CONCEPTUAL DESIGN OF BUILDINGS

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

Performed by:
Project group 10
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European Erasmus Mundus Master Course
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 characteristics:
   
   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, 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:

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

  b) Current floor

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

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.

<table>
<thead>
<tr>
<th>Loaded area</th>
<th>qk, kN/m²</th>
<th>Qk, kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floors</td>
<td>2.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

In the project we will use only the imposed load on floor \( q_k = 2.0 \text{ kN/m}^2 \) 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 \( q_k = 0.4 \text{ kN/m}^2 \).

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 \( \alpha = 0^\circ \), second wind load will act with angle \( \alpha = 90^\circ \). 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:

\[
S = \mu_k C_s C_t s_k = 0.8 \cdot 1.0 \cdot 1.0 \cdot 1.44 = 1.15 \text{ kN/m}^2
\]

where \( \mu_k = 0.8 \) (since \( 0^\circ \leq \alpha \leq 30^\circ \)); \( C_s = 1.0 \); \( C_t = 1.0 \);

and

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

where \( Z = 2 \), \( A = 1056 \text{ m} \) (for Guarda, Portugal).

3. Summary of basic actions.
The resulting actions are summarized in following table:

<table>
<thead>
<tr>
<th>Action No</th>
<th>Description</th>
<th>Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>LC1</td>
<td>Self-weight of steel structure</td>
<td>Permanent action</td>
<td>Obtained in ROBOT</td>
</tr>
<tr>
<td></td>
<td>Weight of floor slab</td>
<td>Permanent action</td>
<td>3.70 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Weight of roof slab</td>
<td>Permanent action</td>
<td>4.48 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Weight of partition walls</td>
<td>Permanent action</td>
<td>1.0 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Weight of external walls</td>
<td>Permanent action</td>
<td>1.0 kN/m²</td>
</tr>
<tr>
<td>LC2</td>
<td>Imposed load on floor</td>
<td>Variable action</td>
<td>2.0 kN/m²</td>
</tr>
<tr>
<td>LC3</td>
<td>Imposed load on roof</td>
<td>Variable action</td>
<td>0.4 kN/m²</td>
</tr>
<tr>
<td>LC4</td>
<td>Wind load 1 (( \alpha = 0^\circ ))</td>
<td>Variable action</td>
<td>1.5 kN/m² – one façade; -0.6 kN/m² – opposite façade</td>
</tr>
<tr>
<td>LC5</td>
<td>Wind load 2 (( \alpha = 90^\circ ))</td>
<td>Variable action</td>
<td>1.5 kN/m² – one façade; -0.6 kN/m² – opposite façade</td>
</tr>
<tr>
<td>LC6</td>
<td>Snow load</td>
<td>Variable action</td>
<td>1.15 kN/m²</td>
</tr>
</tbody>
</table>
2.2 Load combinations.

Ultimate limit state.

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

\[ E_d = y_{G,i} \cdot G_{k,j} + y_g \cdot P + y_{Q,1} \cdot Q_{k,1} + \sum_{i=2}^{n} y_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \]

Reduction coefficients for variable actions.

<table>
<thead>
<tr>
<th>Action</th>
<th>( \psi_0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imposed loads (category A)</td>
<td>0.7</td>
</tr>
<tr>
<td>Snow loads (sites located at altitude H&gt;1000 m a.s.l)</td>
<td>0.7</td>
</tr>
<tr>
<td>Wind load</td>
<td>0.6</td>
</tr>
</tbody>
</table>

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: \( y_{G,i} = 1.35; y_{Q,1} = y_{Q,3} = 1.5 \).

The following combinations are considered for the Ultimate Limit State:

- **Combination 1** - Imposed load as a leading variable action; wind load - \( \alpha = 0^\circ \).
  \[ E_{d1} = 1.35 \cdot G + 1.5 \cdot Q_{\text{imposed}} + 1.5 \cdot 0.7 \cdot Q_{\text{snow}} + 1.5 \cdot 0.6 \cdot Q_{\text{wind,0}} \]

- **Combination 2** - Imposed load as a leading variable action; wind load - \( \alpha = 90^\circ \).
  \[ E_{d2} = 1.35 \cdot G + 1.5 \cdot Q_{\text{imposed}} + 1.5 \cdot 0.7 \cdot Q_{\text{snow}} + 1.5 \cdot 0.6 \cdot Q_{\text{wind,0}} \]

- **Combination 3** - Snow load as a leading variable action; wind load - \( \alpha = 0^\circ \).
  \[ E_{d3} = 1.35 \cdot G + 1.5 \cdot Q_{\text{snow}} + 1.5 \cdot 0.7 \cdot Q_{\text{imposed}} + 1.5 \cdot 0.6 \cdot Q_{\text{wind,0}} \]

- **Combination 4** - Snow load as a leading variable action; wind load - \( \alpha = 90^\circ \).
  \[ E_{d4} = 1.35 \cdot G + 1.5 \cdot Q_{\text{snow}} + 1.5 \cdot 0.7 \cdot Q_{\text{imposed}} + 1.5 \cdot 0.6 \cdot Q_{\text{wind,0}} \]

- **Combination 5** - Wind load as a leading variable action; \( \alpha = 0^\circ \).
  \[ E_{d5} = 1.35 \cdot G + 1.5 \cdot Q_{\text{wind,0}} + 1.5 \cdot 0.7 \cdot Q_{\text{imposed}} + 1.5 \cdot 0.7 \cdot Q_{\text{snow}} \]

- **Combination 6** - Wind load as a leading variable action; \( \alpha = 90^\circ \).
  \[ E_{d6} = 1.35 \cdot G + 1.5 \cdot Q_{\text{wind,0}} + 1.5 \cdot 0.7 \cdot Q_{\text{imposed}} + 1.5 \cdot 0.7 \cdot Q_{\text{snow}} \]

Serviceability limit state.

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

\[ E_d = \sum_{i \geq 1} G_{k,i} + P + \sum_{i > 1} \psi_{0,i} \cdot Q_{k,i} \]

- **Combination 7** - Imposed load as a leading variable action; wind load - \( \alpha = 0^\circ \).
  \[ E_{d7} = G + Q_{\text{imposed}} + 0.7 \cdot Q_{\text{snow}} + 0.6 \cdot Q_{\text{wind,0}} \]

- **Combination 8** - Imposed load as a leading variable action; wind load - \( \alpha = 90^\circ \).
  \[ E_{d8} = G + Q_{\text{imposed}} + 0.7 \cdot Q_{\text{snow}} + 0.6 \cdot Q_{\text{wind,0}} \]

- **Combination 9** - Snow load as a leading variable action; wind load - \( \alpha = 0^\circ \).
  \[ E_{d9} = G + Q_{\text{snow}} + 0.7 \cdot Q_{\text{imposed}} + 0.6 \cdot Q_{\text{wind,0}} \]

- **Combination 10** - Snow load as a leading variable action; wind load - \( \alpha = 90^\circ \).
  \[ E_{d10} = G + Q_{\text{snow}} + 0.7 \cdot Q_{\text{imposed}} + 0.6 \cdot Q_{\text{wind,0}} \]

- **Combination 11** - Wind load as a leading variable action; wind load - \( \alpha = 0^\circ \).
  \[ E_{d11} = G + Q_{\text{wind,0}} + 0.7 \cdot Q_{\text{imposed}} + 0.7 \cdot Q_{\text{snow}} \]

- **Combination 12** - Wind load as a leading variable action; wind load - \( \alpha = 90^\circ \).
  \[ E_{d12} = G + Q_{\text{wind,0}} + 0.7 \cdot Q_{\text{imposed}} + 0.7 \cdot Q_{\text{snow}} \]

where \( G = L C_1; Q_{\text{imposed}} = L C_2 + L C_3; Q_{\text{wind,0}} = L C_4; Q_{\text{wind,90}} = L C_5; Q_{\text{snow}} = L C_6. \)
Substituting $G$, $Q_{\text{imposed}}$, $Q_{\text{wind}}$, $Q_{\text{wind,90^\circ}}$, $Q_{\text{snow}}$ with appropriate actions according to our project we obtain:

**Combination 7** - Imposed load as a leading variable action; wind load - $\alpha = 0^\circ$.

$$E_{d4} = LC1 + (LC2 + LC3) + 0.7 \cdot LC6 + 0.6 \cdot LC4$$

**Combination 8** - Imposed load as a leading variable action; wind load - $\alpha = 90^\circ$.

$$E_{d4} = LC1 + (LC2 + LC3) + 0.7 \cdot LC6 + 0.6 \cdot LC5$$

**Combination 9** - Snow load as a leading variable action; wind load - $\alpha = 0^\circ$.

$$E_{d5} = LC1 + LC6 + 0.7 \cdot (LC2 + LC3) + 0.6 \cdot LC4$$

**Combination 10** - Snow load as a leading variable action; wind load - $\alpha = 90^\circ$.

$$E_{d5} = LC1 + LC6 + 0.7 \cdot (LC2 + LC3) + 0.6 \cdot LC5$$

**Combination 11** - Wind load as a leading variable action; wind load - $\alpha = 0^\circ$.

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

**Combination 12** - Wind load as a leading variable action; wind load - $\alpha = 90^\circ$.

$$E_{d6} = LC1 + LC5 + 0.7 \cdot (LC2 + LC3) + 0.7 \cdot LC6$$
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:

<table>
<thead>
<tr>
<th>Type of columns</th>
<th>Grid</th>
<th>Cross-section</th>
<th>Steel grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perimeter</td>
<td>A1, A4, D1, D4, A2, A3, D2, D3, B1, B4, C1, C4</td>
<td>HEA 300</td>
<td>S355</td>
</tr>
<tr>
<td>Center</td>
<td>B2, B3, C2, C3</td>
<td>HEA 400</td>
<td>S 355</td>
</tr>
</tbody>
</table>

Tab.100. Initial assumed geometric characteristics of columns.

<table>
<thead>
<tr>
<th>Type of beams</th>
<th>Beams</th>
<th>Cross-section</th>
<th>Steel grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main beams</td>
<td>A1-D1, A2-D2, A3-D3, A4-D4</td>
<td>IPE 300</td>
<td>S 355</td>
</tr>
<tr>
<td>Secondary beams</td>
<td>A1-A4, B1-B4, C1-C4, D1-D4</td>
<td>IPE 200</td>
<td>S 355</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Bracing</th>
<th>Cross-section</th>
<th>Steel grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1-A2, D1-D2, A3-A4, D3-D4</td>
<td>CHS 168x8</td>
<td>S 355</td>
</tr>
</tbody>
</table>

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:

**Beams**

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

\[ M_{ed} = 127.99 \text{ kNm} \leq W_{pl,y} \cdot 355 \cdot 10^3 / 1.0 \]
In order to satisfy this condition IPE 240 section ($W_{pl,y} = 367 \text{ cm}^3$) is selected.

2. Main beams.
Assuming class 1 or 2 cross sections, the following solution is obtained:
\[ M_{ed} = 241.12 \text{ kNm} \leq W_{pl,y} \cdot 355 \cdot 10^3 / 1.0 \]
\[ W_{pl,y} \geq 679.2 \text{ cm}^2 \]
In order to satisfy this condition IPE 360 section ($W_{pl,y} = 1019 \text{ cm}^3$) is selected. IPE 330 ($W_{pl,y} = 804 \text{ cm}^3$) does not meet the requirements as long as the moment in cross section increases after changing the cross section in ROBOT model to $M_{ed}=318 \text{ kNm}$.

Columns
1. Central columns.
Assuming class 1, 2 or 3 cross sections, the following solution is obtained:
\[ N_{ed} = 1030 \text{ kN} \leq N_{c,Rd} = 0.4 \frac{A f_y}{\gamma_{m0}} = 0.4 \cdot 355 \cdot 10^3 / 1.0 \]
\[ A \geq 72.5 \text{ cm}^2 \]
As it is expected that buckling resistance will govern the member design, a member HEA 320 (A=124.4 cm²)

2. Perimeter columns.
Assuming class 1, 2 or 3 cross sections, the following solution is obtained:
\[ N_{ed} = 732 \text{ kN} \leq N_{c,Rd} = 0.4 \frac{A f_y}{\gamma_{m0}} = A \cdot 0.4 \cdot 355 \cdot 10^3 / 1.0 \]
\[ A \geq 51.5 \text{ cm}^2 \]
As it is expected that buckling resistance will govern the member design, a member HEA 220 (A=64.3 cm²)

Bracing
Assuming class 1, 2 or 3 cross sections, the following solution is obtained:
\[ N_{ed} = 216 \text{ kN} \leq N_{c,Rd} = 0.4 \frac{A f_y}{\gamma_{m0}} = A \cdot 355 \cdot 10^3 / 1.0 \]
\[ A \geq 6.1 \text{ cm}^2 \]
In order to satisfy this condition TRON 88x2.5 section ($A = 67.9 \text{ cm}^2$) is selected.

Pre-design sections are summarized in following table:

<table>
<thead>
<tr>
<th>Type of columns</th>
<th>Grid</th>
<th>Cross-section</th>
<th>Steel grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Marginal</td>
<td>A1, A4, D1, D4, A2, A3, D2, D3, B1, B4, C1, C4</td>
<td>HEA 220</td>
<td>S355</td>
</tr>
<tr>
<td>Center</td>
<td>B2, B3, C2, C3</td>
<td>HEA 320</td>
<td>S 355</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Type of beams</th>
<th>Beams</th>
<th>Cross-section</th>
<th>Steel grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main beams</td>
<td>A1-D1, A2-D2, A3-D3, A4-D4</td>
<td>IPE 360</td>
<td>S 355</td>
</tr>
<tr>
<td>Secondary beams</td>
<td>A1-A4, B1-B4, C1-C4, D1-D4</td>
<td>IPE 240</td>
<td>S 355</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Bracing</th>
<th>Cross-section</th>
<th>Steel grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1-A2, D1-D2, A3-A4, D3-D4</td>
<td>TRON 88x2.5</td>
<td>S 355</td>
</tr>
</tbody>
</table>

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:

\[ u_i \leq \frac{H_i}{300} \]
\[ u \leq \frac{H}{500} \]

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.

<table>
<thead>
<tr>
<th></th>
<th>Limit values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Secondary beam</td>
<td>IPE 360</td>
</tr>
<tr>
<td>Main beam</td>
<td>IPE 450</td>
</tr>
<tr>
<td>Central column</td>
<td>HEA 450</td>
</tr>
<tr>
<td>Perimeter column</td>
<td>HEA 300</td>
</tr>
<tr>
<td>Bracing</td>
<td>TRON 139x8</td>
</tr>
<tr>
<td>Horizontal deflection ((\Delta))</td>
<td>2</td>
</tr>
<tr>
<td>Horizontal deflection first floor ((\delta_1))</td>
<td>1.3</td>
</tr>
<tr>
<td>Horizontal deflection first floor ((\delta_2))</td>
<td>0.5</td>
</tr>
<tr>
<td>Horizontal deflection first floor ((\delta_2))</td>
<td>0.2</td>
</tr>
<tr>
<td>Vertical deflection main beam</td>
<td>2</td>
</tr>
<tr>
<td>Vertical deflection secondary beam</td>
<td>2.3</td>
</tr>
</tbody>
</table>
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 $\alpha_{cr}$ for ultimate limit state combinations.

<table>
<thead>
<tr>
<th>Combination</th>
<th>$\alpha_{cr}$ (mode 1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Combination 1</td>
<td>10.95</td>
</tr>
<tr>
<td>Combination 2</td>
<td>10.95</td>
</tr>
<tr>
<td>Combination 3</td>
<td>11.46</td>
</tr>
<tr>
<td>Combination 4</td>
<td>11.46</td>
</tr>
<tr>
<td>Combination 5</td>
<td>11.66</td>
</tr>
<tr>
<td>Combination 6</td>
<td>11.66</td>
</tr>
</tbody>
</table>

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

- \( h = 360\text{mm} \)
- \( d = 298.6\text{mm} \)
- \( b = 170\text{mm} \)
- \( t_w = 8\text{mm} \)
- \( t_f = 12.7\text{mm} \)
- \( r = 18\text{mm} \)
- \( A_1 = 72.7\text{cm}^2 \)
- \( h_i = 334.6\text{mm} \)
- \( L_{\text{beam}} = 6\text{m} \)
- \( f_{yd} = 355\text{MPa} \)
- \( \gamma_{M0} = 1.0 \)

Internal forces

- \( M_{Ed} = 129.56\text{kN}\cdot\text{m} \)
V_{Ed} := 86.37\,\text{kN}

Fy and Fz are low so we can neglect them.

