CONCEPTUAL DESIGN OF BUILDINGS

Project
Conception, analysis and design of a 3D steel building

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Sustainable Constructions
under Natural Hazards and Catastrophic Events
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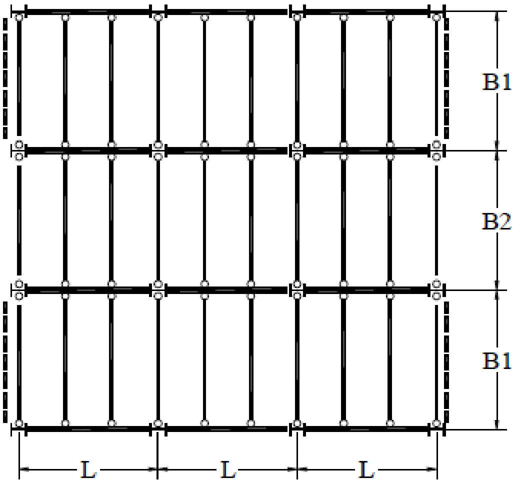
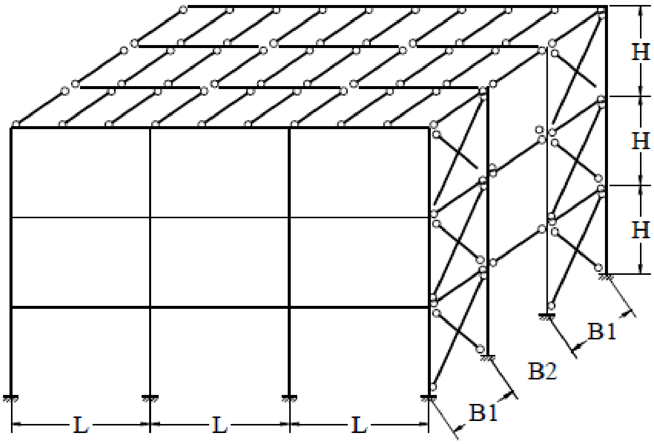
1. Introduction

The building analyzed in this project is based in steel-framed structure and has got following characterisitics:

1) Type of use – residential building;
2) Location – Guarda, Portugal;
3) Span – L=6m;
4) Bay – B1=6m, B2=6m;
5) Number of floors – 3;
6) Floor height – 4 m.

Scheme of the building is given on the following visualization:





2. General safety criteria, actions and combinations of actions

The quantification of the actions and their combinations was made according to EN 1990, EN 1991-1-1, 1991-1-3, considering the permanent actions that correspond to the self-weight of the structure and non-structural members, the variable actions corresponding to imposed loads, snow and wind loads.

2.1 Loads evaluation.

1. Permanent actions:

According to EN 1991-1-1 permanent actions include the self-weight of the structural elements and non-structural elements.
Self-weight of structural elements includes the weight of steel structure (weight is obtained during the calculations in software ROBOT).
Self-weight of non-structural elements includes following positions:

a) Roof slab:

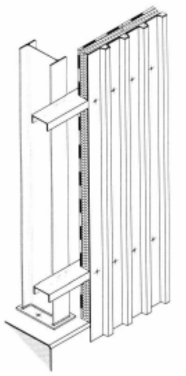
Layer	Thickness, mm	Specific weight, kN/m3	Weight, kN/m2
Hydro insulation			0.003
Thermal insulation	80	1.4	0.112
Slope concrete	50	24	1.2
Vapors foil	0.3	0.2	0.00006
RC slab	115	25	2.87
Steel sheeting	1	78	0.078
Gypsum plaster board	12	15	0.18
Total			**4.48**

b) Current floor

Layer	Thickness, mm	Specific weight, kN/m3	Weight, kN/m2
Sandstone	12	20	0.24
Adhesive support	5	21	0.105
Flooring	20	22	0.44
Vapors foil	0.3	0.2	0.00006
RC slab	115	25	2.875
Steel sheeting	1	78	0.078
Gypsum plaster board	12	15	0.18
Total			**3.70**

c) Partition walls (lightweight walls) = 1.0 kN/m2 (of slab). This weight is represented by uniformly distributed load on the floor slabs.

d) External walls (lightweight walls) = 1.0 kN/m2 (of wall – linear load). We assume that secondary beams (for external lightweight walls) will be installed in horizontal position. The way of installation is shown on the visualization below:



Vis.1. Installation of secondary beams for external walls.

Therefore, the load from external walls will be applied to columns. In order to apply this load in ROBOT we will use claddings working in one-way (horizontal) direction.

2. Variable actions.

   a) Imposed loads.

For residential building according to table 6.1 of EN 1991-1-1, category of use is A (areas for domestic and residential activities). From this this table we take underlined recommended characteristic values of imposed loads.

Loaded area	q_k, kN/m^2	Q_k, kN
Floors	2.0	2.0

In the project we will use only the **imposed load on floor** (q_k=2.0 kN/m^2) which is intended for determination of global effects. We neglect the loads given for representing of stairs and balconies as long as they are not considered for calculation in our project.

According to table 6.9 of EN 1991-1-1 roof is categorized category H (roofs not accessible except for normal maintenance and repair). Therefore, according to recommended values in table 6.10, **imposed load on roof** is q_k=0.4 kN/m^2.

   b) Wind loads.

Surface horizontal load = 1.5 kN/m2 in one façade and -0.6 kN/m2 in the opposite façade. Wind load will be divided into two orthogonal loads which can not act simultaneously. First wind load will act with angle $\propto = 0°$, second wind load will act with angle $\propto = 90°$. Value of the loads will be taken identical in both directions.

   c) Snow loads.

According to EN 1991-1-3 for Iberian Peninsula (Guarda, Portugal, 1056 m a.s.l.) region and stating that in this region exceptional snow falls and exceptional snow drifts are unlikely to occur we obtain:

$$\mathbf{S = \mu_i C_e C_t s_k = 0.8 \cdot 1.0 \cdot 1.0 \cdot 1.44 = 1.15\ \frac{kN}{m^2}},$$

where $\mu_i = 0.8$ (since $0° \leq \propto \leq 30°$); $C_e = 1.0$; $C_t = 1.0$;

and

$$s_k = (0.190Z - 0.095)\left[1 + \left(\frac{A}{524}\right)^2\right] = (0.190 \cdot 2 - 0.095)\left[1 + \left(\frac{1056}{524}\right)^2\right] = 1.44\ \frac{kN}{m^2},$$

where $Z = 2,\ A = 1056$ m (for Guarda, Portugal)

3. Summary of basic actions.

The resulting actions are summarized in following table:

Action No	Description	ype	Value
LC1	Self-weight of steel structure	Permanent action	Obtained in ROBOT
	Weight of floor slab	Permanent action	3.70 kN/m2
	Weight of roof slab	Permanent action	4.48 kN/m2
	Weight of partition walls	Permanent action	1.0 kN/m2
	Weight of external walls	Permanent action	1.0 kN/m2
LC2	Imposed load on floor	Variable action	2.0 kN/m2
LC3	Imposed load on roof	Variable action	0.4 kN/m2
LC4	Wind load 1 ($\propto = 0°$)	Variable action	1.5 kN/m2 – one façade; -0.6 kN/m2 – opposite façade
LC5	Wind load 2 ($\propto = 90°$)	Variable action	1.5 kN/m2 – one façade; -0.6 kN/m2 – opposite façade
LC6	Snow load	Variable action	1.15 kN/m2

2.2 Load combinations.

Ultimate limit state.

The design values of the applied forces are obtained from the fundamental combinations, given by (EN 1990):

$$E_d = \gamma_{G,j} \cdot G_{k,j} + \gamma_g \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i=2}^{n} \gamma_{Q,i} \cdot \Psi_{0,i} \cdot Q_{k,i}$$

Reduction coefficients for variable actions.

Action	ψ_0
Imposed loads (category A)	0.7
Snow loads (sites located at altitude H>1000 m a.s.l)	0.7
Wind load	0.6

According to EN 1990 Table A1.2(A) we consider verification of static equilibrium which involves the resistance of structural members. Permanent actions are considered to be unfavorable part. Following coefficients apply to these considerations: $\gamma_{G,j} = 1.35; \gamma_{Q,1} = \gamma_{Q,i} = 1.5$.

The following combinations are considered for the Ultimate Limit State:

Combination 1-Imposed load as a leading variable action; wind load - $\propto = 0°$.

$E_{d1} = 1.35 \cdot G + 1.5 \cdot Q_{imposed} + 1.5 \cdot 0.7 \cdot Q_{snow} + 1.5 \cdot 0.6 \cdot Q_{wind,0}$

Combination 2-Imposed load as a leading variable action; wind load - $\propto = 90°$.

$E_{d1} = 1.35 \cdot G + 1.5 \cdot Q_{imposed} + 1.5 \cdot 0.7 \cdot Q_{snow} + 1.5 \cdot 0.6 \cdot Q_{wind,90}$

Combination 3-Snow load as a leading variable action; wind load - $\propto = 0°$.

$E_{d2} = 1.35 \cdot G + 1.5 \cdot Q_{snow} + 1.5 \cdot 0.7 \cdot Q_{imposed} + 1.5 \cdot 0.6 \cdot Q_{wind,0}$

Combination 4-Snow load as a leading variable action; wind load - $\propto = 90°$.

$E_{d2} = 1.35 \cdot G + 1.5 \cdot Q_{snow} + 1.5 \cdot 0.7 \cdot Q_{imposed} + 1.5 \cdot 0.6 \cdot Q_{wind,90}$

Combination 5-Wind load as a leading variable action; $\propto = 0°$.

$E_{d3} = 1.35 \cdot G + 1.5 \cdot Q_{wind,0} + 1.5 \cdot 0.7 \cdot Q_{imposed} + 1.5 \cdot 0.7 \cdot Q_{snow}$

Combination 6-Wind load as a leading variable action; $\propto = 90°$.

$E_{d3} = 1.35 \cdot G + 1.5 \cdot Q_{wind,90} + 1.5 \cdot 0.7 \cdot Q_{imposed} + 1.5 \cdot 0.7 \cdot Q_{snow}$

Serviceability limit state.

For serviceability limit state we apply characteristic combinations (irreversible limit states) (EN 1990):

$$E_d = \sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} \cdot Q_{k,i}$$

Combination 7-Imposed load as a leading variable action; wind load - $\propto = 0°$.

$E_{d4} = G + Q_{imposed} + 0.7 \cdot Q_{snow} + 0.6 \cdot Q_{wind,0}$

Combination 8-Imposed load as a leading variable action; wind load - $\propto = 90°$.

$E_{d4} = G + Q_{imposed} + 0.7 \cdot Q_{snow} + 0.6 \cdot Q_{wind,90}$

Combination 9-Snow load as a leading variable action; wind load - $\propto = 0°$.

$E_{d5} = G + Q_{snow} + 0.7 \cdot Q_{imposed} + 0.6 \cdot Q_{wind,0}$

Combination 10-Snow load as a leading variable action; wind load - $\propto = 90°$.

$E_{d5} = G + Q_{snow} + 0.7 \cdot Q_{imposed} + 0.6 \cdot Q_{wind,90}$

Combination 11-Wind load as a leading variable action; wind load - $\propto = 0°$.

$$E_{d6} = G + Q_{wind,0} + 0.7 \cdot Q_{imposed} + 0.7 \cdot Q_{snow}$$

Combination 12-Wind load as a leading variable action; wind load - $\propto = 90°$.

$$E_{d6} = G + Q_{wind,90} + 0.7 \cdot Q_{imposed} + 0.7 \cdot Q_{snow}$$

where G=LC1; $Q_{imposed}$=LC2+LC3; $Q_{wind,0}$=LC4; $Q_{wind,90}$=LC5; Q_{snow}=LC6.

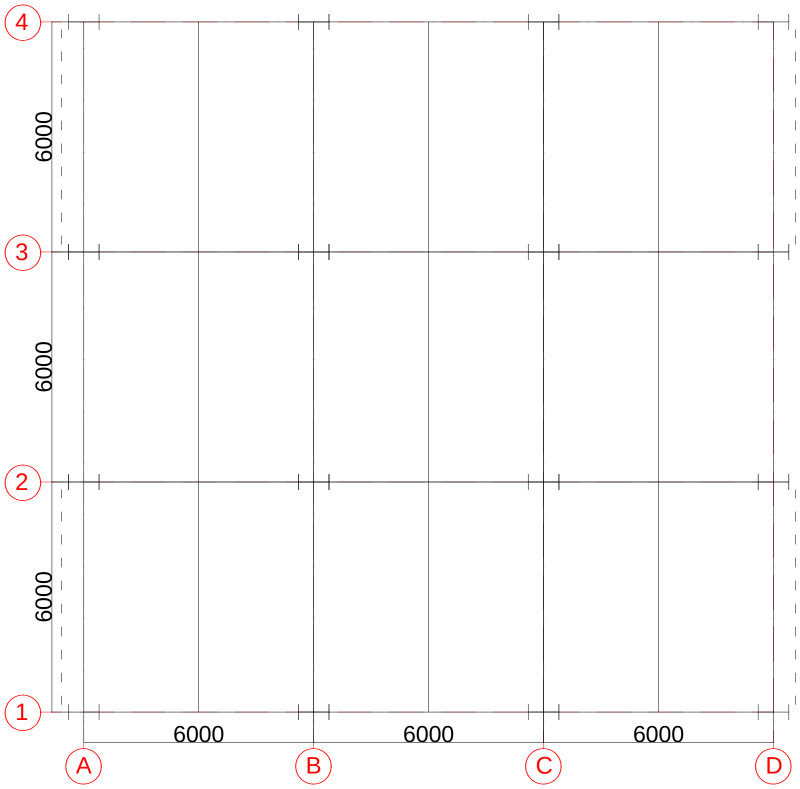
Substituting G, $Q_{imposed}$, $Q_{wind,0}$, $Q_{wind,90}$, Q_{snow} with appropriate actions according to our project we obtain:

Combination 7-Imposed load as a leading variable action; wind load - $\propto = 0°$.
$E_{d4} = LC1 + (LC2 + LC3) + 0.7 \cdot LC6 + 0.6 \cdot LC4$
Combination 8-Imposed load as a leading variable action; wind load - $\propto = 90°$.
$E_{d4} = LC1 + (LC2 + LC3) + 0.7 \cdot LC6 + 0.6 \cdot LC5$
Combination 9-Snow load as a leading variable action; wind load - $\propto = 0°$.
$E_{d5} = LC1 + LC6 + 0.7 \cdot (LC2 + LC3) + 0.6 \cdot LC4$
Combination 10-Snow load as a leading variable action; wind load - $\propto = 90°$.
$E_{d5} = LC1 + LC6 + 0.7 \cdot (LC2 + LC3) + 0.6 \cdot LC5$
Combination 11-Wind load as a leading variable action; wind load - $\propto = 0°$.

$$E_{d6} = LC1 + LC4 + 0.7 \cdot (LC2 + LC3) + 0.7 \cdot LC6$$

Combination 12-Wind load as a leading variable action; wind load - $\propto = 90°$.
$E_{d6} = LC1 + LC5 + 0.7 \cdot (LC2 + LC3) + 0.7 \cdot LC6$

3. **Pre-design**
3.1 Preliminary evaluation of cross sections.



In order to obtain acting internal forces in the ROBOT model we assumed following cross-sections of members:

Type of columns	Grid	Cross-section	Steel grade
Perimeter	A1, A4, D1, D4, A2, A3, D2, D3, B1, B4, C1, C4	HEA 300	S355
Center	B2, B3, C2, C3	HEA 400	S 355

Tab.100. Initial assumed geometric characteristics of columns.

Type of beams	Beams	Cross-section	Steel grade
Main beams	A1-D1, A2-D2, A3-D3, A4-D4	IPE 300	S 355
Secondary beams	A1-A4, B1-B4, C1-C4, D1-D4	IPE 200	S 355

Tab.100. Geometric characteristics of beams on all floors.

Bracing	Cross-section	Steel grade
A1-A2, D1-D2, A3-A4, D3-D4	CHS 168x8	S 355

Tab.100. Geometric characteristics of bracings.

After obtaining internal forces we perform preliminary design of members considering the biggest internal forces in members of a type:

<u>Beams</u>

1. Secondary beams.
   Assuming class 1 or 2 cross sections, the following solution is obtained:
   $M_{ed} = 127.99\ kNm \leq W_{pl,y} \cdot 355 \cdot 10^3/1.0$

$W_{pl,y} \geq 359.13\ cm^3$

In order to satisfy this condition IPE 240 section ($W_{pl,y} = 367\ cm^3$) is selected.

2. Main beams.

   Assuming class 1 or 2 cross sections, the following solution is obtained:

   $M_{ed} = 241.12\ kNm \leq W_{pl,y} \cdot 355 \cdot 10^3/1.0$

   $W_{pl,y} \geq 679.2\ cm^3$

   In order to satisfy this condition IPE 360 section ($W_{pl,y} = 1019\ cm^3$) is selected. IPE 330 ($W_{pl,y} = 804 cm^3$) does not meet the requirements as long as the moment in cross section increases after changing the cross section in ROBOT model to M_{ed}=318 kNm.

Columns

1. Central columns.

   Assuming class 1, 2 or 3 cross sections, the following solution is obtained:

   $$N_{ed} = 1030\ kN \leq N_{C,Rd} = 0.4\frac{Af_y}{\gamma_{m0}} = 0.4 \cdot A \cdot 355 \cdot 10^3/1.0$$

   $A \geq 72.5\ cm^2$

   As it is expected that buckling resistance will govern the member design, a member HEA 320 (A=124.4 cm^2)

2. Perimeter columns.

   Assuming class 1, 2 or 3 cross sections, the following solution is obtained:

   $$N_{ed} = 732\ kN \leq N_{C,Rd} = 0.4\frac{Af_y}{\gamma_{m0}} = A \cdot 0.4 \cdot 355 \cdot 10^3/1.0$$

   $A \geq 51.5\ cm^2$

   As it is expected that buckling resistance will govern the member design, a member HEA 220 (A=64.3 cm^2)

Bracing

Assuming class 1, 2 or 3 cross sections, the following solution is obtained:

$$N_{ed} = 216\ kN \leq N_{C,Rd} = \frac{Af_y}{\gamma_{m0}} = A \cdot 355 \cdot 10^3/1.0$$

$A \geq 6.1\ cm^2$

In order to satisfy this condition TRON 88x2.5 section ($A = 6.79\ cm^2$) is selected.

Pre-design sections are summarized in following table:

Type of c□lumns	Grid	Cross-section	Steel grade
Marginal	A1, A4, D1, D4, A2, A3, D2, D3, B1, B4, C1, C4	HEA 220	S355
Center	B2, B3, C2, C3	HEA 320	S 355

Tab.100. Geometric characteristics of pre-designed columns.

Type of beams	Beams	Cross-section	Steel grade
Main beams	A1-D1, A2-D2, A3-D3, A4-D4	IPE 360	S 355
Secondary beams	A1-A4, B1-B4, C1-C4, D1-D4	IPE 240	S 355

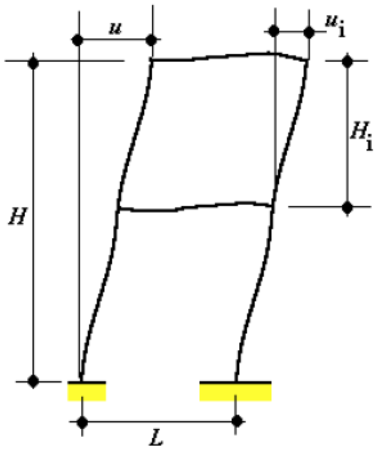
Tab.100. Geometric characteristics of pre-design beams sections.

Bracing	Cross-section	Steel grade
A1-A2, D1-D2, A3-A4, D3-D4	TRON 88x2.5	S 355

Tab.100. Geometric characteristics of pre-design bracings sections.

3.2 Consideration of horizontal and vertical deformations.

In this chapter the verification of horizontal deformations in the building will be made. Limiting values for horizontal displacements in frames:



$$u_i \leq H_i/300$$
$$u \leq H/500$$

This verification is made for serviceability limit states.

H=12 m=1200 cm => H/500=2.4 cm

H_i=4 m=400 cm => H_i/300=1.33 cm

Vertical deformations in beams.

Limiting value for main and secondary beams:

L/250=600/250=2.4 cm

The structure with the members which were chosen in pre-design stage do not satisfy the requirements of horizontal and vertical deformations.

Therefore, we change the cross section of members to satisfy the deformation requirements.

		Limit values
Secondary beam	IPE 360	
Main beam	IPE 450	
Central column	HEA 450	
Perimeter column	HEA 300	
Bracing	TRON 139x8	
Horizontal deflection (Δ)	2	2.4
Horizontal deflection first floor (δ1)	1.3	1.33
Horizontal deflection first floor (δ2)	0.5	1.33
Horizontal deflection first floor (δ2)	0.2	1.33
Vertical deflection main beam	2	2.4
Vertical deflection secondary beam	2.3	2.4

4. Structural analysis

The structural model for the analysis was created in software ROBOT. Following input data is used for model consideration:

1. Beams in plane xz are rigidly connected to the steel columns.
2. The beams in plane yz are hinged at both ends. Releases for hinged connections are indicated in following directions: Ry, Rz.
3. Elements defining bracing system are also hinged at both ends.
4. Supports are pinned. Fixed directions of pinned support: Ux, Uy, Uz, Rz.
5. Bracings in axis A1-A2, D1-D2, A3-A4, D3-D4 are represented by one bar per frame assuming that it will work in tension and compression.
6. The concrete slab has a strong influence on the global stiffness of the structure. In ROBOT 3D model concrete slab was modeled by a horizontal bracing system, connected to main columns. Connection of these bracings are hinged.

To identify the type of analysis which should be performed (1st or 2nd order) we calculate α_{cr} for ultimate limit state combinations.

	α_{cr} (mode 1)
Combination 1	10.95
Combination 2	10.95
Combination 3	11.46
Combination 4	11.46
Combination 5	11.66
Combination 6	11.66

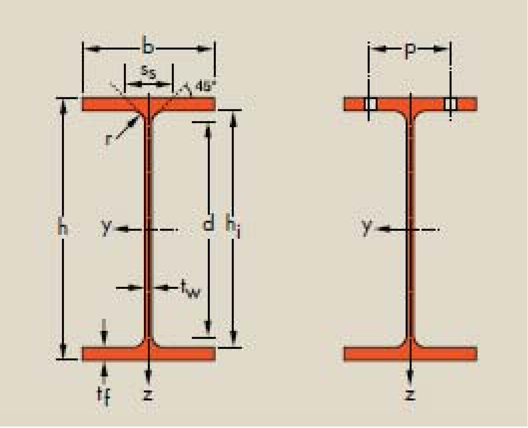
In all combinations $\alpha_{cr}>10$. Therefore, according to EN 1993-1-1 1st order elastic analysis should be performed.

5. Checking of elements

5.1 Verification of beams

5.1.1 Secondary beams (IPE 360)

Cross section characteristics



$h := 360mm$

$b := 170mm$

$t_w := 8mm$

$t_f := 12.7mm$

$r := 18mm$

$A_1 := 72.7cm^2$

$h_i := 334.6mm$

$L_{beam} := 6m$

$d := 298.6mm$

$I_y := 16270cm^4$

$W_{el.y} := 904cm^3$

$W_{pl.y} := 1019cm^3$

$i_y := 15cm$

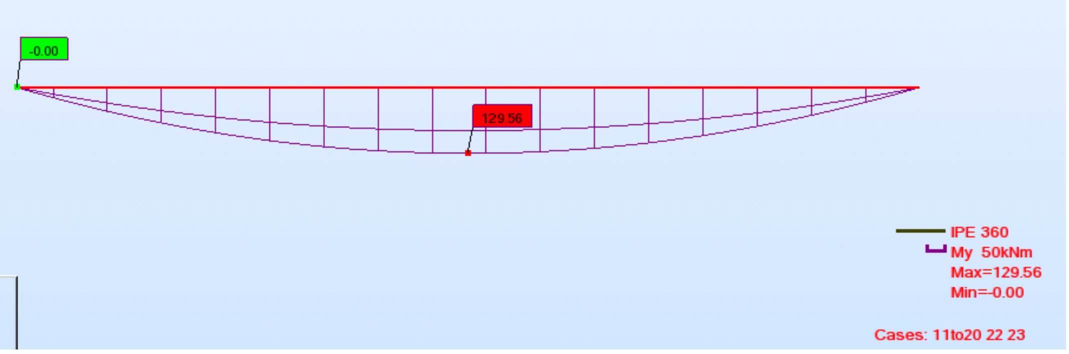
$A_{vz} := 35.1cm^2$

$f_{yd} := 355MPa$

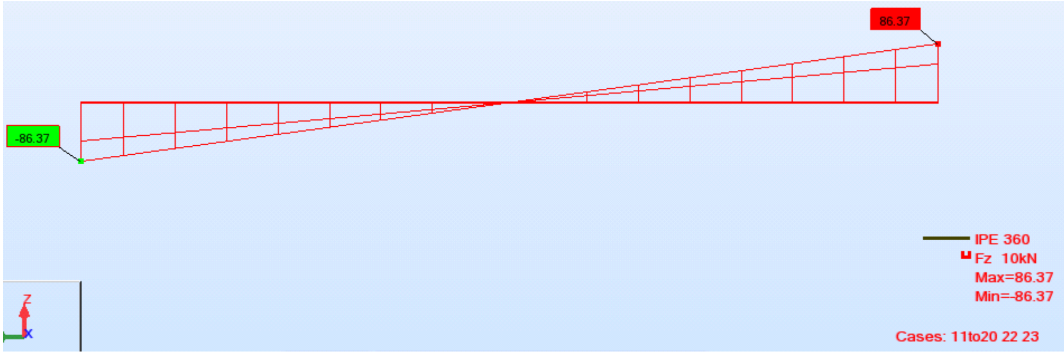
$\gamma_{M0} := 1.0$

Internal forces

$M_{Ed} := 129.56kN \cdot m$



$V_{Ed} := 86.37kN$



Fy and Fz are low so we can neglect them.

Cross section classification

As long as steel class of the beam is S355 we

$$\varepsilon := \sqrt{\frac{235\frac{N}{mm^2}}{f_{yd}}} = 0.814$$

Web in bending:

$$c_w := d$$

$$\frac{c_w}{t_w} = 37.325 \quad < \quad 72\varepsilon = 58.58$$

Web is class 1

Flanges in compression:

$$c_f := \frac{b - t_w - 2r}{2} = 63 \cdot mm$$

$$\frac{c_f}{t_f} = 4.961 \quad < \quad 9\varepsilon = 7.323$$

Flanges are class 1

The class of cross section is **class 1**

Resistance of cross section

1. Bending moment

For class 1 the design resistance for bending according to EC3-1-1 chapter 6.2.5:

$$M_{Rd} := \frac{W_{pl.y} \cdot f_{yd}}{\gamma_{M0}} = 361.745 \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.358 \quad < \quad 1$$

2. Shear resistance according to EC3-1-1 chapter 6.2.6:

Design plastic shear resistance:

$$V_{Rd} := \frac{\left(A_{vz} \cdot f_{yd}\right)}{\gamma_{M0} \cdot \sqrt{3}} = 719.407 \cdot kN$$

$$\frac{V_{Ed}}{V_{Rd}} = 0.12 < 1$$

Shear buckling resistance classification:

$\eta := 1$ (conservatively taken)

$$\frac{h}{t_w} = 45 < \frac{72\varepsilon}{\eta} = 58.58$$

Therefore, shear buckling resistance of the web does not have to be verified

3. Bending and shear force according to EC3-1-1 chapter 6.2.8:

$$V_{Ed} = 86.37 \cdot kN < 0.5V_{Rd} = 359.704 \cdot kN$$

Therefore, effect of shear force on te moment resistance can be neglected

Lateral torsional buckling

Secondary beam is not susceptible to lateral-torsional buckling as long as it is laterally restrained with reinforced concrete slabs on the floor and roof. The slab prevents lateral displacements of the compres parts of the cross section.

