frac{0,8 \cdot 1,25}{\frac{355}{\sqrt{2}}} = 2,95 \text{ mm} \quad a = 3 \text{ mm}$$

$$F_{w,Rd} = \frac{f_u \cdot 4 \cdot a \cdot l}{\sqrt{3} \cdot 0,8 \cdot \gamma_{M0}} = \frac{360 \cdot 4 \cdot 3 \cdot 50}{\sqrt{3} \cdot 0,8 \cdot 1,25} = 124,7 \text{ kN} \gg V_{Ed} = 35,41 \text{ kN}$$

Longitudal profile satisfies.

7.4. 1 – 1 Axis Bracing



N_{Ed} = 62,86 kN

$$F_{1,Ed} = R_{Ed} \cdot \frac{c_{peD}}{0,8} = 35,41 \cdot \frac{0,7}{0,8} = 31 \text{ kN}$$

$F_{2,Ed} = F_1 \cdot \frac{c_{peE}}{c_{peD}} = 31 \cdot \frac{0,3}{0,7} = 13,3$ kN

Imperfections

Reduction factor:

$\alpha_h = \frac{2}{\sqrt{h}} = \frac{2}{\sqrt{6}} = 0,82 > \frac{2}{3} = 0,67$

$\alpha_m = \sqrt{0,5(1+\frac{1}{m})} = \sqrt{0,5(1+\frac{1}{5})} = 0,77$

where;

h . . . is the height of the structure in meters

m . . . is the number of columns in selected axis

$\phi = \phi_0 \cdot \alpha_h \cdot \alpha_m = \frac{2}{3} \cdot 0,82 \cdot \frac{1}{200} = 0,003162$

$H_\phi = \phi\sum N = 5 \cdot 62,82 \cdot 0,003162 = 0,99$ kN

$R_{H,Ed} = \frac{\sum F}{2} = \frac{14,74 + 13,22 + 14,89}{2} = 22,63$ kN

$R_{V,Ed} = \frac{(14,74 + 13,22 + 14,89) \cdot 6}{5,155} = 52,67$ kN

$L = \sqrt{6038^2 + 5155^2} = 7940$ mm

$D_{Ed} = \frac{5,155}{7,94} \cdot 22,63 = 34,72$ kN

CHS60x6,3

$A = 1069$ mm^2

$i = 19,2$ mm

$L_{cry} = \frac{L}{2} = \frac{7940}{2} = 3970$ mm

$L_{crz} = L_{cry} \cdot 0,9 = 3970 \cdot 0,9 = 3573$ mm

$\lambda = \frac{L_{cry}}{i} = \frac{3970}{19,2} = 206,77$ $\bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{206,77}{76,4} = 2,70$ $\chi = 0,12$ (a line)

where; $\lambda_1 = 93,9 \cdot \sqrt{235/f_y} = 93,9 \cdot \sqrt{235/355} = 76,4$

$N_{b,Rd} = \chi \cdot A \cdot f_{yd} = 0,12 \cdot 1069 \cdot 355 = 45,54$ kN $> D_{Ed} = 34,72$ kN

Bolts

M16 8.8

$F_{v,Rd}$ = 66 kN (from profile table)

n = $\frac{34,72}{66}$ = 0,53

Design: 4 bolts

Slenderness check

λ = 206,77 < 250 is suggested.