**Cross section classification**

As long as steel class of the beam is S355 we

\[
\varepsilon := \frac{235}{f_{yd}} = 0.814
\]

Web in bending:

\[
c_w := \frac{d}{t_w} = 37.325 < 72\varepsilon = 58.58
\]

Web is class 1

Flanges in compression:

\[
c_f := \frac{b - t_w - 2r}{2} = 63\text{ mm}
\]

\[
c_f = 4.961 < 9\varepsilon = 7.323
\]

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:

\[
M_{Rd} := \frac{W_{pl} f_{yd}}{\gamma_M0} = 361.745\,\text{kN}\cdot\text{m}
\]

\[
\frac{M_{Ed}}{M_{Rd}} = 0.358 < 1
\]
2. Shear resistance according to EC3-1-1 chapter 6.2.6:

Design plastic shear resistance:

\[ V_{Rd} := \frac{(A_{Vz} f_{yd})}{\gamma_{M0} \sqrt{3}} = 719.407 \text{kN} \]

\[ \frac{V_{Ed}}{V_{Rd}} = 0.12 < 1 \]

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:

\[ V_{Ed} = 86.37 \text{kN} < 0.5V_{Rd} = 359.704 \text{kN} \]

Therefore, effect of shear force on the 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 compressed parts of the cross section.

**Verification of serviceability limit state**

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

\[ \delta_{max} < \frac{L_{beam}}{250} \]

For IPE 360:

\[ \delta_{max} := 2.3 \text{cm} < \frac{L_{beam}}{250} = 2.4 \text{cm} \]

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)

\[ h := 450\text{mm} \]
\[ b := 190\text{mm} \]
\[ t_w := 9.4\text{mm} \]
\[ t_f := 14.6\text{mm} \]
\[ r := 21\text{mm} \]
\[ A_1 := 98.8\text{cm}^2 \]
\[ h_i := 420.8\text{mm} \]
\[ L_{\text{beam}} := 6\text{m} \]
\[ \gamma_{M0} := 1.0 \]

**Internal forces**

\[ M_{Ed} := 275.78\text{kN}\cdot\text{m} \]
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:

\[ \varepsilon = \sqrt{\frac{235 \text{ N}}{\text{mm}^2 \cdot f_{yd}}} = 0.814 \]

Web in bending:

\[ c_w := d \]

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

Web is class 1

Flanges in compression:

\[ c_f := \frac{b - t_w - 2r}{2} = 69.3 \text{ mm} \]

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

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:

$$M_{Rd} := \frac{W_p l_y f_y d}{\gamma_{M0}} = 604.21 \text{kN} \cdot \text{m}$$

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

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

Design plastic shear resistance:

$$V_{Rd} := \frac{(A_v z f_y d)}{\gamma_{M0} \sqrt{3}} = 1.043 \times 10^3 \text{kN}$$

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

Shear buckling resistance classification:

$$\eta := 1 \text{ (conservatively taken)}$$

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

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:

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

Therefore, effect of shear force on the 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:

$$\delta_{max} < \frac{L_{\text{beam}}}{250}$$
For IPE 450:

\[ \delta_{\text{max}} = 2.0\text{cm} < \frac{L_{\text{beam}}}{250} = 2.4\text{cm} \]

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

\[ h = 440 \text{mm} \]
\[ b = 300 \text{mm} \]
\[ t_w = 11.5 \text{mm} \]
\[ t_f = 21 \text{mm} \]
\[ r = 27 \text{mm} \]
\[ A_1 = 178 \text{cm}^2 \]
\[ h_i = 398 \text{mm} \]
\[ d = 344 \text{mm} \]
\[ I_y = 63720 \text{cm}^4 \]
\[ W_{el,y} = 2896 \text{cm}^3 \]

Coefficients and other values

\[ \gamma_{M0} := 1 \]
\[ \gamma_{M1} := 1 \]
\[ E := 210000 \text{MPa} \]
\[ v := 0.3 \]
\[ G := 80700 \text{MPa} \]
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

\[ f_{yd} := 355\text{MPa} \quad \varepsilon := \sqrt{\frac{235\text{MPa}}{f_{yd}}} \]

1. Flange in compression:

\[ c_f := \frac{b - t_w - 2r}{2} = 117.25\text{·mm} \]

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

Flange is Class 1

2. Web in bending and compression

\[ c_w := d \]

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

as long as \( \alpha = 0.875 > 0.5 \)

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

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:

\[ L_{Z,cr} := L_{column} = 4\text{m} \]

\[ L_{Y,cr} := L_{column} = 4\text{m} \]

**Verification of cross section resistance**

1. Axial force

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

\[ \frac{N_{Ed}}{N_{pl,Rd}} = 0.17 < 1 \]

2. Axial force and bending

According to EN1993-1-1 chapter 6.2.9.1

\[ N_{Ed} = 1.073 \times 10^3\cdot\text{kN} < 0.25 \cdot N_{pl,Rd} = 1.58 \times 10^3\cdot\text{kN} \]
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:

\[
N_{Ed} = 1.073 \times 10^3 \cdot \text{kN} > \frac{0.5 \cdot h \cdot t_w \cdot f_{yd}}{\gamma_{M0}} = 898.15 \cdot \text{kN}
\]

Reduced plastic resistance is given by:

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

\[
M_{yEd} = 291.07 \cdot \text{kN} \cdot \text{m} < M_{n,y,Rd} = 1.11 \times 10^3 \cdot \text{kN} \cdot \text{m}
\]

3. Shear force

\[
V_{pl,Rd} := \frac{A_{vz} \cdot f_{yd}}{\gamma_{M0} \cdot \sqrt{3}} = 1.348 \times 10^3 \cdot \text{kN}
\]

\[
\frac{V_{Ed}}{V_{pl,Rd}} = 0.054 < 1
\]

Shear buckling resistance classification:

\[
\eta := 1 \quad \text{(conservatively taken)}
\]

\[
\frac{h}{t_w} = 38.261 < \frac{72 \varepsilon}{\eta} = 58.58
\]

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

4. Bending and shear force

\[
V_{Ed} = 72.77 \cdot \text{kN} < 0.5 \cdot V_{pl,Rd} = 674.111 \cdot \text{kN}
\]

Therefore, effect of shear force on the 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:

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

**Determination of the reduction factor due to lateral-torsional buckling**

\[ k_Z := 1 \]

\[ c_1 := 1.77 \]

\[ c_2 := 0 \]

\[ c_3 := 0 \]

\[ k_w := 1 \]

\[
M_{cr} := c_1 \frac{\left(\pi \cdot E \cdot I_z\right)}{\left(k_z \cdot L_{column}\right)^2} \left[\left(\frac{k_z}{k_w}\right) \cdot I_w + \left(\frac{k_z \cdot L_{column}}{\pi^2 E \cdot I_z}\right) \cdot I^2_{w} \right]^{0.5} = 1.69 \times 10^3 \text{kN\cdotm}
\]

According to EN 1993-1-1 chapter 6.3.2.2:

\[ \lambda_{LT} := \sqrt{\frac{\left(W_{pl} \cdot y^f \cdot y_d\right)}{M_{cr}}} = 0.822 \text{ non-dimensional slenderness} \]

\[ \frac{h}{b} = 1.467 < 2 \Rightarrow \text{we use buckling curve a} \quad \text{(from table 6.4)} \]

For buckling curve a

\[ \alpha_{LT} := 0.21 \quad \text{(from table 6.3)} \]

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

\[ \chi_{LT} := \frac{1}{LT + \sqrt{LT^2 - \lambda_{LT}^2}} = 0.783 \text{ but } \chi_{LT} < 1 \]

**Determination of the reduction factors due to flexural buckling**

Calculation of non-dimensional slenderness for flexural buckling according to EN 1993-1-1 chapter 6.3.1.3
\[ \lambda_1 := 93.9 \cdot \sqrt{\left( \frac{235\,\text{MPa}}{f_{yd}} \right)} = 76.399 \quad \text{for class 1} \]

\[ \lambda_y := \frac{L_{y,\text{cr}}}{i_y \lambda_1} = 0.277 \]

\[ \lambda_z := \frac{L_{z,\text{cr}}}{i_z \lambda_1} = 0.718 \]

Calculation of the reduction factor \( \chi_y \) and \( \chi_z \) according to chapter 6.3.1.2

\[ \frac{h}{b} = 1.467 > 1.2 \]

\[ t_f = 21 \cdot \text{mm} \quad 100 \text{ mm} \]

As a result for \( y-y \) we use curve \( b \), for \( z-z \) curve \( c \) (table 6.2 EC3-1-1)

\[ \alpha_y := 0.34 \]

\[ \alpha_z := 0.49 \]

\[ y := 0.5 \left[ 1 + \alpha_y (\lambda_y - 0.2) + \lambda_y^2 \right] = 0.551 \]

\[ z := 0.5 \left[ 1 + \alpha_z (\lambda_z - 0.2) + \lambda_z^2 \right] = 0.885 \]

\[ X_y := \frac{1}{y + \sqrt{\left( \frac{2}{y} - \lambda_y^2 \right)}} = 0.973 \]

\[ X_z := \frac{1}{z + \sqrt{\left( \frac{2}{z} - \lambda_z^2 \right)}} = 0.713 \]

Calculation of \( \text{N}_{Rk}, \text{M}_{i,rk} \) for class 1

\[ \text{N}_{Rk} := f_{yd} A_1 = 6.319 \times 10^6 \text{ N} \]

\[ \text{M}_{y,Rk} := f_{yd} W_{pl,y} = 1.142 \times 10^3 \text{ kN} \cdot \text{m} \]

\[ \text{M}_{z,Rk} := f_{yd} W_{pl,z} = 342.753 \text{ kN} \cdot \text{m} \]

Calculation of interaction factors according to Method 2

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

\[ \psi := 0 \quad \text{because of triangular shape of bending moment diagram} \]

\[ C_{my} := 0.6 \]

\[ C_{mz} := 0.6 \]

\[ C_{mLt} := 0.6 \]
\[ k_{yy.1} := C_{my} \left[ 1 + \left( \lambda_y - 0.2 \right) \cdot \left( \frac{N_{Ed}}{X_y \cdot N_{Rk}} \right) \right] = 0.608 < \]

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

then \( k_{yy} := 0.608 \)

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

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

then \( k_{zz} := 0.721 \)

\( k_{yz} := 0.6 \cdot k_{zz} = 0.433 \)

\[ k_{zy1} := 1 - 0.1 \cdot \frac{\lambda_z \cdot N_{Ed}}{\left( C_{mLt} - 0.25 \right) \left( X_z \cdot N_{Rk} \right)} = 0.951 \]

\[ k_{zy2} := 1 - 0.1 \cdot \frac{N_{Ed}}{\left( C_{mLt} - 0.25 \right) \left( X_z \cdot N_{Rk} \right)} = 0.932 \]

\( k_{zy1} > k_{zy2} \)

then \( k_{zy} := 0.951 \)
Based on the determined parameters we obtain:

\[
\frac{N_{Ed}}{\chi_y N_{Rk}} + \frac{k_{yy} M_{yEd}}{\gamma M_1} + \frac{k_{yz} M_{zEd}}{\gamma M_1} = 0.373 < 1
\]

\[
\frac{N_{Ed}}{\chi_z N_{Rk}} + \frac{k_{zy} M_{yEd}}{\gamma M_1} + \frac{k_{zz} M_{zEd}}{\gamma M_1} = 0.548 < 1
\]

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 ROBOT software for serviceability limit states. Following limiting values apply for horizontal displacement:

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

\[
\Delta := 2.0 < \frac{H_{\text{building}}}{500} = 2.4 \cdot \text{cm}
\]

2. Verification of horizontal displacement for the each floor:

   - first floor - \( \delta_1 := 1.3 < \frac{L_{\text{column}}}{300} = 1.333 \cdot \text{cm} \)
   - second floor - \( \delta_2 := 0.5 < \frac{L_{\text{column}}}{300} = 1.333 \cdot \text{cm} \)
   - third floor - \( \delta_3 := 0.2 < \frac{L_{\text{column}}}{300} = 1.333 \cdot \text{cm} \)

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 requirements of ULS and SLS.**
5.2.2 Perimeter column HEA 300

Cross section characteristics

\[
\begin{align*}
\text{h} & := 290\text{mm} & \text{W}_{\text{pl.y}} & := 1383\text{cm}^3 \\
\text{b} & := 300\text{mm} & \text{i}_y & := 12.74\text{cm} \\
\text{t}_w & := 8.5\text{mm} & \text{A}_{\text{VZ}} & := 37.28\text{cm}^2 \\
\text{t}_f & := 14\text{mm} & \text{I}_z & := 6310\text{cm}^4 \\
\text{r} & := 27\text{mm} & \text{W}_{\text{el.z}} & := 420.6\text{cm}^3 \\
\text{A}_1 & := 112.5\text{cm}^2 & \text{W}_{\text{pl.z}} & := 641.2\text{cm}^3 \\
\text{h}_1 & := 262\text{mm} & \text{i}_z & := 7.49\text{cm} \\
\text{d} & := 208\text{mm} & \text{I}_w & := 1200000\text{cm}^6 \\
\text{l}_y & := 18260\text{cm}^4 & \text{I}_t & := 85.17\text{cm}^4 \\
\text{W}_{\text{el.y}} & := 1260\text{cm}^3 & \\
\end{align*}
\]

Material

\[
\begin{align*}
\gamma_{\text{M0}} & := 1 \\
\gamma_{\text{M1}} & := 1 \\
\text{E} & := 210000\text{MPa} \\
\nu & := 0.3 \\
\text{G} & := 80700\text{MPa} \\
\text{Building} \\
\text{L}_{\text{column}} & := 4\text{m} \\
\text{H}_{\text{building}} & := 12\text{m} \\
\end{align*}
\]
Verification of perimetr column will be made using SemiComp+ software. For input data we need to calculate Mcr.