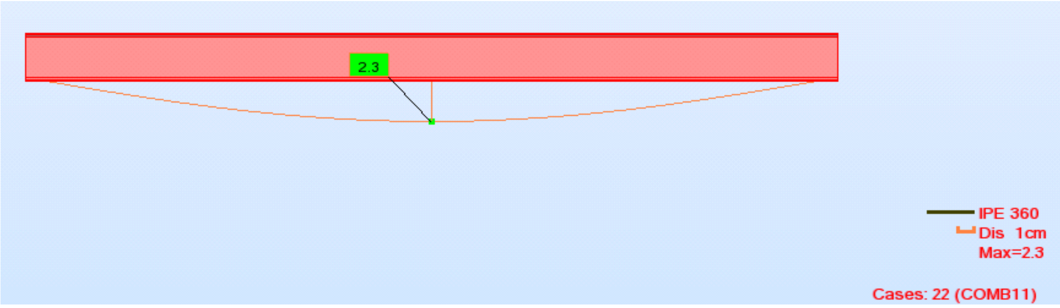
Verification of serviceability limit state

The verification of the maximum vertical deflection is performed using deformations from ROBOTsof for serviceability limit states. For floors general limiting value for the vertical displacement according 1990 Annex A1.4, National annex, figure A1.1 is following:

$$\delta_{max} < \frac{L_{beam}}{250}$$

For IPE 360:

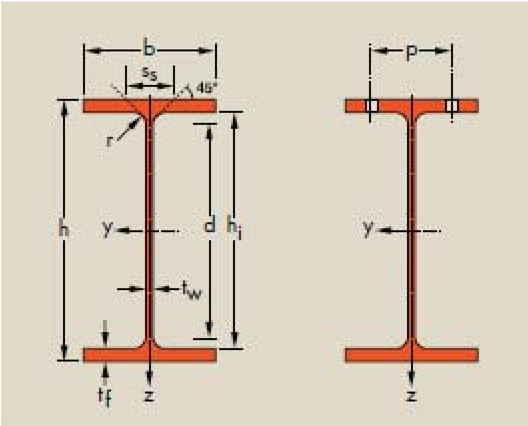
$$\delta_{max} := 2.3cm < \frac{L_{beam}}{250} = 2.4 \cdot cm$$



Cross section size is governed by deformation requirements. As long as vertical deformation values are 2.3<2.4, the cross section satisfies the requirement and can not be reduced more.

Cross section IPE 360 verifies ULS and SLS requirements

5.1.2 Main beam (IPE 450)



$h := 450mm$

$b := 190mm$

$t_w := 9.4mm$

$t_f := 14.6mm$

$r := 21mm$

$A_1 := 98.8cm^2$

$h_i := 420.8mm$

$L_{beam} := 6m$

$\gamma_{M0} := 1.0$

$d := 378.8mm$

$I_y := 33740cm^4$

$W_{el.y} := 1500cm^3$

$W_{pl.y} := 1702cm^3$

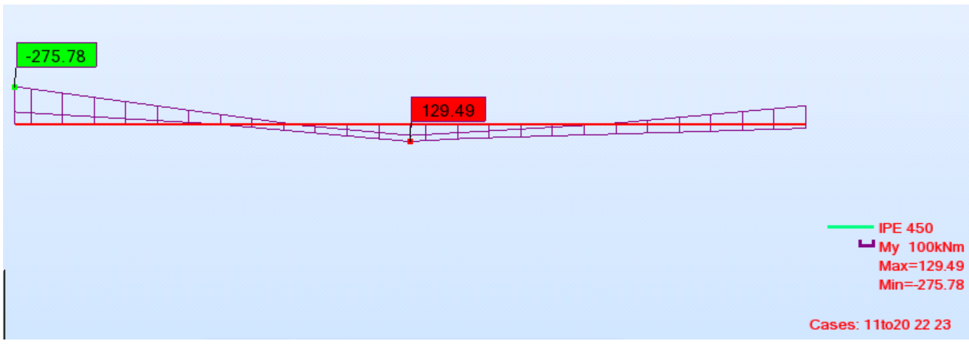
$i_y := 18.5cm$

$A_{vz} := 50.9cm^2$

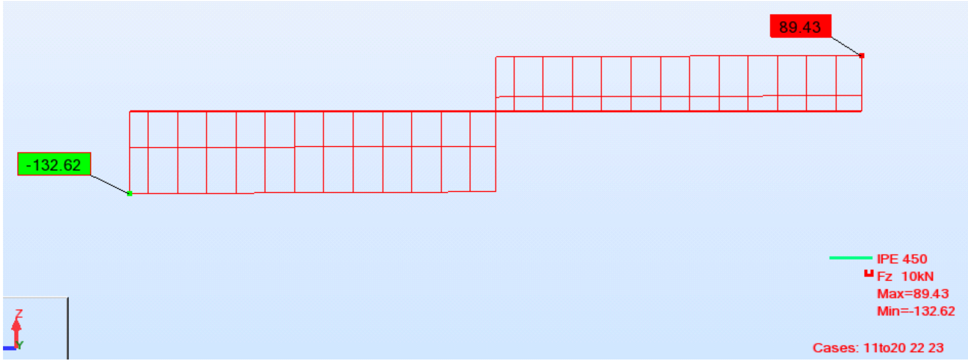
$f_{yd} := 355MPa$

Internal forces

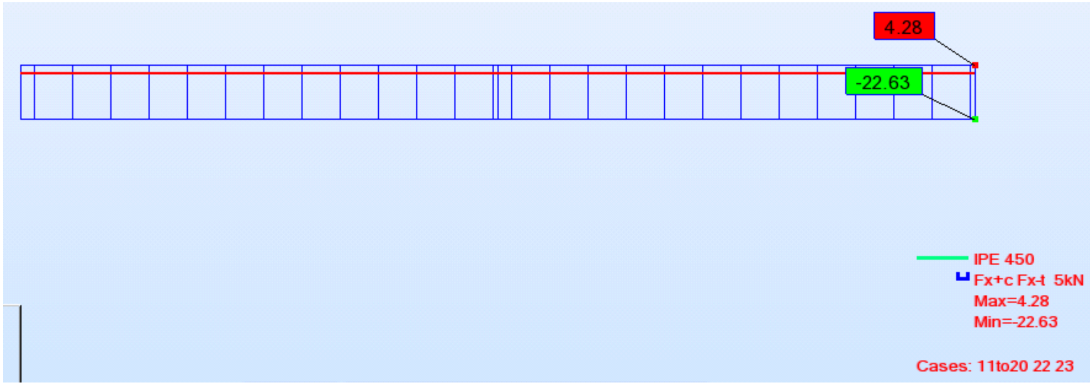
$M_{Ed} := 275.78kN\cdot m$



$V_{Ed} := 132.62kN$



$N_{Ed} := 22.63kN$



Value of axial force is low so we can neglect it.

Cross section classification

As long as steel class of the beam is S355 we obtain:

$$\varepsilon := \sqrt{\frac{235\frac{N}{mm^2}}{f_{yd}}} = 0.814$$

<u>Web in bending</u>:

$$c_w := d$$

$$\frac{c_w}{t_w} = 40.298 < 72\varepsilon = 58.58$$

Web is class 1

<u>Flanges in compression:</u>

$$c_f := \frac{b - t_w - 2r}{2} = 69.3 \cdot mm$$

$$\frac{c_f}{t_f} = 4.747 < 9\varepsilon = 7.323$$

Flanges are class 1

The class of cross section is **class 1**

Resistance of cross section

1. Bending moment

For class 1 the design resistance for bending according to EC3-1-1 chapter 6.2.5:

$$M_{Rd} := \frac{W_{pl.y} \cdot f_{yd}}{\gamma_{M0}} = 604.21 \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.456 \; < \; 1$$

2. Shear resistance according to EC3-1-1 chapter 6.2.6:

Design plastic shear resistance:

$$V_{Rd} := \frac{\left(A_{vz} \cdot f_{yd}\right)}{\gamma_{M0} \cdot \sqrt{3}} = 1.043 \times 10^{3} \cdot kN$$

$$\frac{V_{Ed}}{V_{Rd}} = 0.127 \; < \; 1$$

Shear buckling resistance classification:

$\eta := 1$ (conservatively taken)

$$\frac{h}{t_w} = 47.872 \; < \; \frac{72\varepsilon}{\eta} = 58.58$$

Therefore, shear buckling resistance of the web does not have to be verified

3. Bending and shear force according to EC3-1-1 chapter 6.2.8:

$$V_{Ed} = 132.62 \cdot kN \; < \; 0.5V_{Rd} = 521.622 \cdot kN$$

Therefore, effect of shear force on te moment resistance can be neglected

Laterral torsional buckling

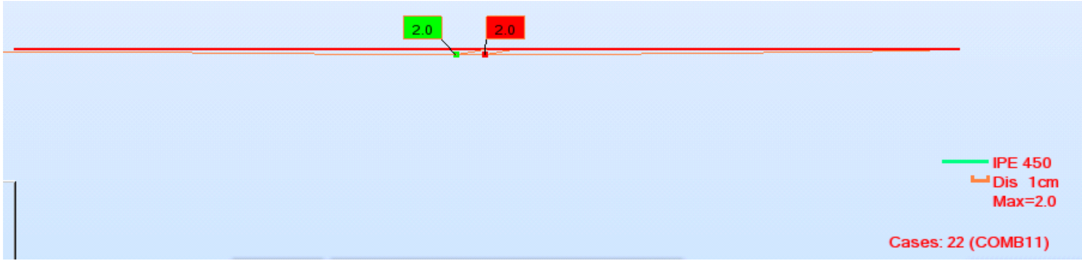
Secondary beam is not susceptible to lateral-torsional buckling as long as it is laterally restrained with reinforced concrete slabs on the floor and roof. The slab prevents lateral displacements of the compressed parts of the cross section.

Verification of serviceability limit state

The verification of the maximum vertical deflection is performed using deformations from ROBOTsoftware for serviceability limit states. For floors general limiting value for the vertical displacement according to EN 1990 Annex A1.4, National annex, figure A1.1 is following:

$$\delta_{max} < \frac{L_{beam}}{250}$$

For IPE 450:



$$\delta_{max} := 2.0\text{cm} < \frac{L_{beam}}{250} = 2.4 \cdot \text{cm}$$

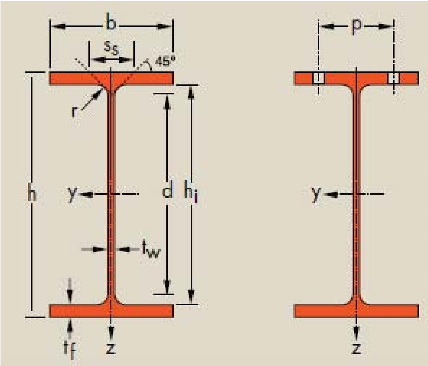
Cross section size is governed by deformation requirements. The deformation can not be increased till 2.4 cm because it results in increase of deformations in secondary beams. Deformation in secondary beams can not be increased (this is verified in verification of secondary beam).

Cross section IPE 450 verifies ULS and SLS requirements

5.2 Verification of columns

5.2.1 Central column (HEA 450)

Cross section characteristics



$h := 440mm$

$b := 300mm$

$t_w := 11.5mm$

$t_f := 21mm$

$r := 27mm$

$A_1 := 178cm^2$

$h_i := 398mm$

$d := 344mm$

$I_y := 63720cm^4$

$W_{el.y} := 2896cm^3$

$W_{pl.y} := 3216cm^3$

$i_y := 18.92cm$

$A_{vz} := 65.78cm^2$

$I_z := 9465cm^4$

$W_{el.z} := 631cm^3$

$W_{pl.z} := 965.5cm^3$

$i_z := 7.29cm$

$I_w := 4148000cm^6$

$I_t := 243.8cm^4$

Coefficients and other values

$\gamma_{M0} := 1$

$\gamma_{M1} := 1$

$L_{column} := 4m$

$H_{building} := 12m$

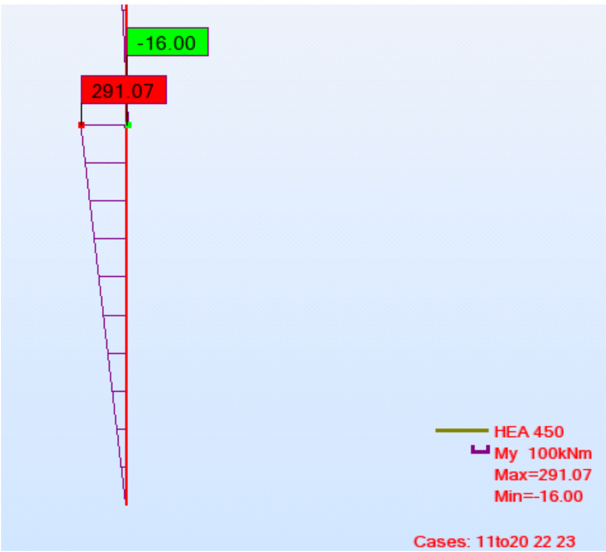
$E := 210000MPa$

$\nu := 0.3$

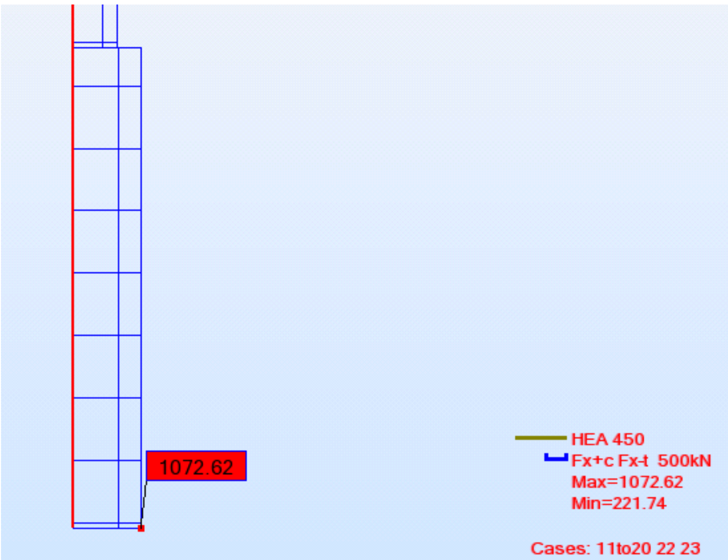
$G := 80700MPa$

Internal forces

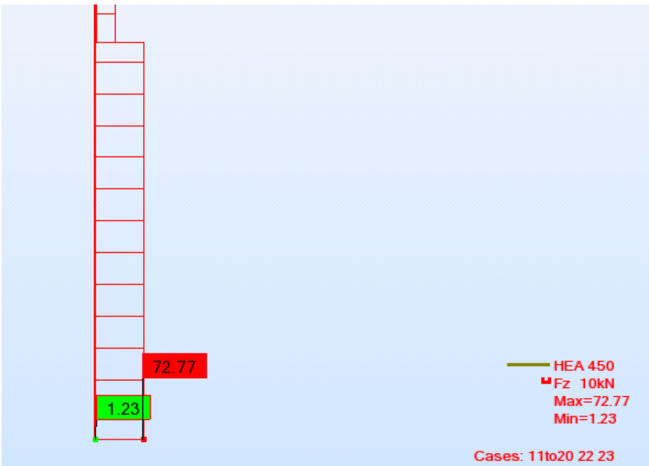
$M_{yEd} := 291.07\text{kN}\cdot\text{m}$



$N_{Ed} := 1072.62\text{kN}$



$V_{Ed} := 72.77\text{kN}$



$M_{zEd} := 0$

Cross section classification

$$f_{yd} := 355MPa \quad \varepsilon := \sqrt{\left(\frac{235MPa}{f_{yd}}\right)}$$

1. Flange in compression:

$$c_f := \frac{b - t_w - 2r}{2} = 117.25 \cdot mm$$

$$\frac{c_f}{t_f} = 5.583 < 9\varepsilon = 7.323$$

Flange is Class 1

2. Web in bending and compression

$$c_w := d$$

$$\alpha := \left(\frac{1}{d}\right) \cdot \left[\left(\frac{h}{2}\right) + \left[\frac{N_{Ed}}{\left(2 \cdot t_w \cdot f_{yd}\right)}\right] - \left(t_f + r\right)\right] = 0.882 \quad < \quad 1$$

as long as α=0.875>0.5

$$\frac{c_w}{t_w} = 29.913 \quad < \quad \frac{(396\varepsilon)}{13\alpha - 1} = 30.789$$

Therefore, web is Class 1

Cross section is Class 1.

Buckling length of the column

As long as the column is pinned we obtain following buckling length:

$$L_{z.cr} := L_{column} = 4\,m$$

$$L_{y.cr} := L_{column} = 4\,m$$

Verification of cross section resistance

1. Axial force

$$N_{pl.Rd} := \frac{\left(A_1 \cdot f_{yd}\right)}{\gamma_{M0}} = 6.319 \times 10^3 \cdot kN$$

$$\frac{N_{Ed}}{N_{pl.Rd}} = 0.17 < 1$$

2. Axial force and bending

According to EN1993-1-1 chapter 6.2.9.1

$$N_{Ed} = 1.073 \times 10^3 \cdot kN \quad < \quad 0.25 \cdot N_{pl.Rd} = 1.58 \times 10^3 \cdot kN$$

$$N_{Ed} = 1.073 \times 10^{3} \cdot kN \quad > \quad \frac{(0.5 \cdot h \cdot t_w \cdot f_{yd})}{\gamma_{M0}} = 898.15 \cdot kN$$

As a result, axial force has an effect on plastic moment resistance.

The resistance to bending combined with axial force is obtained from following expressions according to clause 6.2.9.1:

$$n := \frac{N_{Ed}}{N_{pl.Rd}} = 0.17$$

$$a := \frac{(A_1 - 2 \cdot b \cdot t_f)}{A_1} = 0.292 \quad < \quad 0.5$$

$$M_{pl.y.Rd} := W_{pl.y} \cdot \frac{f_{yd}}{\gamma_{M0}} = 1.142 \times 10^{3} \cdot kN \cdot m$$

Reduced plastic resistance is given by:

$$M_{n.y.Rd} := M_{pl.y.Rd} \cdot \frac{(1 - n)}{1 - 0.5 \cdot a} = 1.11 \times 10^{3} \cdot kN \cdot m$$

$$M_{yEd} = 291.07 \cdot kN \cdot m \quad < \quad M_{n.y.Rd} = 1.11 \times 10^{3} \cdot kN \cdot m$$

3. Shear force

$$V_{pl.Rd} := \frac{(A_{vz} \cdot f_{yd})}{\gamma_{M0} \cdot \sqrt{3}} = 1.348 \times 10^{3} \cdot kN$$

$$\frac{V_{Ed}}{V_{pl.Rd}} = 0.054 < 1$$

Shear buckling resistance classification:

$\eta := 1$ (conservatively taken)

$$\frac{h}{t_w} = 38.261 \quad < \quad \frac{72\varepsilon}{\eta} = 58.58$$

Therefore, shear buckling resistance of the web does not have to be verified

4. Bending and shear force

$$V_{Ed} = 72.77 \cdot kN \quad < \quad 0.5 V_{pl.Rd} = 674.111 \cdot kN$$

Therefore, effect of shear force on te moment resistance can be neglected

Verification of the stability of the member

According to EN 1993-1-1 chapter 6.3.3 members which are subjected to combined bending and axial compression should satisfy:

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

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

As long as members with open sections are susceptible to torsional deformation verification of lateral-tersional buckling is needed.

<u>Deternination of the reduction factor due to lateral-torsional buckling</u>

$k_z := 1$

$c_1 := 1.77$

$c_2 := 0$

$c_3 := 0$

$k_w := 1$

$$M_{cr} := c_1 \cdot \frac{(\pi \cdot E \cdot I_z)}{(k_z \cdot L_{column})^2} \cdot \left[\left(\frac{k_z}{k_w}\right)^2 \cdot \frac{I_w}{I_z} + \frac{\left[(k_z \cdot L_{column})^2 \cdot G \cdot I_t\right]}{\pi^2 E \cdot I_z}\right]^{0.5} = 1.69 \times 10^3 \cdot kN \cdot m$$

According to EN1993-1-1 chapter 6.3.2.2:

$\lambda_{LT} := \sqrt{\dfrac{(W_{pl.y} \cdot f_{yd})}{M_{cr}}} = 0.822$ non-dimensional slenderness

$\dfrac{h}{b} = 1.467$ < 2 => we use buckling curve a (from table 6.4)

For buckling curve a

$\alpha_{LT} := 0.21$ (from table 6.3)

$\Box_{LT} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot (\lambda_{LT} - 0.2) + \lambda_{LT}^2\right] = 0.903$

$\chi_{LT} := \dfrac{1}{\Box_{LT} + \sqrt{\Box_{LT}^2 - \lambda_{LT}^2}} = 0.783$ but $\chi_{LT} < 1$

<u>Determination of the reduction factors due to flexural buckling</u>

Calculation of non-dimensional slenderness for flexural buckling accordinf to EN1993-1-1 chapter 6.3.1.3

$$\lambda_1 := 93.9 \cdot \sqrt{\left(\frac{235MPa}{f_{yd}}\right)} = 76.399 \quad \text{for class 1}$$

$$\lambda_y := \frac{L_{y.cr}}{i_y \cdot \lambda_1} = 0.277$$

$$\lambda_z := \frac{L_{z.cr}}{i_z \cdot \lambda_1} = 0.718$$

Calculation of the reduction factor χy and χz according to chapter 6.3.1.2

$$\frac{h}{b} = 1.467 \quad > \quad 1.2$$

$$t_f = 21 \cdot mm < 100 \text{ mm}$$

As a result for <u>y-y</u> we use curve <u>b</u>, for <u>z-z</u> curve <u>c (table 6.2 EC3-1-1)</u>

$$\alpha_y := 0.34$$
$$\alpha_z := 0.49$$

$$\square_y := 0.5 \cdot \left[1 + \alpha_y \cdot \left(\lambda_y - 0.2\right) + \lambda_y^{\,2}\right] = 0.551$$

$$\square_z := 0.5 \cdot \left[1 + \alpha_z \cdot \left(\lambda_z - 0.2\right) + \lambda_z^{\,2}\right] = 0.885$$

$$\chi_y := \frac{1}{\left[\square_y + \sqrt{\left(\square_y^{\,2} - \lambda_y^{\,2}\right)}\right]} = 0.973$$

$$\chi_z := \frac{1}{\left[\square_z + \sqrt{\left(\square_z^{\,2} - \lambda_z^{\,2}\right)}\right]} = 0.713$$

<u>Calculation of Nrk, Mi,rk for class 1</u>

$$N_{Rk} := f_{yd} \cdot A_1 = 6.319 \times 10^6 N$$

$$M_{yRk} := f_{yd} \cdot W_{pl.y} = 1.142 \times 10^3 \cdot kN \cdot m$$

$$M_{zRk} := f_{yd} \cdot W_{pl.z} = 342.753 \cdot kN \cdot m$$

<u>Calculation of interaction factors according to Method 2</u>

Calculation is made according to Annex B EC3-1-1.

$\psi := 0$ because of triangular shape of bending moment diagram

$$C_{my} := 0.6$$

$$C_{mz} := 0.6$$

$$C_{mLt} := 0.6$$

$$k_{yy.1} := C_{my} \cdot \left[1 + (\lambda_y - 0.2) \cdot \left[\frac{N_{Ed}}{\frac{(\chi_y \cdot N_{Rk})}{\gamma_{M1}}} \right] \right] = 0.608 <$$

$$< k_{yy.2} := C_{my} \cdot \left[1 + 0.8 \cdot \left[\frac{N_{Ed}}{\frac{(\chi_y \cdot N_{Rk})}{\gamma_{M1}}} \right] \right] = 0.684$$

then $k_{yy} := 0.608$

$$k_{zz.1} := C_{mz} \cdot \left[1 + (2\lambda_z - 0.6) \cdot \left[\frac{N_{Ed}}{\frac{(\chi_z \cdot N_{Rk})}{\gamma_{M1}}} \right] \right] = 0.719 <$$

$$< k_{zz.2} := C_{mz} \cdot \left[1 + 1.4 \cdot \left[\frac{N_{Ed}}{\frac{(\chi_z \cdot N_{Rk})}{\gamma_{M1}}} \right] \right] = 0.8$$

then $k_{zz} := 0.721$

$$k_{yz} := 0.6 \cdot k_{zz} = 0.433$$

$$k_{zy1} := 1 - 0.1 \cdot \frac{\lambda_z \cdot N_{Ed}}{(C_{mLt} - 0.25) \cdot \left(\chi_z \cdot \frac{N_{Rk}}{\gamma_{M1}} \right)} = 0.951$$

$$k_{zy2} := 1 - 0.1 \cdot \frac{N_{Ed}}{(C_{mLt} - 0.25) \cdot \left(\chi_z \cdot \frac{N_{Rk}}{\gamma_{M1}} \right)} = 0.932$$

$k_{zy1} > k_{zy2}$

then $k_{zy} := 0.951$

Based on the determined parameters we obtain:

$$\frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} + \frac{k_{yy} \cdot M_{yEd}}{\frac{\chi_{LT} \cdot M_{yRk}}{\gamma_{M1}}} + \frac{k_{yz} \cdot M_{zEd}}{\frac{\chi_{LT} \cdot M_{zRk}}{\gamma_{M1}}} = 0.373 \quad <1$$

$$\frac{N_{Ed}}{\frac{\chi_z \cdot N_{Rk}}{\gamma_{M1}}} + \frac{k_{zy} \cdot M_{yEd}}{\frac{\chi_{LT} \cdot M_{yRk}}{\gamma_{M1}}} + \frac{k_{zz} \cdot M_{zEd}}{\frac{\chi_{LT} \cdot M_{zRk}}{\gamma_{M1}}} = 0.548 \quad <1$$

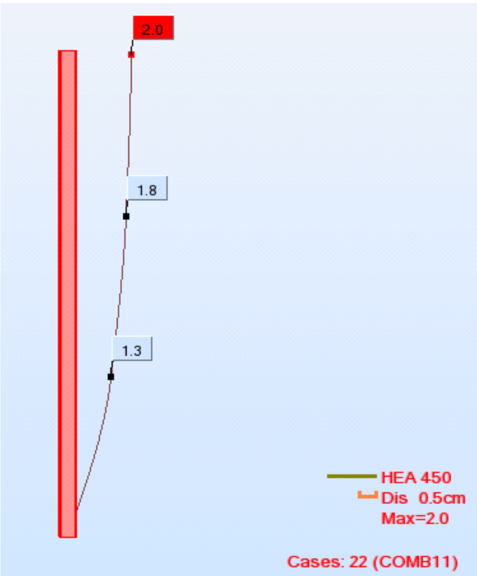
The stability of column with cross section HEA 450 is verified.

Verification of serviceability limit state

The verification of the maximum horizontal deflection is performed using deformations from ROBOTsoftware for serviceability limit states. Following limiting values apply for horizontal displacement:

1. Verification of horizontal displacement for the whole building height (H=12 m):

$$\Delta := 2.0 \quad < \quad \frac{H_{building}}{500} = 2.4 \cdot cm$$



2. Verification of horizontal displacement for the each floor:

first floor - $\delta_1 := 1.3 \quad < \quad \frac{L_{column}}{300} = 1.333 \cdot cm$

second floor - $\delta_2 := 0.5 \quad < \quad \frac{L_{column}}{300} = 1.333 \cdot cm$

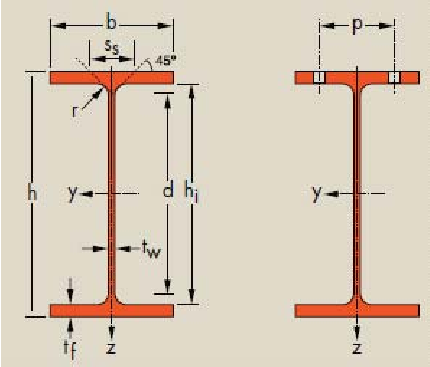
third floor - $\delta_3 := 0.2 \quad < \quad \frac{L_{column}}{300} = 1.333 \cdot cm$

Due to the horizontal displacement on the first floor which are very close to the limit section can not be reduced. Moreover, decreasing of column section results in increase of the displacement in secondary beams which are also close to the limit.