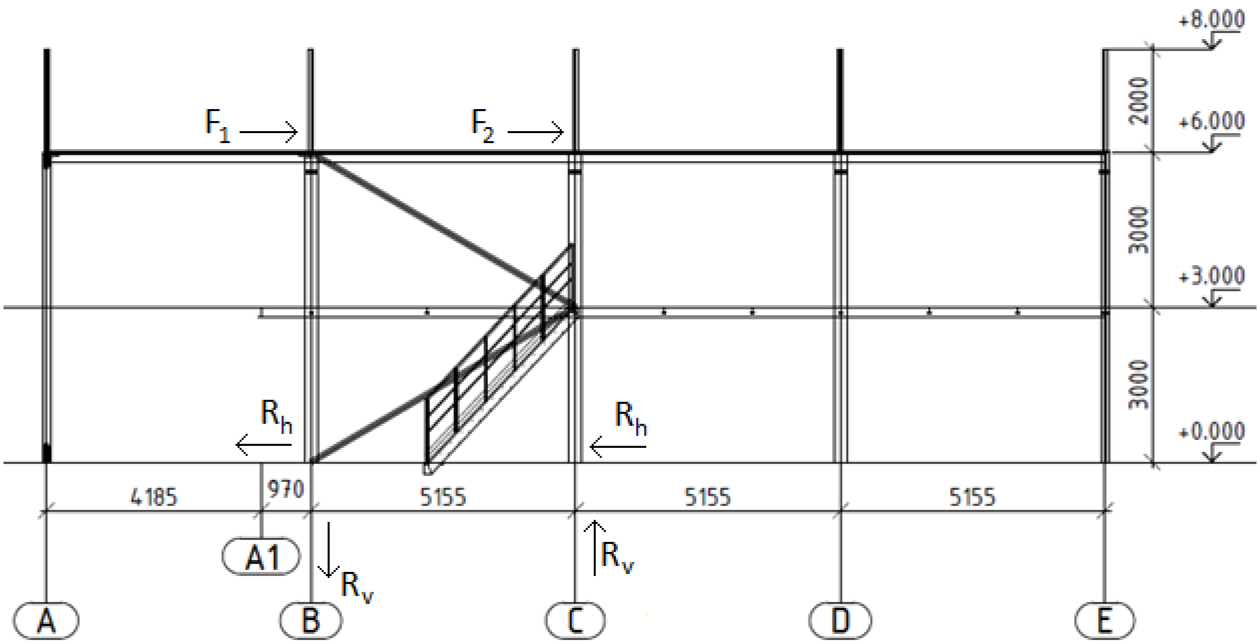
Weld design

$$a \geq \frac{t}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{6,3}{2} \frac{355}{1,0} \frac{0,8 \cdot 1,25}{\frac{355}{\sqrt{2}}} = 2,95 \text{ mm} \quad a = 3 \text{ mm}$$

$$F_{w,Rd} = \frac{f_u \cdot 4 \cdot a \cdot l}{\sqrt{3} \cdot 0,8 \cdot \gamma_{M0}} = \frac{360 \cdot 4 \cdot 3 \cdot 50}{\sqrt{3} \cdot 0,8 \cdot 1,25} = 124,7 \text{ kN} >> V_{Ed} = 34,72 \text{ kN}$$

Bracing satisfies.

7.5. 5 – 5 Axis Bracing and Shear Plate



N_{Ed} = 135 kN (Co 2.4)

$F_{1,Ed} = R_{Ed} \cdot \frac{c_{peD}}{0,8} = 35,41 \cdot \frac{0,7}{0,8} = 31$ kN

$F_{2,Ed} = F_1 \cdot \frac{c_{peE}}{c_{peD}} = 31 \cdot \frac{0,3}{0,7} = 13,3$ kN

Imperfections

Reduction factor:

$\alpha_h = \frac{2}{\sqrt{h}} = \frac{2}{\sqrt{3}} = 1{,}15$ $\alpha_h = 1$ (cannot be more than 1)

$\alpha_m = \sqrt{0{,}5(1+\frac{1}{m})} = \sqrt{0{,}5(1+\frac{1}{5})} = 0{,}77$

where;

h . . . is the height of the structure in meters

m . . . is the number of columns in selected axis

$\phi = \phi_0 \cdot \alpha_h \cdot \alpha_m = \frac{2}{3} \cdot 0{,}82 \cdot \frac{1}{200} = 0{,}003162$

$H_\phi = \phi \sum N = 5 \cdot 135 \cdot 0{,}003162 = 2{,}61$ kN

$R_{H,Ed} = \sum F = 14{,}74 + 13{,}22 + 14{,}89 = 46{,}87$ kN

$R_{V,Ed} = \frac{(14{,}74 + 13{,}22 + 14{,}89) \cdot 3}{5{,}155} = 27{,}28$ kN

$L = \sqrt{3000^2 + 5155^2} = 5960$ mm

$D_{Ed} = \frac{5{,}96}{5{,}155} \cdot 46{,}87 = 54{,}23$ kN

<u>CHS89x6,3</u>

$A = 1637$ mm^2

$i = 29{,}3$ mm

$L_{cry} = L = 5960$ mm

$L_{crz} = L_{cry} \cdot 0{,}9 = 5960 \cdot 0{,}9 = 5364$ mm

$\lambda = \frac{L_{cry}}{i} = \frac{5960}{29{,}3} = 203{,}56$ $\bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{203{,}56}{76{,}4} = 2{,}66$ $\chi = 0{,}12$ (a line)

where; $\lambda_1 = 93{,}9 \cdot \sqrt{235/f_y} = 93{,}9 \cdot \sqrt{235/355} = 76{,}4$

$N_{b,Rd} = \chi \cdot A \cdot f_{yd} = 0{,}12 \cdot 1637 \cdot 355 = 69{,}74 \text{ kN} > D_{Ed} = 54{,}23$ kN

Bolts

M16 8.8

$F_{v,Rd} = 66$ kN

$n = \frac{54{,}23}{66} = 0{,}82$

Design: 4 bolts

Slenderness check

λ = 203,56 < 250 is suggested.