**Determination of critical moment:**

\[ k_z := 1 \]
\[ c_1 := 1.77 \]
\[ c_2 := 0 \]
\[ c_3 := 0 \]
\[ k_w := 1 \]

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

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

### Cross-section data
- **H** = 280.0 [mm]
- **B** = 300.0 [mm]
- **Tw** = 8.5 [mm]
- **Tf** = 14.0 [mm]
- **R** = 27.0 [mm]

### Material
- **Steel grade** = S395
- **f_y** = 356.0 N/mm²
- **E** = 210000.0 N/mm²

### Cross-sectional properties
- \( A \) [cm²] = 112.53
- \( I_y \) [cm⁴] = 18263.50
- \( I_z \) [cm⁴] = 6309.58
- \( W_{x,y} \) [cm³] = 1259.55
- \( W_{x,z} \) [cm³] = 420.64
- \( W_{y,z} \) [cm³] = 1393.27
- \( W_{z,x} \) [cm³] = 641.17

### Boundary conditions
- **L** = 4.000 m
- **n** = 0

### Loading in z-x-plane
- \( N_{ed} \) = -594.30 kN
- \( q_{k,x} \) = 5.00 kN/m
- \( M_{k,x} \) = 0.00 kNm
- \( M_{k,y} \) = 117.37 kNm
- \( P_{k,z} \) = 0.00 kN
- Distance of Loading to shear center \( z_{0} \) = -146.00 mm

### Loading in y-x-plane
- \( q_{k,y} \) = 0.00 kN/m
- \( M_{k,y} \) = 0.00 kNm
- \( M_{k,x} \) = 0.00 kNm
- \( P_{k,z} \) = 0.00 kN

### Additional notes
- \( M_{ud} \) = 762.68 kNm
- \( M_{ue} \) = 0.00 kNm

**Specify path of LTBeam.exe file:**
- C:\Program Files (x86)\LTBeam_v10101\LTBeam.exe

*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.*
## SEMICOMP Member Check

**Choose method for member check**  
Method 2 (EN 1993-1-1 Annex B)  

**Choose method for cross-section resistance**  
EN 1993-1-1:2010-12  

### Section classification for member design check

(based on 1. order cross-section forces)

Reference values for classification in the worst section along the member

<table>
<thead>
<tr>
<th>c/t_p</th>
<th>(e_{w,2})</th>
<th>(\lambda_{w,2})</th>
<th>(\psi_{w,2})</th>
<th>(\epsilon)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24.471</td>
<td>1.000</td>
<td>-0.040</td>
<td>0.814</td>
<td></td>
</tr>
<tr>
<td>8.482</td>
<td>1.000</td>
<td>1.000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Boundaries

<table>
<thead>
<tr>
<th>Class 1</th>
<th>Class 2</th>
<th>Class 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>(c/t_p) ≤ (c/t_{pl,\text{max}})</td>
<td>26.849</td>
<td>30.917</td>
</tr>
<tr>
<td>(c/t_f) ≤ (c/t_{fl,\text{max}})</td>
<td>7.323</td>
<td>8.136</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>N_{Ed}</th>
<th>M_{pl,Rd}</th>
<th>M_{pl,E,Rd}</th>
<th>M_{pl,E,Ed,max}</th>
<th>N_{Ed}</th>
</tr>
</thead>
<tbody>
<tr>
<td>3994.737 kN</td>
<td>491.061 kNm</td>
<td>227.614 kNm</td>
<td>117.370 kNm</td>
<td>-694.300 kN</td>
</tr>
<tr>
<td>447.141 kNm</td>
<td>447.141 kNm</td>
<td>149.326 kNm</td>
<td>0.000 kNm</td>
<td></td>
</tr>
</tbody>
</table>

**Strong axis buckling**

- \(L_{c,y} = 4.000\) m
- \(N_{pl,y} = 23658.272\) kN
- \(\alpha_{y} = 0.34\) [-]
- \(\lambda_{y} = 0.411\) [-]
- \(X_{y} = 0.922\) [-]

**Weak axis buckling**

- \(L_{c,x} = 4.000\) m
- \(N_{pl,x} = 8173.312\) kN
- \(\alpha_{x} = 0.49\) [-]
- \(\lambda_{x} = 0.699\) [-]
- \(X_{x} = 0.725\) [-]

**Lateral torsional buckling**

- \(M_{pl} = 762.660\) kNm
- \(\alpha_{LT} = 0.34\) [-]
- \(\lambda_{LT} = 0.766\) [-]
- \(X_{LT,mod} = 0.924\) [-]
- \(f_{mod} = 0.904\) [-]
Reference values

Correction factor \( k_e \) table 6.6
\[
k_e = 0.000 [-]
\]

Method 1 auxiliary terms (if applicable):

\[
\begin{align*}
\mu_1 &= 0.000 [-] \\
\mu_2 &= 0.000 [-] \\
w_1 &= 0.000 [-] \\
w_2 &= 0.000 [-] \\
n_1 &= 0.000 [-] \\
a_1 &= 0.000 [-] \\
b_1 &= 0.000 [-] \\
c_1 &= 0.000 [-] \\
d_1 &= 0.000 [-] \\
e_1 &= 0.000 [-] \\
m_{1o} &= 0.000 [-] \\
c_{1o} &= 0.000 [-] \\
c_{1k} &= 0.000 [-]
\end{align*}
\]

Method 2 auxiliary terms (if applicable):

\[
\begin{align*}
c_{1o} &= 0.668 [-] \\
c_{1k} &= 0.000 [-] \\
c_{m1} &= 0.668 [-]
\end{align*}
\]

---

**EN 1993-1-1, 6.3.3**

*Uniform member in bending and axial compression*

*Global interaction factors*

\[
\begin{align*}
\text{Eq. (6.61)}: & \quad U = 0.387 \leq 1.0 \quad \text{ok} \\
\text{Eq. (6.62)}: & \quad U = 0.518 \leq 1.0 \quad \text{ok}
\end{align*}
\]

*Cross-section check at each end of the member*

\[
\begin{align*}
\text{Left end:} & \quad U = 0.174 \leq 1.0 \quad \text{ok} \quad \text{UF} = 0.174 \\
\text{Right end:} & \quad U = 0.436 \leq 1.0 \quad \text{ok} \quad \text{UF} = 0.436
\end{align*}
\]

**Additional member checks**

**EN 1993-1-1, 6.3.1**

*Strong axis flexural buckling check*

\[
\frac{N_{kr}}{N_{E,cr}} = 0.189 \leq 1.0 \quad \text{ok}
\]

*Weak axis flexural buckling check*

\[
\frac{N_{kr}}{N_{E,cr}} = 0.240 \leq 1.0 \quad \text{ok}
\]

**EN 1993-1-1, 6.3.2**

*Lateral torsional buckling*

\[
\frac{M_{kr}}{M_{E,cr}} = 0.284 \leq 1.0 \quad \text{ok}
\]
Verification of serviceability limit state

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

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

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

2. Verification of horizontal displacement for each floor:

- first floor - $\delta_1 := 1.3 \ < \ \frac{L_{\text{column}}}{300} = 1.333\cdot\text{cm}$
- second floor - $\delta_2 := 0.5 \ < \ \frac{L_{\text{column}}}{300} = 1.333\cdot\text{cm}$
- third floor - $\delta_3 := 0.2 \ < \ \frac{L_{\text{column}}}{300} = 1.333\cdot\text{cm}$

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 requirements of ULS and SLS.**
5.3 Bracing

**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**

- $f_{yd} := 355\text{MPa}$
- $\gamma_{M0} := 1$
- $\gamma_{M1} := 1$
- $E := 210000\text{MPa}$

**Internal forces**

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

**Cross section classification**

$$\varepsilon := \sqrt{\frac{235\text{MPa}}{f_{yd}}}$$

Section in compression

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

The cross section is class 1.
Cross section resistance

Axial force
\[ N_{Ed} = 216.8 \text{kN} < N_{cRd} := A_X \cdot \frac{f_{yd}}{Y_{M0}} = 1.175 \times 10^3 \text{kN} \]

Verification of buckling resistance

Flexural buckling
\[ \lambda_1 := \pi \cdot \sqrt{\frac{E}{f_{yd}}} = 76.409 \]
\[ L_E := 1 \cdot L_b = 7.21 \text{ m} \quad \text{- buckling length} \]
\[ i := \sqrt{\frac{L}{A_X}} = 4.665 \text{ cm} \quad \text{- radius of giration} \]
\[ \lambda := \frac{L_E}{i} = 154.559 \]

\[ \lambda_n := \frac{\lambda}{\lambda_1} = 2.023 \quad \text{- nondimensional skenderness coefficient} \]

hot finished hollow section => curve a, so \( \alpha_1 := 0.21 \)
\[ := 0.5 \left[ 1 + \alpha_1 \left( \lambda_n - 0.2 \right) + \lambda_n^2 \right] = 2.737 \]
\[ X := \frac{1}{\left[ 2 - \lambda_n \right]} = 0.197 \]
\[ N_{bRd} := \frac{X \cdot A_X \cdot f_{yd}}{Y_{M1}} = 231.494 \text{kN} \]
\[ N_{Ed} = 216.8 \text{kN} < N_{bRd} = 231.494 \text{kN} \]

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.

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 minimize 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

\[ A_x := 33.1 \text{cm}^2 \text{- cross section area} \]
\[ I_{y} := 720.29 \text{cm}^4 \text{- moment of inertia of a section around y-axis} \]
\[ I_{z} := 720.29 \text{cm}^4 \text{- moment of inertia of a section around y-axis} \]
\[ h := 14 \text{cm} \text{- diameter} \]
\[ t := 0.8 \text{cm} \text{- web thickness} \]
\[ L_b := 7.21 \text{m} \text{- length of bracing element} \]

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 type 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:

\[ F_{vRd} := \frac{\alpha_v f_{ub} A_b}{\gamma M_2} \]

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

\( \alpha_v := 0.6 \) - for class 8.8

\( f_{ub} := 800 \text{ N/mm}^2 \) - for class 8.8

\( F_{vRd} := 216.8 \text{kN} \)

We obtain the required cross section of the bolts:

\[ A_b := \frac{F_{vRd} \gamma M_2}{\alpha_v f_{ub}} = 564.583 \text{ mm}^2 \]

Taking the bolts with \( d=20\text{mm} \), class 8.8 required quantity of bolts:

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

\[ n := \frac{A_b}{A_{20}} = 1.798 \]

As a result we take 2 bolts class 8.8,

\( d := 20\text{mm} \)

\( d_0 := d + 2\text{mm} = 22\text{mm} \)

Now \( A_{b1} := 2 \cdot A_{20} = 628 \text{mm}^2 \)

Shear resistance of bolts:

\[ F_{vRd} := \frac{\alpha_v f_{ub} A_{b1}}{\gamma M_2} = 241.152 \text{kN} \]

\[ \frac{N_{Ed}}{F_{vRd}} = 0.899 < 1 \]

Verification of bearing resistance

\[ F_{bRd} := \frac{k_1 a_b f_{uac} d \cdot t_p}{\gamma M_2} \]
Characteristics of bolts location:

\[ e_1 := 40\text{mm} \]
\[ e_2 := 40\text{mm} \]
\[ p_2 := 80\text{mm} \]

\[ \alpha_b := \frac{e_1}{3d_0} = 0.606 \]

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

\[ t_p := 10\text{mm} - \text{thickness of plate welded to bracing} \]

\[ t_g := 15\text{mm} - \text{thickness of gusset plate} \]

\[ F_{bRd} := \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t_p}{\gamma M2} = 113.939\cdot\text{kN} \]

Shear force for one bolt:

\[ N_{Ed} := \frac{N_{Ed}}{2} = 108.4\cdot\text{kN} < F_{bRd} = 113.939\cdot\text{kN} \]

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 \( a := 4\text{mm} \)

\[ \beta_w := 0.95 \]

\[ l_c := 320\text{mm} \]

\[ N_{rdw} := 2F_{wRd}^{-1} \]

\[ F_{wRd} := f_{vw} \cdot a \]

\[ f_{vw} := \frac{f_u}{\sqrt{3}} = 228.509 \cdot \frac{N}{\text{mm}^2} \]

\[ F_{wRd} := f_{vw} \cdot a = 914.037 \cdot \frac{N}{\text{mm}} \]

\[ N_{rdw} := 2F_{wRd}^{-1} l_c = 584.983\cdot\text{kN} \]

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

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

Therefore, the horizontal weld is OK.
2. The bracing is welded to the plate bolted to the gusset plate.

we propose \( a_3 := 4 \text{mm} \)
\[ \beta_{w3} := 0.95 \]
\[ l_3 := 150 \text{mm} \]

\[ f_{vw3} := \frac{f_u}{\sqrt{3}} = \frac{228.509}{0.95 \cdot 0.150} \text{ N/mm}^2 \]

\[ F_{wRd3} := f_{vw} \cdot a_3 = 914.037 \text{ N/mm} \]

\[ N_{rdw3} := 4F_{wRd1c} = 1.17 \times 10^3 \text{ kN} \]

\[ N_{Ed} = 216.8 \text{ kN} < N_{rdw3} = 1.17 \times 10^3 \text{ kN} \]

So the welding is OK.
7. References.

5. EN 1990 Basis of structural design.
Annex 1. Calculation of column base joint in software ROBOT.
Pinned column base design


GENERAL

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

GEOMETRY

COLUMN

Section: HEA 450
Bar no.: 29

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lc</td>
<td>12.00 [m]</td>
</tr>
<tr>
<td>α</td>
<td>0.0 [Deg]</td>
</tr>
<tr>
<td>hc</td>
<td>440 [mm]</td>
</tr>
<tr>
<td>bfc</td>
<td>300 [mm]</td>
</tr>
<tr>
<td>twc</td>
<td>12 [mm]</td>
</tr>
<tr>
<td>tfc</td>
<td>21 [mm]</td>
</tr>
<tr>
<td>rc</td>
<td>27 [mm]</td>
</tr>
<tr>
<td>Ac</td>
<td>178.03 [cm²]</td>
</tr>
<tr>
<td>Iyc</td>
<td>63721.60 [cm⁴]</td>
</tr>
</tbody>
</table>

Material: S 355

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>fy</td>
<td>355.00 [MPa]</td>
</tr>
<tr>
<td>fy</td>
<td>470.00 [MPa]</td>
</tr>
</tbody>
</table>

COLUMN BASE

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>lpd</td>
<td>470 [mm]</td>
</tr>
<tr>
<td>bpd</td>
<td>340 [mm]</td>
</tr>
<tr>
<td>tpd</td>
<td>15 [mm]</td>
</tr>
</tbody>
</table>

Material: S 355

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>fy</td>
<td>355.00 [MPa]</td>
</tr>
<tr>
<td>fu</td>
<td>470.00 [MPa]</td>
</tr>
</tbody>
</table>

ANCHORAGE

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

Class = 4.6

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>fy</td>
<td>240.00 [MPa]</td>
</tr>
<tr>
<td>fu</td>
<td>400.00 [MPa]</td>
</tr>
<tr>
<td>d</td>
<td>8 [mm]</td>
</tr>
</tbody>
</table>

Effective section area of a bolt
As = [cm²]  
Aw = 0.50 [cm²] Area of bolt section  
\( n_V = 2 \) Number of bolt columns  
\( n_H = 2 \) Number of bolt rows  
\( \delta_H = 220 \) [mm] Horizontal spacing  
\( \delta_V = 150 \) [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**  
\( \gamma_M = 1.00 \) Partial safety factor  
\( \gamma_M2 = 1.25 \) Partial safety factor  
\( \gamma_C = 1.50 \) 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  
\( f = 9.88 \) [MPa] Design bearing resistance under the base plate  
\( c = \sqrt{\frac{f_p}{3(\gamma_M2)}} \) Additional width of the bearing pressure zone  
\( b_{ef} = 88 \) [mm] Effective width of the bearing pressure zone under the flange  
\( l_{ef} = 340 \) [mm] Effective length of the bearing pressure zone under the flange  
\( A_{c0} = 298.86 \) [cm²] Area of the joint between the base plate and the foundation  
\( A_{c1} = 524.02 \) [cm²] Maximum design area of load distribution  
\( F_{n,rb} = A_{c0} \times f_{cd} \times (A_{c1}/A_{c0}) \leq 3A_{c0} f_{cd} \) Reduction factor for compression  

\( A_{c0} = 524.02 \) [cm²] Maximum design area of load distribution  
\( \delta = 0.67 \) Reduction factor for compression  

\( F_{n,rb} = F_{n,rb}/(b_{atn} \times \delta) \)
RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

\[ N_{j,Rd} = F_{c,Rd,n} = 1102.80 \text{ [kN]} \]