Cross section HEA 450 for column verifies the reuirements of ULS and SLS.

5.2.2 Perimeter column HEA 300

Cross section characteristics



$h := 290mm$

$b := 300mm$

$t_w := 8.5mm$

$t_f := 14mm$

$r := 27mm$

$A_1 := 112.5cm^2$

$h_i := 262mm$

$d := 208mm$

$I_y := 18260cm^4$

$W_{el.y} := 1260cm^3$

$W_{pl.y} := 1383cm^3$

$i_y := 12.74cm$

$A_{vz} := 37.28cm^2$

$I_z := 6310cm^4$

$W_{el.z} := 420.6cm^3$

$W_{pl.z} := 641.2cm^3$

$i_z := 7.49cm$

$I_w := 1200000cm^6$

$I_t := 85.17cm^4$

Material

$\gamma_{M0} := 1$

$\gamma_{M1} := 1$

$E := 210000MPa$

$\nu := 0.3$

$G := 80700MPa$

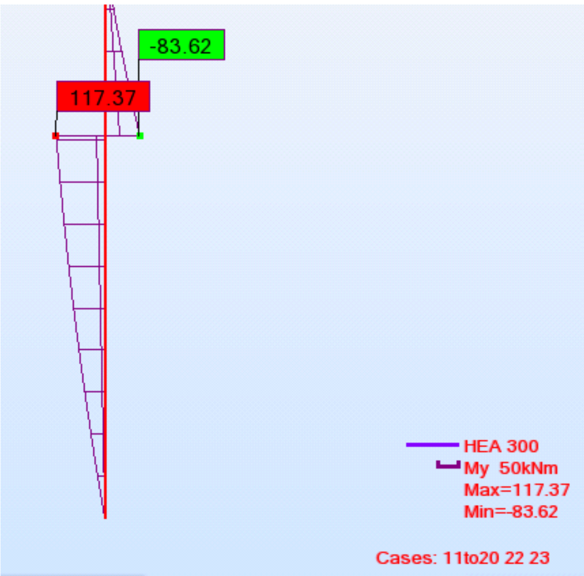
Building

$L_{column} := 4m$

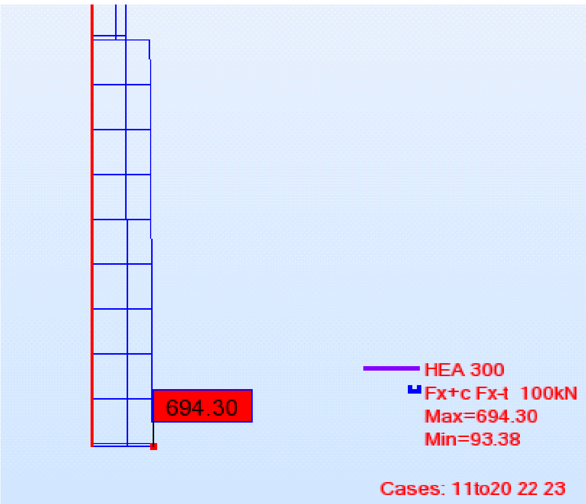
$H_{building} := 12m$

Internal forces

$M_{yEd} := 117.37 kN \cdot m$



$N_{Ed} := 694.3 kN$



Verification of perimetr column will be made using SemiComp+ software. For input data we need to calculate Mcr.

Deternination of critical moment:

$k_z := 1$

$c_1 := 1.77$

$c_2 := 0$

$c_3 := 0$

$k_w := 1$

$$M_{cr} := c_1 \cdot \frac{(\pi \cdot E \cdot I_z)}{(k_z \cdot L_{column})^2} \cdot \left[\left(\frac{k_z}{k_w} \right)^2 \cdot \frac{I_w}{I_z} + \frac{\left[(k_z \cdot L_{column})^2 \cdot G \cdot I_t \right]}{\pi^2 E \cdot I_z} \right]^{0.5} = 762.666 \cdot kN \cdot m$$

Following verification were obtained (using Method 2):

SEMICOMP Member Design

Cross-section type I- or H-Section

Finishing Rolled

Select from library (optional) HEA 300

Partial factors γ_M

γ_{M0} = 1.00

γ_{M1} = 1.00

Cross-section data

H = 290.0 [mm]

B = 300.0 [mm]

Tw = 8.5 [mm]

Tf = 14.0 [mm]

R = 27.0 [mm]

Material

Steel grade S 355

f_y = 355.0 N/mm²

E = 210000.0 N/mm²

A [cm²]	I_{yy} [cm⁴]	I_{zz} [cm⁴]	$W_{el,y}$ [cm³]	$W_{el,z}$ [cm³]	$W_{pl,y}$ [cm³]	$W_{pl,z}$ [cm³]
112.53	18263.50	6309.56	1259.55	420.64	1383.27	641.17

Boundary conditions

L_{beam}= 4.000 m

n_{fork}= 0 [-]

Torsion restrained

strong Axis buckling restrained

weak Axis Buckling restrained

I_t [cm⁴]	I_w [cm⁶]
82.74	1201589.67

Loading in z-x-plane

N_{Ed} = -694.30 kN

$q_{z,Ed}$ (*) = 5.00 kN/m

$M_{y,left,Ed}$ = 0.00 kNm

$M_{y,right,Ed}$ = 117.37 kNm

$P_{z,Ed}$ (*) = 0.00 kN

Distance of Loading to shear center

z_{Si} = -145.00 mm

Loading in y-x-plane

$q_{y,Ed}$ (*) = 0.00 kN/m

$M_{z,left,Ed}$ = 0.00 kNm

$M_{z,right,Ed}$ = 0.00 kNm

$P_{y,Ed}$ (*) = 0.00 kN

Calculate Forces!

(*)

M_{cr} = 762.66 kNm

$M_{cr,2}$ = 0.00 kNm

Enter Mcr manually

Use LTBeam

Specify path of LTBeam.exe file:

C:\Program Files (x86)\LTBeam_v1010\LTBeam.ex

Note: LTBeam is a tool developed by CTICM to calculate the lateral torsional buckling moment of beams. You can download it for free at www.cticm.com.

Definition of axes

SEMICOMP Member Check

Choose method for member check	*Choose method for cross-section resistance*	
Method 2 (EN 1993-1-1 Annex B)	EN 1993-1-1:2010-12	Perform member design check

Section classification for member design check *(based on I. order cross-section forces)*

Reference values for classification in the worst section along the member

c/t_w =	24.471	α_{web} =	1.000	ψ_{web} =	-0.040	ε =	0.814
c/t_f =	8.482	α_{flange} =	1.000	ψ_{flange} =	1.000		

Boundaries

		Class 1	Class 2	Class 3
$c/t_w \leq$	$c/t_{w,\,max}$ =	26.849	30.917	52.027
$c/t_f \leq$	$c/t_{f,\,max}$ =	7.323	8.136	11.204

Member class = 3

Note: This tool is only applicable to Class 1 to 3. For Class 4 elastic cross-section values are used for all calculations. The user should check if parts of member are Class 4 in the "Additional info"-sheet.

Member Check

			$M_{pl,y,Rd}$ =	491.061	kNm			
N_{Rd} =	3994.737	kN	$M_{pl,z,Rd}$ =	227.614	kNm	N_{Ed} =	-694.300	kN
$M_{y,Rd}$ =	447.141	kNm	$M_{el,y,Rd}$ =	447.141	kNm	$M_{y,Ed,max}$ =	117.370	kNm
$M_{z,Rd}$ =	149.326	kNm	$M_{el,z,Rd}$ =	149.326	kNm	$M_{z,Ed,max}$ =	0.000	kNm

Strong axis buckling			*Weak axis buckling*			*Lateral torsional buckling*		
$L_{cr,y}$ =	4.000	m	$L_{cr,z}$ =	4.000	m	M_{cr} =	762.660	kNm
$N_{cr,y}$ =	23658.272	kN	$N_{cr,z}$ =	8173.312	kN	α_{LT} =	0.34	[-]
α_y =	0.34	[-]	α_z =	0.49	[-]	$\bar{\lambda}_{LT}$ =	0.766	[-]
$\bar{\lambda}_y$ =	0.411	[-]	$\bar{\lambda}_z$ =	0.699	[-]	$\chi_{LT,mod}$ =	0.924	[-]
χ_y =	0.922	[-]	χ_z =	0.725	[-]	f_{mod} =	0.904	[-]

Reference values

Correction factor k_c, table 6.6

k_c = 0.808 [-]

Method 1 auxiliary terms (if applicable):

μ_y =	0.000	[-]	λ_{max} =	0.000	[-]
μ_z =	0.000	[-]	λ_0 =	0.000	[-]
w_y =	0.000	[-]	$\lambda_{0,lim}$ =	0.000	[-]
w_z =	0.000	[-]	λ_{LT} =	0.000	[-]
n_{pl} =	0.000	[-]	ε_y =	0.000	[-]
a_{LT} =	0.000	[-]	$C_{my,0}$ =	0.000	[-]
b_{LT} =	0.000	[-]	$C_{mz,0}$ =	0.000	[-]
c_{LT} =	0.000	[-]	C_{my} =	0.000	[-]
d_{LT} =	0.000	[-]	C_{mz} =	0.000	[-]
e_{LT} =	0.000	[-]	C_{mLT} =	0.000	[-]
$M_{cr,0}$ =	0.000	[-]	C_1 =	0.000	[-]
C_{yy} =	0.000	[-]	C_{zy} =	0.000	[-]
C_{yz} =	0.000	[-]	C_{zz} =	0.000	[-]

Method 2 auxiliary terms (if applicable):

C_{my} =	0.668	[-]
C_{mz} =	0.000	[-]
C_{mLT} =	0.668	[-]

EN 1993-1-1, 6.3.3

Uniform member in bending and axial compression

Eq. (6.61):	U = 0.387 ≤ 1,0	ok	
Eq. (6.62):	U = 0.518 ≤ 1,0	ok	

Global interaction factors

k_{yy} =	0.699
k_{yz} =	0.000
k_{zy} =	0.980
k_{zz} =	0.000

Cross-section check at each end of the member

Left end:	U = 0.174 ≤ 1,0	ok	UF = 0.174
Right end:	U = 0.436 ≤ 1,0	ok	UF = 0.436

Additional member checks

EN 1993-1-1, 6.3.1

Strong axis flexural buckling check

Eq. (6.46): $N_{Ed}/N_{b,Rd}$ = 0.189 ≤ 1,0 ok

Weak axis flexural buckling check

Eq. (6.46): $N_{Ed}/N_{b,Rd}$ = 0.240 ≤ 1,0 ok

EN 1993-1-1, 6.3.2

Lateral torsional buckling

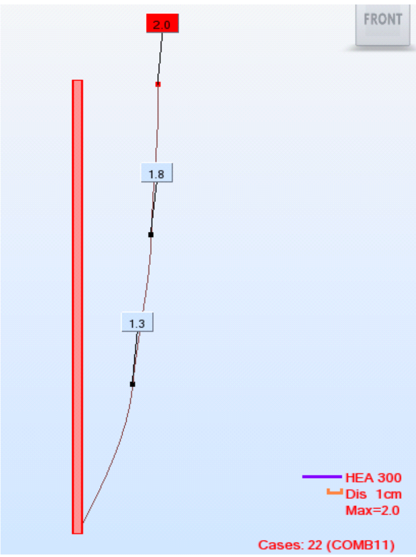
Eq. (6.54): $M_{Ed}/M_{b,Rd}$ = 0.284 ≤ 1,0 ok

Verification of serviceability limit state

The verification of the maximum horizontal deflection is performed using deformations from ROBOTsoftware for serviceability limit states. Following limiting values apply for horizontal displace

1. Verification of horizontal displacement for the whole building height (H=12 m):

$$\Delta := 2.0 < \frac{H_{building}}{500} = 2.4 \cdot cm$$



2. Verification of horizontal displacement for the each floor:

first floor - $\delta_1 := 1.3 < \frac{L_{column}}{300} = 1.333 \cdot cm$

second floor - $\delta_2 := 0.5 < \frac{L_{column}}{300} = 1.333 \cdot cm$

third floor - $\delta_3 := 0.2 < \frac{L_{column}}{300} = 1.333 \cdot cm$

Due to the horizontal displacement on the first floor which are very close to the limit section can not be reduced. Moreover, decreasing of column section results in increase of the displacement in secondary beams which are also close to the limit.

Cross section HEA 300 for column verifies the reuirements of ULS and SLS.

5.3 Bracing

Cross section characteristics TRON 139x8

$A_x := 33.1cm^2$ - cross section area

$I_y := 720.29cm^4$ - moment of inertia of a section around y-axis

$I_z := 720.29cm^4$ - moment of inertia of a section around y-axis

$h := 14cm$ –diameter

$t := 0.8cm$ - web thickness

$L_b := 7.21m$ - length of bracing element

Material

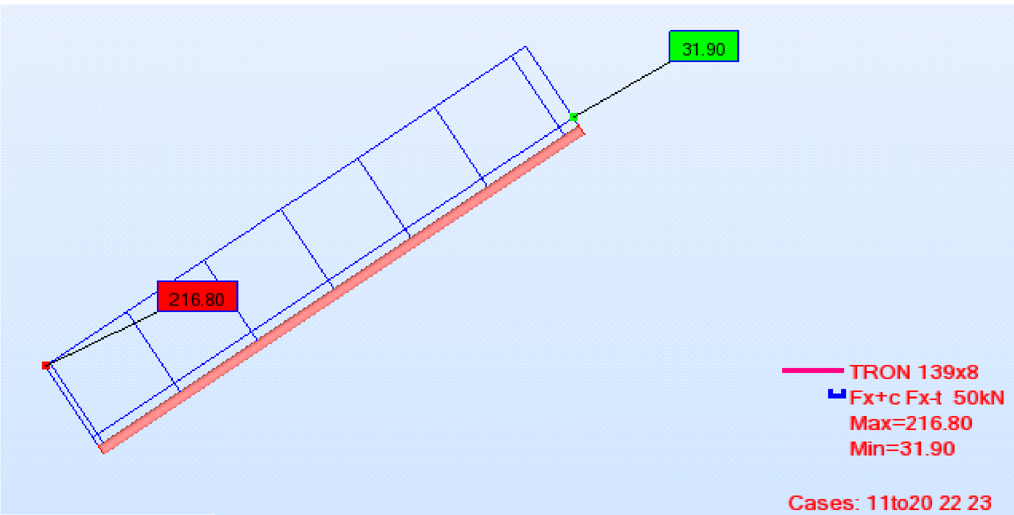
$f_{yd} := 355MPa$

$\gamma_{M0} := 1$

$\gamma_{M1} := 1$

$E := 210000MPa$

Internal forces



$N_{Ed} := 216.8kN$

Cross section classification

$$\varepsilon := \sqrt{\left(\frac{235MPa}{f_{yd}}\right)}$$

Section in compression

$$\frac{h}{t} = 17.5 \;<\; 50\varepsilon^2 = 33.099$$

The cross section is class 1.

Cross section resistance

Axial force

$$N_{Ed} = 216.8 \cdot kN < \quad N_{cRd} := A_x \cdot \frac{f_{yd}}{\gamma_{M0}} = 1.175 \times 10^3 \cdot kN$$

Verification of buckling resistance

<u>Flexural buckling</u>

$$\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yd}}} = 76.409$$

$L_E := 1 \cdot L_b = 7.21\,m$ - buckling length

$i := \sqrt{\frac{I_y}{A_x}} = 4.665 \cdot cm$ - radius of giration

$$\lambda := \frac{L_E}{i} = 154.559$$

$\lambda_n := \frac{\lambda}{\lambda_1} = 2.023$ - nondimensional skenderness coefficient

hot finished hollow section => curve a, s $\alpha_1 := 0.21$

$$\square := 0.5 \cdot \left[1 + \alpha_1 \cdot \left(\lambda_n - 0.2\right) + \lambda_n{}^2\right] = 2.737$$

$$\chi := \frac{1}{\square + \sqrt{\square^2 - \lambda_n}} = 0.197$$

$$N_{bRd} := \frac{\chi \cdot A_x \cdot f_{yd}}{\gamma_{M1}} = 231.494 \cdot kN$$

$$N_{Ed} = 216.8 \cdot kN \quad < \quad N_{bRd} = 231.494 \cdot kN$$

So section TRON 139x8 is adopted.

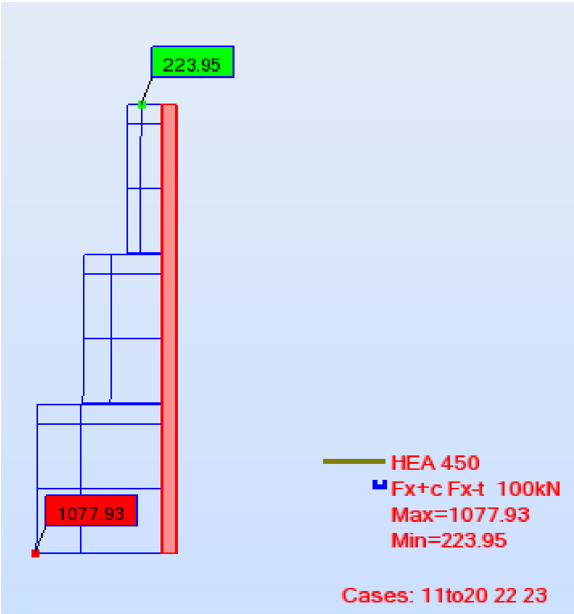
6. Verification of joints

6.1 Column base

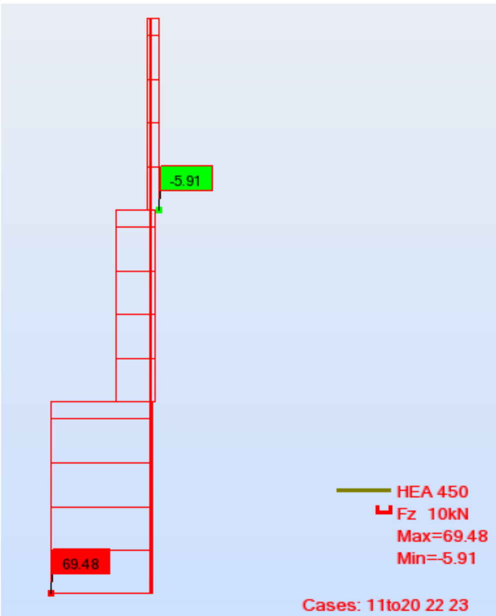
The design of column base joint is performed in software ROBOT.

a) Pinned column base joint for column HEA 450.

For designing was chosen a column with the maximum axial load.



Axial force diagram



Shear force diagram

For anchoring of the column two anchor bolts are sufficient. However, for better execution it is chosen to install 4 anchor bolts.

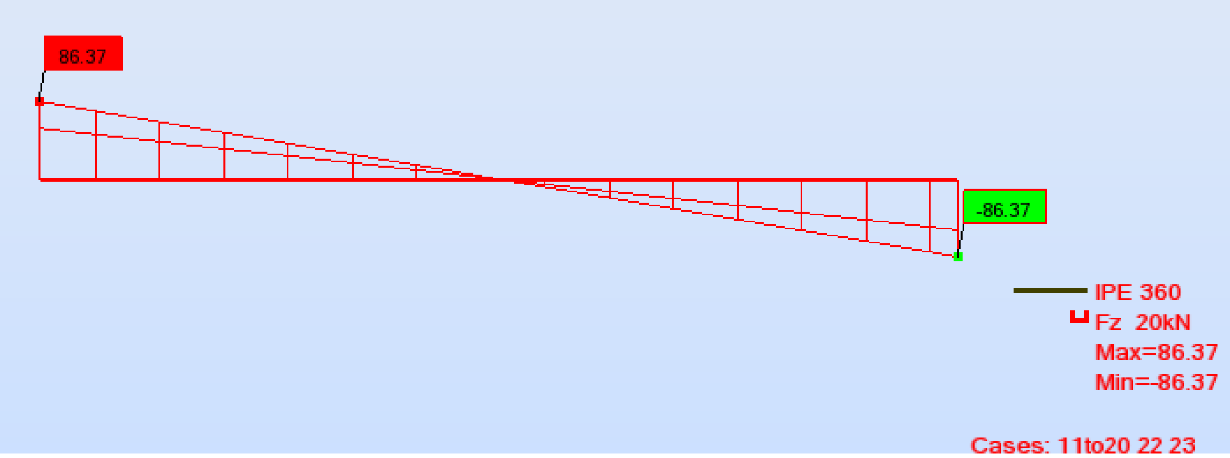
Results of calculation can be found in Annex 1.

6.2 Beam-beam connection

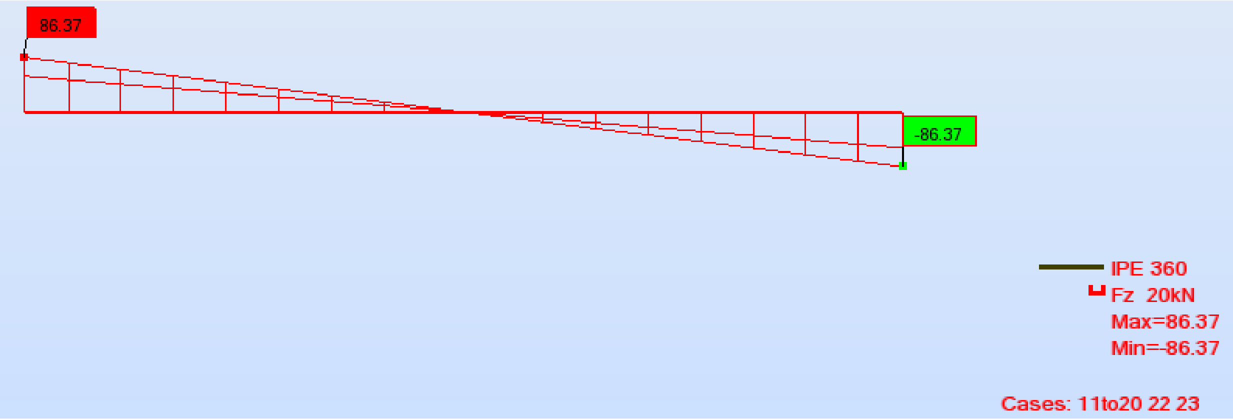
The design of beam-beam joint is performed in software ROBOT.

Connection is made for two secondary beams IPE 360 and main beam IPE 450.

As long as the secondary beam has pinned connection to main beam the governing internal force is shear. Therefore, we chose the beam with maximum shear force for design.



Shear force diagram for right secondary beam IPE 360.



Shear force diagram for left secondary beam IPE 360.

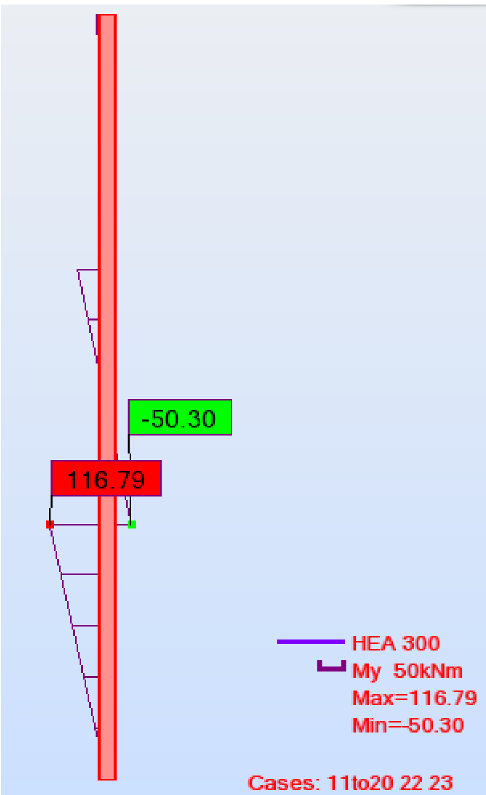
Results of calculation can be found in Annex 2.

6.3 Beam-to-column connection

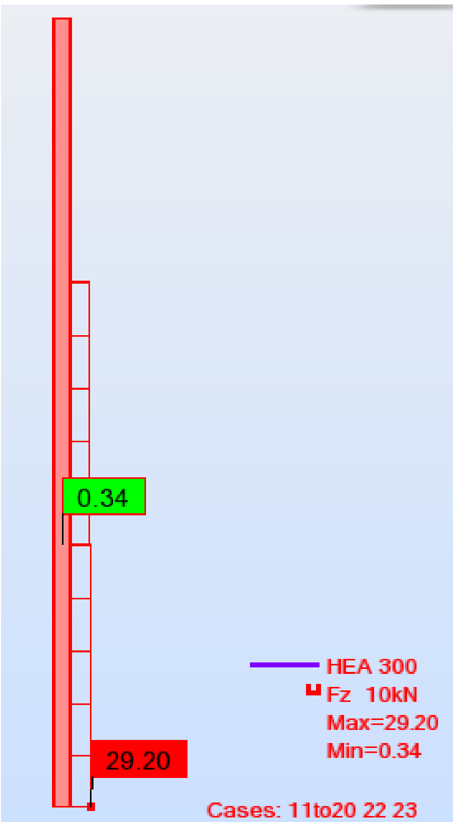
The design of beam-beam joint is performed in software ROBOT.

a) Connection of column HEA 300 (flange) to beam IPE 450.

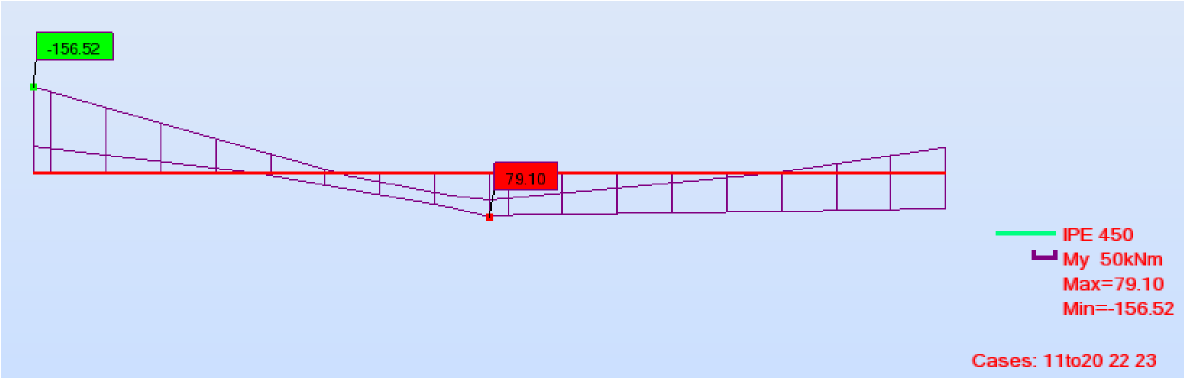
This joint is a moment resisting one. We chose the connection with the biggest moment and shear force.