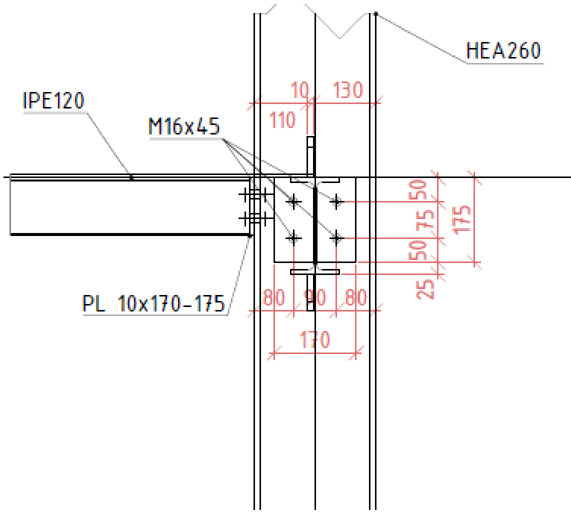
Weld design

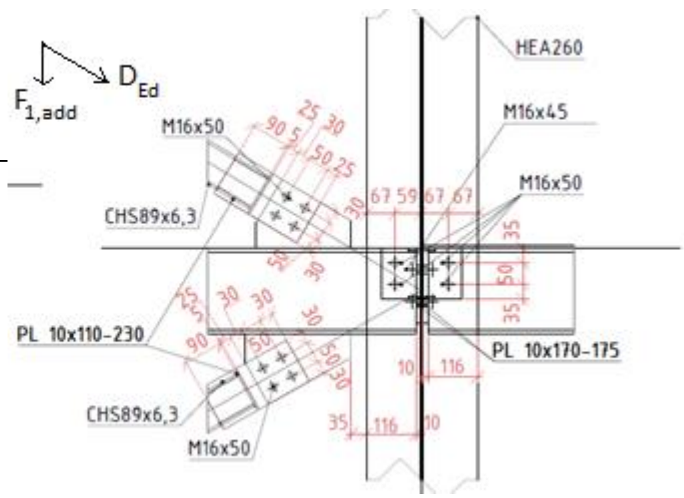
$$a \geq \frac{t}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{6,3}{2} \frac{355}{1,0} \frac{0,8 \cdot 1,25}{\frac{355}{\sqrt{2}}} = 2,95 \text{ mm} \quad a = 3 \text{ mm}$$

$$F_{w,Rd} = \frac{f_u \cdot 4 \cdot a \cdot l}{\sqrt{3} \cdot 0,8 \cdot \gamma_{M0}} = \frac{360 \cdot 4 \cdot 3 \cdot 90}{\sqrt{3} \cdot 0,8 \cdot 1,25} = 224,46 \text{ kN} >> V_{Ed} = 34,72 \text{ kN}$$

Selected profile of bracing satisfies.

C5 Connection





D_{Ed} = 54,23 kN

Arctan(3000/5155) = 30,19°

$F_{1,add}$ = 2 · 54,23 · sin(30) = 54,23 kN

M16 8.8 $F_{v,Rd}$ = 66 kN

For primary beam	For secondary beam
V_{Ed} = 41,02 kN	V_{Ed} = 50,22 kN
M_{Ed} = 0	M_{Ed} = 0

$V_{Edp} = R_{Edp}$ = 39,77 + 54,23 = 94 kN

$V_{Eds} = R_{Edp}$ = 53,07 kN

$n_p = \frac{94}{66} = 1,42 \quad n_s = \frac{53,07}{66} = 0,80$

Design: 4 bolts

L_{we} = 90 mm for secondary beam 2x3 mm

L_{we} = 150 mm for secondary beam 2x3 mm

Web weld

$$a_p = \frac{t_w}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{5{,}9}{2} \frac{355}{1{,}0} \frac{0{,}8 \cdot 1{,}25}{\frac{355}{\sqrt{2}}} = 2{,}76 \text{ mm} \qquad a_p = 3 \text{ mm}$$

$$a_s = \frac{t_w}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{4{,}4}{2} \frac{355}{1{,}0} \frac{0{,}8 \cdot 1{,}25}{\frac{355}{\sqrt{2}}} = 2{,}06 \text{ mm} \qquad a_s = 3 \text{ mm}$$

$$f_{vw,d} = \frac{f_u}{\sqrt{3}\ \beta_w\ \gamma_{M2}} = \frac{490}{\sqrt{3} \cdot 0{,}9 \cdot 1{,}25} = 251{,}5 \text{ MPa}$$

Weld design

Fillet weld 2 x a_p = 6 mm L_{wep} = 150 mm (for primary beam)

Fillet weld 2 x a_s = 6 mm L_{wes} = 90 mm (for secondary beam)