Shear force \( V_{j,Ed} \)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{bd} )</td>
<td>0.37</td>
<td>-</td>
<td>Coeff. for resistance calculation ( F_{1,vb,Rd} )</td>
</tr>
<tr>
<td>( A_{bd} )</td>
<td>0.50</td>
<td>[cm²]</td>
<td>Area of bolt section</td>
</tr>
<tr>
<td>( f_{ub} )</td>
<td>400.00</td>
<td>[MPa]</td>
<td>Tensile strength of the anchor material</td>
</tr>
<tr>
<td>( \gamma_{M2} )</td>
<td>1.25</td>
<td>-</td>
<td>Partial safety factor</td>
</tr>
<tr>
<td>( F_{2,vb,Rd} )</td>
<td>3.52</td>
<td>[kN]</td>
<td>Shear resistance of a bolt - without lever arm</td>
</tr>
<tr>
<td>( M_{Rk,s} )</td>
<td>0.02</td>
<td>[kN*m]</td>
<td>Characteristic bending resistance of an anchor</td>
</tr>
<tr>
<td>( l_{sm} )</td>
<td>29</td>
<td>[mm]</td>
<td>Lever arm length</td>
</tr>
<tr>
<td>( \gamma_{Ma} )</td>
<td>1.20</td>
<td>-</td>
<td>Partial safety factor</td>
</tr>
<tr>
<td>( F_{v,Rd,sm} )</td>
<td>1.04</td>
<td>[kN]</td>
<td>Shear resistance of a bolt - with lever arm</td>
</tr>
<tr>
<td>( N_{Rk,c} )</td>
<td>26.34</td>
<td>[kN]</td>
<td>Design uplift capacity</td>
</tr>
<tr>
<td>( k_{3} )</td>
<td>2.00</td>
<td>-</td>
<td>Factor related to the anchor length</td>
</tr>
<tr>
<td>( \gamma_{Mc} )</td>
<td>2.16</td>
<td>-</td>
<td>Partial safety factor</td>
</tr>
<tr>
<td>( F_{v,Rd,cp} )</td>
<td>24.39</td>
<td>[kN]</td>
<td>Concrete resistance for pry-out failure</td>
</tr>
</tbody>
</table>

CONCRETE PRY-OUT FAILURE

Shear force \( V_{j,Ed,y} \)

\[ V_{j,Ed,y} = 44.33 \text{ [kN]} \]

Shear force \( V_{j,Ed,z} \)

\[ V_{j,Ed,z} = 96.00 \text{ [kN]} \]

Shear of an anchor bolt

\[ \alpha_b = 0.37 \]

\[ \alpha_{ub} = 0.85 \]

\[ k_{1,y} = 2.50 \]

\[ F_{1,vb,Rd,y} = k_{1,y} A_{vb,y} f_{up,d} \]

\[ F_{1,vb,Rd,z} = k_{1,z} A_{vb,z} f_{up,d} \]

Concrete edge failure

Shear force \( V_{j,Ed,y} \)

\[ V_{j,Ed,y} = 0.87 \]

\[ V_{j,Ed,z} = 1.00 \]

\[ V_{j,Ed,ec} = 1.00 \]

\[ V_{j,Ed,fa} = 1.00 \]

\[ V_{j,Ed,fr} = 1.00 \]

\[ V_{j,Ed,ft} = 1.00 \]

\[ V_{j,Ed,fv} = 1.00 \]

\[ V_{j,Ed,fw} = 1.00 \]
Partial safety factor $\gamma_M = 2.16$ CEB [3.2.3.1]

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

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

Characteristic resistance of an anchor $V_{R,k,c,z} = 66.03$ [kN] CEB [9.3.4.(a)]

Factor related to anchor spacing and edge distance $A_{V,z} = 0.51$

Factor related to the foundation thickness $h_{V,z} = 1.00$

Factor related to the influence of edges parallel to the shear load direction $s_{V,z} = 0.85$

Factor related to the angle at which the shear load is applied $\alpha_{V,z} = 1.00$

Factor related to the type of edge reinforcement used $u_{cr, V,z} = 1.00$

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

SPLITTING RESISTANCE

Coeff. of friction between the base plate and concrete $C_f,d = 0.30$ CEB [6.2.2.(6)]

Compressive force $N_{c,Ed} = 1077.93$ [kN] CEB [6.2.2.(6)]

Slip resistance $F_{f, Rd} = C_f,d \times N_{c,Ed}$

SLIDING RESISTANCE

Connection resistance for shear $V_{j,Rd,y} = nb \times \min (F_1, v_{b,Rd,y}, F_2, v_{b,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,y}) + F_{f,Rd}$

Connection resistance for shear $V_{j,Rd,z} = nb \times \min (F_1, v_{b,Rd,z}, F_2, v_{b,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$

RESILIENT COEFFICIENT $\beta_W = 0.90$ CEB [4.5.3.(7)]

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

Normal stress in a weld $\sigma_1 = 72.44$ [MPa] CEB [4.5.3.(7)]

Perpendicular tangent stress $\tau_1 = 72.44$ [MPa] CEB [4.5.3.(7)]

Tangent stress parallel to $V_{j,Ed,y}$ $\tau_{yII} = 0.00$ [MPa] CEB [4.5.3.(7)]

Tangent stress parallel to $V_{j,Ed,z}$ $\tau_{zII} = 21.82$ [MPa] CEB [4.5.3.(7)]

Resistance-dependent coefficient $\beta_W = 0.90$ CEB [4.5.3.(7)]

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


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_p = 450 [mm] Height of the principal beam section
b_f = 190 [mm] Width of the flange of the principal beam section
t_w = 9 [mm] Thickness of the web of the principal beam section
t_f = 15 [mm] Thickness of the flange of the principal beam section
r_f = 21 [mm] Fillet radius of the web of the principal beam section
A_p = 98.82 [cm²] Cross-sectional area of a principal beam
I_{yp} = 33742.90 [cm⁴] Moment of inertia of the principal beam section
Material: S 355
f_y = 355.00 [MPa] Design resistance
f_u = 470.00 [MPa] Tensile resistance

LEFT SIDE

BEAM

Section: IPE 360
Bar no.: 68

α = 0.0 [Deg] Inclination angle
h_p = 360 [mm] Height of beam section
b_f = 170 [mm] Width of beam section
t_w = 8 [mm] Thickness of the web of beam section
t_f = 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

$\alpha = 0.0$ [Deg] Inclination angle

$h_{bd} = 80$ [mm] Height of angle section

$b_{bd} = 80$ [mm] Width of angle section

$t_{bd} = 8$ [mm] Thickness of the flange of angle section

$f_{bd} = 10$ [mm] Fillet radius of the web of angle section

$l_{bd} = 150$ [mm] Angle length

Material: S 355

$f_{yd} = 355.00$ [MPa] Design resistance

$f_{ukd} = 470.00$ [MPa] Tensile resistance

**BOLTS**

BOLTS CONNECTING ANGLE WITH BEAM

The shear plane passes through the UNTHEREDED 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

$\rho_1 = 45$ [mm] Vertical spacing

**RIGHT SIDE**

**BEAM**

Section: IPE 360

Bar no.: 67

$\alpha = 0.0$ [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
**ANGLE**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_{kr}$</td>
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<td>[mm]</td>
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<tr>
<td>$b_{kr}$</td>
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<td>[mm]</td>
</tr>
<tr>
<td>$t_{kr}$</td>
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<td>[mm]</td>
</tr>
<tr>
<td>$r_{kr}$</td>
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<td>[mm]</td>
</tr>
<tr>
<td>$l_{kr}$</td>
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<td>[mm]</td>
</tr>
<tr>
<td><strong>Height of angle section</strong></td>
<td>80</td>
<td>[mm]</td>
</tr>
<tr>
<td><strong>Width of angle section</strong></td>
<td>80</td>
<td>[mm]</td>
</tr>
<tr>
<td><strong>Thickness of the flange of angle section</strong></td>
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<td>[mm]</td>
</tr>
<tr>
<td><strong>Fillet radius of the web of angle section</strong></td>
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<td><strong>Angle length</strong></td>
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<td>[mm]</td>
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**Material:** S 355

<table>
<thead>
<tr>
<th>Parameter</th>
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<tbody>
<tr>
<td>$f_{ykr}$</td>
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<td>$f_{ukr}$</td>
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**BOLTS CONNECTING ANGLE WITH PRINCIPAL BEAM**

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

**Class =** 5.8

<table>
<thead>
<tr>
<th>Parameter</th>
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<tbody>
<tr>
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<tr>
<td>$d_0$</td>
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<td>[mm]</td>
</tr>
<tr>
<td>$A_e$</td>
<td>1.57</td>
<td>[cm²]</td>
</tr>
<tr>
<td>$A_v$</td>
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<td>[cm²]</td>
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<tr>
<td>$f_{ub}$</td>
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<td>[MPa]</td>
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<tr>
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<tr>
<td>$w$</td>
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<tr>
<td>$e_1$</td>
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<tr>
<td>$p_1$</td>
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**BOLTS CONNECTING ANGLE WITH BEAM**

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

**Class =** 5.8

<table>
<thead>
<tr>
<th>Parameter</th>
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<tr>
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<td>[MPa]</td>
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<tr>
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<tr>
<td>$e_1$</td>
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<tr>
<td>$p_1$</td>
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<td>[mm]</td>
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**MATERIAL FACTORS**

<table>
<thead>
<tr>
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<th>Value</th>
<th>Unit</th>
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</thead>
<tbody>
<tr>
<td>$\gamma_{M0}$</td>
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<tr>
<td>$\gamma_{M2}$</td>
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</table>

**LOADS**

Case: 11: COMB1 \((1+2+3+4+5)\ast1.35+(6+7)\ast1.50+10+0+8\ast0.90\)

**LEFT SIDE**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
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</thead>
<tbody>
<tr>
<td>$N_{b2,Ed}$</td>
<td>0.06</td>
<td>[kN]</td>
</tr>
<tr>
<td>$V_{b2,Ed}$</td>
<td>86.37</td>
<td>[kN]</td>
</tr>
<tr>
<td>$M_{b2,Ed}$</td>
<td>0.00</td>
<td>[kN*m]</td>
</tr>
</tbody>
</table>

**RIGHT SIDE**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{b1,Ed}$</td>
<td>-0.06</td>
<td>[kN]</td>
</tr>
<tr>
<td>$V_{b1,Ed}$</td>
<td>86.37</td>
<td>[kN]</td>
</tr>
</tbody>
</table>
RESULTS

LEFT SIDE

BOLTS CONNECTING ANGLE WITH PRINCIPAL BEAM

BOLT CAPACITIES

\[ F_{v,Rd} = 48.25 \text{ [kN]} \quad \text{Shear resistance of the shank of a single bolt} \]
\[ F_{t,Rd} = 56.52 \text{ [kN]} \quad \text{Tensile resistance of a single bolt} \]

Bolt bearing on the angle

**Direction x**

\[ k_{1x} = 1.80 \]  
\[ k_{1x} > 0.0 \]  
\[ \alpha_{bx} = 0.65 \]  
\[ \alpha_{bx} > 0.0 \]  
\[ F_{b,Rd2x} = 56.15 \text{ [kN]} \quad \text{Bearing resistance of a single bolt} \]

**Direction z**

\[ k_{1z} = 2.50 \]  
\[ k_{1z} > 0.0 \]  
\[ \alpha_{bz} = 0.56 \]  
\[ \alpha_{bz} > 0.0 \]  
\[ F_{b,Rd2z} = 66.84 \text{ [kN]} \quad \text{Bearing resistance of a single bolt} \]

FORCES ACTING ON BOLTS IN THE PRINCIPAL BEAM - ANGLE CONNECTION

**Bolt shear**

\[ e = 49 \text{ [mm]} \quad \text{Distance between centroid of a bolt group of an angle and center of the beam web} \]
\[ M_0 = 2.12 \text{ [kN*m]} \quad \text{Real bending moment} \]
\[ F_{Vz} = 14.40 \text{ [kN]} \quad \text{Component force in a bolt due to influence of the shear force} \]
\[ F_{Mx} = 23.51 \text{ [kN]} \quad \text{Component force in a bolt due to influence of the moment} \]
\[ F_{Mx} = \min(2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5) \]
\[ F_{Vz} = \frac{0.5}{M_0}M_{0}^{2}V_{b2,Ed}e \]
\[ F_{Mx} = \frac{0.5}{M_0}M_{0}^{2}Z_{2x}^{2} + 0.5N_{b2,Ed} \]
\[ F_{Rdx} = 48.25 \text{ [kN]} \quad \text{Effective design capacity of a bolt on the direction x} \]
\[ F_{Rdz} = 48.25 \text{ [kN]} \quad \text{Effective design capacity of a bolt on the direction z} \]

**Bolt tension**

\[ e = 50 \text{ [mm]} \quad \text{Distance between centroid of a bolt group and center of the principal beam web} \]
\[ M_{ut} = 2.15 \text{ [kN*m]} \quad \text{Real bending moment} \]
\[ F_{led} = 23.86 \text{ [kN]} \quad \text{Tensile force in the outermost bolt} \]
\[ F_{led} = \frac{0.5}{M_0}M_{0}^{2}Z_{2x}^{2} + 0.5N_{b2,Ed} \]
\[ F_{led} = \frac{0.5}{M_0}M_{0}^{2}Z_{2x}^{2} \]

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

\[ F_{v,Ed} = 27.57 \text{ [kN]} \quad \text{Resultant shear force in a bolt} \]
\[ F_{v,Ed} = \sqrt{F_{Vz}^{2} + F_{Mx}^{2}} \]
\[ F_{v,Ed} = \frac{0.5}{M_0}M_{0}^{2}Z_{2x}^{2} + 0.5N_{b2,Ed} \]

**BOLTS CONNECTING ANGLE WITH BEAM**

**BOLT CAPACITIES**

\[ F_{v,Rd} = 96.51 \text{ [kN]} \quad \text{Shear resistance of the shank of a single bolt} \]

Bolt bearing on the beam

**Direction x**

\[ k_{1x} = 1.80 \]  
\[ k_{1x} > 0.0 \]  
\[ \alpha_{bx} = 0.65 \]  
\[ \alpha_{bx} > 0.0 \]  
\[ F_{b,Rd2x} = k_{1x}^{*}u_{bx}^{*}d_{t}^{*}t_{i}^{*}/M_{2}^{*} \]

mhtml:file://C:\Documents and Settings\Администратор\Мои документы\Autodesk\... 04.11.2012
k_{ix} > 0.0
\alpha_{bx} = 0.65 \quad \text{Coefficient for calculation of } F_{b,Rd}
\alpha_{bx} > 0.0 \quad \text{verified}
F_{b,Rd1x} = 56.15 \quad \text{[kN]} \quad \text{Bearing resistance of a single bolt}

Direction z
k_{iz} = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd}
\alpha_{bz} = 0.58 \quad \text{verified}
F_{b,Rdz} = 70.19 \quad \text{[kN]} \quad \text{Bearing resistance of a single bolt}

Bolt bearing on the angle

FORCES ACTING ON BOLTS IN THE ANGLE - BEAM CONNECTION

Bolt shear
\varepsilon = 50 \quad \text{[mm]} \quad \text{Distance between centroid of a bolt group and center of the principal beam web}
M_0 = 4.29 \quad \text{[kN*m]} \quad \text{Real bending moment}
F_{Nx} = 0.02 \quad \text{[kN]} \quad \text{Component force in a bolt due to influence of the longitudinal force}
F_{Vz} = 28.79 \quad \text{[kN]} \quad \text{Component force in a bolt due to influence of the shear force}
F_{Mx} = 47.70 \quad \text{[kN]} \quad \text{Component force in a bolt due to influence of the moment on the x direction}
F_{Mz} = 0.00 \quad \text{[kN]} \quad \text{Component force in a bolt due to influence of the moment on the z direction}
F_{x,Ed} = 47.72 \quad \text{[kN]} \quad \text{Design total force in a bolt on the direction x}
F_{z,Ed} = 28.79 \quad \text{[kN]} \quad \text{Design total force in a bolt on the direction z}
F_{Rdx} = 56.15 \quad \text{[kN]} \quad \text{Effective design capacity of a bolt on the direction x}
F_{Rdz} = 70.19 \quad \text{[kN]} \quad \text{Effective design capacity of a bolt on the direction z}
|F_{x,Ed}| \leq |F_{Rdx}| \quad \text{verified (0.85)}
|F_{z,Ed}| \leq |F_{Rdz}| \quad \text{verified (0.41)}

VERIFICATION OF THE SECTION DUE TO BLOCK TEARING

ANGLE
A_{ox} = 2.08 \quad \text{[cm}^2\text{]} \quad \text{Net area of the section in tension}
A_{oz} = 6.00 \quad \text{[cm}^2\text{]} \quad \text{Area of the section in shear}
V_{effRd} = 162.08 \quad \text{[kN]} \quad \text{Design capacity of a section weakened by openings}
|0.5V_{b2,Ed}| \leq V_{effRd} \quad |43.19| < 162.08 \quad \text{verified (0.27)}