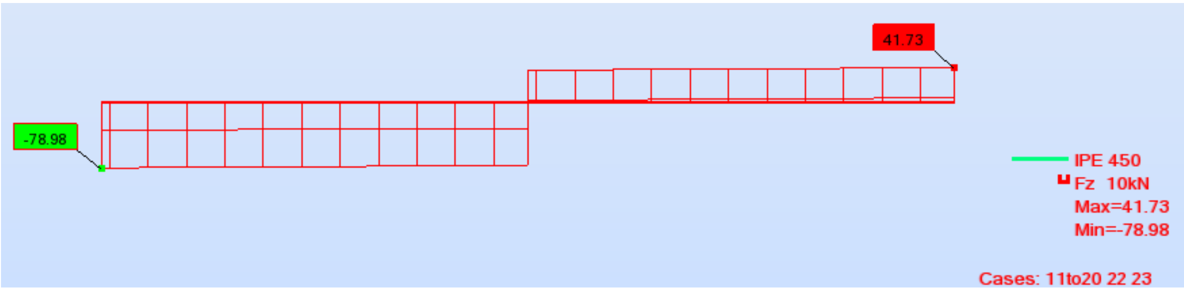
Moment diagram of column



Shear force diagram of column.



Moment diagram of beam

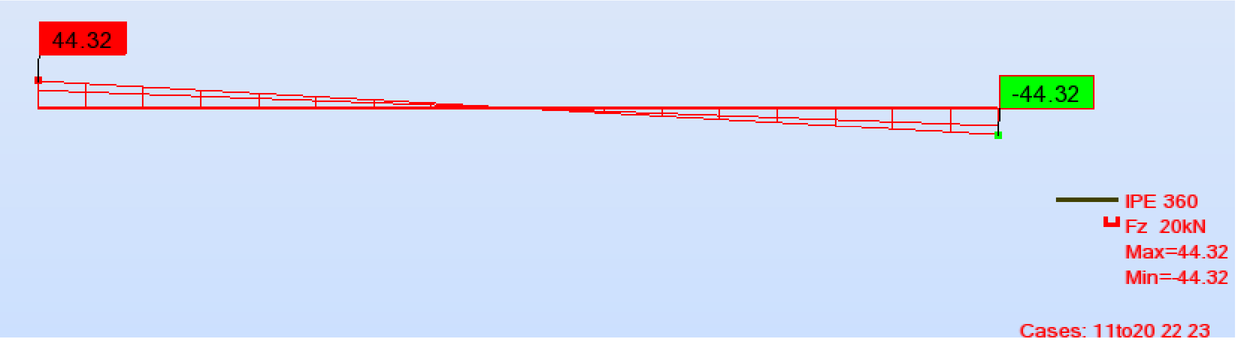


Shear diagram for beam

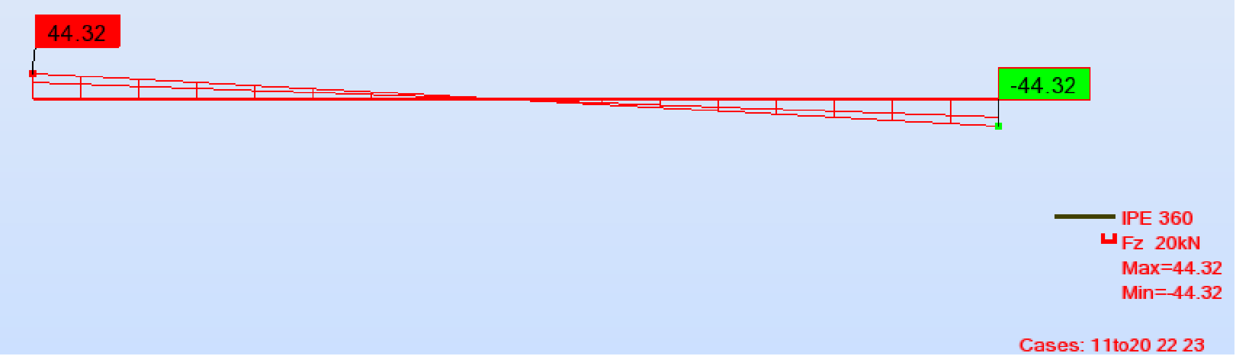
Results of calculation can be found in Annex 3.

b) Connection of column HEA 300 (web) to two secondary beams IPE 360.

As long as the secondary beam has pinned connection to column, the governing internal force is shear. Therefore, we chose the beam with maximum shear force for design.



Shear force diagram of the left secondary beam IPE 360.



Shear force diagram of the right secondary beam IPE 360.

Results of calculation can be found in Annex 4.

6.4 Bracing joint (gusset plate connection)

The gusset plate is welded to the beam using double fillet welds. Joint is designed in a way to minimiz the eccentricity between the bracing member and the column axis.

Main joint data:

Column - HEA 300, S355

Beam - IPE 360, S355

Bracing - TRON 139x8, S355

Type - plate welded to bracing and then bolted to gusset plate; gusset plate is welded to beam.

Cross section characteristics TRON 139x8

$A_x := 33.1\text{cm}^2$ - cross section area

$I_y := 720.29\text{cm}^4$ - moment of inertia of a section around y-axis

$I_z := 720.29\text{cm}^4$ - moment of inertia of a section around y-axis

$h := 14\text{cm}$ –diameter

$t := 0.8\text{cm}$ - web thickness

$L_b := 7.21\text{m}$ - length of bracing element

Material

$f_{yd} := 355\text{MPa}$

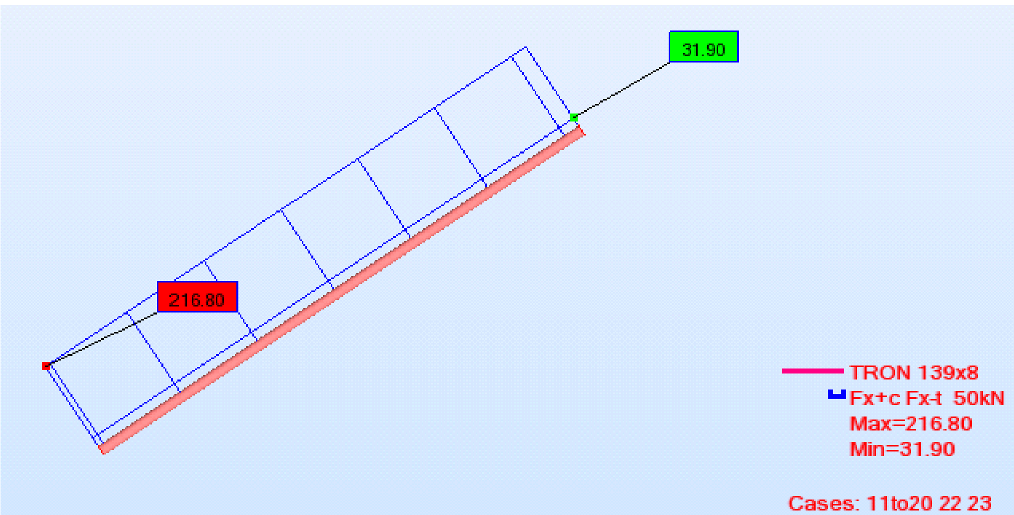
$f_u := 470\text{MPa}$

$\gamma_{M0} := 1$

$\gamma_{M1} := 1$

$\gamma_{M2} := 1.25$

Internal forces



$N_{Ed} := 216.8\text{kN}$

Shear resistance of bolts

In order to evaluate the tpe and quantity of bolts for fastening the bracing plate to gusset plate we use table 3.4 EN 1993-1-8.

Shear resistance per shear plane:

$$F_{vRd} := \frac{\alpha_v \cdot f_{ub} \cdot A_b}{\gamma_{M2}}$$

We choose class of bolts 8.8. As a result we obtain following input data:

$\alpha_v := 0.6$ - for class 8.8

$f_{ub} := 800 \frac{N}{mm^2}$ - for class 8.8

$$F_{vRd} := 216.8kN$$

We obtain the required cross section of the bolts:

$$A_b := \frac{F_{vRd} \cdot \gamma_{M2}}{\alpha_v \cdot f_{ub}} = 564.583 \cdot mm^2$$

Taking the bolts with d=20mm, class 8.8 required quantity of bolts:

$A_{20} := 314mm^2$ - area of one bolt with d=20mm in accordance with EN ISO 898.

$$n := \frac{A_b}{A_{20}} = 1.798$$

As a result we take 2 bolts class 8.8,

$d := 20mm$

$d_0 := d + 2mm = 22 \cdot mm$

Now $A_{b1} := 2 \cdot A_{20} = 628 \cdot mm^2$

Shear resistance of bolts:

$$F_{vRd} := \frac{\alpha_v \cdot f_{ub} \cdot A_{b1}}{\gamma_{M2}} = 241.152 \cdot kN$$

$$\frac{N_{Ed}}{F_{vRd}} = 0.899 \quad < 1$$

Verification of bearing resistance

$$F_{bRd} := \frac{k_1 \cdot \alpha_b \cdot f_{uac} \cdot d \cdot t_p}{\gamma_{M2}}$$

Characteristics of bolts location:

$e_1 := 40mm$

$e_2 := 40mm$

$p_2 := 80mm$

$$\alpha_b := \frac{e_1}{3d_0} = 0.606$$

$$k_1 := \min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right) = 2.5$$

$t_p := 10mm$ - thickness of plate welded to bracing

$t_g := 15mm$ - thickness of gusset plate

$$F_{bRd} := \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t_p}{\gamma_{M2}} = 113.939 \cdot kN$$

Shear force for one bolt:

$$N_{Ed1} := \frac{N_{Ed}}{2} = 108.4 \cdot kN \ < \ F_{bRd} = 113.939 \cdot kN$$

Bearing resistance is sufficient.

Weld design

Weld design is as follows:

1. The gusset plate is welded to the beam using double fillets.

a)Weld design for gusset plate and a beam according to simplified method:

we propose $a := 4mm$

$\beta_w := 0.95$

$l_c := 320mm$

$N_{rdw} := 2F_{wRd} \cdot l$

$F_{wRd} := f_{vw} \cdot a$

$$f_{vw} := \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = 228.509 \cdot \frac{N}{mm^2}$$

$$F_{wRd} := f_{vw} \cdot a = 914.037 \frac{N}{mm}$$

$$N_{rdw} := 2F_{wRd} \cdot l_c = 584.983 \cdot kN$$

It supports the horizontal component of the force acting in the bracing:

$$N_{Edhor} := N_{Ed} \cdot \sin(56deg) = 179.735 \cdot kN$$

Therefore, the horizontal weld is OK.

2. The bracing is welded to the plate bolted to the gusset plate.

we propose $a_3 := 4mm$

$\beta_{w3} := 0.95$

$l_3 := 150mm$

$$f_{vw3} := \frac{\frac{f_u}{\sqrt{3}}}{\beta_{w3} \cdot \gamma_{M2}} = 228.509 \cdot \frac{N}{mm^2}$$

$$F_{wRd3} := f_{vw} \cdot a_3 = 914.037 \frac{N}{mm}$$

$$N_{rdw3} := 4F_{wRd} \cdot l_c = 1.17 \times 10^3 kN$$

$$N_{Ed} = 216.8 \cdot kN < N_{rdw3} = 1.17 \times 10^3 \cdot kN$$

So the welding is OK.

7. References.

1. Simoes da Silva L., Simoes R., Gervasio H.: Design of steel structures. ECCS Eurocode Design Manuals, ECCS and Ernst & Sohn, 2010, 438 p.
2. Jaspart J.P. and Weynand K.: Design of joints for steel and steel-concrete structures, ECCS, Ernst & Sohn and Wiley, 2012.
3. EN 1993-1-1 Design of steel structures – Part 1.1: General rules and rules for buildings.
4. EN 1993-1-8 Design of steel structures – Part 1.8: Design of joints.
5. EN 1990 Basis of structural design.
6. EN 1991-1-1 Actions on structures – Part 1.1: General actions – Densities, self-weight, imposed loads for buildings.
7. EN 1991-1-3 Actions on structures – Part 1.3: General actions – Snow loads.

Annex 1. Calculation of column base joint in software ROBOT.

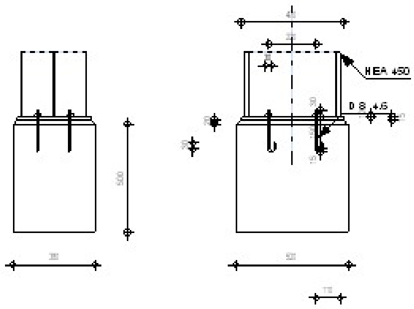
Autodesk Robot Structural Analysis Professional 2013-Student Version

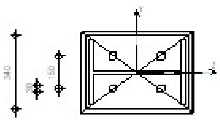
Pinned column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009 + CEB Design Guide: Design of fastenings on concrete

OK

Ratio
0.98





GENERAL

Connection no.: 2
Connection name: Pinned column base
Structure node: 37
Structure bars: 29

GEOMETRY

COLUMN

Section: HEA 450
Bar no.: 29

L_c =	12.00	[m]	Column length
α =	0.0	[Deg]	Inclination angle
h_c =	440	[mm]	Height of column section
b_{fc} =	300	[mm]	Width of column section
t_{wc} =	12	[mm]	Thickness of the web of column section
t_{fc} =	21	[mm]	Thickness of the flange of column section
r_c =	27	[mm]	Radius of column section fillet
A_c =	178.03	[cm^2]	Cross-sectional area of a column
I_{yc} =	63721.60	[cm^4]	Moment of inertia of the column section
Material:	S 355		
f_{yc} =	355.00	[MPa]	Resistance
f_{uc} =	470.00	[MPa]	Yield strength of a material

COLUMN BASE

l_{pd} =	470	[mm]	Length
b_{pd} =	340	[mm]	Width
t_{pd} =	15	[mm]	Thickness
Material:	S 355		
f_{ypd} =	355.00	[MPa]	Resistance
f_{upd} =	470.00	[MPa]	Yield strength of a material

ANCHORAGE

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	4.6		Anchor class
f_{yb} =	240.00	[MPa]	Yield strength of the anchor material
f_{ub} =	400.00	[MPa]	Tensile strength of the anchor material
d =	8	[mm]	Bolt diameter
	0.37		Effective section area of a bolt

A_s =		[cm^2]	
A_v =	0.50	[cm^2]	Area of bolt section
n_V =	2		Number of bolt columns
n_H =	2		Number of bolt rows
e_H =	220	[mm]	Horizontal spacing
e_V =	150	[mm]	Vertical spacing

Anchor dimensions

L_1 =	30	[mm]
L_2 =	150	[mm]
L_3 =	30	[mm]
L_4 =	30	[mm]

Washer

l_{wd} =	30	[mm]	Length
b_{wd} =	30	[mm]	Width
t_{wd} =	10	[mm]	Thickness

MATERIAL FACTORS

γ_{M0} =	1.00	Partial safety factor
γ_{M2} =	1.25	Partial safety factor
γ_C =	1.50	Partial safety factor

SPREAD FOOTING

L =	520	[mm]	Spread footing length
B =	380	[mm]	Spread footing width
H =	500	[mm]	Spread footing height

Concrete

Class C20/25

f_{ck} =	20.00	[MPa]	Characteristic resistance for compression

Grout layer

t_g =	20	[mm]	Thickness of leveling layer (grout)
$f_{ck,g}$ =	12.00	[MPa]	Characteristic resistance for compression
$C_{f,d}$ =	0.30		Coeff. of friction between the base plate and concrete

WELDS

a_p =	4	[mm]	Footing plate of the column base

LOADS

Case: Manual calculations.

$N_{j,Ed}$ =	-1077.93	[kN]	Axial force
$V_{j,Ed,y}$ =	0.01	[kN]	Shear force
$V_{j,Ed,z}$ =	69.48	[kN]	Shear force

RESULTS

COMPRESSION ZONE

COMPRESSION OF CONCRETE

f_{cd} =	13.33	[MPa]	Design compressive resistance	EN 1992-1:[3.1.6.(1)]
f_j =	9.88	[MPa]	Design bearing resistance under the base plate	[6.2.5.(7)]

$c = t_p \sqrt{(f_{yp}/(3*f_j*\gamma_{M0}))}$

c =	52	[mm]	Additional width of the bearing pressure zone	[6.2.5.(4)]
b_{eff} =	88	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
l_{eff} =	340	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]
A_{c0} =	298.86	[cm^2]	Area of the joint between the base plate and the foundation	EN 1992-1:[6.7.(3)]
A_{c1} =	524.02	[cm^2]	Maximum design area of load distribution	EN 1992-1:[6.7.(3)]

$F_{rdu} = A_{c0}*f_{cd}*\sqrt{(A_{c1}/A_{c0})} \leq 3*A_{c0}*f_{cd}$

A_{c1} =	524.02	[cm^2]	Maximum design area of load distribution	EN 1992-1:[6.7.(3)]
β_j =	0.67		Reduction factor for compression	[6.2.5.(7)]

$f_{jd} = \beta_j*F_{rdu}/(b_{eff}*l_{eff})$

f_{jd} = 11.77 [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,n}$ = 936.93 [cm^2] Bearing area for compression [6.2.8.2.(1)]
$F_{c,Rd,i} = A_{C,i}*f_{jd}$
$F_{c,Rd,n}$ = 1102.80 [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$N_{j,Rd} = F_{c,Rd,n}$
$N_{j,Rd}$ = 1102.80 [kN] Resistance of a spread footing for axial compression [6.2.8.2.(1)]

CONNECTION CAPACITY CHECK

$N_{j,Ed} / N_{j,Rd} \le 1,0$ (6.24) 0.98 < 1.00 verified (0.98)

SHEAR

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

Shear force $V_{j,Ed,y}$

$\alpha_{d,y}$ = 3.17 Coeff. taking account of the bolt position - in the direction of shear [Table 3.4]
$\alpha_{b,y}$ = 0.85 Coeff. for resistance calculation $F_{1,vb,Rd}$ [Table 3.4]
$k_{1,y}$ = 2.50 Coeff. taking account of the bolt position - perpendicularly to the direction of shear [Table 3.4]
$F_{1,vb,Rd,y} = k_{1,y}*\alpha_{b,y}*f_{up}*d*t_p / \gamma_{M2}$
$F_{1,vb,Rd,y}$ = 96.00 [kN] Resistance of an anchor bolt for bearing pressure onto the base plate [6.2.2.(7)]

Shear force $V_{j,Ed,z}$

$\alpha_{d,z}$ = 4.17 Coeff. taking account of the bolt position - in the direction of shear [Table 3.4]
$\alpha_{b,z}$ = 0.85 Coeff. for resistance calculation $F_{1,vb,Rd}$ [Table 3.4]
$k_{1,z}$ = 2.50 Coeff. taking account of the bolt position - perpendicularly to the direction of shear [Table 3.4]
$F_{1,vb,Rd,z} = k_{1,z}*\alpha_{b,z}*f_{up}*d*t_p / \gamma_{M2}$
$F_{1,vb,Rd,z}$ = 96.00 [kN] Resistance of an anchor bolt for bearing pressure onto the base plate [6.2.2.(7)]

SHEAR OF AN ANCHOR BOLT

α_b = 0.37 Coeff. for resistance calculation $F_{2,vb,Rd}$ [6.2.2.(7)]
A_{vb} = 0.50 [cm^2] Area of bolt section [6.2.2.(7)]
f_{ub} = 400.00 [MPa] Tensile strength of the anchor material [6.2.2.(7)]
γ_{M2} = 1.25 Partial safety factor [6.2.2.(7)]
$F_{2,vb,Rd} = \alpha_b*f_{ub}*A_{vb}/\gamma_{M2}$
$F_{2,vb,Rd}$ = 5.92 [kN] Shear resistance of a bolt - without lever arm [6.2.2.(7)]

α_M = 2.00 Factor related to the fastening of an anchor in the foundation CEB [9.3.2.2]
$M_{Rk,s}$ = 0.02 [kN*m] Characteristic bending resistance of an anchor CEB [9.3.2.2]
l_{sm} = 29 [mm] Lever arm length CEB [9.3.2.2]
γ_{Ms} = 1.20 Partial safety factor CEB [3.2.3.2]
$F_{v,Rd,sm} = \alpha_M*M_{Rk,s}/(l_{sm}*\gamma_{Ms})$
$F_{v,Rd,sm}$ = 1.04 [kN] Shear resistance of a bolt - with lever arm CEB [9.3.1]

CONCRETE PRY-OUT FAILURE

$N_{Rk,c}$ = 26.34 [kN] Design uplift capacity CEB [9.2.4]
k_3 = 2.00 Factor related to the anchor length CEB [9.3.3]
γ_{Mc} = 2.16 Partial safety factor CEB [3.2.3.1]
$F_{v,Rd,cp} = k_3*N_{Rk,c}/\gamma_{Mc}$
$F_{v,Rd,cp}$ = 24.39 [kN] Concrete resistance for pry-out failure CEB [9.3.1]

CONCRETE EDGE FAILURE

Shear force $V_{j,Ed,y}$

$V_{Rk,c,y}^0$ = 44.33 [kN] Characteristic resistance of an anchor CEB [9.3.4.(a)]
$\psi_{A,V,y}$ = 0.87 Factor related to anchor spacing and edge distance CEB [9.3.4]
$\psi_{h,V,y}$ = 1.00 Factor related to the foundation thickness CEB [9.3.4.(c)]
$\psi_{s,V,y}$ = 0.96 Factor related to the influence of edges parallel to the shear load direction CEB [9.3.4.(d)]
$\psi_{ec,V,y}$ = 1.00 Factor taking account a group effect when different shear loads are acting on the individual anchors in a group CEB [9.3.4.(e)]
$\psi_{\alpha,V,y}$ = 1.00 Factor related to the angle at which the shear load is applied CEB [9.3.4.(f)]
$\psi_{ucr,V,y}$ = 1.00 Factor related to the type of edge reinforcement used CEB [9.3.4.(g)]

γ_{Mc} =	2.16		Partial safety factor	CEB [3.2.3.1]

$F_{v,Rd,c,y} = V_{Rk,c,y}{}^{0}*\psi_{A,V,y}*\psi_{h,V,y}*\psi_{s,V,y}*\psi_{ec,V,y}*\psi_{\alpha,V,y}*\psi_{ucr,V,y}/\gamma_{Mc}$

$F_{v,Rd,c,y}$ =	17.15	[kN]	Concrete resistance for edge failure	CEB [9.3.1]

Shear force $V_{j,Ed,z}$

$V_{Rk,c,z}{}^{0}$ =	66.03	[kN]	Characteristic resistance of an anchor	CEB [9.3.4.(a)]
$\psi_{A,V,z}$ =	0.51		Factor related to anchor spacing and edge distance	CEB [9.3.4]
$\psi_{h,V,z}$ =	1.00		Factor related to the foundation thickness	CEB [9.3.4.(c)]
$\psi_{s,V,z}$ =	0.85		Factor related to the influence of edges parallel to the shear load direction	CEB [9.3.4.(d)]
$\psi_{ec,V,z}$ =	1.00		Factor taking account a group effect when different shear loads are acting on the individual anchors in a group	CEB [9.3.4.(e)]
$\psi_{\alpha,V,z}$ =	1.00		Factor related to the angle at which the shear load is applied	CEB [9.3.4.(f)]
$\psi_{ucr,V,z}$ =	1.00		Factor related to the type of edge reinforcement used	CEB [9.3.4.(g)]
γ_{Mc} =	2.16		Partial safety factor	CEB [3.2.3.1]

$F_{v,Rd,c,z} = V_{Rk,c,z}{}^{0}*\psi_{A,V,z}*\psi_{h,V,z}*\psi_{s,V,z}*\psi_{ec,V,z}*\psi_{\alpha,V,z}*\psi_{ucr,V,z}/\gamma_{Mc}$

$F_{v,Rd,c,z}$ =	13.33	[kN]	Concrete resistance for edge failure	CEB [9.3.1]

SPLITTING RESISTANCE

$C_{f,d}$ =	0.30		Coeff. of friction between the base plate and concrete	[6.2.2.(6)]
$N_{c,Ed}$ =	1077.93	[kN]	Compressive force	[6.2.2.(6)]

$F_{f,Rd} = C_{f,d}*N_{c,Ed}$

$F_{f,Rd}$ =	323.38	[kN]	Slip resistance	[6.2.2.(6)]

SHEAR CHECK

$V_{j,Rd,y} = n_b*\min(F_{1,vb,Rd,y},F_{2,vb,Rd},F_{v,Rd,sm},F_{v,Rd,cp},F_{v,Rd,c,y}) + F_{f,Rd}$

$V_{j,Rd,y}$ =	327.54 [kN]	Connection resistance for shear			CEB [9.3.1]
$V_{j,Ed,y} / V_{j,Rd,y} \leq 1,0$			0.00 < 1.00	verified	(0.00)

$V_{j,Rd,z} = n_b*\min(F_{1,vb,Rd,z},F_{2,vb,Rd},F_{v,Rd,sm},F_{v,Rd,cp},F_{v,Rd,c,z}) + F_{f,Rd}$

$V_{j,Rd,z}$ =	327.54 [kN]	Connection resistance for shear			CEB [9.3.1]
$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$			0.21 < 1.00	verified	(0.21)
$V_{j,Ed,y} / V_{j,Rd,y} + V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$			0.21 < 1.00	verified	(0.21)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp}$ =	72.44 [MPa]	Normal stress in a weld			[4.5.3.(7)]
$\tau_{\perp}$ =	72.44 [MPa]	Perpendicular tangent stress			[4.5.3.(7)]
τ_{yII} =	0.00 [MPa]	Tangent stress parallel to $V_{j,Ed,y}$			[4.5.3.(7)]
τ_{zII} =	21.82 [MPa]	Tangent stress parallel to $V_{j,Ed,z}$			[4.5.3.(7)]
β_W =	0.90	Resistance-dependent coefficient			[4.5.3.(7)]
$\sigma_{\perp} / (0.9*f_u/\gamma_{M2}) \leq 1.0$ (4.1)			0.21 < 1.00	verified	(0.21)
$\sqrt{(\sigma_{\perp}{}^2 + 3.0 (\tau_{yII}{}^2 + \tau_{\perp}{}^2))} / (f_u/(\beta_W*\gamma_{M2})) \leq 1.0$ (4.1)			0.35 < 1.00	verified	(0.35)
$\sqrt{(\sigma_{\perp}{}^2 + 3.0 (\tau_{zII}{}^2 + \tau_{\perp}{}^2))} / (f_u/(\beta_W*\gamma_{M2})) \leq 1.0$ (4.1)			0.36 < 1.00	verified	(0.36)

WEAKEST COMPONENT:

FOUNDATION - BEARING PRESSURE ONTO CONCRETE

REMARKS

Anchor curvature radius is too small.	15 [mm] < 24 [mm]
Segment L4 of the hook anchor is too short.	30 [mm] < 40 [mm]

Connection conforms to the code	Ratio 0.98

Annex 2. Calculation of beam-beam joint in software ROBOT.