$F_{w,Rdp} = 2 \cdot a_p \cdot L_{we} \cdot f_{vw,d} = 2 \cdot 3 \cdot 150 \cdot 251{,}5 = 226{,}35 \text{ kN} > R_{Edp} = 56{,}77 \text{ kN}$

$F_{w,Rds} = 2 \cdot a_s \cdot L_{we} \cdot f_{vw,d} = 2 \cdot 3 \cdot 90 \cdot 251{,}5 = 135{,}81 \text{ kN} > R_{Eds} = 53{,}07 \text{ kN}$

Shear strength check of weakened cross section of the primary beam

$A_{vz} = 5 \cdot 150 = 750 \text{ mm}^2$

$V_{pl,Rd} = A_{vz} \cdot f_{yd} / \sqrt{3} = 750 \cdot 355 / \sqrt{3} = 153{,}72 \text{ kN} > R_{Edp} = 56{,}77 \text{ kN}$

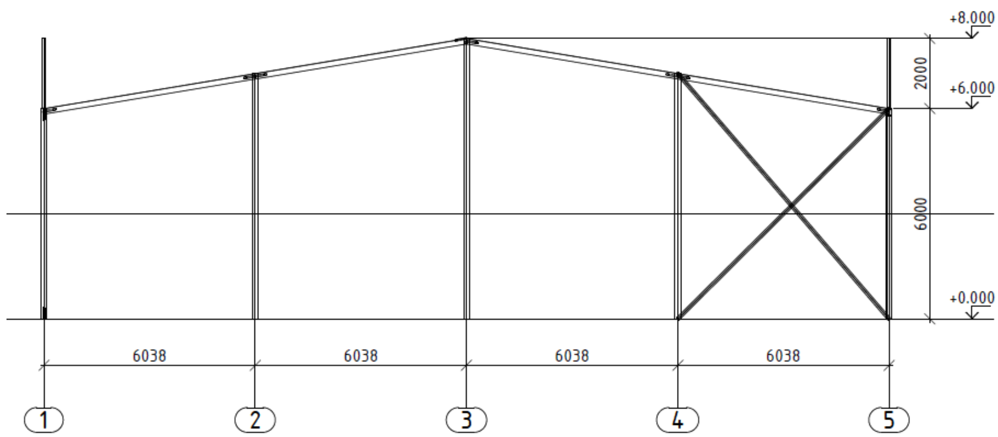
Shear strength check of weakened cross section of the secondary beam

$A_{vz} = 4 \cdot 90 = 360 \text{ mm}^2$

$V_{pl,Rd} = A_{vz} \cdot f_{yd} / \sqrt{3} = 360 \cdot 355 / \sqrt{3} = 73{,}79 \text{ kN} > R_{Edp} = 53{,}07 \text{ kN}$

Suggested connection satisfies.

7.6. A – A Axis Bracing



D_{Ed} = 11,53 kN (Co 2.6)

CHS60,3x5

A = 869 mm^2

i = 19,6 mm

$L_{cry} = \frac{L}{2} = \frac{9,24}{2}$ = 4620 mm

$L_{crz} = L_{cry} \cdot 0,9 = 4620 \cdot 0,9 = 4160$ mm

$\lambda = \frac{L_{cry}}{i} = \frac{4620}{19,6} = 235,82$ $\bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{235,82}{76,4} = 3,09$ χ = 0,08 (a line)

where; $\lambda_1 = 93,9 \cdot \sqrt{235/f_y} = 93,9 \cdot \sqrt{235/355} = 76,4$

$N_{b,Rd} = \chi \cdot A \cdot f_{yd} = 0,08 \cdot 869 \cdot 355 = 24,68$ kN > D_{Ed} = 11,53 kN

Bolts

M16 8.8

$F_{v,Rd}$ = 66 kN

$n = \frac{10,78}{66} = 0,16$

Design: 4 bolts,

Slenderness check

λ = 235,82 < 400 is suggested.

Weld design

$$a \geq \frac{t}{2} \cdot \left(\frac{f_y}{\gamma_{m0}}\right) \cdot \left(\frac{\beta_w \cdot \gamma_{m2}}{f_u / \sqrt{2}}\right) = \frac{5}{2} \frac{355}{1,0} \frac{0,8 \cdot 1,25}{\frac{355}{\sqrt{2}}} = 2,34 \text{ mm} \qquad a = 3 \text{ mm}$$

$$F_{w,Rd} = \frac{f_u \cdot 4 \cdot a \cdot l}{\sqrt{3} \cdot 0,8 \cdot y_{M0}} = \frac{360 \cdot 4 \cdot 3 \cdot 50}{\sqrt{3} \cdot 0,8 \cdot 1,25} = 124,7 \text{ kN} >> V_{Ed} = 11,53 \text{ kN}$$

Selected profile of bracing satisfies.