BEAM
A_{ox} = 2.08 \quad \text{[cm}^2\text{]} \quad \text{Net area of the section in tension}
A_{oz} = 12.40 \quad \text{[cm}^2\text{]} \quad \text{Area of the section in shear}
V_{effRd} = 293.25 \quad \text{[kN]} \quad \text{Design capacity of a section weakened by openings}
|V_{b2,Ed}| \leq V_{effRd} \quad |86.37| < 293.25 \quad \text{verified (0.29)}

RIGHT SIDE
BOLTS CONNECTING ANGLE WITH PRINCIPAL BEAM

**BOLT CAPACITIES**

\[ F_{v,Rd} = 48.25 \text{ [kN]} \quad \text{Shear resistance of the shank of a single bolt} \]
\[ F_{t,Rd} = 56.52 \text{ [kN]} \quad \text{Tensile resistance of a single bolt} \]

**Bolt bearing on the angle**

**Direction x**

- \[ k_{1x} = 1.80 \]
- \[ \alpha_{bx} = 0.65 \]
- \[ F_{b,Rd2x} = 56.15 \text{ [kN]} \quad \text{Bearing resistance of a single bolt} \]

**Direction z**

- \[ k_{1z} = 2.50 \]
- \[ \alpha_{bz} = 0.56 \]
- \[ F_{b,Rd2z} = 66.84 \text{ [kN]} \quad \text{Bearing resistance of a single bolt} \]

**FORCES ACTING ON BOLTS IN THE PRINCIPAL BEAM - ANGLE CONNECTION**

**Bolt shear**

- \[ e = 49 \text{ [mm]} \quad \text{Distance between centroid of a bolt group of an angle and center of the beam web} \]
- \[ M_0 = 2.12 \text{ [kN} \cdot \text{m]} \quad \text{Real bending moment} \]
- \[ F_{Vx} = 14.40 \text{ [kN]} \quad \text{Component force in a bolt due to influence of the shear force} \]
- \[ F_{Mx} = 23.51 \text{ [kN]} \quad \text{Component force in a bolt due to influence of the moment} \]
- \[ F_{Vx1,Ed} = 23.51 \text{ [kN]} \quad \text{Design total force in a bolt on the direction x} \]
- \[ F_{Rdx} = 48.25 \text{ [kN]} \quad \text{Effective design capacity of a bolt on the direction x} \]
- \[ \vert F_{Vx1,Ed} \vert \leq F_{Rdx} \]
  - \[ 23.51 \leq 48.25 \quad \text{verified} \quad (0.49) \]

**Bolt tension**

- \[ e = 50 \text{ [mm]} \quad \text{Distance between centroid of a bolt group and center of the principal beam web} \]
- \[ M_0 = 2.15 \text{ [kN} \cdot \text{m]} \quad \text{Real bending moment} \]
- \[ F_{MEd} = 23.84 \text{ [kN]} \quad \text{Tensile force in the outermost bolt} \]
- \[ F_{MEd} \leq F_{Lrd} \]
  - \[ 23.84 \leq 56.52 \quad \text{verified} \quad (0.42) \]

**BOLTS CONNECTING ANGLE WITH BEAM**

**BOLT CAPACITIES**

\[ F_{v,Rd} = 96.51 \text{ [kN]} \quad \text{Shear resistance of the shank of a single bolt} \]

**Bolt bearing on the beam**

**Direction x**

- \[ k_{1x} = 1.80 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
  - \[ 1.80 > 0.00 \quad \text{verified} \]
- \[ \alpha_{bx} = 0.65 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
  - \[ 0.65 > 0.00 \quad \text{verified} \]
- \[ F_{b,Rdx} = 56.15 \text{ [kN]} \quad \text{Bearing resistance of a single bolt} \]

**Direction z**
\[ k_{1z} = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd} \quad k_{1z} = \min[2.8(\varepsilon_2/d_0)-1.7, 2.5] \]

\[ k_{1z} > 0.0 \quad 2.50 > 0.0 \quad \text{verified} \]

\[ F_{b,Rd1z} = 70.19 \quad [\text{kN}] \quad \text{Bearing resistance of a single bolt} \quad F_{b,Rd1z} = k_{1z} \cdot \alpha_{bz} \cdot \sigma_u \cdot d \cdot t_i / \gamma_{M2} \]

**Bolt bearing on the angle**

**Direction x**

\[ k_{1x} = 1.80 \quad \text{Coefficient for calculation of } F_{b,Rd} \quad k_{1x} = \min[2.8(\varepsilon_1/d_0)-1.7, 1.4(\rho_1/d_0)-1.7, 2.5] \]

\[ k_{1x} > 0.0 \quad 1.80 > 0.0 \quad \text{verified} \]

\[ F_{b,Rd2x} = 112.30 \quad [\text{kN}] \quad \text{Bearing resistance of a single bolt} \quad F_{b,Rd2x} = k_{1x} \cdot \alpha_{bx} \cdot \sigma_u \cdot d \cdot t_i / \gamma_{M2} \]

**Direction z**

\[ k_{1z} = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd} \quad k_{1z} = \min[2.8(\varepsilon_2/d_0)-1.7, 2.5] \]

\[ k_{1z} > 0.0 \quad 2.50 > 0.0 \quad \text{verified} \]

\[ F_{b,Rd2z} = 133.69 \quad [\text{kN}] \quad \text{Bearing resistance of a single bolt} \quad F_{b,Rd2z} = k_{1z} \cdot \alpha_{bz} \cdot \sigma_u \cdot d \cdot t_i / \gamma_{M2} \]

**FORCES ACTING ON BOLTS IN THE ANGLE - BEAM CONNECTION**

**Bolt shear**

\[ e = 50 \quad [\text{mm}] \quad \text{Distance between centroid of a bolt group and center of the principal beam web} \]

\[ M_0 = 4.29 \quad [\text{kN} \cdot \text{m}] \quad \text{Real bending moment} \quad M_0 = V_{b1,Ed} \cdot e \]

\[ F_{Nx} = 0.02 \quad [\text{kN}] \quad \text{Component force in a bolt due to influence of the longitudinal force} \quad F_{Nx} = N_{b1,Ed} / n \]

\[ F_{Vz} = 28.79 \quad [\text{kN}] \quad \text{Component force in a bolt due to influence of the shear force} \quad F_{Vz} = V_{b1,Ed} / n \]

\[ F_{mx} = 47.70 \quad [\text{kN}] \quad \text{Component force in a bolt due to influence of the moment on the x direction} \quad F_{mx} = M_0 / z_x / (x^2 + z_x^2) \]

\[ F_{mz} = 0.00 \quad [\text{kN}] \quad \text{Component force in a bolt due to influence of the moment on the z direction} \quad F_{mz} = M_0 / z_z / (x^2 + z_z^2) \]

\[ F_{x,Ed} = 47.72 \quad [\text{kN}] \quad \text{Design total force in a bolt on the direction x} \quad F_{x,Ed} = F_{Nx} + F_{mx} \]

\[ F_{z,Ed} = 28.79 \quad [\text{kN}] \quad \text{Design total force in a bolt on the direction z} \quad F_{z,Ed} = F_{Vz} + F_{mz} \]

\[ F_{Rdx} = 56.15 \quad [\text{kN}] \quad \text{Effective design capacity of a bolt on the direction x} \quad F_{Rdx} = \min(F_{vRd} \cdot F_{Rd1x}, F_{Rd2x}) \]

\[ F_{Rdz} = 70.19 \quad [\text{kN}] \quad \text{Effective design capacity of a bolt on the direction z} \quad F_{Rdz} = \min(F_{vRd} \cdot F_{Rd1z}, F_{Rd2z}) \]

\[ |F_{x,Ed}| \leq F_{Rdx} \quad |47.72| < 56.15 \quad \text{verified} \quad (0.85) \]

\[ |F_{z,Ed}| \leq F_{Rdz} \quad |28.79| < 70.19 \quad \text{verified} \quad (0.41) \]

**VERIFICATION OF THE SECTION DUE TO BLOCK TEARING**

**ANGLE**

\[ A_{b} = 2.08 \quad [\text{cm}^2] \quad \text{Net area of the section in tension} \]

\[ A_{b} = 6.00 \quad [\text{cm}^2] \quad \text{Area of the section in shear} \]

\[ V_{effRd} = 162.08 \quad [\text{kN}] \quad \text{Design capacity of a section weakened by openings} \quad V_{effRd} = 0.5 \cdot \sigma_u \cdot A_{b} / \gamma_{M2} + (1/3) \cdot \sigma_y \cdot A_{b} / \gamma_{M0} \]

\[ |0.5 \cdot V_{b1,Ed}| \leq V_{effRd} \quad |43.19| < 162.08 \quad \text{verified} \quad (0.27) \]

**BEAM**

\[ A_{b} = 2.08 \quad [\text{cm}^2] \quad \text{Net area of the section in tension} \]

\[ A_{b} = 12.40 \quad [\text{cm}^2] \quad \text{Area of the section in shear} \]

\[ V_{effRd} = 293.25 \quad [\text{kN}] \quad \text{Design capacity of a section weakened by openings} \quad V_{effRd} = 0.5 \cdot \sigma_u \cdot A_{b} / \gamma_{M2} + (1/3) \cdot \sigma_y \cdot A_{b} / \gamma_{M0} \]

\[ |V_{b1,Ed}| \leq V_{effRd} \quad |86.37| < 293.25 \quad \text{verified} \quad (0.29) \]

**VERIFICATION OF PRINCIPAL BEAM**

**BOLT BEARING ON THE PRINCIPAL BEAM WEB**

**Direction x**

\[ k_{sx} = 1.80 \quad \text{Coefficient for calculation of } F_{b,Rd} \quad k_{sx} = \min[2.8(\varepsilon_1/d_0)-1.7, 1.4(\rho_1/d_0)-1.7, 2.5] \]

\[ k_{sx} > 0.0 \quad 1.80 > 0.0 \quad \text{verified} \]

\[ \alpha_{sx} = 1.00 \quad \text{Coefficient for calculation of } F_{b,Rd} \quad \alpha_{sx} = \min[\sigma_y/(3d_0), f_u/(3d_0)-0.25, f_{ub}/f_u-1] \]

\[ \alpha_{sx} > 0.0 \quad 1.00 > 0.0 \quad \text{verified} \]
\[ F_{b,Rdx} = 101.79 \text{ [kN]} \] Bearing resistance of a single bolt

Direction z

\[ k_z = 2.50 \]

Coefficient for calculation of \( F_{b,Rd} \)

\[ k_z > 0.0 \]

\( k_z = \min\{2.8 \cdot (e_2/d_0) - 1.7, 2.5\} \]

\[ \alpha_{bz} = 0.58 \]

Coefficient for calculation of \( F_{b,Rd} \)

\[ \alpha_{bz} = \min\{e_1/(3d_0), p_1/(3d_0) - 0.25, f_{ub}/f_u, 1\} \]

\[ F_{b,Rdz} = 82.47 \text{ [kN]} \] Bearing resistance of a single bolt

RESULTANT FORCE ACTING ON THE OUTERMOST BOLT

\[ F_{x,Ed} = 47.03 \text{ [kN]} \] Design total force in a bolt on the direction x

\[ F_{z,Ed} = 28.79 \text{ [kN]} \] Design total force in a bolt on the direction z

\[ |F_{x,Ed}| \leq F_{b,Rdx} \]

\[ |47.03| < 101.79 \quad \text{verified} \]

\[ (0.46) \]

\[ |F_{z,Ed}| \leq F_{b,Rdz} \]

\[ |28.79| < 82.47 \quad \text{verified} \]

\[ (0.35) \]

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

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
hc = 290 [mm] Height of column section
bfc = 300 [mm] Width of column section
twc = 9 [mm] Thickness of the web of column section
tfc = 14 [mm] Thickness of the flange of column section
rc = 27 [mm] Radius of column section fillet
Ac = 112.53 [cm²] Cross-sectional area of a column
Ixc = 18263.50 [cm⁴] Moment of inertia of the column section
Material: S 355
fyc = 355.00 [MPa] Resistance

BEAM
Section: IPE 450
Bar no.: 62
α = -0.0 [Deg] Inclination angle
**hb** = 450 [mm] Height of beam section
**bf** = 190 [mm] Width of beam section
**twb** = 9 [mm] Thickness of the web of beam section
**tfb** = 15 [mm] Thickness of the flange of beam section
**rb** = 21 [mm] Radius of beam section fillet
**Ab** = 98.82 [cm²] Cross-sectional area of a beam
**Ixb** = 33742.90 [cm⁴] Moment of inertia of the beam section

**Material:** S 355
**fyb** = 355.00 [MPa] Resistance

**BOLTS**
The shear plane passes through the UNTREATED portion of the bolt.
**d** = 20 [mm] Bolt diameter
**Class** = 9.8 Bolt class
**FtRd** = 158.76 [kN] Tensile resistance of a bolt
**nh** = 2 Number of bolt columns
**nv** = 3 Number of bolt rows
**h1** = 80 [mm] Distance between first bolt and upper edge of front plate
**Horizontal spacing e1** = 100 [mm]
**Vertical spacing p1** = 110,180 [mm]

**PLATE**
**hp** = 490 [mm] Plate height
**bp** = 210 [mm] Plate width
**tp** = 20 [mm] Plate thickness
**Material:** S 355
**fyp** = 355.00 [MPa] Resistance

**FILLET WELDS**
**aw** = 7 [mm] Web weld
**af** = 11 [mm] Flange weld

**MATERIAL FACTORS**
**γM0** = 1.00 Partial safety factor [2.2]
**γM1** = 1.00 Partial safety factor [2.2]
**γM2** = 1.25 Partial safety factor [2.2]
**γM3** = 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

**M̄b1,Ed** = 156.52 [kN*m] Bending moment in the right beam
**V̄b1,Ed** = 78.98 [kN] Shear force in the right beam
**N̄b1,Ed** = -4.32 [kN] Axial force in the right beam
**M̄b2,Ed** = -10.67 [kN*m] Bending moment in the left beam
**V̄b2,Ed** = 17.64 [kN] Shear force in the left beam
**N̄b2,Ed** = 1.35 [kN] Axial force in the left beam
**M̄c1,Ed** = -116.79 [kN*m] Bending moment in the lower column
\[ V_{c1,Ed} = -29.20 \text{ [kN]} \]  Shear force in the lower column
\[ N_{c1,Ed} = -575.28 \text{ [kN]} \]  Axial force in the lower column
\[ M_{c2,Ed} = 50.30 \text{ [kN*m]} \]  Bending moment in the upper column
\[ V_{c2,Ed} = 27.49 \text{ [kN]} \]  Shear force in the upper column
\[ N_{c2,Ed} = -398.93 \text{ [kN]} \]  Axial force in the upper column

**RESULTS**

**BEAM RESISTANCES**

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

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

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

**BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT**
\[ W_{pl} = 1701.92 \text{ [cm}^3\text{]} \]  Plastic section modulus
\[ M_{cb,Rd} = W_{pl} f_{yb} / \gamma_M \]  Design resistance of the section for bending
\[ M_{cb,Rd} = 604.18 \text{ [kN*m]} \]  EN1993-1-1:[6.2.5]

**FLANGE AND WEB - COMPRESSION**
\[ M_{cb,Rd} = 604.18 \text{ [kN*m]} \]  Design resistance of the section for bending
\[ h_f = 328 \text{ [mm]} \]  Distance between the centroids of flanges
\[ F_{c,fb,Rd} = M_{cb,Rd} / h_f \]  Resistance of the compressed flange and web
\[ F_{c,fb,Rd} = 1387.65 \text{ [kN]} \]  EN1993-1-1:[6.2.6.7.(1)]