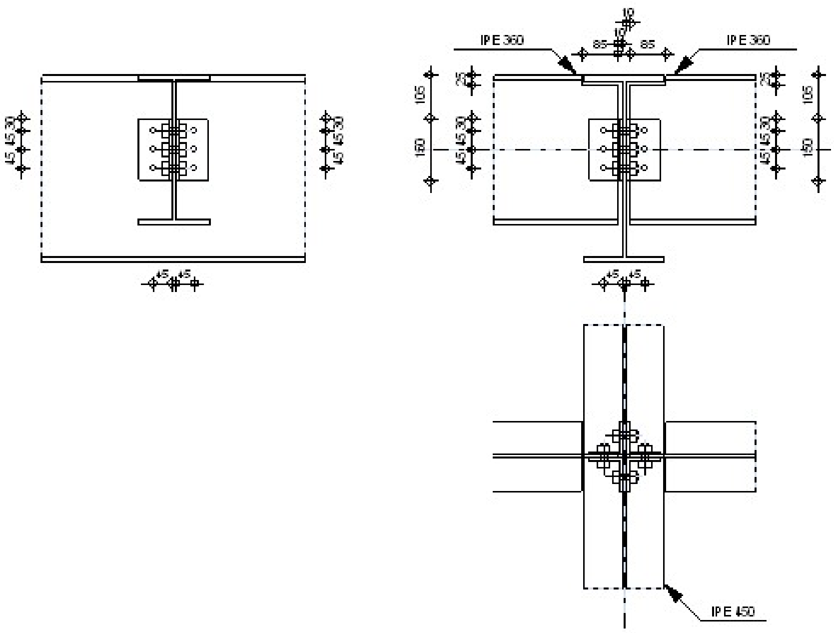
Autodesk Robot Structural Analysis Professional 2013-Student Version

Calculation of the beam-to-beam (web) connection

EN 1993-1-8:2005/AC:2009

OK

Ratio
0.87



GENERAL

Connection no.:	9
Connection name:	Beam-beam (web)
Structure node:	68
Structure bars:	58, 68, 67

GEOMETRY

PRINCIPAL BEAM

Section: IPE 450
Bar no.: 58

α =	-90.0	[Deg]	Inclination angle
h_g =	450	[mm]	Height of the principal beam section
b_{fg} =	190	[mm]	Width of the flange of the principal beam section
t_{wg} =	9	[mm]	Thickness of the web of the principal beam section
t_{fg} =	15	[mm]	Thickness of the flange of the principal beam section
r_g =	21	[mm]	Fillet radius of the web of the principal beam section
A_p =	98.82	[cm^2]	Cross-sectional area of a principal beam
I_{yp} =	33742.90	[cm^4]	Moment of inertia of the principal beam section

Material: S 355

f_{yg} =	355.00	[MPa]	Design resistance
f_{ug} =	470.00	[MPa]	Tensile resistance

LEFT SIDE

BEAM

Section: IPE 360
Bar no.: 68

α =	0.0	[Deg]	Inclination angle
h_{bl} =	360	[mm]	Height of beam section
b_{bl} =	170	[mm]	Width of beam section
t_{wbl} =	8	[mm]	Thickness of the web of beam section
t_{fbl} =	13	[mm]	Thickness of the flange of beam section

r_{bl} =	18	[mm]	Radius of beam section fillet
A_b =	72.73	[cm^2]	Cross-sectional area of a beam
I_{ybl} =	16265.60	[cm^4]	Moment of inertia of the beam section
Material:	S 355		
f_{ybl} =	355.00	[MPa]	Design resistance
f_{ubl} =	470.00	[MPa]	Tensile resistance

BEAM CUT

h_1 =	25	[mm]	Top cut-out
h_2 =	0	[mm]	Bottom cut-out
l =	85	[mm]	Cut-out length

ANGLE

Section:	CAE 80x8		
α =	0.0	[Deg]	Inclination angle
h_{kl} =	80	[mm]	Height of angle section
b_{kl} =	80	[mm]	Width of angle section
t_{fkl} =	8	[mm]	Thickness of the flange of angle section
r_{kl} =	10	[mm]	Fillet radius of the web of angle section
l_{kl} =	150	[mm]	Angle length
Material:	S 355		
f_{ykl} =	355.00	[MPa]	Design resistance
f_{ukl} =	470.00	[MPa]	Tensile resistance

BOLTS

BOLTS CONNECTING ANGLE WITH BEAM

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	5.8		Bolt class
d =	16	[mm]	Bolt diameter
d_0 =	18	[mm]	Bolt opening diameter
A_s =	1.57	[cm^2]	Effective section area of a bolt
A_v =	2.01	[cm^2]	Area of bolt section
f_{ub} =	500.00	[MPa]	Tensile resistance
k =	1		Number of bolt columns
w =	3		Number of bolt rows
e_1 =	30	[mm]	Level of first bolt
p_1 =	45	[mm]	Vertical spacing

RIGHT SIDE

BEAM

Section:	IPE 360		
Bar no.:	67		
α =	0.0	[Deg]	Inclination angle
h_{br} =	360	[mm]	Height of beam section
b_{br} =	170	[mm]	Width of beam section
t_{wbr} =	8	[mm]	Thickness of the web of beam section
t_{fbr} =	13	[mm]	Thickness of the flange of beam section
r_{br} =	18	[mm]	Radius of beam section fillet
A_{br} =	72.73	[cm^2]	Cross-sectional area of a beam
I_{ybr} =	16265.60	[cm^4]	Moment of inertia of the beam section
Material:	S 355		
f_{ybr} =	355.00	[MPa]	Design resistance
f_{ubr} =	470.00	[MPa]	Tensile resistance

BEAM CUT

h_1 =	25	[mm]	Top cut-out

h_2 =	0	[mm]	Bottom cut-out
l =	85	[mm]	Cut-out length

ANGLE

Section: CAE 80x8

h_{kr} =	80	[mm]	Height of angle section
b_{kr} =	80	[mm]	Width of angle section
t_{fkr} =	8	[mm]	Thickness of the flange of angle section
r_{kr} =	10	[mm]	Fillet radius of the web of angle section
l_{kr} =	150	[mm]	Angle length

Material: S 355

f_{ykr} =	355.00	[MPa]	Design resistance
f_{ukr} =	470.00	[MPa]	Tensile resistance

BOLTS

BOLTS CONNECTING ANGLE WITH PRINCIPAL BEAM

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	5.8		Bolt class
d =	16	[mm]	Bolt diameter
d_0 =	18	[mm]	Bolt opening diameter
A_s =	1.57	[cm^2]	Effective section area of a bolt
A_v =	2.01	[cm^2]	Area of bolt section
f_{ub} =	500.00	[MPa]	Tensile resistance
k =	1		Number of bolt columns
w =	3		Number of bolt rows
e_1 =	30	[mm]	Level of first bolt
p_1 =	45	[mm]	Vertical spacing

BOLTS CONNECTING ANGLE WITH BEAM

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	5.8		Bolt class
d =	16	[mm]	Bolt diameter
d_0 =	18	[mm]	Bolt opening diameter
A_s =	1.57	[cm^2]	Effective section area of a bolt
A_v =	2.01	[cm^2]	Area of bolt section
f_{ub} =	500.00	[MPa]	Tensile resistance
k =	1		Number of bolt columns
w =	3		Number of bolt rows
e_1 =	30	[mm]	Level of first bolt
p_1 =	45	[mm]	Vertical spacing

MATERIAL FACTORS

γ_{M0} =	1.00		Partial safety factor	[2.2]
γ_{M2} =	1.25		Partial safety factor	[2.2]

LOADS

Case: 11: COMB1 (1+2+3+4+5)*1.35+(6+7)*1.50+10*1.05+8*0.90

LEFT SIDE

$N_{b2,Ed}$ =	0.06	[kN]	Axial force
$V_{b2,Ed}$ =	86.37	[kN]	Shear force
$M_{b2,Ed}$ =	0.00	[kN*m]	Bending moment

RIGHT SIDE

$N_{b1,Ed}$ =	-0.06	[kN]	Axial force
$V_{b1,Ed}$ =	86.37	[kN]	Shear force

$M_{b1,Ed}$ =	0.00	[kN*m]	Bending moment

RESULTS

LEFT SIDE

BOLTS CONNECTING ANGLE WITH PRINCIPAL BEAM

BOLT CAPACITIES

$F_{v,Rd}$ =	48.25	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$
$F_{t,Rd}$ =	56.52	[kN]	Tensile resistance of a single bolt	$F_{t,Rd}= 0.9*f_u*A_s/\gamma_{M2}$

Bolt bearing on the angle

Direction x

k_{1x} =	1.80		Coefficient for calculation of $F_{b,Rd}$		$k_{1x}=\min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$			1.80 > 0.00	verified	
α_{bx} =	0.65		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bx}=\min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$			0.65 > 0.00	verified	
$F_{b,Rd2x}$ =	56.15	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2x}=k_{1x}*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		$k_{1z}=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$			2.50 > 0.00	verified	
α_{bz} =	0.56		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$			0.56 > 0.00	verified	
$F_{b,Rd2z}$ =	66.84	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

FORCES ACTING ON BOLTS IN THE PRINCIPAL BEAM - ANGLE CONNECTION

Bolt shear

e =	49	[mm]	Distance between centroid of a bolt group of an angle and center of the beam web		
M_0 =	2.12	[kN*m]	Real bending moment		$M_0=0.5*V_{b2,Ed}*e$
F_{Vz} =	14.40	[kN]	Component force in a bolt due to influence of the shear force		$F_{Vz}=0.5*V_{b1,Ed}/n$
F_{Mx} =	23.51	[kN]	Component force in a bolt due to influence of the moment		$F_{Mx}=M_0*z_i/\Sigma z_i^2$
$F_{x2,Ed}$ =	23.51	[kN]	Design total force in a bolt on the direction x		$F_{x2,Ed} = F_{Mx}$
$F_{z2,Ed}$ =	14.40	[kN]	Design total force in a bolt on the direction z		$F_{z2,Ed} = F_{Vz} + F_{Mz}$
F_{Rdx} =	48.25	[kN]	Effective design capacity of a bolt on the direction x		$F_{Rdx}=\min(F_{vRd}, F_{bRd2x})$
F_{Rdz} =	48.25	[kN]	Effective design capacity of a bolt on the direction z		$F_{Rdz}=\min(F_{vRd}, F_{bRd2z})$
$\lvert F_{x2,Ed}\rvert \le F_{Rdx}$			|23.51| < 48.25	verified	(0.49)
$\lvert F_{z2,Ed}\rvert \le F_{Rdz}$			|14.40| < 48.25	verified	(0.30)

Bolt tension

e =	50	[mm]	Distance between centroid of a bolt group and center of the principal beam web		
M_{0t} =	2.15	[kN*m]	Real bending moment		$M_{0t}=0.5*V_{b2,Ed}*e$
$F_{t,Ed}$ =	23.86	[kN]	Tensile force in the outermost bolt		$F_{t,Ed}=M_{0t}*z_{max}/\Sigma z_i^2 + 0.5*N_{b2,Ed}/n$
$F_{t,Ed} \le F_{tRd}$			23.86 < 56.52	verified	(0.42)

Simultaneous action of a tensile force and a shear force in a bolt

$F_{v,Ed}$ =	27.57	[kN]	Resultant shear force in a bolt		$F_{v,Ed} = \sqrt{[F_{x,Ed}^2 + F_{z,Ed}^2]}$
$F_{v,Ed}/F_{vRd} + F_{t,Ed}/(1.4*F_{t,Rd}) \le 1.0$			0.87 < 1.00	verified	(0.87)

BOLTS CONNECTING ANGLE WITH BEAM

BOLT CAPACITIES

$F_{v,Rd}$ =	96.51	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the beam

Direction x

k_{1x} =	1.80	Coefficient for calculation of $F_{b,Rd}$	$k_{1x} = \min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$

$k_{1x} > 0.0$			1.80 > 0.00	verified	
α_{bx} =	0.65		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bx}=\min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$			0.65 > 0.00	verified	
$F_{b,Rd1x}$ =	56.15	[kN]	Bearing resistance of a single bolt		$F_{b,Rd1x}=k_{1x}*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		$k_{1z}=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$			2.50 > 0.00	verified	
α_{bz} =	0.58		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$			0.58 > 0.00	verified	
$F_{b,Rd1z}$ =	70.19	[kN]	Bearing resistance of a single bolt		$F_{b,Rd1z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Bolt bearing on the angle

Direction x

k_{1x} =	1.80		Coefficient for calculation of $F_{b,Rd}$		$k_{1x}=\min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$			1.80 > 0.00	verified	
α_{bx} =	0.65		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bx}=\min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$			0.65 > 0.00	verified	
$F_{b,Rd2x}$ =	112.30	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2x}=k_{1x}*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		$k_{1z}=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$			2.50 > 0.00	verified	
α_{bz} =	0.56		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$			0.56 > 0.00	verified	
$F_{b,Rd2z}$ =	133.69	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Forces acting on bolts in the angle - beam connection

Bolt shear

e =	50	[mm]	Distance between centroid of a bolt group and center of the principal beam web		
M_0 =	4.29	[kN*m]	Real bending moment		$M_0=M_{b2,Ed}+V_{b2,Ed}*e$
F_{Nx} =	0.02	[kN]	Component force in a bolt due to influence of the longitudinal force		$F_{Nx}=N_{b2,Ed}/n$
F_{Vz} =	28.79	[kN]	Component force in a bolt due to influence of the shear force		$F_{Vz}=V_{b2,Ed}/n$
F_{Mx} =	47.70	[kN]	Component force in a bolt due to influence of the moment on the x direction		$F_{Mx}=M_0*z_i/\Sigma(x_i^2+z_i^2)$
F_{Mz} =	0.00	[kN]	Component force in a bolt due to influence of the moment on the z direction		$F_{Mz}=M_0*x_i/\Sigma(x_i^2+z_i^2)$
$F_{x,Ed}$ =	47.72	[kN]	Design total force in a bolt on the direction x		$F_{x,Ed} = F_{Nx} + F_{Mx}$
$F_{z2,Ed}$ =	28.79	[kN]	Design total force in a bolt on the direction z		$F_{z2,Ed} = F_{Vz} + F_{Mz}$
F_{Rdx} =	56.15	[kN]	Effective design capacity of a bolt on the direction x		$F_{Rdx}=\min(F_{vRd}, F_{bRd1x}, F_{bRd2x})$
F_{Rdz} =	70.19	[kN]	Effective design capacity of a bolt on the direction z		$F_{Rdz}=\min(F_{vRd}, F_{bRd1z}, F_{bRd2z})$
$\lvert F_{x,Ed}\rvert \le F_{Rdx}$			|47.72| < 56.15	verified	(0.85)
$\lvert F_{z,Ed}\rvert \le F_{Rdz}$			|28.79| < 70.19	verified	(0.41)

Verification of the section due to block tearing

Angle

A_{nt} =	2.08	[cm^2]	Net area of the section in tension		
A_{nv} =	6.00	[cm^2]	Area of the section in shear		
V_{effRd} =	162.08	[kN]	Design capacity of a section weakened by openings		$V_{effRd}=0.5*f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$
$\lvert 0.5*V_{b2,Ed}\rvert \le V_{effRd}$			|43.19| < 162.08	verified	(0.27)

Beam

A_{nt} =	2.08	[cm^2]	Net area of the section in tension		
A_{nv} =	12.40	[cm^2]	Area of the section in shear		
V_{effRd} =	293.25	[kN]	Design capacity of a section weakened by openings		$V_{effRd}=0.5*f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$
$\lvert V_{b2,Ed}\rvert \le V_{effRd}$			|86.37| < 293.25	verified	(0.29)

Right side

BOLTS CONNECTING ANGLE WITH PRINCIPAL BEAM

BOLT CAPACITIES

$F_{v,Rd}$ =	48.25	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}$= 0.6*f_{ub}*A_v*m/γ_{M2}
$F_{t,Rd}$ =	56.52	[kN]	Tensile resistance of a single bolt	$F_{t,Rd}$= 0.9*f_u*A_s/γ_{M2}

Bolt bearing on the angle

Direction x

k_{1x} =	1.80		Coefficient for calculation of $F_{b,Rd}$		k_{1x}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1x} > 0.0			1.80 > 0.00	verified	
α_{bx} =	0.65		Coefficient for calculation of $F_{b,Rd}$		α_{bx}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bx} > 0.0			0.65 > 0.00	verified	
$F_{b,Rd2x}$ =	56.15	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		k_{1z}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1z} > 0.0			2.50 > 0.00	verified	
α_{bz} =	0.56		Coefficient for calculation of $F_{b,Rd}$		α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bz} > 0.0			0.56 > 0.00	verified	
$F_{b,Rd2z}$ =	66.84	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

FORCES ACTING ON BOLTS IN THE PRINCIPAL BEAM - ANGLE CONNECTION

Bolt shear

e =	49	[mm]	Distance between centroid of a bolt group of an angle and center of the beam web		
M_0 =	2.12	[kN*m]	Real bending moment		M_0=0.5*$V_{b2,Ed}$*e
F_{Vz} =	14.40	[kN]	Component force in a bolt due to influence of the shear force		F_{Vz}=0.5*$V_{b2,Ed}$/n
F_{Mx} =	23.51	[kN]	Component force in a bolt due to influence of the moment		F_{Mx}=M_0*z_i/Σz_i^2
$F_{x1,Ed}$ =	23.51	[kN]	Design total force in a bolt on the direction x		$F_{x1,Ed}$ = F_{Mx}
$F_{z1,Ed}$ =	14.40	[kN]	Design total force in a bolt on the direction z		$F_{z1,Ed}$ = F_{Vz} + F_{Mz}
F_{Rdx} =	48.25	[kN]	Effective design capacity of a bolt on the direction x		F_{Rdx}=min(F_{vRd}, F_{bRd2x})
F_{Rdz} =	48.25	[kN]	Effective design capacity of a bolt on the direction z		F_{Rdz}=min(F_{vRd}, F_{bRd2z})
$\lvert F_{x1,Ed}\rvert \le F_{Rdx}$			|23.51| < 48.25	verified	(0.49)
$\lvert F_{z1,Ed}\rvert \le F_{Rdz}$			|14.40| < 48.25	verified	(0.30)

Bolt tension

e =	50	[mm]	Distance between centroid of a bolt group and center of the principal beam web		
M_{0t} =	2.15	[kN*m]	Real bending moment		M_{0t}=0.5*$V_{b1,Ed}$*e
$F_{t,Ed}$ =	23.84	[kN]	Tensile force in the outermost bolt		$F_{t,Ed}$=M_{0t}*z_{max}/Σz_i^2 + 0.5*$N_{b2,Ed}$/n
$F_{t,Ed} \le F_{t,Rd}$			23.84 < 56.52	verified	(0.42)

Simultaneous action of a tensile force and a shear force in a bolt

$F_{v,Ed}$ =	27.57	[kN]	Resultant shear force in a bolt		$F_{v,Ed}$ = √[$F_{x,Ed}^2$ + $F_{z,Ed}^2$]
$F_{v,Ed}/F_{v,Rd} + F_{t,Ed}/(1.4*F_{t,Rd}) \le 1.0$			0.87 < 1.00	verified	(0.87)

BOLTS CONNECTING ANGLE WITH BEAM

BOLT CAPACITIES

$F_{v,Rd}$ =	96.51	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}$= 0.6*f_{ub}*A_v*m/γ_{M2}

Bolt bearing on the beam

Direction x

k_{1x} =	1.80		Coefficient for calculation of $F_{b,Rd}$		k_{1x} = min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1x} > 0.0			1.80 > 0.00	verified	
α_{bx} =	0.65		Coefficient for calculation of $F_{b,Rd}$		α_{bx}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bx} > 0.0			0.65 > 0.00	verified	
$F_{b,Rd1x}$ =	56.15	[kN]	Bearing resistance of a single bolt		$F_{b,Rd1x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		k_{1z}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1z} > 0.0			2.50 > 0.00	verified	
α_{bz} =	0.58		Coefficient for calculation of $F_{b,Rd}$		α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bz} > 0.0			0.58 > 0.00	verified	
$F_{b,Rd1z}$ =	70.19	[kN]	Bearing resistance of a single bolt		$F_{b,Rd1z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Bolt bearing on the angle

Direction x

k_{1x} =	1.80		Coefficient for calculation of $F_{b,Rd}$		k_{1x}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1x} > 0.0			1.80 > 0.00	verified	
α_{bx} =	0.65		Coefficient for calculation of $F_{b,Rd}$		α_{bx}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bx} > 0.0			0.65 > 0.00	verified	
$F_{b,Rd2x}$ =	112.30	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		k_{1z}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1z} > 0.0			2.50 > 0.00	verified	
α_{bz} =	0.56		Coefficient for calculation of $F_{b,Rd}$		α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bz} > 0.0			0.56 > 0.00	verified	
$F_{b,Rd2z}$ =	133.69	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Forces acting on bolts in the angle - beam connection

Bolt shear

e =	50	[mm]	Distance between centroid of a bolt group and center of the principal beam web		
M_0 =	4.29	[kN*m]	Real bending moment		M_0=$V_{b1,Ed}$*e
F_{Nx} =	0.02	[kN]	Component force in a bolt due to influence of the longitudinal force		F_{Nx}=$N_{b1,Ed}$/n
F_{Vz} =	28.79	[kN]	Component force in a bolt due to influence of the shear force		F_{Vz}=$V_{b1,Ed}$/n
F_{Mx} =	47.70	[kN]	Component force in a bolt due to influence of the moment on the x direction		F_{Mx}=M_0*z_i/Σ(x_i^2+z_i^2)
F_{Mz} =	0.00	[kN]	Component force in a bolt due to influence of the moment on the z direction		F_{Mz}=M_0*x_i/Σ(x_i^2+z_i^2)
$F_{x,Ed}$ =	47.72	[kN]	Design total force in a bolt on the direction x		$F_{x,Ed}$ = F_{Nx} + F_{Mx}
$F_{z1,Ed}$ =	28.79	[kN]	Design total force in a bolt on the direction z		$F_{z1,Ed}$ = F_{Vz} + F_{Mz}
F_{Rdx} =	56.15	[kN]	Effective design capacity of a bolt on the direction x		F_{Rdx}=min(F_{vRd}, F_{bRd1x}, F_{bRd2x})
F_{Rdz} =	70.19	[kN]	Effective design capacity of a bolt on the direction z		F_{Rdz}=min(F_{vRd}, F_{bRd1z}, F_{bRd2z})
$\lvert F_{x,Ed}\rvert \le F_{Rdx}$			|47.72| < 56.15	verified	(0.85)
$\lvert F_{z,Ed}\rvert \le F_{Rdz}$			|28.79| < 70.19	verified	(0.41)

Verification of the section due to block tearing

Angle

A_{nt} =	2.08	[cm^2]	Net area of the section in tension		
A_{nv} =	6.00	[cm^2]	Area of the section in shear		
V_{effRd} =	162.08	[kN]	Design capacity of a section weakened by openings		V_{effRd}=0.5*f_u*A_{nt}/γ_{M2} + (1/√3)*f_y*A_{nv}/γ_{M0}
$\lvert 0.5 V_{b1,Ed}\rvert \le V_{effRd}$			|43.19| < 162.08	verified	(0.27)

Beam

A_{nt} =	2.08	[cm^2]	Net area of the section in tension		
A_{nv} =	12.40	[cm^2]	Area of the section in shear		
V_{effRd} =	293.25	[kN]	Design capacity of a section weakened by openings		V_{effRd}=0.5*f_u*A_{nt}/γ_{M2} + (1/√3)*f_y*A_{nv}/γ_{M0}
$\lvert V_{b1,Ed}\rvert \le V_{effRd}$			|86.37| < 293.25	verified	(0.29)

Verification of principal beam

Bolt bearing on the principal beam web

Direction x

k_x =	1.80		Coefficient for calculation of $F_{b,Rd}$		k_x = min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_x > 0.0			1.80 > 0.00	verified	
α_{bx} =	1.00		Coefficient for calculation of $F_{b,Rd}$		α_{bx}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_b > 0.0			1.00 > 0.00	verified	

$F_{b,Rdx}$ =	101.79	[kN]	Bearing resistance of a single bolt		$F_{b,Rdx}=k_x*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

k_z =	2.50		Coefficient for calculation of $F_{b,Rd}$		$k_z=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_z > 0.0$			2.50 > 0.00	verified	
α_{bz} =	0.58		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$			0.58 > 0.00	verified	
$F_{b,Rdz}$ =	82.47	[kN]	Bearing resistance of a single bolt		$F_{b,Rdz}=k_z*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

Resultant force acting on the outermost bolt

$F_{x,Ed}$ =	47.03	[kN]	Design total force in a bolt on the direction x		$F_{x,Ed} = F_{x1,Ed} + F_{x2,Ed}$
$F_{z,Ed}$ =	28.79	[kN]	Design total force in a bolt on the direction z		$F_{z,Ed} = F_{z1,Ed} + F_{z2,Ed}$
$\lvert F_{x,Ed}\rvert \le F_{b,Rdx}$			|47.03| < 101.79	verified	(0.46)
$\lvert F_{z,Ed}\rvert \le F_{b,Rdz}$			|28.79| < 82.47	verified	(0.35)

Connection conforms to the code	Ratio 0.87

Annex 3. Calculation of column (flange) to beam joint in software ROBOT.

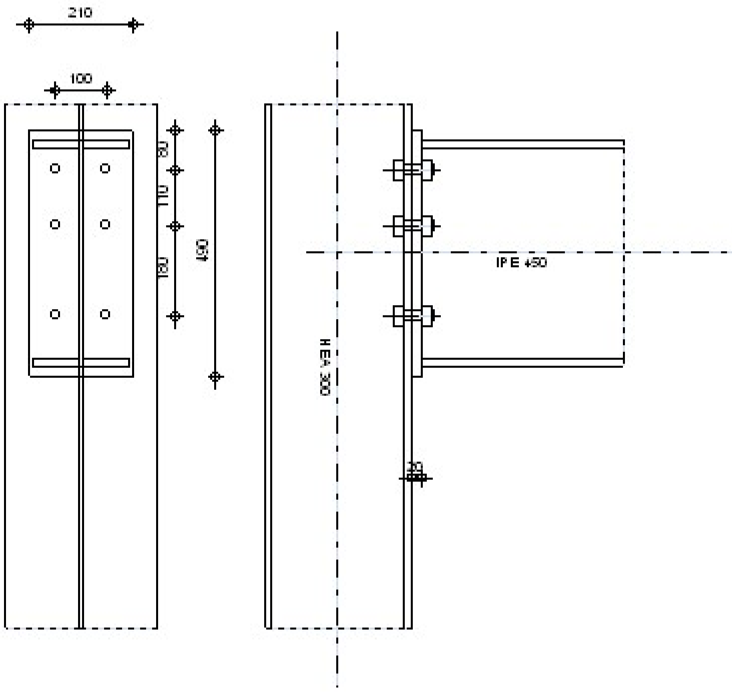
Autodesk Robot Structural Analysis Professional 2013-Student Version

Design of fixed beam-to-column connection

EN 1993-1-8:2005/AC:2009

OK

Ratio
0.92



GENERAL

Connection no.: 19
Connection name: Column-Beam
Structure node: 31
Structure bars: 17, 62

GEOMETRY

COLUMN

Section: HEA 300
Bar no.: 17

α =	-90.0	[Deg]	Inclination angle
h_c =	290	[mm]	Height of column section
b_{fc} =	300	[mm]	Width of column section
t_{wc} =	9	[mm]	Thickness of the web of column section
t_{fc} =	14	[mm]	Thickness of the flange of column section
r_c =	27	[mm]	Radius of column section fillet
A_c =	112.53	[cm^2]	Cross-sectional area of a column
I_{xc} =	18263.50	[cm^4]	Moment of inertia of the column section

Material: S 355

f_{yc} =	355.00	[MPa]	Resistance

BEAM

Section: IPE 450
Bar no.: 62

α =	-0.0	[Deg]	Inclination angle

h_b =	450	[mm]	Height of beam section
b_f =	190	[mm]	Width of beam section
t_{wb} =	9	[mm]	Thickness of the web of beam section
t_{fb} =	15	[mm]	Thickness of the flange of beam section
r_b =	21	[mm]	Radius of beam section fillet
r_b =	21	[mm]	Radius of beam section fillet
A_b =	98.82	[cm^2]	Cross-sectional area of a beam
I_{xb} =	33742.90	[cm^4]	Moment of inertia of the beam section
Material:	S 355		
f_{yb} =	355.00	[MPa]	Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.

d =	20	[mm]	Bolt diameter
Class =	9.8		Bolt class
F_{tRd} =	158.76	[kN]	Tensile resistance of a bolt
n_h =	2		Number of bolt columns
n_v =	3		Number of bolt rows
h_1 =	80	[mm]	Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 100 [mm]

Vertical spacing p_i = 110;180 [mm]

PLATE

h_p =	490	[mm]	Plate height
b_p =	210	[mm]	Plate width
t_p =	20	[mm]	Plate thickness
Material:	S 355		
f_{yp} =	355.00	[MPa]	Resistance

FILLET WELDS

a_w =	7	[mm]	Web weld
a_f =	11	[mm]	Flange weld

MATERIAL FACTORS

γ_{M0} =	1.00	Partial safety factor	[2.2]
γ_{M1} =	1.00	Partial safety factor	[2.2]
γ_{M2} =	1.25	Partial safety factor	[2.2]
γ_{M3} =	1.25	Partial safety factor	[2.2]

LOADS

Ultimate limit state

Case: 15: COMB5 (1+2+3+4+5)*1.35+8*1.50+(6+7+10)*1.05

$M_{b1,Ed}$ =	156.52	[kN*m]	Bending moment in the right beam
$V_{b1,Ed}$ =	78.98	[kN]	Shear force in the right beam
$N_{b1,Ed}$ =	-4.32	[kN]	Axial force in the right beam
$M_{b2,Ed}$ =	-10.67	[kN*m]	Bending moment in the left beam
$V_{b2,Ed}$ =	17.64	[kN]	Shear force in the left beam
$N_{b2,Ed}$ =	1.35	[kN]	Axial force in the left beam
$M_{c1,Ed}$ =	-116.79	[kN*m]	Bending moment in the lower column

$V_{c1,Ed}$ = -29.20 [kN] Shear force in the lower column
$N_{c1,Ed}$ = -575.28 [kN] Axial force in the lower column
$M_{c2,Ed}$ = 50.30 [kN*m] Bending moment in the upper column
$V_{c2,Ed}$ = 27.49 [kN] Shear force in the upper column
$N_{c2,Ed}$ = -398.93 [kN] Axial force in the upper column

RESULTS

BEAM RESISTANCES

COMPRESSION

A_b = 98.82 [cm^2] Area EN1993-1-1:[6.2.4]
$N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$
$N_{cb,Rd}$ = 3508.14 [kN] Design compressive resistance of the section EN1993-1-1:[6.2.4]

SHEAR

A_{vb} = 50.85 [cm^2] Shear area EN1993-1-1:[6.2.6.(3)]
$V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3}) / \gamma_{M0}$
$V_{cb,Rd}$ = 1042.12 [kN] Design sectional resistance for shear EN1993-1-1:[6.2.6.(2)]
$V_{b1,Ed} / V_{cb,Rd} \le 1,0$ 0.08 < 1.00 verified (0.08)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W_{plb} = 1701.92 [cm^3] Plastic section modulus EN1993-1-1:[6.2.5.(2)]
$M_{b,pl,Rd} = W_{plb} f_{yb} / \gamma_{M0}$
$M_{b,pl,Rd}$ = 604.18 [kN*m] Plastic resistance of the section for bending (without stiffeners) EN1993-1-1:[6.2.5.(2)]

BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

W_{pl} = 1701.92 [cm^3] Plastic section modulus EN1993-1-1:[6.2.5]
$M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$
$M_{cb,Rd}$ = 604.18 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd}$ = 604.18 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]
h_f = 435 [mm] Distance between the centroids of flanges [6.2.6.7.(1)]
$F_{c,fb,Rd} = M_{cb,Rd} / h_f$
$F_{c,fb,Rd}$ = 1387.65 [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

COLUMN RESISTANCES

WEB PANEL - SHEAR

$M_{b1,Ed}$ = 156.52 [kN*m] Bending moment (right beam) [5.3.(3)]
$M_{b2,Ed}$ = -10.67 [kN*m] Bending moment (left beam) [5.3.(3)]
$V_{c1,Ed}$ = -29.20 [kN] Shear force (lower column) [5.3.(3)]
$V_{c2,Ed}$ = 27.49 [kN] Shear force (upper column) [5.3.(3)]
z = 328 [mm] Lever arm [6.2.5]
$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$
$V_{wp,Ed}$ = 538.51 [kN] Shear force acting on the web panel [5.3.(3)]

A_{vs} = 37.28 [cm^2] Shear area of the column web EN1993-1-1:[6.2.6.(3)]
A_{vc} = 37.28 [cm^2] Shear area EN1993-1-1:[6.2.6.(3)]
$V_{wp,Rd} = 0.9*(f_{y,wc}*A_{vc}+f_{y,wp}*A_{vp}+f_{ys}*A_{vd}) / (\sqrt{3}\ \gamma_{M0})$

$V_{wp,Rd}$ = 687.64 [kN] Resistance of the column web panel for shear [6.2.6.1]

$V_{wp,Ed} / V_{wp,Rd} \leq 1,0$ 0.78 < 1.00 verified (0.78)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

t_{wc} =	9	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	291	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	37.28	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0.78		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	117.63	[MPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
k_{wc} =	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]

$F_{c,wc,Rd1} = \omega\ k_{wc}\ b_{eff,c,wbc}\ t_{wc}\ f_{yc} / \gamma_{M0}$

$F_{c,wc,Rd1}$ = 685.12 [kN] Column web resistance [6.2.6.2.(1)]

Buckling:

d_{wc} =	208	[mm]	Height of compressed web	[6.2.6.2.(1)]
λ_p =	1.11		Plate slenderness of an element	[6.2.6.2.(1)]
ρ =	0.74		Reduction factor for element buckling	[6.2.6.2.(1)]

$F_{c,wb,Rd2} = \omega\ k_{wc}\ \rho\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M1}$

$F_{c,wc,Rd2}$ = 506.52 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,low} = \mathrm{Min}\ (F_{c,wc,Rd1}\ ,\ F_{c,wc,Rd2})$

$F_{c,wc,Rd}$ = 506.52 [kN] Column web resistance [6.2.6.2.(1)]

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

Bearing:

t_{wc} =	9	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	291	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	37.28	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0.78		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	117.63	[MPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
k_{wc} =	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]

$F_{c,wc,Rd1} = \omega\ k_{wc}\ b_{eff,c,wbc}\ t_{wc}\ f_{yc} / \gamma_{M0}$

$F_{c,wc,Rd1}$ = 685.12 [kN] Column web resistance [6.2.6.2.(1)]

Buckling:

d_{wc} =	208	[mm]	Height of compressed web	[6.2.6.2.(1)]
λ_p =	1.11		Plate slenderness of an element	[6.2.6.2.(1)]
ρ =	0.74		Reduction factor for element buckling	[6.2.6.2.(1)]

$F_{c,wb,Rd2} = \omega\ k_{wc}\ \rho\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M1}$

$F_{c,wc,Rd2}$ = 506.52 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,upp} = \mathrm{Min}\ (F_{c,wc,Rd1}\ ,\ F_{c,wc,Rd2})$

$F_{c,wc,Rd,upp}$ = 506.52 [kN] Column web resistance [6.2.6.2.(1)]

GEOMETRICAL PARAMETERS OF A CONNECTION

EFFECTIVE LENGTHS AND PARAMETERS - COLUMN FLANGE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	24	-	100	-	110	152	191	152	191	186	135	135	135
2	24	-	100	-	145	152	222	152	222	220	110	110	110
3	24	-	100	-	180	152	222	152	222	256	201	201	201

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	37	-	55	-	110	235	253	235	253	227	199	199	199
2	37	-	55	-	145	235	218	218	218	290	145	145	145
3	37	-	55	-	180	235	218	218	218	297	199	199	199

m – Bolt distance from the web
m_x – Bolt distance from the beam flange
e – Bolt distance from the outer edge
e_x – Bolt distance from the horizontal outer edge
p – Distance between bolts
$l_{eff,cp}$ – Effective length for a single bolt in the circular failure mode
$l_{eff,nc}$ – Effective length for a single bolt in the non-circular failure mode
$l_{eff,1}$ – Effective length for a single bolt for mode 1
$l_{eff,2}$ – Effective length for a single bolt for mode 2
$l_{eff,cp,g}$ – Effective length for a group of bolts in the circular failure mode
$l_{eff,nc,g}$ – Effective length for a group of bolts in the non-circular failure mode
$l_{eff,1,g}$ – Effective length for a group of bolts for mode 1
$l_{eff,2,g}$ – Effective length for a group of bolts for mode 2

CONNECTION RESISTANCE FOR COMPRESSION

$N_{j,Rd} = \mathrm{Min}\,(N_{cb,Rd}, 2F_{c,wc,Rd,low}, 2F_{c,wc,Rd,upp})$

$N_{j,Rd} =$ 1013.05 [kN] Connection resistance for compression [6.2]

$N_{b1,Ed} / N_{j,Rd} \le 1{,}0$ 0.00 < 1.00 verified (0.00)

CONNECTION RESISTANCE FOR BENDING

$F_{t,Rd} =$ 158.76 [kN] Bolt resistance for tension [Table 3.4]

$B_{p,Rd} =$ 297.67 [kN] Punching shear resistance of a bolt [Table 3.4]

$F_{t,fc,Rd}$ – column flange resistance due to bending
$F_{t,wc,Rd}$ – column web resistance due to tension
$F_{t,ep,Rd}$ – resistance of the front plate due to bending
$F_{t,wb,Rd}$ – resistance of the web in tension

$F_{t,fc,Rd} = \mathrm{Min}\,(F_{T,1,fc,Rd}, F_{T,2,fc,Rd}, F_{T,3,fc,Rd})$ [6.2.6.4] , [Tab.6.2]

$F_{t,wc,Rd} = \omega\, b_{eff,t,wc}\, t_{wc}\, f_{yc} / \gamma_{M0}$ [6.2.6.3.(1)]

$F_{t,ep,Rd} = \mathrm{Min}\,(F_{T,1,ep,Rd}, F_{T,2,ep,Rd}, F_{T,3,ep,Rd})$ [6.2.6.5] , [Tab.6.2]

$F_{t,wb,Rd} = b_{eff,t,wb}\, t_{wb}\, f_{yb} / \gamma_{M0}$ [6.2.6.8.(1)]

RESISTANCE OF THE BOLT ROW NO. 1

$F_{t1,Rd,comp}$ - Formula	$F_{t1,Rd,comp}$	Component
$F_{t,fc,Rd(1)} = 298.56$	298.56	Column flange - tension
$F_{t,wc,Rd(1)} = 425.93$	425.93	Column web - tension
$F_{t,ep,Rd(1)} = 317.52$	317.52	Front plate - tension
$F_{t,wb,Rd(1)} = 783.75$	783.75	Beam web - tension
$B_{p,Rd} = 595.34$	595.34	Bolts due to shear punching
$V_{wp,Rd}/\beta = 643.77$	643.77	Web panel - shear
$F_{c,wc,Rd} = 506.52$	506.52	Column web - compression
$F_{c,fb,Rd} = 1387.65$	1387.65	Beam flange - compression
$F_{t1,Rd} = \text{Min}(F_{t1,Rd,comp})$	298.56	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t,fc,Rd(2)} = 317.52$	317.52	Column flange - tension
$F_{t,wc,Rd(2)} = 425.93$	425.93	Column web - tension
$F_{t,ep,Rd(2)} = 317.52$	317.52	Front plate - tension
$F_{t,wb,Rd(2)} = 728.37$	728.37	Beam web - tension
$B_{p,Rd} = 595.34$	595.34	Bolts due to shear punching
$V_{wp,Rd}/\beta - \Sigma_1^1 F_{ti,Rd} = 643.77 - 298.56$	345.21	Web panel - shear
$F_{c,wc,Rd} - \Sigma_1^1 F_{tj,Rd} = 506.52 - 298.56$	207.96	Column web - compression
$F_{c,fb,Rd} - \Sigma_1^1 F_{tj,Rd} = 1387.65 - 298.56$	1089.09	Beam flange - compression
$F_{t,fc,Rd(2+1)} - \Sigma_1^1 F_{tj,Rd} = 509.66 - 298.56$	211.10	Column flange - tension - group
$F_{t,wc,Rd(2+1)} - \Sigma_1^1 F_{tj,Rd} = 623.54 - 298.56$	324.98	Column web - tension - group
$F_{t,ep,Rd(2+1)} - \Sigma_1^1 F_{tj,Rd} = 635.04 - 298.56$	336.48	Front plate - tension - group
$F_{t,wb,Rd(2+1)} - \Sigma_1^1 F_{tj,Rd} = 1148.08 - 298.56$	849.52	Beam web - tension - group
$F_{t2,Rd} = \text{Min}(F_{t2,Rd,comp})$	207.96	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 3

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t,fc,Rd(3)} = 317.52$	317.52	Column flange - tension
$F_{t,wc,Rd(3)} = 425.93$	425.93	Column web - tension
$F_{t,ep,Rd(3)} = 317.52$	317.52	Front plate - tension
$F_{t,wb,Rd(3)} = 728.37$	728.37	Beam web - tension
$B_{p,Rd} = 595.34$	595.34	Bolts due to shear punching
$V_{wp,Rd}/\beta - \Sigma_1^2 F_{ti,Rd} = 643.77 - 506.52$	137.25	Web panel - shear
$F_{c,wc,Rd} - \Sigma_1^2 F_{tj,Rd} = 506.52 - 506.52$	0.00	Column web - compression
$F_{c,fb,Rd} - \Sigma_1^2 F_{tj,Rd} = 1387.65 - 506.52$	881.12	Beam flange - compression
$F_{t,fc,Rd(3+2)} - \Sigma_2^2 F_{tj,Rd} = 551.79 - 207.96$	343.83	Column flange - tension - group
$F_{t,wc,Rd(3+2)} - \Sigma_2^2 F_{tj,Rd} = 729.47 - 207.96$	521.51	Column web - tension - group
$F_{t,fc,Rd(3+2+1)} - \Sigma_2^1 F_{tj,Rd} = 814.63 - 506.52$	308.10	Column flange - tension - group
$F_{t,wc,Rd(3+2+1)} - \Sigma_2^1 F_{tj,Rd} = 878.78 - 506.52$	372.25	Column web - tension - group
$F_{t,ep,Rd(3+2)} - \Sigma_2^2 F_{tj,Rd} = 635.04 - 207.96$	427.08	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \Sigma_2^2 F_{tj,Rd} = 1148.38 - 207.96$	940.42	Beam web - tension - group

$F_{t,ep,Rd(3+2+1)} - \Sigma_2^1 F_{tj,Rd} = 952.56 - 506.52$	446.04	Front plate - tension - group
$F_{t,wb,Rd(3+2+1)} - \Sigma_2^1 F_{tj,Rd} = 1812.60 - 506.52$	1306.08	Beam web - tension - group
$F_{t3,Rd} = \mathrm{Min}\,(F_{t3,Rd,comp})$	0.00	Bolt row resistance

SUMMARY TABLE OF FORCES

Nr	h_j	$F_{tj,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
1	383	298.56	298.56	425.93	317.52	783.75	317.52	595.34
2	273	207.96	317.52	425.93	317.52	728.37	317.52	595.34
3	93	-	317.52	425.93	317.52	728.37	317.52	595.34

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$M_{j,Rd} = \Sigma h_j F_{tj,Rd}$

$M_{j,Rd} =$ 170.97 [kN*m] Connection resistance for bending [6.2]

$M_{b1,Ed} / M_{j,Rd} \le 1,0$ 0.92 < 1.00 verified (0.92)

CONNECTION RESISTANCE FOR SHEAR

$\alpha_v =$	0.60		Coefficient for calculation of $F_{v,Rd}$	[Table 3.4]
$F_{v,Rd} =$	135.72	[kN]	Shear resistance of a single bolt	[Table 3.4]
$F_{t,Rd,max} =$	158.76	[kN]	Tensile resistance of a single bolt	[Table 3.4]
$F_{b,Rd,int} =$	263.20	[kN]	Bearing resistance of an intermediate bolt	[Table 3.4]
$F_{b,Rd,ext} =$	263.20	[kN]	Bearing resistance of an outermost bolt	[Table 3.4]

Nr	$F_{tj,Rd,N}$	$F_{tj,Ed,N}$	$F_{tj,Rd,M}$	$F_{tj,Ed,M}$	$F_{tj,Ed}$	$F_{vj,Rd}$
1	317.52	-1.44	298.56	273.32	271.88	105.42
2	317.52	-1.44	207.96	190.38	188.94	156.07
3	317.52	-1.44	317.52	0.00	-1.44	271.43

$F_{tj,Rd,N}$ – Bolt row resistance for simple tension
$F_{tj,Ed,N}$ – Force due to axial force in a bolt row
$F_{tj,Rd,M}$ – Bolt row resistance for simple bending
$F_{tj,Ed,M}$ – Force due to moment in a bolt row
$F_{tj,Ed}$ – Maximum tensile force in a bolt row
$F_{vj,Rd}$ – Reduced bolt row resistance

$F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}$

$F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}$

$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$

$F_{vj,Rd} = \mathrm{Min}\,(n_h F_{v,Rd} (1 - F_{tj,Ed}/ (1.4 n_h F_{t,Rd,max}), n_h F_{v,Rd} , n_h F_{b,Rd}))$

$V_{j,Rd} = n_h \Sigma_1^n F_{vj,Rd}$ [Table 3.4]

$V_{j,Rd} =$ 532.92 [kN] Connection resistance for shear [Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \le 1,0$ 0.15 < 1.00 verified (0.15)

WELD RESISTANCE

$A_w =$ 115.19 [cm^2] Area of all welds [4.5.3.2(2)]

$A_{wy} =$ 62.16 [cm^2] Area of horizontal welds [4.5.3.2(2)]

A_{wz} = 53.03 [cm^2] Area of vertical welds [4.5.3.2(2)]
I_{wy} = 36281.76 [cm^4] Moment of inertia of the weld arrangement with respect to the hor. axis [4.5.3.2(5)]
$\sigma_{\perp max}=\tau_{\perp max}$ = -70.69 [MPa] Normal stress in a weld [4.5.3.2(5)]
$\sigma_{\perp}=\tau_{\perp}$ = -58.15 [MPa] Stress in a vertical weld [4.5.3.2(5)]
τ_{II} = 14.89 [MPa] Tangent stress [4.5.3.2(5)]
β_w = 0.90 Correlation coefficient [4.5.3.2(7)]

$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$ 141.37 < 417.78 verified (0.34)
$\sqrt{[\sigma_{\perp}^2 + 3*(\tau_{\perp}^2+\tau_{II}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$ 119.13 < 417.78 verified (0.29)
$\sigma_{\perp} \leq 0.9*f_u/\gamma_{M2}$ 70.69 < 338.40 verified (0.21)

CONNECTION STIFFNESS

t_{wash} = 4 [mm] Washer thickness [6.2.6.3.(2)]
h_{head} = 14 [mm] Bolt head height [6.2.6.3.(2)]
h_{nut} = 20 [mm] Bolt nut height [6.2.6.3.(2)]
L_b = 59 [mm] Bolt length [6.2.6.3.(2)]
k_{10} = 7 [mm] Stiffness coefficient of bolts [6.3.2.(1)]

STIFFNESSES OF BOLT ROWS

Nr	hj	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j}\ h_j$	$k_{eff,j}\ h_j^2$
1	383	3	24	27	2	6.89	263.71
2	273	2	19	20	2	4.18	113.94
3	93	3	27	27	2	1.80	16.70
					Sum	12.87	394.34

$k_{eff,j} = 1 / (\Sigma_3^5 (1 / k_{i,j}))$ [6.3.3.1.(2)]

$z_{eq} = \Sigma_j k_{eff,j} h_j^2 / \Sigma_j k_{eff,j} h_j$
z_{eq} = 306 [mm] Equivalent force arm [6.3.3.1.(3)]

$k_{eq} = \Sigma_j k_{eff,j} h_j / z_{eq}$
k_{eq} = 4 [mm] Equivalent stiffness coefficient of a bolt arrangement [6.3.3.1.(1)]

A_{vc} = 37.28 [cm^2] Shear area EN1993-1-1:[6.2.6.(3)]
β = 1.07 Transformation parameter [5.3.(7)]
z = 306 [mm] Lever arm [6.2.5]
k_1 = 4 [mm] Stiffness coefficient of the column web panel subjected to shear [6.3.2.(1)]

$b_{eff,c,wc}$ = 275 [mm] Effective width of the web for compression [6.2.6.2.(1)]
t_{wc} = 9 [mm] Effective thickness of the column web [6.2.6.2.(6)]
d_c = 262 [mm] Height of compressed web [6.2.6.2.(1)]
k_2 = 6 [mm] Stiffness coefficient of the compressed column web [6.3.2.(1)]

$S_{j,ini} = E\ z_{eq}^2 / \Sigma_i (1 / k_1 + 1 / k_2 + 1 / k_{eq})$ [6.3.1.(4)]
$S_{j,ini}$ = 31337.20 [kN*m] Initial rotational stiffness [6.3.1.(4)]

μ = 2.35 Stiffness coefficient of a connection [6.3.1.(6)]
$S_j = S_{j,ini} / \mu$ [6.3.1.(4)]

S_j = 13310.58 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

$S_{j,rig}$ = 94480.12 [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,pin}$ = 5905.01 [kN*m] Stiffness of a pinned connection [5.2.2.5]

$S_{j,pin} \le S_{j,ini} < S_{j,rig}$ SEMI-RIGID

WEAKEST COMPONENT:

COLUMN WEB - COMPRESSION

REMARKS

Distance of bolts from an edge is too large. 120 [mm] > 120 [mm]

Connection conforms to the code	Ratio 0.92

Annex 4. Calculation of column (web) to beam joint in software ROBOT.

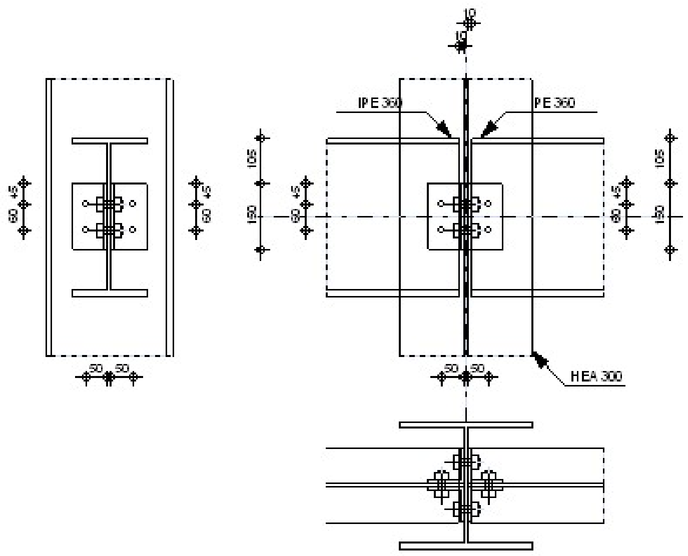
Autodesk Robot Structural Analysis Professional 2013-Student Version

Calculation of the beam-column (web) connection

EN 1993-1-8:2005/AC:2009

OK

Ratio
0.84



GENERAL

Connection no.: 7
Connection name: Beam-column (web)

GEOMETRY

COLUMN

Section: HEA 300

α =	-90.0	[Deg]	Inclination angle
h_c =	290	[mm]	Height of column section
b_{fc} =	300	[mm]	Width of column section
t_{wc} =	9	[mm]	Thickness of the web of column section
t_{fc} =	14	[mm]	Thickness of the flange of column section
r_c =	27	[mm]	Radius of column section fillet
A_c =	112.53	[cm^2]	Cross-sectional area of a column
I_{yc} =	18263.50	[cm^4]	Moment of inertia of the column section

Material: S 355

f_{yc} =	355.00	[MPa]	Design resistance
f_{uc} =	470.00	[MPa]	Tensile resistance

LEFT SIDE

BEAM

Section: IPE 360

α =	0.0	[Deg]	Inclination angle
h_{bl} =	360	[mm]	Height of beam section
b_{bl} =	170	[mm]	Width of beam section
t_{wbl} =	8	[mm]	Thickness of the web of beam section
t_{fbl} =	13	[mm]	Thickness of the flange of beam section
r_{bl} =	18	[mm]	Radius of beam section fillet
A_b =	72.73	[cm^2]	Cross-sectional area of a beam
I_{ybl} =	16265.60	[cm^4]	Moment of inertia of the beam section

Material: S 355

f_{ybl} =	355.00	[MPa]	Design resistance
f_{ubl} =	470.00	[MPa]	Tensile resistance

ANGLE

Section: CAE 80x8

α =	0.0	[Deg]	Inclination angle
h_{kl} =	80	[mm]	Height of angle section
b_{kl} =	80	[mm]	Width of angle section
t_{fkl} =	8	[mm]	Thickness of the flange of angle section
r_{kl} =	10	[mm]	Fillet radius of the web of angle section
l_{kl} =	150	[mm]	Angle length

Material: S 355

f_{ykl} =	355.00	[MPa]	Design resistance
f_{ukl} =	470.00	[MPa]	Tensile resistance

BOLTS

BOLTS CONNECTING ANGLE WITH BEAM

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	5.8		Bolt class
d =	16	[mm]	Bolt diameter
d_0 =	18	[mm]	Bolt opening diameter
A_s =	1.57	[cm^2]	Effective section area of a bolt
A_v =	2.01	[cm^2]	Area of bolt section
f_{ub} =	500.00	[MPa]	Tensile resistance
k =	1		Number of bolt columns
w =	2		Number of bolt rows
e_1 =	45	[mm]	Level of first bolt
p_1 =	60	[mm]	Vertical spacing

RIGHT SIDE

BEAM

Section: IPE 360

α =	0.0	[Deg]	Inclination angle
h_{br} =	360	[mm]	Height of beam section
b_{br} =	170	[mm]	Width of beam section
t_{wbr} =	8	[mm]	Thickness of the web of beam section
t_{fbr} =	13	[mm]	Thickness of the flange of beam section
r_{br} =	18	[mm]	Radius of beam section fillet
A_{br} =	72.73	[cm^2]	Cross-sectional area of a beam
I_{ybr} =	16265.60	[cm^4]	Moment of inertia of the beam section

Material: S 355

f_{ybr} =	355.00	[MPa]	Design resistance
f_{ubr} =	470.00	[MPa]	Tensile resistance

ANGLE

Section: CAE 80x8

h_{kr} =	80	[mm]	Height of angle section
b_{kr} =	80	[mm]	Width of angle section
t_{fkr} =	8	[mm]	Thickness of the flange of angle section
r_{kr} =	10	[mm]	Fillet radius of the web of angle section
l_{kr} =	150	[mm]	Angle length

Material: S 355

f_{ykr} =	355.00	[MPa]	Design resistance
f_{ukr} =	470.00	[MPa]	Tensile resistance

BOLTS

BOLTS CONNECTING COLUMN WITH ANGLE

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	5.8		Bolt class
d =	16	[mm]	Bolt diameter
d_0 =	18	[mm]	Bolt opening diameter
A_s =	1.57	[cm^2]	Effective section area of a bolt
A_v =	2.01	[cm^2]	Area of bolt section
f_{ub} =	500.00	[MPa]	Tensile resistance
k =	1		Number of bolt columns
w =	2		Number of bolt rows
e_1 =	45	[mm]	Level of first bolt
p_1 =	60	[mm]	Vertical spacing

BOLTS CONNECTING ANGLE WITH BEAM

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	5.8		Bolt class
d =	16	[mm]	Bolt diameter
d_0 =	18	[mm]	Bolt opening diameter
A_s =	1.57	[cm^2]	Effective section area of a bolt
A_v =	2.01	[cm^2]	Area of bolt section
f_{ub} =	500.00	[MPa]	Tensile resistance
k =	1		Number of bolt columns
w =	2		Number of bolt rows
e_1 =	45	[mm]	Level of first bolt
p_1 =	60	[mm]	Vertical spacing

MATERIAL FACTORS

γ_{M0} =	1.00	Partial safety factor	[2.2]
γ_{M2} =	1.25	Partial safety factor	[2.2]

LOADS

Case: Manual calculations.