**COLUMN RESISTANCES**

**WEB PANEL - SHEAR**
\[ M_{b1,Ed} = 156.52 \text{ [kN*m]} \]  Bending moment (right beam)
\[ M_{b2,Ed} = -10.67 \text{ [kN*m]} \]  Bending moment (left beam)
\[ V_{c1,Ed} = -29.20 \text{ [kN]} \]  Shear force (lower column)
\[ V_{c2,Ed} = 27.49 \text{ [kN]} \]  Shear force (upper column)
\[ z = 328 \text{ [mm]} \]  Lever arm
\[ V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2 \]  Shear force acting on the web panel
\[ V_{wp,Ed} = 538.51 \text{ [kN]} \]  EN1993-1-1:[5.3.3]
\[ A_{sv} = 37.28 \text{ [cm}^2\text{]} \]  Shear area of the column web
\[ A_{sc} = 37.28 \text{ [cm}^2\text{]} \]  Shear area
\[ V_{wp,Rd} = 0.9(f_{y,wc}A_{vc} + f_{y,wp}A_{vp} + f_{ys}A_{vd}) / (\sqrt{3} \gamma_M) \]  EN1993-1-1:[6.2.6.3]
\[ V_{wp,Rd} = 687.64 \text{ [kN]} \] Resistance of the column web panel for shear [6.2.6.1]

\[ V_{wp,Ed} / V_{wp,Rd} \leq 1.0 \]
\[ 0.78 < 1.00 \quad \text{verified} \] (0.78)

**WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE**

**Bearing:**
- \( t_{wc} = 9 \text{ [mm]} \) Effective thickness of the column web [6.2.6.2.(6)]
- \( b_{eff,c,wc} = 291 \text{ [mm]} \) Effective width of the web for compression [6.2.6.2.(1)]
- \( A_{vc} = 37.28 \text{ [cm}^2\text{]} \) Shear area EN1993-1-1:6.2.6.(3)]
- \( \omega = 0.78 \) Reduction factor for interaction with shear [6.2.6.2.(1)]
- \( \sigma_{com,Ed} = 117.63 \text{ [MPa]} \) 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)]

\[ F_{c,wc,Rd1} = \omega k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0} \]
\[ F_{c,wc,Rd1} = 685.12 \text{ [kN]} \] Column web resistance [6.2.6.2.(1)]

**Buckling:**
- \( d_{wc} = 208 \text{ [mm]} \) Height of compressed web [6.2.6.2.(1)]
- \( \lambda_p = 1.11 \) Plate slenderness of an element [6.2.6.2.(1)]
- \( \rho = 0.74 \) Reduction factor for element buckling [6.2.6.2.(1)]

\[ F_{c,wb,Rd2} = \omega k_{wc} \rho b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M1} \]
\[ F_{c,wc,Rd2} = 506.52 \text{ [kN]} \] Column web resistance [6.2.6.2.(1)]

**Final resistance:**
\[ F_{c,wc,Rd,low} = \text{Min} (F_{c,wc,Rd1} , F_{c,wc,Rd2}) \]
\[ F_{c,wc,Rd,upp} = 506.52 \text{ [kN]} \] Column web resistance [6.2.6.2.(1)]

**WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE**

**Bearing:**
- \( t_{wc} = 9 \text{ [mm]} \) Effective thickness of the column web [6.2.6.2.(6)]
- \( b_{eff,c,wc} = 291 \text{ [mm]} \) Effective width of the web for compression [6.2.6.2.(1)]
- \( A_{vc} = 37.28 \text{ [cm}^2\text{]} \) Shear area EN1993-1-1:6.2.6.(3)]
- \( \omega = 0.78 \) Reduction factor for interaction with shear [6.2.6.2.(1)]
- \( \sigma_{com,Ed} = 117.63 \text{ [MPa]} \) 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)]

\[ F_{c,wc,Rd1} = \omega k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0} \]
\[ F_{c,wc,Rd1} = 685.12 \text{ [kN]} \] Column web resistance [6.2.6.2.(1)]

**Buckling:**
- \( d_{wc} = 208 \text{ [mm]} \) Height of compressed web [6.2.6.2.(1)]
- \( \lambda_p = 1.11 \) Plate slenderness of an element [6.2.6.2.(1)]
- \( \rho = 0.74 \) Reduction factor for element buckling [6.2.6.2.(1)]

\[ F_{c,wb,Rd2} = \omega k_{wc} \rho b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M1} \]
\[ F_{c,wc,Rd2} = 506.52 \text{ [kN]} \] Column web resistance [6.2.6.2.(1)]

**Final resistance:**
\[ F_{c,wc,Rd,upp} = \text{Min} (F_{c,wc,Rd1} , F_{c,wc,Rd2}) \]
\[ F_{c,wc,Rd,upp} = 506.52 \text{ [kN]} \] Column web resistance [6.2.6.2.(1)]

**GEOMETRICAL PARAMETERS OF A CONNECTION**
EFFECTIVE LENGTHS AND PARAMETERS - COLUMN FLANGE

<table>
<thead>
<tr>
<th>Nr</th>
<th>m</th>
<th>(m_x)</th>
<th>e</th>
<th>(e_x)</th>
<th>p</th>
<th>(l_{\text{eff,cp}})</th>
<th>(l_{\text{eff,nc}})</th>
<th>(l_{\text{eff,1}})</th>
<th>(l_{\text{eff,2}})</th>
<th>(l_{\text{eff,cp,g}})</th>
<th>(l_{\text{eff,nc,g}})</th>
<th>(l_{\text{eff,1,g}})</th>
<th>(l_{\text{eff,2,g}})</th>
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<tr>
<td>1</td>
<td>24</td>
<td>-</td>
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<td>-</td>
<td>110</td>
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<td>191</td>
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<td>152</td>
<td>222</td>
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<td>222</td>
<td>256</td>
<td>201</td>
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</tbody>
</table>

\(m\) – 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
\(l_{\text{eff,cp}}\) – Effective length for a single bolt in the circular failure mode
\(l_{\text{eff,nc}}\) – Effective length for a single bolt in the non-circular failure mode
\(l_{\text{eff,1}}\) – Effective length for a single bolt for mode 1
\(l_{\text{eff,2}}\) – Effective length for a single bolt for mode 2
\(l_{\text{eff,cp,g}}\) – Effective length for a group of bolts in the circular failure mode
\(l_{\text{eff,nc,g}}\) – Effective length for a group of bolts in the non-circular failure mode
\(l_{\text{eff,1,g}}\) – Effective length for a group of bolts for mode 1
\(l_{\text{eff,2,g}}\) – Effective length for a group of bolts for mode 2

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

<table>
<thead>
<tr>
<th>Nr</th>
<th>m</th>
<th>(m_x)</th>
<th>e</th>
<th>(e_x)</th>
<th>p</th>
<th>(l_{\text{eff,cp}})</th>
<th>(l_{\text{eff,nc}})</th>
<th>(l_{\text{eff,1}})</th>
<th>(l_{\text{eff,2}})</th>
<th>(l_{\text{eff,cp,g}})</th>
<th>(l_{\text{eff,nc,g}})</th>
<th>(l_{\text{eff,1,g}})</th>
<th>(l_{\text{eff,2,g}})</th>
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<td>180</td>
<td>235</td>
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<td>218</td>
<td>297</td>
<td>199</td>
<td>199</td>
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</tr>
</tbody>
</table>

\(m\) – 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
\(l_{\text{eff,cp}}\) – Effective length for a single bolt in the circular failure mode
\(l_{\text{eff,nc}}\) – Effective length for a single bolt in the non-circular failure mode
\(l_{\text{eff,1}}\) – Effective length for a single bolt for mode 1
\(l_{\text{eff,2}}\) – Effective length for a single bolt for mode 2
\(l_{\text{eff,cp,g}}\) – Effective length for a group of bolts in the circular failure mode
\(l_{\text{eff,nc,g}}\) – Effective length for a group of bolts in the non-circular failure mode
\(l_{\text{eff,1,g}}\) – Effective length for a group of bolts for mode 1
\(l_{\text{eff,2,g}}\) – Effective length for a group of bolts for mode 2

CONNECTION RESISTANCE FOR COMPRESSION

\[
N_{j,Rd} = \min (N_{j,Rd} \cdot 2 F_{c,wc,Rd,low} \cdot 2 F_{c,wc,Rd,upp})
\]

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

\[
N_{b1,Ed} / N_{j,Rd} \leq 1.0 \quad 0.00 < 1.00 \quad \text{verified} \quad (0.00)
\]

CONNECTION RESISTANCE FOR BENDING

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

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

\[
F_{t,fc,Rd} \quad \text{– column flange resistance due to bending}
\]

\[
F_{t,wc,Rd} \quad \text{– column web resistance due to tension}
\]

\[
F_{t,ep,Rd} \quad \text{– resistance of the front plate due to bending}
\]

\[
F_{t,wb,Rd} \quad \text{– resistance of the web in tension}
\]

\[
F_{t,fc,Rd} = \min (F_{T,1,fc,Rd} \cdot F_{T,2,fc,Rd} \cdot F_{T,3,fc,Rd}) \quad [6.2.6.4], [Tab.6.2]
\]

\[
F_{t,wc,Rd} = b_{eff,t,wc} f_{yc} / \gamma_{M0} \quad [6.2.6.3.1(1)]
\]

\[
F_{t,ep,Rd} = \min (F_{T,1,ep,Rd} \cdot F_{T,2,ep,Rd} \cdot F_{T,3,ep,Rd}) \quad [6.2.6.5], [Tab.6.2]
\]

\[
F_{t,wb,Rd} = b_{eff,t,wb} f_{yb} / \gamma_{M0} \quad [6.2.6.8.1(1)]
\]
### RESISTANCE OF THE BOLT ROW NO. 2

<table>
<thead>
<tr>
<th>Formula</th>
<th>Component</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{t_{1,Rd,comp}} = 298.56$</td>
<td>Column flange - tension</td>
</tr>
<tr>
<td>$F_{t_{2,Rd,comp}} = 317.52$</td>
<td>Column flange - tension</td>
</tr>
<tr>
<td>$F_{t_{3,Rd,comp}} = 317.52$</td>
<td>Column flange - tension</td>
</tr>
<tr>
<td>$B_{p,Rd} = 595.34$</td>
<td>Bolts due to shear punching</td>
</tr>
<tr>
<td>$V_{wp,Rd} = 643.77$</td>
<td>Web panel - shear</td>
</tr>
<tr>
<td>$F_{c,wc,Rd} = 506.52$</td>
<td>Column web - compression</td>
</tr>
<tr>
<td>$F_{c,fb,Rd} = 1387.65$</td>
<td>Beam web - compression</td>
</tr>
<tr>
<td>$F_{t_{1,Rd}} = \min(F_{t_{1,Rd,comp}})$</td>
<td>Bolt row resistance</td>
</tr>
<tr>
<td>$F_{t_{2,Rd}} = \min(F_{t_{2,Rd,comp}})$</td>
<td>Bolt row resistance</td>
</tr>
<tr>
<td>$F_{t_{3,Rd}} = \min(F_{t_{3,Rd,comp}})$</td>
<td>Bolt row resistance</td>
</tr>
</tbody>
</table>

### RESISTANCE OF THE BOLT ROW NO. 3

<table>
<thead>
<tr>
<th>Formula</th>
<th>Component</th>
</tr>
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<tbody>
<tr>
<td>$F_{t_{1,Rd,comp}} = 298.56$</td>
<td>Column flange - tension</td>
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<tr>
<td>$F_{t_{2,Rd,comp}} = 317.52$</td>
<td>Column flange - tension</td>
</tr>
<tr>
<td>$F_{t_{3,Rd,comp}} = 317.52$</td>
<td>Column flange - tension</td>
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<tr>
<td>$B_{p,Rd} = 595.34$</td>
<td>Bolts due to shear punching</td>
</tr>
<tr>
<td>$V_{wp,Rd} = 643.77$</td>
<td>Web panel - shear</td>
</tr>
<tr>
<td>$F_{c,wc,Rd} = 506.52$</td>
<td>Column web - compression</td>
</tr>
<tr>
<td>$F_{c,fb,Rd} = 1387.65$</td>
<td>Beam web - compression</td>
</tr>
<tr>
<td>$F_{t_{1,Rd}} = \min(F_{t_{1,Rd,comp}})$</td>
<td>Bolt row resistance</td>
</tr>
<tr>
<td>$F_{t_{2,Rd}} = \min(F_{t_{2,Rd,comp}})$</td>
<td>Bolt row resistance</td>
</tr>
<tr>
<td>$F_{t_{3,Rd}} = \min(F_{t_{3,Rd,comp}})$</td>
<td>Bolt row resistance</td>
</tr>
</tbody>
</table>
SUMMARY TABLE OF FORCES

<table>
<thead>
<tr>
<th>Nr</th>
<th>( h_j )</th>
<th>( F_{t,\text{ep},Rd} )</th>
<th>( F_{t,\text{fc},Rd} )</th>
<th>( F_{t,\text{wc},Rd} )</th>
<th>( F_{t,\text{ep},Rd} )</th>
<th>( F_{t,\text{wb},Rd} )</th>
<th>( F_{t,\text{Rd}} )</th>
<th>( B_{p,Rd} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>383</td>
<td>298.56</td>
<td>298.56</td>
<td>425.93</td>
<td>317.52</td>
<td>783.75</td>
<td>317.52</td>
<td>595.34</td>
</tr>
<tr>
<td>2</td>
<td>273</td>
<td>207.96</td>
<td>317.52</td>
<td>425.93</td>
<td>317.52</td>
<td>728.37</td>
<td>317.52</td>
<td>595.34</td>
</tr>
<tr>
<td>3</td>
<td>93</td>
<td>-</td>
<td>317.52</td>
<td>425.93</td>
<td>317.52</td>
<td>728.37</td>
<td>317.52</td>
<td>595.34</td>
</tr>
</tbody>
</table>

CONNECTION RESISTANCE FOR BENDING \( M_{j,Rd} \)
\[
M_{j,Rd} = \sum h_j F_{t,j,Rd}
\]

Connection resistance for bending \([6.2]\)

\[
M_{b1,Ed}/M_{j,Rd} \leq 1.0
\]

Verifiable \((0.92)\)

CONNECTION RESISTANCE FOR SHEAR

\[
\alpha_v = 0.60 \quad \text{Coefficient for calculation of } F_{v,Rd}
\]

Shear resistance of a single bolt \([\text{Table 3.4}]\)

\[
F_{v,Rd} = 135.72 \quad [\text{kN}]
\]

Tensile resistance of a single bolt \([\text{Table 3.4}]\)

\[
F_{t,\text{Rd,max}} = 158.76 \quad [\text{kN}]
\]

Bearing resistance of an intermediate bolt \([\text{Table 3.4}]\)

\[
F_{b,Rd,int} = 263.20 \quad [\text{kN}]
\]

Bearing resistance of an outermost bolt \([\text{Table 3.4}]\)

\[
F_{b,Rd,ext} = 263.20 \quad [\text{kN}]
\]

<table>
<thead>
<tr>
<th>Nr</th>
<th>( F_{t,j,Rd,N} )</th>
<th>( F_{t,j,Ed,N} )</th>
<th>( F_{t,j,Rd,M} )</th>
<th>( F_{t,j,Ed,M} )</th>
<th>( F_{t,j,Ed} )</th>
<th>( F_{v,j,Rd} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>317.52</td>
<td>-1.44</td>
<td>298.56</td>
<td>273.32</td>
<td>271.88</td>
<td>105.42</td>
</tr>
<tr>
<td>2</td>
<td>317.52</td>
<td>-1.44</td>
<td>207.96</td>
<td>190.38</td>
<td>188.94</td>
<td>156.07</td>
</tr>
<tr>
<td>3</td>
<td>317.52</td>
<td>-1.44</td>
<td>317.52</td>
<td>0.00</td>
<td>-1.44</td>
<td>271.43</td>
</tr>
</tbody>
</table>

\( F_{t,j,Rd,N} \) – Bolt row resistance for simple tension

\( F_{t,j,Ed,N} \) – Force due to axial force in a bolt row

\( F_{t,j,Rd,M} \) – Bolt row resistance for simple bending

\( F_{t,j,Ed,M} \) – Force due to moment in a bolt row

\( F_{t,j,Ed} \) – Maximum tensile force in a bolt row

\( F_{v,j,Rd} \) – Reduced bolt row resistance

\[
F_{t,j,Ed,N} = N_{j,Ed} F_{t,j,Rd,N} / N_{j,Rd}
\]

\[
F_{t,j,Ed,M} = M_{j,Ed} F_{t,j,Rd,M} / M_{j,Rd}
\]

\[
F_{t,j,Ed} = F_{t,j,Ed,N} + F_{t,j,Ed,M}
\]

\[
F_{v,j,Rd} = \min (n_h F_{v,Rd} (1 - F_{t,j,Ed} / (1.4 n_h F_{t,Rd,max}) n_h F_{v,Rd} \cdot n_h F_{b,Rd}))
\]