LEFT SIDE

$N_{b2,Ed}$ =	23.44	[kN]	Axial force
$V_{b2,Ed}$ =	44.32	[kN]	Shear force
$M_{b2,Ed}$ =	0.00	[kN*m]	Bending moment

RIGHT SIDE

$N_{b1,Ed}$ =	37.38	[kN]	Axial force
$V_{b1,Ed}$ =	-44.32	[kN]	Shear force
$M_{b1,Ed}$ =	0.00	[kN*m]	Bending moment

RESULTS

LEFT SIDE

BOLTS CONNECTING COLUMN WITH ANGLE

BOLT CAPACITIES

$F_{v,Rd}$ =	48.25	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd} = 0.6*f_{ub}*A_v*m/\gamma_{M2}$
$F_{t,Rd}$ =	56.52	[kN]	Tensile resistance of a single bolt	$F_{t,Rd} = 0.9*f_u*A_s/\gamma_{M2}$

Bolt bearing on the angle

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$		k_{1x}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1x} > 0.0			2.50 > 0.00	verified	
α_{bx} =	0.56		Coefficient for calculation of $F_{b,Rd}$		α_{bx}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bx} > 0.0			0.56 > 0.00	verified	
$F_{b,Rd2x}$ =	66.84	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		k_{1z}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1z} > 0.0			2.50 > 0.00	verified	
α_{bz} =	0.83		Coefficient for calculation of $F_{b,Rd}$		α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bz} > 0.0			0.83 > 0.00	verified	
$F_{b,Rd2z}$ =	100.27	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Forces acting on bolts in the column - angle connection

Bolt shear

e =	54	[mm]	Distance between centroid of a bolt group of an angle and center of the beam web		
M_0 =	1.20	[kN*m]	Real bending moment		M_0=0.5*$V_{b2,Ed}$*e
F_{Vz} =	11.08	[kN]	Component force in a bolt due to influence of the shear force		F_{Vz}=0.5*$V_{b1,Ed}$/n
F_{Mx} =	19.94	[kN]	Component force in a bolt due to influence of the moment		F_{Mx}=M_0*$z_i/\Sigma z_i^2$
$F_{x2,Ed}$ =	19.94	[kN]	Design total force in a bolt on the direction x		$F_{x2,Ed}$ = F_{Mx}
$F_{z2,Ed}$ =	11.08	[kN]	Design total force in a bolt on the direction z		$F_{z2,Ed}$ = F_{Vz} + F_{Mz}
F_{Rdx} =	48.25	[kN]	Effective design capacity of a bolt on the direction x		F_{Rdx}=min(F_{vRd}, F_{bRd2x})
F_{Rdz} =	48.25	[kN]	Effective design capacity of a bolt on the direction z		F_{Rdz}=min(F_{vRd}, F_{bRd2z})
$\lvert F_{x2,Ed}\rvert \le F_{Rdx}$			|19.94| < 48.25	verified	(0.41)
$\lvert F_{z2,Ed}\rvert \le F_{Rdz}$			|11.08| < 48.25	verified	(0.23)

Bolt tension

e =	54	[mm]	Distance between centroid of a bolt group and center of column web		
M_{0t} =	1.20	[kN*m]	Real bending moment		M_{0t}=0.5*$V_{b2,Ed}$*e
$F_{t,Ed}$ =	25.90	[kN]	Tensile force in the outermost bolt		$F_{t,Ed}$=M_{0t}*$z_{max}/\Sigma z_i^2$ + 0.5*$N_{b2,Ed}$/n
$F_{t,Ed} \le F_{tRd}$			25.90 < 56.52	verified	(0.46)

Simultaneous action of a tensile force and a shear force in a bolt

$F_{v,Ed}$ =	22.82	[kN]	Resultant shear force in a bolt		$F_{v,Ed}$ = √[$F_{x,Ed}^2$ + $F_{z,Ed}^2$]
$F_{v,Ed}/F_{vRd}$ + $F_{t,Ed}$/(1.4*$F_{t,Rd}$) ≤ 1.0			0.80 < 1.00	verified	(0.80)

Bolts connecting angle with beam

Bolt capacities

$F_{v,Rd}$ =	96.51	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}$= 0.6*f_{ub}*A_v*m/γ_{M2}

Bolt bearing on the beam

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$		k_{1x} = min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1x} > 0.0			2.50 > 0.00	verified	
α_{bx} =	0.74		Coefficient for calculation of $F_{b,Rd}$		α_{bx}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bx} > 0.0			0.74 > 0.00	verified	
$F_{b,Rd1x}$ =	89.13	[kN]	Bearing resistance of a single bolt		$F_{b,Rd1x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		k_{1z}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1z} > 0.0			2.50 > 0.00	verified	
α_{bz} =	0.86		Coefficient for calculation of $F_{b,Rd}$		α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bz} > 0.0			0.86 > 0.00	verified	
$F_{b,Rd1z}$ =	103.61	[kN]	Bearing resistance of a single bolt		$F_{b,Rd1z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Bolt bearing on the angle

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1x} > 0.0			2.50 > 0.00	verified
α_{bx} =	0.56		Coefficient for calculation of $F_{b,Rd}$	α_{bx}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bx} > 0.0			0.56 > 0.00	verified
$F_{b,Rd2x}$ =	133.69	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1z} > 0.0			2.50 > 0.00	verified
α_{bz} =	0.83		Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bz} > 0.0			0.83 > 0.00	verified
$F_{b,Rd2z}$ =	200.53	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

FORCES ACTING ON BOLTS IN THE ANGLE - BEAM CONNECTION

Bolt shear

e =	54	[mm]	Distance between centroid of a bolt group and center of column web		
M_0 =	2.40	[kN*m]	Real bending moment		M_0=$M_{b2,Ed}$+$V_{b2,Ed}$*e
F_{Nx} =	11.72	[kN]	Component force in a bolt due to influence of the longitudinal force		F_{Nx}=$N_{b2,Ed}$/n
F_{Vz} =	22.16	[kN]	Component force in a bolt due to influence of the shear force		F_{Vz}=$V_{b2,Ed}$/n
F_{Mx} =	40.07	[kN]	Component force in a bolt due to influence of the moment on the x direction		F_{Mx}=M_0*z_i/Σ(x_i^2+z_i^2)
F_{Mz} =	0.00	[kN]	Component force in a bolt due to influence of the moment on the z direction		F_{Mz}=M_0*x_i/Σ(x_i^2+z_i^2)
$F_{x,Ed}$ =	51.79	[kN]	Design total force in a bolt on the direction x		$F_{x,Ed}$ = F_{Nx} + F_{Mx}
$F_{z2,Ed}$ =	22.16	[kN]	Design total force in a bolt on the direction z		$F_{z2,Ed}$ = F_{Vz} + F_{Mz}
F_{Rdx} =	89.13	[kN]	Effective design capacity of a bolt on the direction x		F_{Rdx}=min(F_{vRd}, F_{bRd1x}, F_{bRd2x})
F_{Rdz} =	96.51	[kN]	Effective design capacity of a bolt on the direction z		F_{Rdz}=min(F_{vRd}, F_{bRd1z}, F_{bRd2z})
$\lvert F_{x,Ed}\rvert \le F_{Rdx}$			|51.79| < 89.13	verified	(0.58)
$\lvert F_{z,Ed}\rvert \le F_{Rdz}$			|22.16| < 96.51	verified	(0.23)

VERIFICATION OF THE SECTION DUE TO BLOCK TEARING

ANGLE

A_{nt} =	1.68	[cm^2]	Net area of the section in tension		
A_{nv} =	6.24	[cm^2]	Area of the section in shear		
V_{effRd} =	159.48	[kN]	Design capacity of a section weakened by openings		V_{effRd}=0.5*f_u*A_{nt}/γ_{M2} + (1/√3)*f_y*A_{nv}/γ_{M0}
$\lvert 0.5*V_{b2,Ed}\rvert \le V_{effRd}$			|22.16| < 159.48	verified	(0.14)

BEAM

A_{nt} =	2.48	[cm^2]	Net area of the section in tension		
A_{nv} =	14.64	[cm^2]	Area of the section in shear		
V_{effRd} =	346.68	[kN]	Design capacity of a section weakened by openings		V_{effRd}=0.5*f_u*A_{nt}/γ_{M2} + (1/√3)*f_y*A_{nv}/γ_{M0}
$\lvert V_{b2,Ed}\rvert \le V_{effRd}$			|44.32| < 346.68	verified	(0.13)

RIGHT SIDE

BOLTS CONNECTING COLUMN WITH ANGLE

BOLT CAPACITIES

$F_{v,Rd}$ =	48.25	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}$= 0.6*f_{ub}*A_v*m/γ_{M2}
$F_{t,Rd}$ =	56.52	[kN]	Tensile resistance of a single bolt	$F_{t,Rd}$= 0.9*f_u*A_s/γ_{M2}

Bolt bearing on the angle

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1x} > 0.0			2.50 > 0.00	verified

α_{bx} =	0.56		Coefficient for calculation of $F_{b,Rd}$		α_{bx}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bx} > 0.0			0.56 > 0.00	verified	
$F_{b,Rd2x}$ =	66.84	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		k_{1z}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1z} > 0.0			2.50 > 0.00	verified	
α_{bz} =	0.83		Coefficient for calculation of $F_{b,Rd}$		α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bz} > 0.0			0.83 > 0.00	verified	
$F_{b,Rd2z}$ =	100.27	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

FORCES ACTING ON BOLTS IN THE COLUMN - ANGLE CONNECTION

Bolt shear

e =	54	[mm]	Distance between centroid of a bolt group of an angle and center of the beam web		
M_0 =	1.20	[kN*m]	Real bending moment		M_0=0.5*$V_{b2,Ed}$*e
F_{Vz} =	11.08	[kN]	Component force in a bolt due to influence of the shear force		F_{Vz}=0.5*$V_{b2,Ed}$/n
F_{Mx} =	19.94	[kN]	Component force in a bolt due to influence of the moment		F_{Mx}=M_0*z_i/Σz_i^2
$F_{x1,Ed}$ =	19.94	[kN]	Design total force in a bolt on the direction x		$F_{x1,Ed}$ = F_{Mx}
$F_{z1,Ed}$ =	11.08	[kN]	Design total force in a bolt on the direction z		$F_{z1,Ed}$ = F_{Vz} + F_{Mz}
F_{Rdx} =	48.25	[kN]	Effective design capacity of a bolt on the direction x		F_{Rdx}=min(F_{vRd}, F_{bRd2x})
F_{Rdz} =	48.25	[kN]	Effective design capacity of a bolt on the direction z		F_{Rdz}=min(F_{vRd}, F_{bRd2z})
$\|F_{x1,Ed}\| \le F_{Rdx}$			|19.94| < 48.25	verified	(0.41)
$\|F_{z1,Ed}\| \le F_{Rdz}$			|11.08| < 48.25	verified	(0.23)

Bolt tension

e =	54	[mm]	Distance between centroid of a bolt group and center of column web		
M_{0t} =	1.20	[kN*m]	Real bending moment		M_{0t}=0.5*$V_{b1,Ed}$*e
$F_{t,Ed}$ =	29.38	[kN]	Tensile force in the outermost bolt		$F_{t,Ed}$=M_{0t}*z_{max}/Σz_i^2 + 0.5*$N_{b2,Ed}$/n
$F_{t,Ed} \le F_{t,Rd}$			29.38 < 56.52	verified	(0.52)

Simultaneous action of a tensile force and a shear force in a bolt

$F_{v,Ed}$ =	22.82	[kN]	Resultant shear force in a bolt		$F_{v,Ed}$ = $\sqrt{[F_{x,Ed}^2 + F_{z,Ed}^2]}$
$F_{v,Ed}/F_{v,Rd} + F_{t,Ed}/(1.4*F_{t,Rd}) \le 1.0$			0.84 < 1.00	verified	(0.84)

BOLTS CONNECTING ANGLE WITH BEAM

BOLT CAPACITIES

$F_{v,Rd}$ =	96.51	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}$= 0.6*f_{ub}*A_v*m/γ_{M2}

Bolt bearing on the beam

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$		k_{1x} = min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1x} > 0.0			2.50 > 0.00	verified	
α_{bx} =	0.74		Coefficient for calculation of $F_{b,Rd}$		α_{bx}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{bx} > 0.0			0.74 > 0.00	verified	
$F_{b,Rd1x}$ =	89.13	[kN]	Bearing resistance of a single bolt		$F_{b,Rd1x}$=k_{1x}*α_{bx}*f_u*d*t_i/γ_{M2}

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		k_{1z}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1z} > 0.0			2.50 > 0.00	verified	
α_{bz} =	0.86		Coefficient for calculation of $F_{b,Rd}$		α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bz} > 0.0			0.86 > 0.00	verified	
$F_{b,Rd1z}$ =	103.61	[kN]	Bearing resistance of a single bolt		$F_{b,Rd1z}$=k_{1z}*α_{bz}*f_u*d*t_i/γ_{M2}

Bolt bearing on the angle

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]

$k_{1x} > 0.0$			2.50 > 0.00	verified	
α_{bx} =	0.56		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bx}=\min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$			0.56 > 0.00	verified	
$F_{b,Rd2x}$ =	133.69	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2x}=k_{1x}*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$		$k_{1z}=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$			2.50 > 0.00	verified	
α_{bz} =	0.83		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$			0.83 > 0.00	verified	
$F_{b,Rd2z}$ =	200.53	[kN]	Bearing resistance of a single bolt		$F_{b,Rd2z}=k_{1z}*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

FORCES ACTING ON BOLTS IN THE ANGLE - BEAM CONNECTION

Bolt shear

e =	54	[mm]	Distance between centroid of a bolt group and center of column web		
M_0 =	-2.40	[kN*m]	Real bending moment		$M_0=V_{b1,Ed}*e$
F_{Nx} =	18.69	[kN]	Component force in a bolt due to influence of the longitudinal force		$F_{Nx}=N_{b1,Ed}/n$
F_{Vz} =	22.16	[kN]	Component force in a bolt due to influence of the shear force		$F_{Vz}=V_{b1,Ed}/n$
F_{Mx} =	-40.07	[kN]	Component force in a bolt due to influence of the moment on the x direction		$F_{Mx}=M_0*z_i/\Sigma(x_i^2+z_i^2)$
F_{Mz} =	0.00	[kN]	Component force in a bolt due to influence of the moment on the z direction		$F_{Mz}=M_0*x_i/\Sigma(x_i^2+z_i^2)$
$F_{x,Ed}$ =	-21.38	[kN]	Design total force in a bolt on the direction x		$F_{x,Ed} = F_{Nx} + F_{Mx}$
$F_{z1,Ed}$ =	22.16	[kN]	Design total force in a bolt on the direction z		$F_{z1,Ed} = F_{Vz} + F_{Mz}$
F_{Rdx} =	89.13	[kN]	Effective design capacity of a bolt on the direction x		$F_{Rdx}=\min(F_{vRd}, F_{bRd1x}, F_{bRd2x})$
F_{Rdz} =	96.51	[kN]	Effective design capacity of a bolt on the direction z		$F_{Rdz}=\min(F_{vRd}, F_{bRd1z}, F_{bRd2z})$
$\lvert F_{x,Ed}\rvert \le F_{Rdx}$			|-21.38| < 89.13	verified	(0.24)
$\lvert F_{z,Ed}\rvert \le F_{Rdz}$			|22.16| < 96.51	verified	(0.23)

VERIFICATION OF THE SECTION DUE TO BLOCK TEARING

ANGLE

A_{nt} =	3.28	[cm^2]	Net area of the section in tension		
A_{nv} =	6.24	[cm^2]	Area of the section in shear		
V_{effRd} =	189.56	[kN]	Design capacity of a section weakened by openings		$V_{effRd}=0.5*f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$
$\lvert 0.5*V_{b1,Ed}\rvert \le V_{effRd}$			|-22.16| < 189.56	verified	(0.12)

BEAM

A_{nt} =	2.48	[cm^2]	Net area of the section in tension		
A_{nv} =	14.64	[cm^2]	Area of the section in shear		
V_{effRd} =	346.68	[kN]	Design capacity of a section weakened by openings		$V_{effRd}=0.5*f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$
$\lvert V_{b1,Ed}\rvert \le V_{effRd}$			|-44.32| < 346.68	verified	(0.13)

COLUMN VERIFICATION

BOLT BEARING ON THE COLUMN WEB

Direction x

k_x =	2.50		Coefficient for calculation of $F_{b,Rd}$		$k_x = \min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_x > 0.0$			2.50 > 0.00	verified	
α_{bx} =	1.00		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bx}=\min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_b > 0.0$			1.00 > 0.00	verified	
$F_{b,Rdx}$ =	127.84	[kN]	Bearing resistance of a single bolt		$F_{b,Rdx}=k_x*\alpha_{bx}*f_u*d*t_i/\gamma_{M2}$

Direction z

k_z =	2.50		Coefficient for calculation of $F_{b,Rd}$		$k_z=\min[2.8*(e_2/d_0)-1.7, 2.5]$
$k_z > 0.0$			2.50 > 0.00	verified	
α_{bz} =	0.86		Coefficient for calculation of $F_{b,Rd}$		$\alpha_{bz}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$			0.86 > 0.00	verified	
$F_{b,Rdz}$ =	110.08	[kN]	Bearing resistance of a single bolt		$F_{b,Rdz}=k_z*\alpha_{bz}*f_u*d*t_i/\gamma_{M2}$

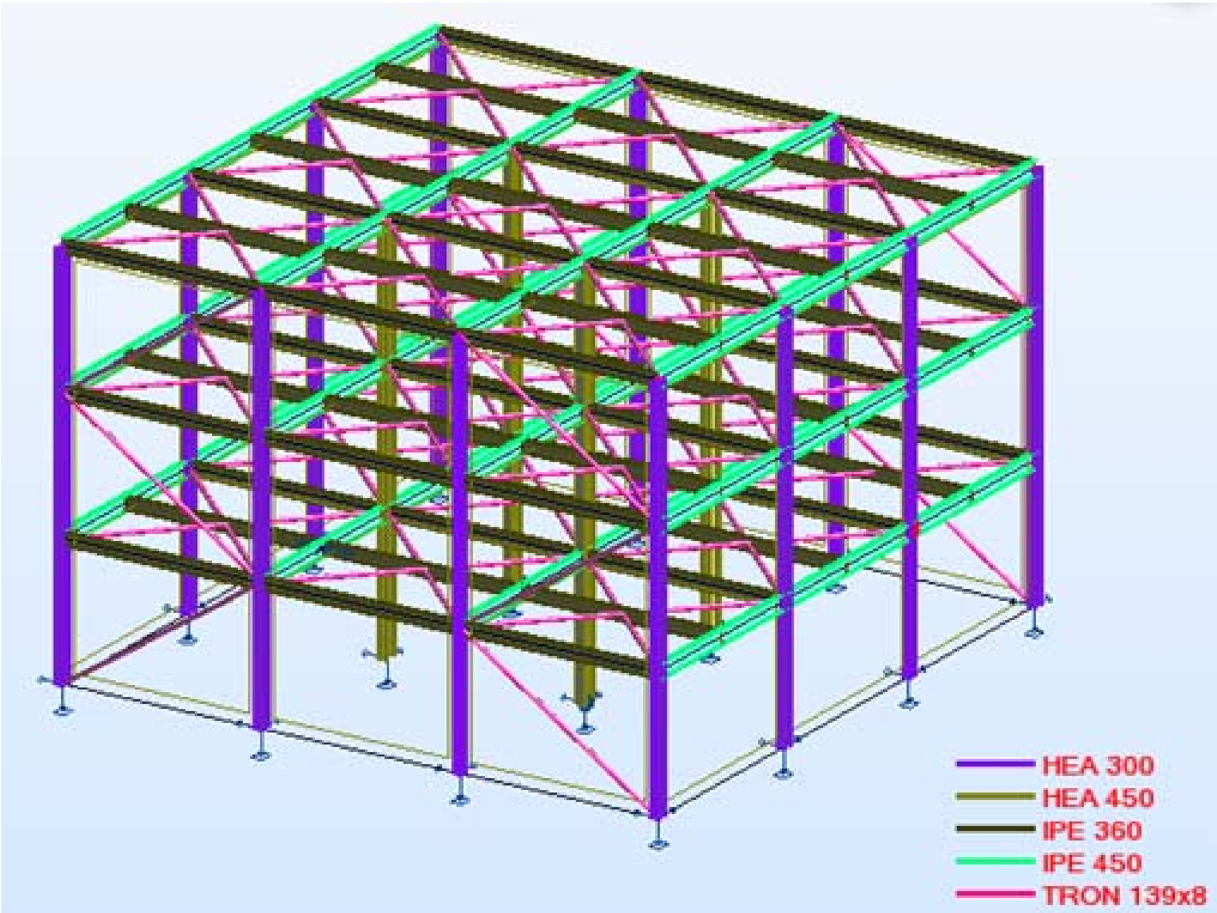
RESULTANT FORCE ACTING ON THE OUTERMOST BOLT

$F_{x,Ed}$ =	39.89	[kN]	Design total force in a bolt on the direction x		$F_{x,Ed} = F_{x1,Ed} + F_{x2,Ed}$
$F_{z,Ed}$ =	22.16	[kN]	Design total force in a bolt on the direction z		$F_{z,Ed} = F_{z1,Ed} + F_{z2,Ed}$
$\lvert F_{x,Ed}\rvert \le F_{b,Rdx}$			|39.89| < 127.84	verified	(0.31)
$\lvert F_{z,Ed}\rvert \le F_{b,Rdz}$			|22.16| < 110.08	verified	(0.20)

Connection conforms to the code Ratio 0.84

Annex 5. Drawn part (3D view, plan view, elevations, execution drawing for a column, execution drawing for a beam, joint details).

3D view



Performed by	Description
Maksym Podgayskyy	3D view
MD Refat Ahmed	
	Page 1

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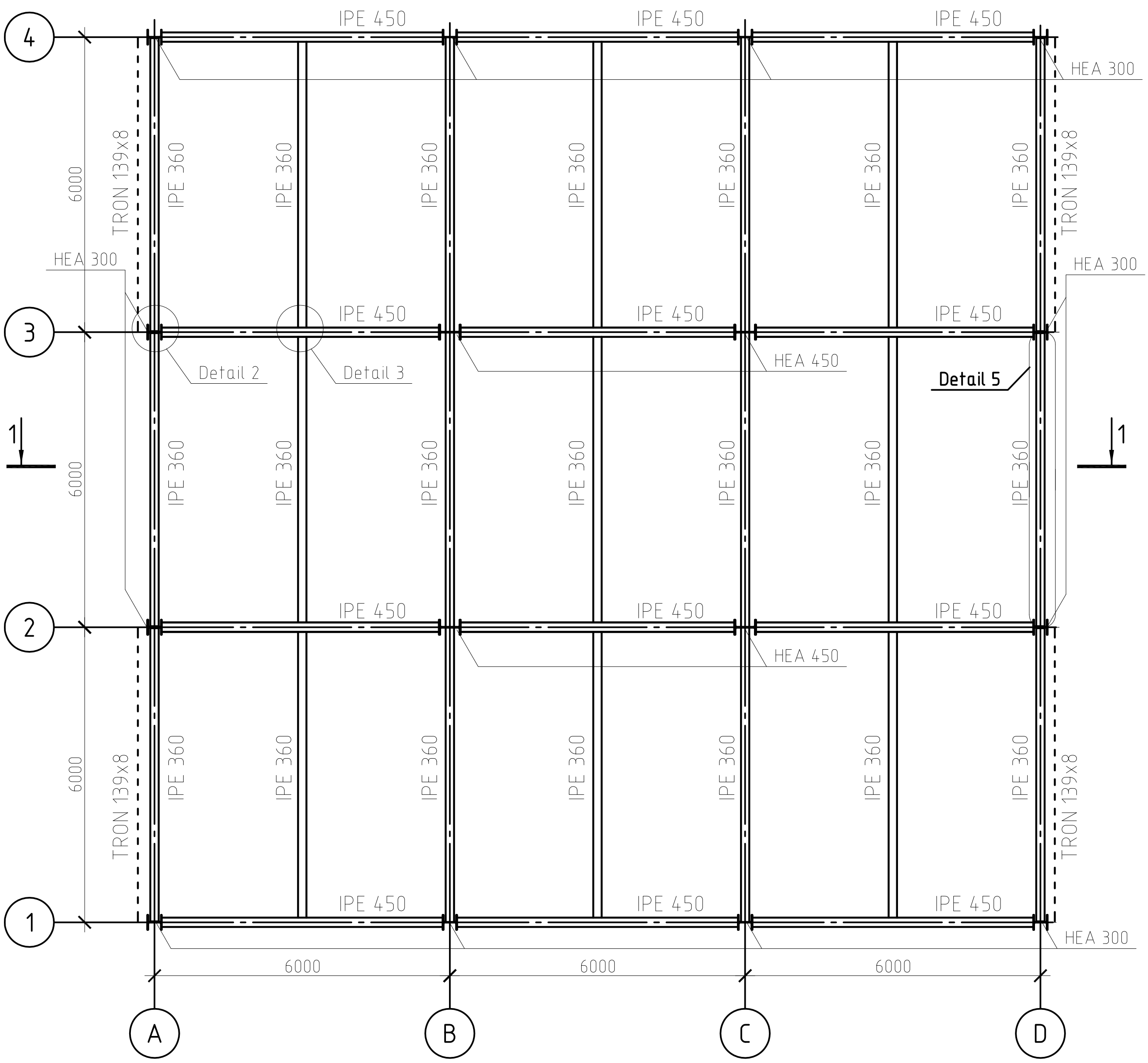
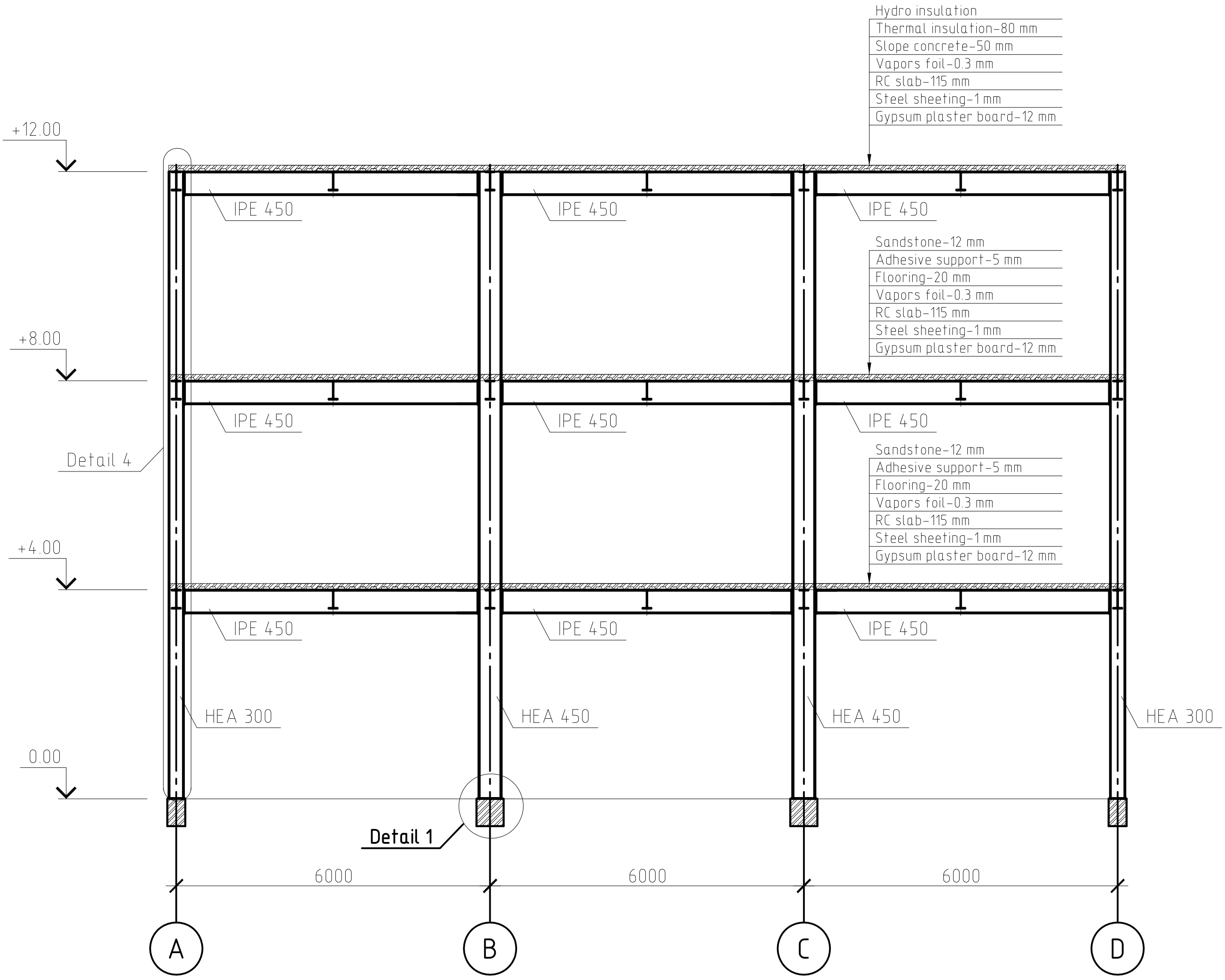
Plan at level +4.00, +8.00, +12.00