Connection resistance for shear \([\text{Table 3.4}]\)

\[
V_{j,Rd} = 532.92 \quad [\text{kN}]
\]

\[
V_{b1,Ed} / V_{j,Rd} \leq 1.0
\]

Verifiable \((0.15)\)

WELD RESISTANCE

\[
A_w = 115.19 \quad [\text{cm}^2] \quad \text{Area of all welds} \quad [4.5.3.2(2)]
\]

\[
A_{wy} = 62.16 \quad [\text{cm}^2] \quad \text{Area of horizontal welds} \quad [4.5.3.2(2)]
\]
\[
\begin{align*}
A_{wz} &= 53.03 \text{ [cm}^2\text{]} \quad \text{Area of vertical welds} \\
I_{wy} &= 36281.76 \text{ [cm}^4\text{]} \quad \text{Moment of inertia of the weld arrangement with respect to the hor. axis} \\
\sigma_{\text{max}} &= -70.69 \text{ [MPa]} \quad \text{Normal stress in a weld} \\
\tau_{\text{max}} &= -58.15 \text{ [MPa]} \quad \text{Stress in a vertical weld} \\
\beta_w &= 0.90 \quad \text{Correlation coefficient} \\
\sqrt{\sigma_{\text{max}}^2 + 3\tau_{\text{max}}^2} &\leq f_u/(\beta_w \gamma M_2) \\
141.37 < 417.78 \quad \text{verified} \\
\sqrt{\sigma^2 + 3(\tau_x^2 + \tau_y^2)} &\leq f_u/(\beta_w \gamma M_2) \\
119.13 < 417.78 \quad \text{verified} \\
\sigma_x &\leq 0.9f_u/\gamma M_2 \\
70.69 < 338.40 \quad \text{verified}
\end{align*}
\]

**CONNECTION STIFFNESS**

\[
\begin{align*}
t_{\text{wash}} &= 4 \text{ [mm]} \quad \text{Washer thickness} \\
h_{\text{head}} &= 14 \text{ [mm]} \quad \text{Bolt head height} \\
h_{\text{nut}} &= 20 \text{ [mm]} \quad \text{Bolt nut height} \\
L_b &= 59 \text{ [mm]} \quad \text{Bolt length} \\
k_{10} &= 7 \text{ [mm]} \quad \text{Stiffness coefficient of bolts}
\end{align*}
\]

**STIFFNESSES OF BOLT ROWS**

<table>
<thead>
<tr>
<th>Nr</th>
<th>hj</th>
<th>k_3</th>
<th>k_4</th>
<th>k_5</th>
<th>k_{eff,j}</th>
<th>k_{eff,j} h_j</th>
<th>k_{eff,j} h_j^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>383</td>
<td>3</td>
<td>24</td>
<td>27</td>
<td>2</td>
<td>6.89</td>
<td>263.71</td>
</tr>
<tr>
<td>2</td>
<td>273</td>
<td>2</td>
<td>19</td>
<td>20</td>
<td>2</td>
<td>4.18</td>
<td>113.94</td>
</tr>
<tr>
<td>3</td>
<td>93</td>
<td>3</td>
<td>27</td>
<td>27</td>
<td>2</td>
<td>1.80</td>
<td>16.70</td>
</tr>
</tbody>
</table>

\[
k_{\text{eff,j}} = 1 / \left( \sum_{i=3}^{5} \left( 1 / k_{ij} \right) \right)
\]

\[
z_{\text{eq}} = \sum_{j} k_{\text{eff,j}} h_j^2 / \sum_{j} k_{\text{eff,j}} h_j
\]

\[
z_{\text{eq}} = 306 \text{ [mm]} \quad \text{Equivalent force arm}
\]

\[
k_{\text{eq}} = \sum_{j} k_{\text{eff,j}} h_j / z_{\text{eq}}
\]

\[
k_{\text{eq}} = 4 \text{ [mm]} \quad \text{Equivalent stiffness coefficient of a bolt arrangement}
\]

\[
A_{\text{vc}} = 37.28 \text{ [cm}^2\text{]} \quad \text{Shear area}
\]

\[
\beta = 1.07 \quad \text{Transformation parameter}
\]

\[
z = 306 \text{ [mm]} \quad \text{Lever arm}
\]

\[
k_1 = 4 \text{ [mm]} \quad \text{Stiffness coefficient of the column web panel subjected to shear}
\]

\[
b_{\text{eff,c,wc}} = 275 \text{ [mm]} \quad \text{Effective width of the web for compression}
\]

\[
t_{\text{wc}} = 9 \text{ [mm]} \quad \text{Effective thickness of the column web}
\]

\[
d_c = 262 \text{ [mm]} \quad \text{Height of compressed web}
\]

\[
k_2 = 6 \text{ [mm]} \quad \text{Stiffness coefficient of the compressed column web}
\]

\[
S_{j,\text{ini}} = E z_{\text{eq}}^2 / \sum \left( 1 / k_1 + 1 / k_2 + 1 / k_{\text{eq}} \right)
\]

\[
S_{j,\text{ini}} = 31337.20 \text{ [kN}^2\text{m]} \quad \text{Initial rotational stiffness}
\]

\[
\mu = 2.35 \quad \text{Stiffness coefficient of a connection}
\]

\[
S_j = S_{j,\text{ini}} / \mu
\]
$S_j = 13310.58 \text{ [kN}\cdot\text{m]}$ Final rotational stiffness

**Connection classification due to stiffness.**

$S_{j,\text{rig}} = 94480.12 \text{ [kN}\cdot\text{m]}$ Stiffness of a rigid connection

$S_{j,\text{pin}} = 5905.01 \text{ [kN}\cdot\text{m]}$ Stiffness of a pinned connection

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

**WEAKEST COMPONENT:**

COLUMN WEB - COMPRESSION

**REMARKS**

Distance of bolts from an edge is too large. $120 \text{ [mm]} > 120 \text{ [mm]}$

| Connection conforms to the code | Ratio | 0.92 |
Annex 4. Calculation of column (web) to beam joint in software ROBOT.
Calculation of the beam-column (web) connection


**GENERAL**

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

**COLUMN**

Section: HEA 300

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$</td>
<td>-90.0</td>
</tr>
<tr>
<td>$h_c$</td>
<td>290 [mm]</td>
</tr>
<tr>
<td>$b_{lc}$</td>
<td>300 [mm]</td>
</tr>
<tr>
<td>$t_{wc}$</td>
<td>9 [mm]</td>
</tr>
<tr>
<td>$t_{tc}$</td>
<td>14 [mm]</td>
</tr>
<tr>
<td>$r_c$</td>
<td>27 [mm]</td>
</tr>
<tr>
<td>$A_c$</td>
<td>112.53 [cm²]</td>
</tr>
<tr>
<td>$I_{yc}$</td>
<td>18263.50 [cm⁴]</td>
</tr>
</tbody>
</table>

Material: S 355

$\sigma_{yc}$ = 355.00 [MPa]  Design resistance
$\sigma_{uc}$ = 470.00 [MPa]  Tensile resistance

**LEFT SIDE**

Section: IPE 360

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$</td>
<td>0.0 [Deg]</td>
</tr>
<tr>
<td>$h_d$</td>
<td>360 [mm]</td>
</tr>
<tr>
<td>$b_{bd}$</td>
<td>170 [mm]</td>
</tr>
<tr>
<td>$t_{wb}$</td>
<td>8 [mm]</td>
</tr>
<tr>
<td>$t_{fb}$</td>
<td>13 [mm]</td>
</tr>
<tr>
<td>$r_{bl}$</td>
<td>18 [mm]</td>
</tr>
<tr>
<td>$A_b$</td>
<td>72.73 [cm²]</td>
</tr>
<tr>
<td>$I_{yb}$</td>
<td>16265.60 [cm⁴]</td>
</tr>
</tbody>
</table>

Material: S 355
**ANGLE**

<table>
<thead>
<tr>
<th>Section: CAE 80x8</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha =$</td>
<td>0.0 [Deg] Inclination angle</td>
</tr>
<tr>
<td>$h_{kl} =$</td>
<td>80 [mm] Height of angle section</td>
</tr>
<tr>
<td>$b_{kl} =$</td>
<td>80 [mm] Width of angle section</td>
</tr>
<tr>
<td>$t_{kl} =$</td>
<td>8 [mm] Thickness of the flange of angle section</td>
</tr>
<tr>
<td>$r_{kl} =$</td>
<td>10 [mm] Fillet radius of the web of angle section</td>
</tr>
<tr>
<td>$l_{kl} =$</td>
<td>150 [mm] Angle length</td>
</tr>
<tr>
<td>Material:</td>
<td>S 355</td>
</tr>
<tr>
<td>$f_{ykl} =$</td>
<td>355.00 [MPa] Design resistance</td>
</tr>
<tr>
<td>$f_{ukl} =$</td>
<td>470.00 [MPa] Tensile resistance</td>
</tr>
</tbody>
</table>

**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**

<table>
<thead>
<tr>
<th>Section: IPE 360</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha =$</td>
<td>0.0 [Deg] Inclination angle</td>
</tr>
<tr>
<td>$h_{br} =$</td>
<td>360 [mm] Height of beam section</td>
</tr>
<tr>
<td>$b_{br} =$</td>
<td>170 [mm] Width of beam section</td>
</tr>
<tr>
<td>$t_{wbr} =$</td>
<td>8 [mm] Thickness of the web of beam section</td>
</tr>
<tr>
<td>$t_{fbr} =$</td>
<td>13 [mm] Thickness of the flange of beam section</td>
</tr>
<tr>
<td>$r_{br} =$</td>
<td>18 [mm] Radius of beam section fillet</td>
</tr>
<tr>
<td>$A_{br} =$</td>
<td>72.73 [cm$^2$] Cross-sectional area of a beam</td>
</tr>
<tr>
<td>$I_{ybr} =$</td>
<td>16265.60 [cm$^4$] Moment of inertia of the beam section</td>
</tr>
<tr>
<td>Material:</td>
<td>S 355</td>
</tr>
<tr>
<td>$f_{ybr} =$</td>
<td>355.00 [MPa] Design resistance</td>
</tr>
<tr>
<td>$f_{ubr} =$</td>
<td>470.00 [MPa] Tensile resistance</td>
</tr>
</tbody>
</table>

**ANGLE**

<table>
<thead>
<tr>
<th>Section: CAE 80x8</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_{kr} =$</td>
<td>80 [mm] Height of angle section</td>
</tr>
<tr>
<td>$b_{kr} =$</td>
<td>80 [mm] Width of angle section</td>
</tr>
<tr>
<td>$t_{kr} =$</td>
<td>8 [mm] Thickness of the flange of angle section</td>
</tr>
<tr>
<td>$r_{kr} =$</td>
<td>10 [mm] Fillet radius of the web of angle section</td>
</tr>
<tr>
<td>$l_{kr} =$</td>
<td>150 [mm] Angle length</td>
</tr>
<tr>
<td>Material:</td>
<td>S 355</td>
</tr>
<tr>
<td>$f_{ykr} =$</td>
<td>355.00 [MPa] Design resistance</td>
</tr>
<tr>
<td>$f_{ukr} =$</td>
<td>470.00 [MPa] Tensile resistance</td>
</tr>
</tbody>
</table>
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 [cm²] [Effective section area of a bolt]
- **A_v =** 2.01 [cm²] [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 [cm²] [Effective section area of a bolt]
- **A_v =** 2.01 [cm²] [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**

\[ \gamma_{M0} = 1.00 \] [Partial safety factor]

\[ \gamma_{M2} = 1.25 \] [Partial safety factor]

**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_{T,Rd} =** 56.52 [kN] [Tensile resistance of a single bolt]

**Bolt bearing on the angle**
Direction x

\[ k_{kx} = 2.50 \]

Coefficient for calculation of \( F_{b,Rd} \)

\[ k_{kx} > 0.0 \]

\[ \alpha_{bx} = 0.56 \]

Coefficient for calculation of \( F_{b,Rd} \)

\[ \alpha_{bx} > 0.0 \]

\[ F_{b,Rd2x} = 66.84 \text{ [kN]} \]

Bearing resistance of a single bolt

\[ F_{b,Rd2x} = k_{kx} \cdot \alpha_{bx} \cdot f_u \cdot d \cdot t_i / M_2 \]

verified

Direction z

\[ k_{kz} = 2.50 \]

Coefficient for calculation of \( F_{b,Rd} \)

\[ k_{kz} > 0.0 \]

\[ \alpha_{bz} = 0.83 \]

Coefficient for calculation of \( F_{b,Rd} \)

\[ \alpha_{bz} > 0.0 \]

\[ F_{b,Rd2z} = 100.27 \text{ [kN]} \]

Bearing resistance of a single bolt

\[ F_{b,Rd2z} = k_{kz} \cdot \alpha_{bz} \cdot f_u \cdot d \cdot t_i / M_2 \]

verified

FORCES ACTING ON BOLTS IN THE COLUMN - ANGLE CONNECTION

Bolt shear

\[ e = 54 \text{ [mm]} \]

Distance between centroid of a bolt group of an angle and center of the beam web

\[ M_0 = 1.20 \text{ [kN*m]} \]

Real bending moment

\[ F_{Vz} = 11.08 \text{ [kN]} \]

Component force in a bolt due to influence of the shear force

\[ F_{Mz} = 9.45 \text{ [kN]} \]

Component force in a bolt due to influence of the moment

\[ F_{x,Ed} = 19.94 \text{ [kN]} \]

Design total force in a bolt on the direction x

\[ F_{z,Ed} = 11.08 \text{ [kN]} \]

Design total force in a bolt on the direction z

\[ F_{Rdx} = 48.25 \text{ [kN]} \]

Effective design capacity of a bolt on the direction x

\[ F_{Rdz} = 48.25 \text{ [kN]} \]

Effective design capacity of a bolt on the direction z

\[ F_{x,Ed} \leq F_{Rdx} \]

verified

\[ F_{z,Ed} \leq F_{Rdz} \]

verified

\[ |F_{x,Ed}| \leq FRdx \]

\[ |19.94| < 48.25 \]

verified

\[ (0.41) \]

\[ |F_{z,Ed}| \leq FRdz \]

\[ |11.08| < 48.25 \]

verified

\[ (0.23) \]

Bolt tension

\[ e = 54 \text{ [mm]} \]

Distance between centroid of a bolt group and center of column web

\[ M_0 = 1.20 \text{ [kN*m]} \]

Real bending moment

\[ F_{t,Ed} = 25.90 \text{ [kN]} \]

Tensile force in the outermost bolt

\[ F_{t,Ed} = M_0 t_{max}^2 / 2 \sum z_i^2 + 0.5 N_{bx,Ed} / n \]

\[ F_{t,Ed} \leq F_{t,Rd} \]

\[ 25.90 < 56.52 \]

verified

\[ (0.46) \]

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

\[ F_{V,Ed} = 22.82 \text{ [kN]} \]

Resultant shear force in a bolt

\[ F_{V,Ed} = \sqrt{F_{x,Ed}^2 + F_{z,Ed}^2} \]

\[ F_{V,Ed} / F_{V,Rd} + F_{t,Ed} / (1.4 * F_{t,Rd}) \leq 1.0 \]

\[ 0.80 < 1.00 \]

verified

\[ (0.80) \]

BOLTS CONNECTING ANGLE WITH BEAM

BOLT CAPACITIES

\[ F_{v,Rd} = 96.51 \text{ [kN]} \]

Shear resistance of the shank of a single bolt

\[ F_{v,Rd} = 0.6 f_u A_v \cdot m_v / M_2 \]

Bolt bearing on the beam

\[ k_{kx} = 2.50 \]