Performed by	Description
Maksym Podgayskyy	Plan at level +4.00, +8.00, +12.00
MD Refat Ahmed	
	Page 2

Section elevation 1-1



Performed by	Description
Maksym Podgayskyy	Section elevation 1-1
MD Refat Ahmed	
	Page 3

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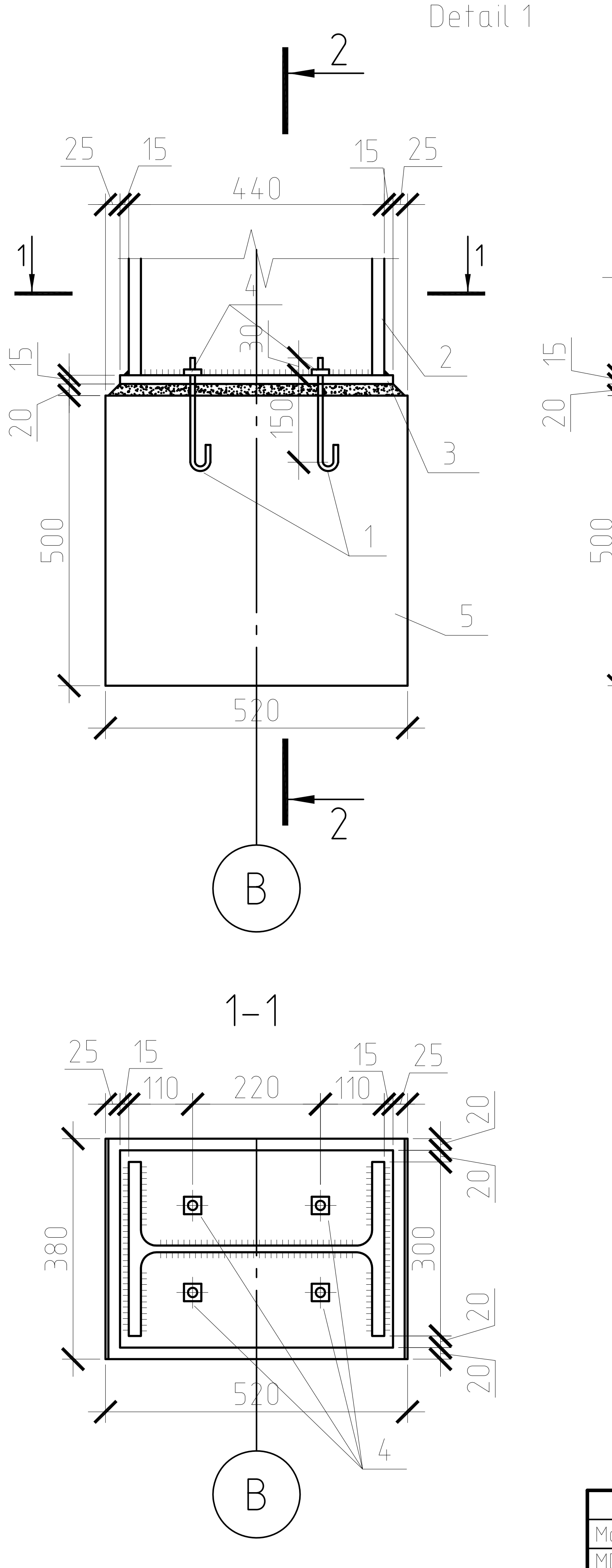
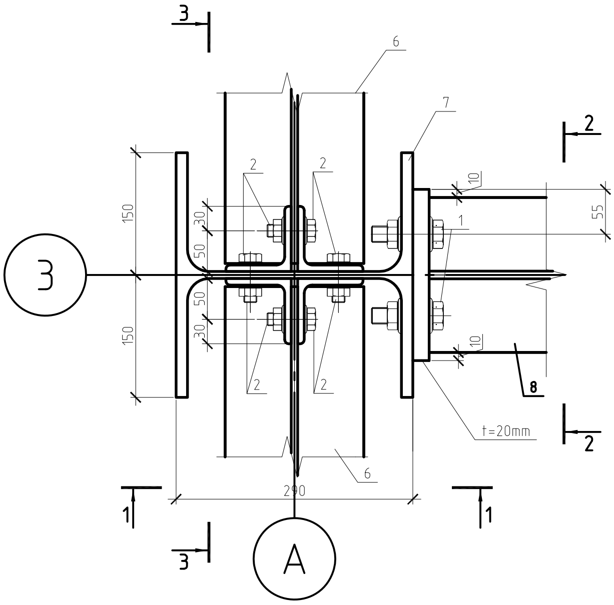


Nº	Description
1	D=8mm, class 4.6
2	HEA 450, S355
3	– 470x340x15, S355
4	Washer 30x30x10, S355
5	Concrete C20/25

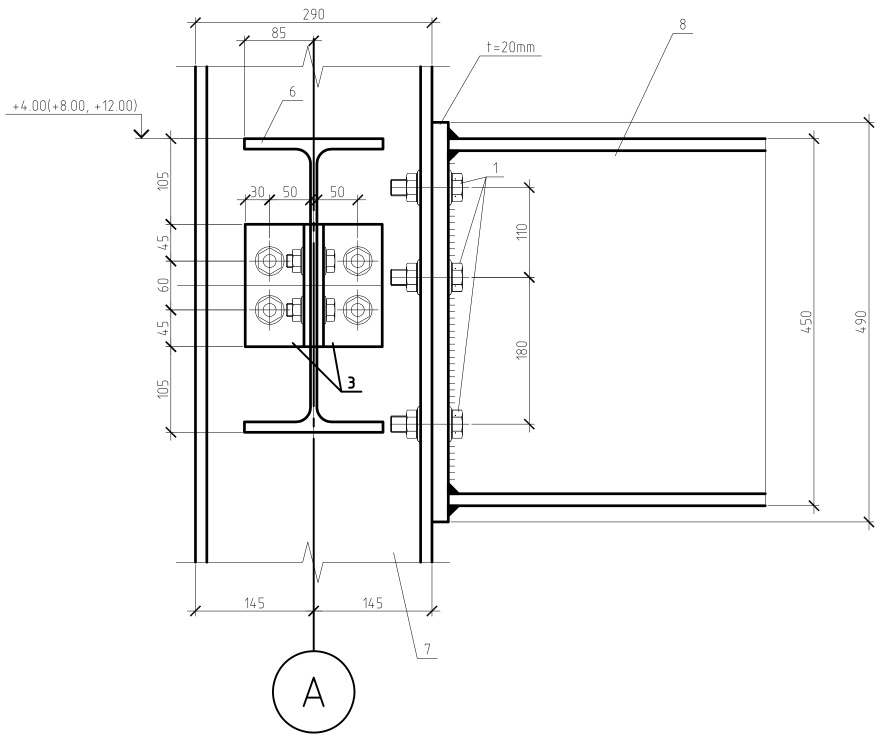
Performed by	Description
Maksym Podgayskyy	Detail 1
MD Refat Ahmed	Column base
	Page 4

Detail 2

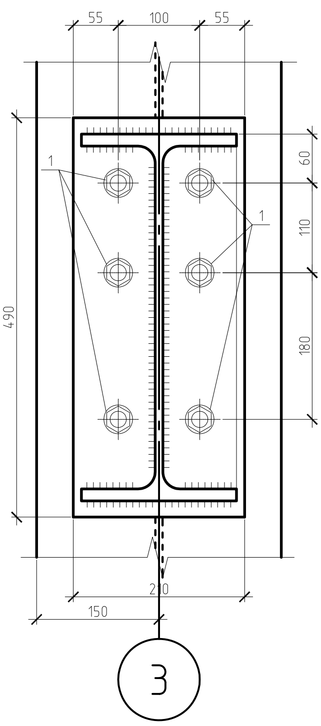


Nº	Description
1	Bolts M20, class 9.8
2	Bolts M16, class 5.8
3	CAE 80x8, S355
4	Bolts M20, class 8.8
5	TRON 139x8, S355
6	IPE 360
7	HEA 300
8	IPE 450

1–1



2–2



Performed by	Description
Maksym Podgayskyy	Beam-column joint
MD Refat Ahmed	
	Page 5

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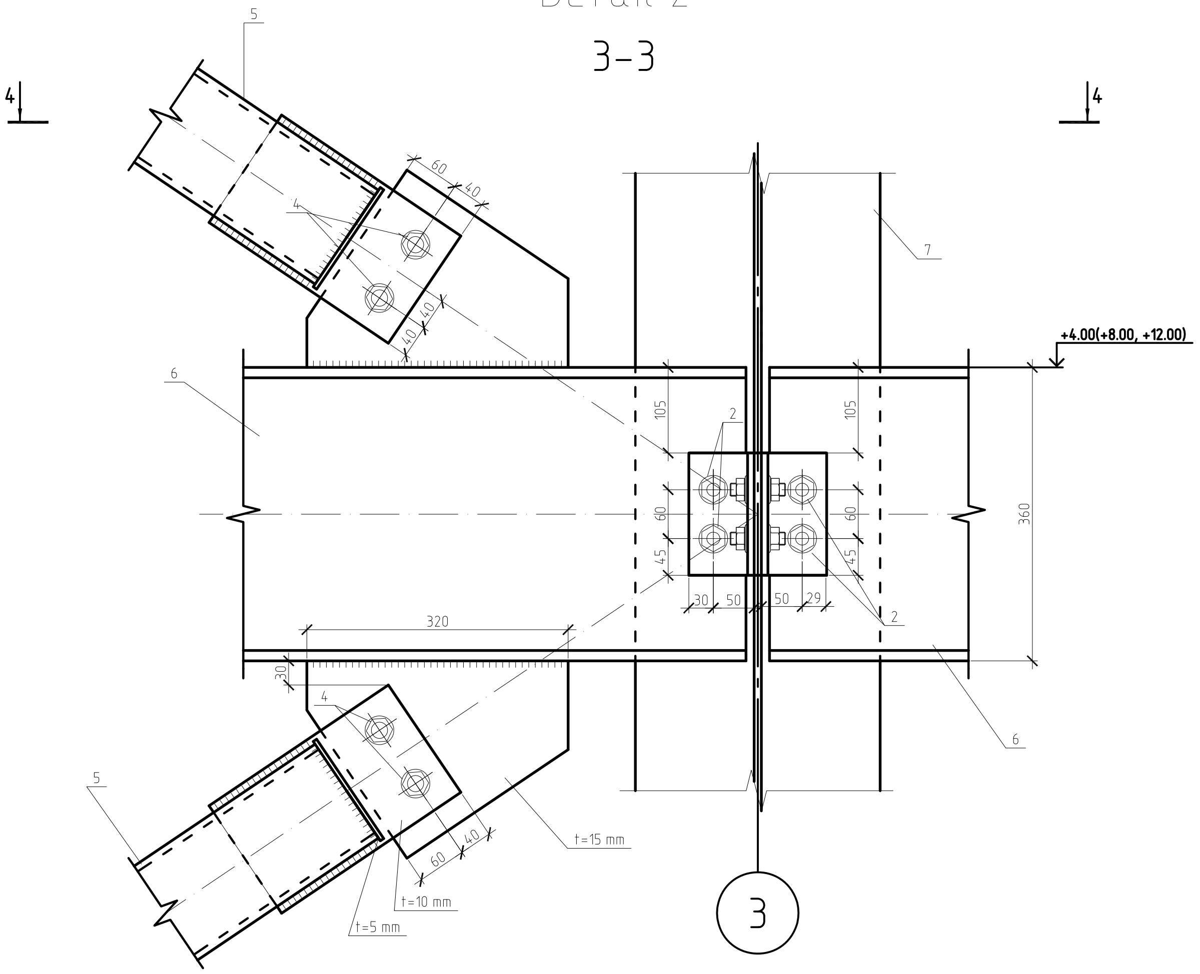
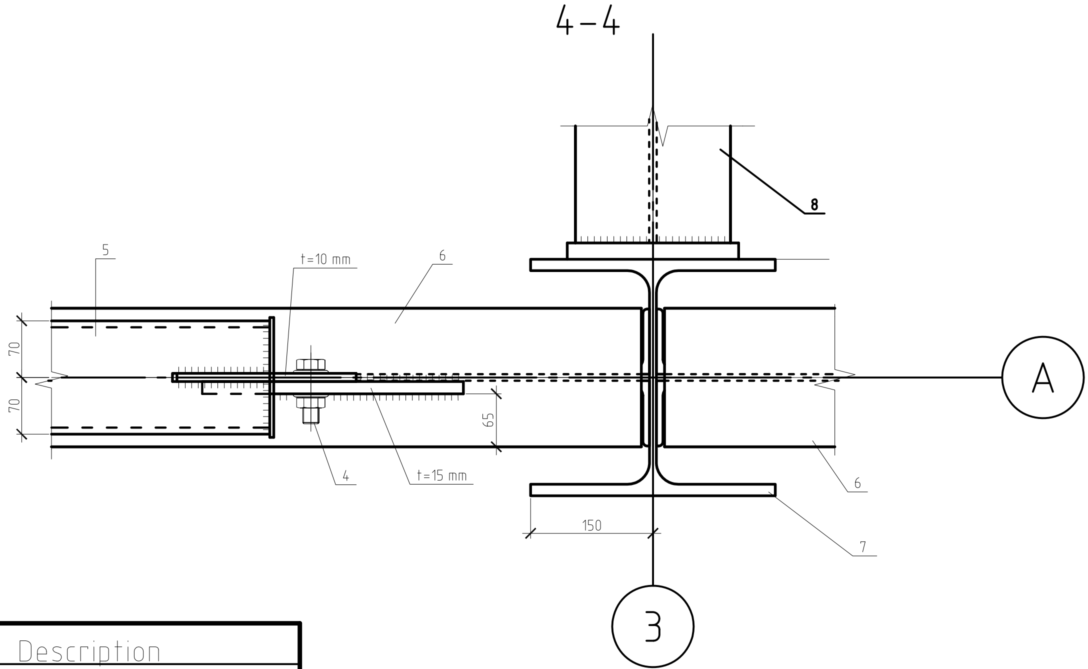
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

Detail 2

3-3



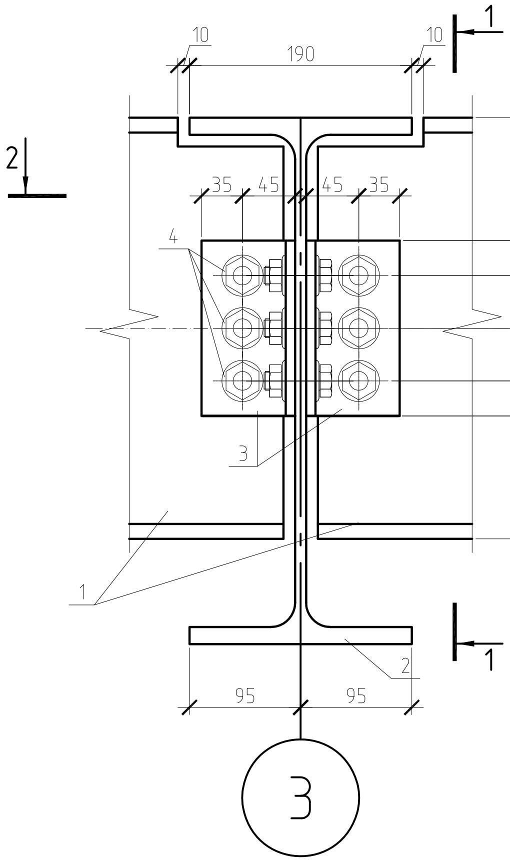
4-4



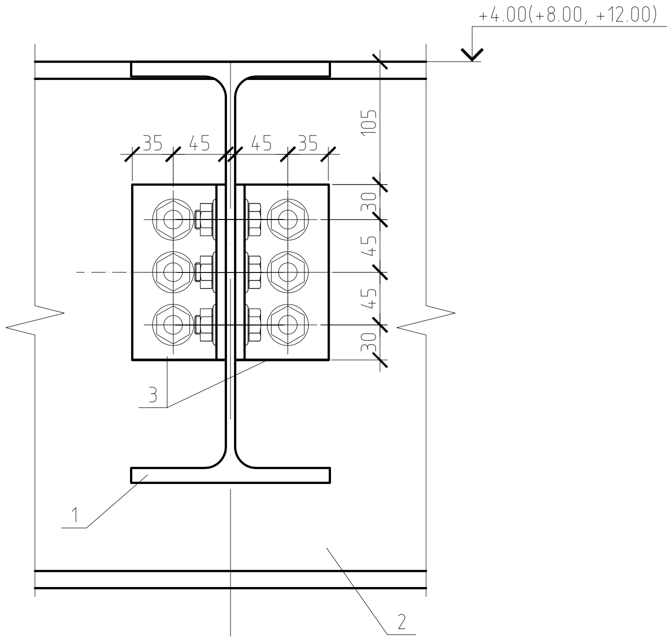
№	Description
1	Bolts M20, class 9.8
2	Bolts M16, class 5.8
3	CAE 80x8, S355
4	Bolts M20, class 8.8
5	TRON 139x8, S355
6	IPE 360
7	HEA 300
8	IPE 450

Performed by	Description
Maksym Podgayskyy	Detail 2
MD Refat Ahmed	Beam-column joint
	Page 6

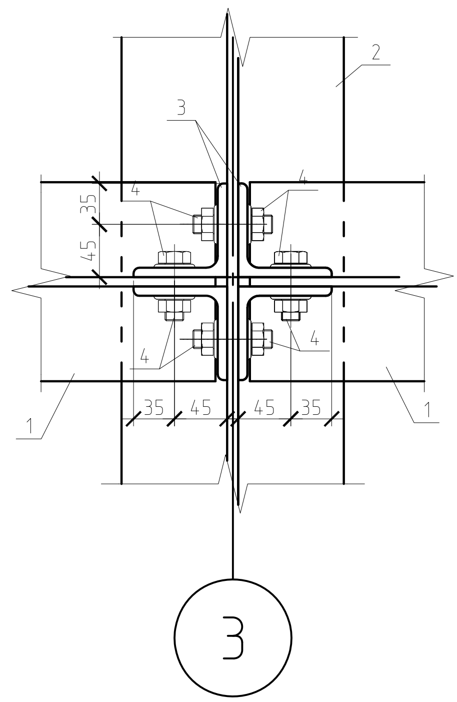
Detail 3



1–1

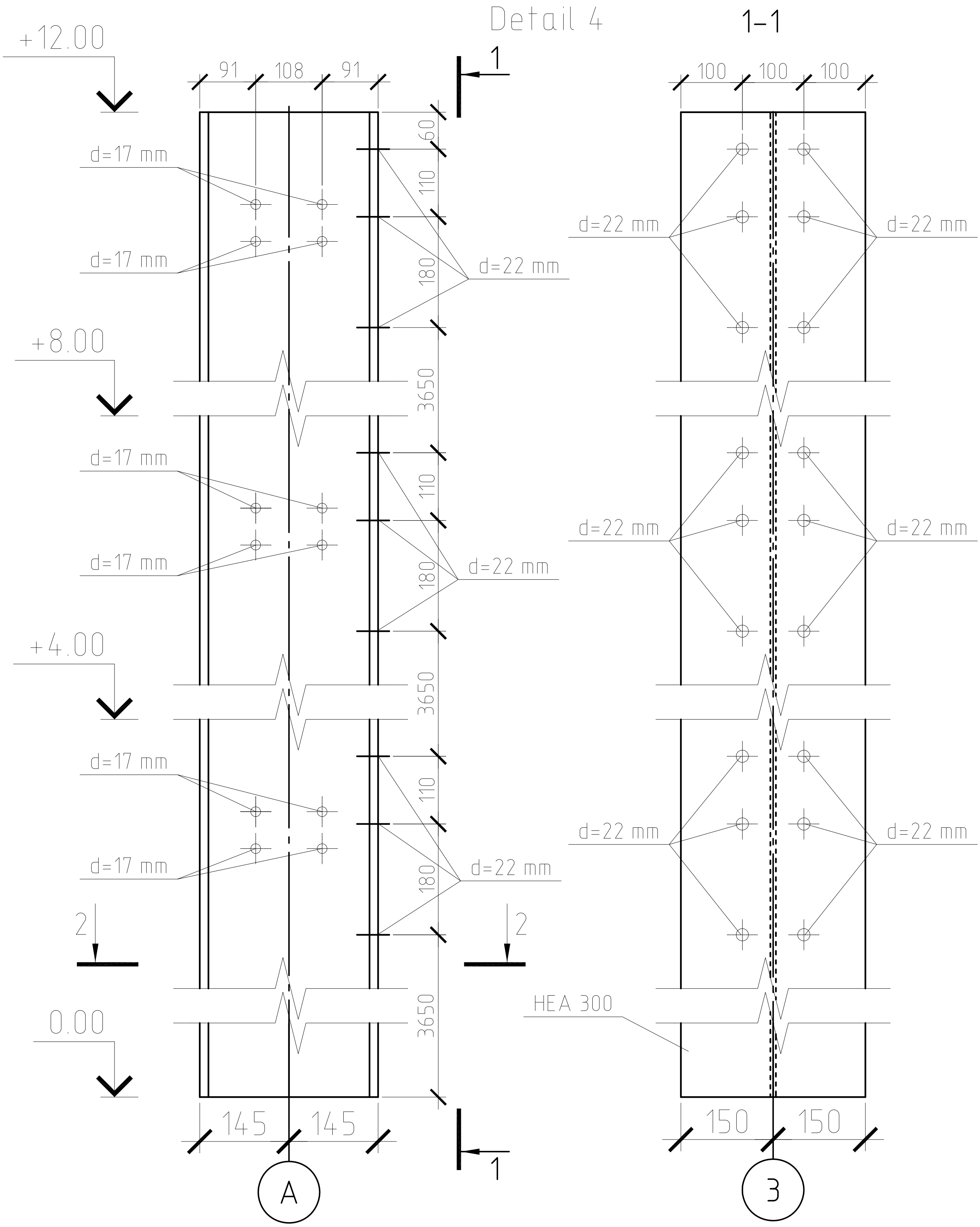


2–2

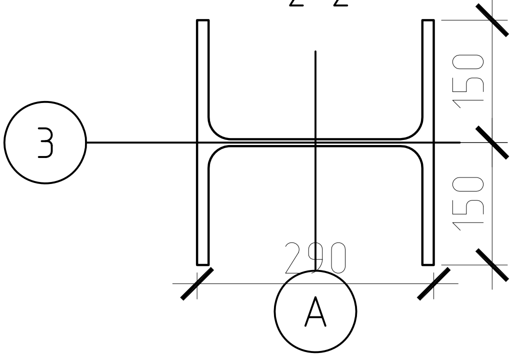


Nº	Description
1	IPE 360
2	IPE 450
3	CAE 80x8, S355
4	Bolts M16, class 5.8

Performed by	Description
Maksym Podgayskyy	Detail 3
MD Refat Ahmed	Beam-beam joint
	Page 7



2–2



Performed by	Description
Maksym Podgayskyy	Detail 4
MD Refat Ahmed	Execution drawing of column HEA 300
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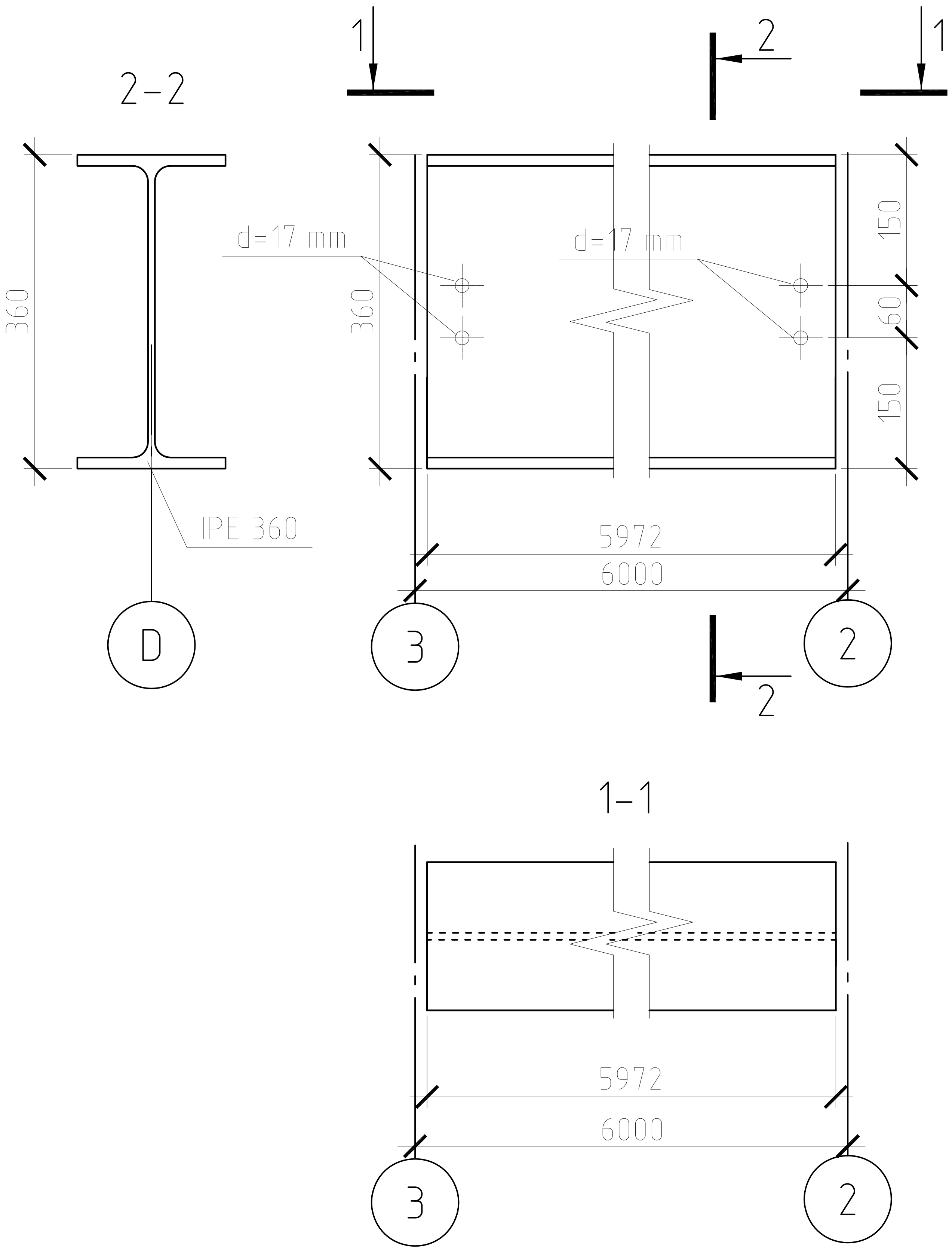
Detail 5



Performed by	Description
Maksym Podgayskyy	Detail 5
MD Refat Ahmed	Execution drawing of secondary beam IPE 360
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