Coefficient for calculation of \( F_{b,Rd} \)

\[ k_{kx} > 0.0 \]

\[ \alpha_{bx} = 0.56 \]

Coefficient for calculation of \( F_{b,Rd} \)

\[ \alpha_{bx} > 0.0 \]

\[ F_{b,Rd1x} = 89.13 \text{ [kN]} \]

Bearing resistance of a single bolt

\[ F_{b,Rd1x} = k_{kx} \cdot \alpha_{bx} \cdot f_u \cdot d \cdot t_i / M_2 \]

verified

\[ k_{kz} = 2.50 \]

Coefficient for calculation of \( F_{b,Rd} \)

\[ k_{kz} > 0.0 \]

\[ \alpha_{bz} = 0.83 \]

Coefficient for calculation of \( F_{b,Rd} \)

\[ \alpha_{bz} > 0.0 \]

\[ F_{b,Rd1z} = 103.61 \text{ [kN]} \]

Bearing resistance of a single bolt

\[ F_{b,Rd1z} = k_{kz} \cdot \alpha_{bz} \cdot f_u \cdot d \cdot t_i / M_2 \]
Bolt bearing on the angle

**Direction x**

- \( k_{tx} = 2.50 \)  
- Coefficient for calculation of \( F_{b,Rd} \)  
- \( k_{tx} > 0.0 \)  
- \( 2.50 > 0.00 \)  
- verified

- \( \alpha_{bx} = 0.56 \)  
- Coefficient for calculation of \( F_{b,Rd} \)  
- \( \alpha_{bx} > 0.0 \)  
- \( 0.56 > 0.00 \)  
- verified

- \( F_{b,Rd2x} = 133.69 \) [kN]  
- Bearing resistance of a single bolt  
- \( F_{b,Rd2x} = k_{tx} \cdot \alpha_{bx} \cdot f_u \cdot d \cdot t_i / \gamma_M2 \)

**Direction z**

- \( k_{tz} = 2.50 \)  
- Coefficient for calculation of \( F_{b,Rd} \)  
- \( k_{tz} > 0.0 \)  
- \( 2.50 > 0.00 \)  
- verified

- \( \alpha_{bz} = 0.83 \)  
- Coefficient for calculation of \( F_{b,Rd} \)  
- \( \alpha_{bz} > 0.0 \)  
- \( 0.83 > 0.00 \)  
- verified

- \( F_{b,Rd2z} = 200.53 \) [kN]  
- Bearing resistance of a single bolt  
- \( F_{b,Rd2z} = k_{tz} \cdot \alpha_{bz} \cdot f_u \cdot d \cdot t_i / \gamma_M2 \)

**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  
- \( M_0 = M_{b2,Ed} + V_{b2,Ed} \cdot e \)

- \( F_{Nx} = 11.72 \) [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  
- \( F_{Mx} = M_0 \cdot z_x / (x_x^2 + z_x^2) \)

- \( F_{Mz} = 0.00 \) [kN]  
- Component force in a bolt due to influence of the moment on the z direction  
- \( F_{Mz} = M_0 \cdot z_z / (x_z^2 + z_z^2) \)

- \( F_{x,Ed} = 51.79 \) [kN]  
- Design total force in a bolt on the direction x  
- \( F_{x,Ed} = F_{Nx} + F_{Mx} \)

- \( F_{x,Ed} = 22.16 \) [kN]  
- Design total force in a bolt on the direction z  
- \( F_{x,Ed} = F_{Vz} + F_{Mz} \)

- \( F_{Rdx} = 89.13 \) [kN]  
- Effective design capacity of a bolt on the direction x  
- \( F_{Rdx} = \min(F_{v,Rd}, F_{b,Rd1x}, F_{b,Rd2x}) \)

- \( F_{Rdz} = 96.51 \) [kN]  
- Effective design capacity of a bolt on the direction z  
- \( F_{Rdz} = \min(F_{v,Rd}, F_{b,Rd1z}, F_{b,Rd2z}) \)

- \(|F_{x,Ed}| \leq F_{Rdx} \)  
- \(|51.79| < 89.13 \)  
- verified  
- \((0.58)\)

- \(|F_{z,Ed}| \leq F_{Rdz} \)  
- \(|22.16| < 96.51 \)  
- 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

- \( A_{sv} = 6.24 \) [cm\(^2\)]  
- Area of the section in shear

- \( V_{effRd} = 159.48 \) [kN]  
- Design capacity of a section weakened by openings  
- \( V_{effRd} = 0.5 \cdot f_y \cdot A_{nt} / \gamma_M2 + (1 / \sqrt{3}) \cdot f_y \cdot A_{sv} / \gamma_M2 \)

- \(|0.5 \cdot V_{b2,Ed}| \leq V_{effRd} \)  
- \(|22.16| < 159.48 \)  
- verified  
- \((0.14)\)

**BEAM**

- \( A_{nt} = 2.48 \) [cm\(^2\)]  
- Net area of the section in tension

- \( A_{sv} = 14.64 \) [cm\(^2\)]  
- Area of the section in shear

- \( V_{effRd} = 346.68 \) [kN]  
- Design capacity of a section weakened by openings  
- \( V_{effRd} = 0.5 \cdot f_y \cdot A_{nt} / \gamma_M2 + (1 / \sqrt{3}) \cdot f_y \cdot A_{sv} / \gamma_M2 \)

- \(|V_{b2,Ed}| \leq V_{effRd} \)  
- \(|44.32| < 346.68 \)  
- 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  
- \( F_{v,Rd} = 0.6 \cdot f_s \cdot A_{sv} \cdot m / \gamma_M2 \)

- \( F_{t,Rd} = 56.52 \) [kN]  
- Tensile resistance of a single bolt  
- \( F_{t,Rd} = 0.9 \cdot f_u \cdot A_{nt} / \gamma_M2 \)

**Bolt bearing on the angle**

**Direction x**

- \( k_{tx} = 2.50 \)  
- Coefficient for calculation of \( F_{b,Rd} \)  
- \( k_{tx} > 0.0 \)  
- \( 2.50 > 0.00 \)  
- verified

- \( \alpha_{bx} = 0.56 \)  
- Coefficient for calculation of \( F_{b,Rd} \)  
- \( \alpha_{bx} > 0.0 \)  
- \( 0.56 > 0.00 \)  
- verified

- \( F_{b,Rd2x} = 133.69 \) [kN]  
- Bearing resistance of a single bolt  
- \( F_{b,Rd2x} = k_{tx} \cdot \alpha_{bx} \cdot f_u \cdot d \cdot t_i / \gamma_M2 \)
\[ \alpha_{bx} = 0.56 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ \alpha_{bx} > 0.0 \quad 0.56 > 0.00 \quad \text{verified} \]
\[ F_{b,Rd2x} = 66.84 \quad [kN] \quad \text{Bearing resistance of a single bolt} \]

\[ k_{1z} = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ k_{1z} > 0.0 \quad 2.50 > 0.00 \quad \text{verified} \]
\[ \alpha_{bz} = 0.83 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ \alpha_{bz} > 0.0 \quad 0.83 > 0.00 \quad \text{verified} \]
\[ F_{b,Rd2z} = 100.27 \quad [kN] \quad \text{Bearing resistance of a single bolt} \]

FORCES ACTING ON BOLTS IN THE COLUMN - ANGLE CONNECTION

Bolt shear
\[ e = 54 \quad [mm] \quad \text{Distance between centroid of a bolt group of an angle and center of the beam web} \]
\[ M_0 = 1.20 \quad [kN*m] \quad \text{Real bending moment} \]
\[ F_{Vz} = 11.08 \quad [kN] \quad \text{Component force in a bolt due to influence of the shear force} \]
\[ F_{Mx} = 19.94 \quad [kN] \quad \text{Component force in a bolt due to influence of the moment} \]
\[ F_{x1,Ed} = 19.94 \quad [kN] \quad \text{Design total force in a bolt on the direction } x \]
\[ F_{z1,Ed} = 11.08 \quad [kN] \quad \text{Design total force in a bolt on the direction } z \]
\[ F_{Rdx} = 48.25 \quad [kN] \quad \text{Effective design capacity of a bolt on the direction } x \]
\[ F_{Rdz} = 48.25 \quad [kN] \quad \text{Effective design capacity of a bolt on the direction } z \]

\[ |F_{x1,Ed}| \leq F_{Rdx} \quad 19.94 < 48.25 \quad \text{verified} \quad (0.41) \]
\[ |F_{z1,Ed}| \leq F_{Rdz} \quad 11.08 < 48.25 \quad \text{verified} \quad (0.23) \]

Bolt tension
\[ e = 54 \quad [mm] \quad \text{Distance between centroid of a bolt group and center of column web} \]
\[ M_{0t} = 1.20 \quad [kN*m] \quad \text{Real bending moment} \]
\[ F_{TelEd} = 29.38 \quad [kN] \quad \text{Tensile force in the outermost bolt} \]
\[ F_{TelEd} \leq F_{Rd} \quad 29.38 < 56.52 \quad \text{verified} \quad (0.52) \]

Simultaneous action of a tensile force and a shear force in a bolt
\[ F_{V,Ed} = 22.82 \quad [kN] \quad \text{Resultant shear force in a bolt} \]
\[ F_{x,Ed}^2 + F_{z,Ed}^2 \leq 1.0 \quad 0.84 < 1.00 \quad \text{verified} \quad (0.84) \]

BOLTS CONNECTING ANGLE WITH BEAM

BOLT CAPACITIES
\[ F_{V,Rd} = 96.51 \quad [kN] \quad \text{Shear resistance of the shank of a single bolt} \]
\[ F_{V,Rd} = 0.6*f_{ub} * A_v * m_{v}/M_2 \]

Bolt bearing on the beam

Direction x
\[ k_{1x} = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ k_{1x} > 0.0 \quad 2.50 > 0.00 \quad \text{verified} \]
\[ \alpha_{bx} = 0.74 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ \alpha_{bx} > 0.0 \quad 0.74 > 0.00 \quad \text{verified} \]
\[ F_{b,Rd1x} = 89.13 \quad [kN] \quad \text{Bearing resistance of a single bolt} \]

Direction z
\[ k_{1z} = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ k_{1z} > 0.0 \quad 2.50 > 0.00 \quad \text{verified} \]
\[ \alpha_{bz} = 0.86 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ \alpha_{bz} > 0.0 \quad 0.86 > 0.00 \quad \text{verified} \]
\[ F_{b,Rd1z} = 103.61 \quad [kN] \quad \text{Bearing resistance of a single bolt} \]

Bolt bearing on the angle

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

Direction z
\[ k_{1z} = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ k_{1z} = \min[2.8*(e_2/d_0)-1.7, 2.5] \]
\[ k_{1x} > 0.0 \quad 2.50 > 0.00 \quad \text{verified} \]
\[ \alpha_{bx} = 0.56 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ \alpha_{bx} > 0.0 \quad 0.56 > 0.00 \quad \text{verified} \]
\[ F_{b,Rd2x} = 133.69 \quad [\text{kN}] \quad \text{Bearing resistance of a single bolt} \]

**Direction z**

\[ k_{1z} = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ k_{1z} > 0.0 \quad 2.50 > 0.00 \quad \text{verified} \]
\[ \alpha_{bz} = 0.83 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ \alpha_{bz} > 0.0 \quad 0.83 > 0.00 \quad \text{verified} \]
\[ F_{b,Rd2z} = 200.53 \quad [\text{kN}] \quad \text{Bearing resistance of a single bolt} \]

**FORCES ACTING ON BOLTS IN THE ANGLE - BEAM CONNECTION**

**Bolt shear**

\[ \varepsilon = 54 \quad [\text{mm}] \quad \text{Distance between centroid of a bolt group and center of column web} \]
\[ M_0 = -2.40 \quad [\text{kN}\cdot\text{m}] \quad \text{Real bending moment} \]
\[ F_{Nz} = 18.69 \quad [\text{kN}] \quad \text{Component force in a bolt due to influence of the longitudinal force} \]
\[ F_{Vz} = 22.16 \quad [\text{kN}] \quad \text{Component force in a bolt due to influence of the shear force} \]
\[ F_{Mx} = -40.07 \quad [\text{kN}] \quad \text{Component force in a bolt due to influence of the moment on the } x \text{ direction} \]
\[ F_{Mz} = 0.00 \quad [\text{kN}] \quad \text{Component force in a bolt due to influence of the moment on the } z \text{ direction} \]
\[ F_{E,z,Ed} = -21.38 \quad [\text{kN}] \quad \text{Design total force in a bolt on the direction } z \]
\[ F_{E,x,Ed} = 22.16 \quad [\text{kN}] \quad \text{Design total force in a bolt on the direction } x \]
\[ F_{Rd,x} = 89.13 \quad [\text{kN}] \quad \text{Effective design capacity of a bolt on the direction } x \]
\[ F_{Rd,z} = 96.51 \quad [\text{kN}] \quad \text{Effective design capacity of a bolt on the direction } z \]

\[ |F_{E,Ed,x}| \leq F_{Rd,x} \quad |-21.38| < 89.13 \quad \text{verified} \]
\[ |F_{E,Ed,z}| \leq F_{Rd,z} \quad |22.16| < 96.51 \quad \text{verified} \]

**VERIFICATION OF THE SECTION DUE TO BLOCK TEARING**

**ANGLE**

\[ A_n = 3.28 \quad [\text{cm}^2] \quad \text{Net area of the section in tension} \]
\[ A_{nv} = 6.24 \quad [\text{cm}^2] \quad \text{Area of the section in shear} \]
\[ V_{eff,Rd} = 189.56 \quad [\text{kN}] \quad \text{Design capacity of a section weakened by openings} \]
\[ |0.5V_{Vb,Ed}| \leq V_{eff,Rd} \quad |-22.16| < 189.56 \quad \text{verified} \]

**BEAM**

\[ A_n = 2.48 \quad [\text{cm}^2] \quad \text{Net area of the section in tension} \]
\[ A_{nv} = 14.64 \quad [\text{cm}^2] \quad \text{Area of the section in shear} \]
\[ V_{eff,Rd} = 346.68 \quad [\text{kN}] \quad \text{Design capacity of a section weakened by openings} \]
\[ |V_{Vb,Ed}| \leq V_{eff,Rd} \quad |44.32| < 346.68 \quad \text{verified} \]

**COLUMN VERIFICATION**

**BOLT BEARING ON THE COLUMN WEB**

**Direction x**

\[ k_x = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ k_x > 0.0 \quad 2.50 > 0.00 \quad \text{verified} \]
\[ \alpha_{bx} = 1.00 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ \alpha_{bx} > 0.0 \quad 1.00 > 0.00 \quad \text{verified} \]
\[ F_{b,Rd2x} = 127.84 \quad [\text{kN}] \quad \text{Bearing resistance of a single bolt} \]

**Direction z**

\[ k_z = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ k_z > 0.0 \quad 2.50 > 0.00 \quad \text{verified} \]
\[ \alpha_{bz} = 0.86 \quad \text{Coefficient for calculation of } F_{b,Rd} \]
\[ \alpha_{bz} > 0.0 \quad 0.86 > 0.00 \quad \text{verified} \]
\[ F_{b,Rd2z} = 110.08 \quad [\text{kN}] \quad \text{Bearing resistance of a single bolt} \]
RESULTANT FORCE ACTING ON THE OUTERMOST BOLT

\[ F_{x,Ed} = 39.89 \text{ [kN]} \quad \text{Design total force in a bolt on the direction x} \]
\[ F_{z,Ed} = 22.16 \text{ [kN]} \quad \text{Design total force in a bolt on the direction z} \]

\[ |F_{x,Ed}| \leq F_{b,Rdx} \quad |39.89| < 127.84 \quad \text{verified} \]
\[ |F_{z,Ed}| \leq F_{b,Rdz} \quad |22.16| < 110.08 \quad \text{verified} \]

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).
### Table

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<thead>
<tr>
<th>No.</th>
<th>Description</th>
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**Performed by:**
- Maksym Podgysky
- MD Reza Ahmed

**Description:**
- Beam-column joint