CZECH TECHNICAL UNIVERSITY IN PRAGUE

FACULTY OF CIVIL ENGINEERING

MASTER THESIS

ICE HOCKEY ARENA WITH ARCHED ROOF

2019

B. HALBERSTADT
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FACULTY OF CIVIL ENGINEERING

MASTER THESIS

ICE HOCKEY ARENA WITH ARCHED ROOF

PART A: TECHNICAL REPORT

2019

B. HALBERSTADT
# DIPLOMA THESIS ASSIGNMENT FORM

## I. PERSONAL AND STUDY DATA

<table>
<thead>
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<tbody>
<tr>
<td>Surname</td>
<td>Halberstadt</td>
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<tr>
<td>Name</td>
<td>Bastiaan</td>
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<tr>
<td>Personal number</td>
<td>477991</td>
</tr>
<tr>
<td>Assigning Department</td>
<td>Department of steel and timber structures</td>
</tr>
<tr>
<td>Study programme</td>
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<td>Branch of study</td>
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## II. DIPLOMA THESIS DATA

<table>
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<tbody>
<tr>
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<tr>
<td>Diploma Thesis title in English</td>
<td>Ice hockey arena with arched roof</td>
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<tr>
<td>The design project is expected to include:</td>
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<tr>
<td>- design of main (typical) load bearing steel members</td>
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<tr>
<td>- design of selected structural details</td>
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<tr>
<td>- drawing documentation (layout, detailing)</td>
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<td>- technical report</td>
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List of recommended literature:
- Design codes - EN 1991, EN 1993

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<tbody>
<tr>
<td>Name of Diploma Thesis Supervisor</td>
<td>Michal Jandera</td>
</tr>
<tr>
<td>DT assignment date</td>
<td>4.10.2018</td>
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<td>DT Supervisor’s signature</td>
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<tr>
<td>Head of Department’s signature</td>
<td></td>
</tr>
</tbody>
</table>

## III. ASSIGNMENT RECEIPT

I declare that I am obliged to write the Diploma Thesis on my own, without anyone’s assistance, except for provided consultations. The list of references, other sources and consultants’ names must be stated in the Diploma Thesis and in referencing I must abide by the CTU methodological manual “How to Write University Final Theses” and the CTU methodological instruction “On the Observation of Ethical Principles in the Preparation of University Final Theses”.

<table>
<thead>
<tr>
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<td>Assignment receipt date</td>
<td></td>
</tr>
<tr>
<td>Student’s name</td>
<td></td>
</tr>
</tbody>
</table>
Statutory Declaration

I declare that I have developed and written the Master Thesis completely by myself and have not used sources or means without declaration in the text. Any thoughts from others or quotations are clearly marked. All the materials I have used for the Master thesis are listed in the references.

…………………………

Signature
Abstract

The thesis deals with the design of the steel load bearing structure for an ice hockey arena with an arched roof. The arena is designed for a location in Alkmaar, The Netherlands. The arena is symmetrical and offers space to two ice hockey fields, each covered under an arched roof part with a span of 35.6 meters. The complete plan of the arena has a size of 66x86 meters.

The design is done based on Eurocodes with Dutch National Annex. Drawing documentation is included to the report.

Keywords: Arch beam, middle beam, gable wall, main frame, bracing system, tension bar.
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Description of object
The arena consist of three parts and has a rectangular plan. The two ice hockey fields are on both sides of a concrete core. The sides of the building have an arched roof and the roof of the core is flat. The walls on the sides are 8 meters high, while the flat roof in the center are on a height of 11 meters. By means of a one meter cantilever the roof sticks out of the wall. The plan is 66x86 meters. The arches are 38.1m long and have a projection length of 35,6m and the core is 14,8m long. The highest point of the center line of the arch stands at 15,3m.

The arena is located in the city of Alkmaar, in the Northwest of The Netherlands. It is located at the edge of the city with a large open (agricultural) space next to it.

Loading
The loads on the roof and walls are considered. The loads on the inner structure are out of the scope of this project.

Permanent actions
Self-weight of ceiling (insulation) 0,2 kN/m²
Ventilation, lights 0,2 kN/m²
Total dead load 0,4 kN/m²

Imposed actions
Imposed roof load from maintenance work 1,0 kN/m² on an area of 10 m²

Wind actions
The Dutch National Annex of EN 1991-1-4 is used to determine the wind load. A reference height of 16 meters is used in wind zone I.

\[ q_p(16) = 1,2 \text{kN/m}^2 \]

Snow actions
The Dutch National Annex of EN 1991-1-3 is used to determine the snow load.

\[ S_k = \text{characteristic value of the snow load} = 0,7 \text{kN/m}^2 \]
\[ \mu_1 = \text{Snow load shape coefficient for for} \ 0^\circ < \alpha < 30 \ = 0,8 [-] \]
\[ C_e = \text{Exposure coefficient} = 1,0 [-] \]
\[ C_p = \text{Thermal coefficient} = 1,0 [-] \]
Material
Based on EN 1993-1-10, steel grade S355 J0 is used for all the elements of the steel structure with a thickness of 30 mm and higher e.g. the plates of the fixed column base. Other, less thick elements will have steel grade S355 JR.
The studs ensuring the shear connection are from steel grade S235. The trapezoidal metal sheet is made of steel grade S320 GD. The bolts used in connection are from steel grade 8.8 or 10.9.

Description of structure
The load bearing structure consists of a steel portal with two columns, 2 arched rafters and a middle beam. Under the arched beams are tension bars with 36mm diameter. The frame is supported by a concrete core underneath the middle beam. The concrete core is out of the scope of this project and is therefore simply modelled as supports in the structure.

Both the columns and the main arch are made from HEA280 profiles, while the middle beam is an IPE450 profile. The span between each frame is 6m. At both ends there is a gable wall consisting of HEA280 profile corner columns with a height of 8m and 10 HEA450 middle columns, varying in height from 11,93m till 15,12m. The arch and middle beam are IPE300 profiles. The cantilevers of one meter are IPE120 profiles.

The stability is generated by 3 bracing systems. The struts connecting the frames horizontally are CHS88.9/3.2 profiles and the diagonals are CHS168/5.0 profiles.

The execution class is EXC3.

Assembly
The length of all the elements aren’t larger than 12 meter and therefore can be transported without the use of exceptional transport. Most of the welding is done in the workshop with exception of some welds around the fixed column base anchors. The assembly of the structure itself happens on the building site by means of a crane for lifting the elements.

Corrosion protection
The class of corrosive environment is C2. The anti-corrosion coating will be designed for this class according to other requirements (it is not a part of this project). The bolts are galvanized in the production workshop.

Fire safety
Fire safety has to be guaranteed and needs to be calculated. This is however out of the scope of this project.
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MASTER THESIS

ICE HOCKEY ARENA WITH ARCHED ROOF

PART B: STATIC CALCULATIONS

2019

B. HALBERSTADT
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**Introduction**

Before defining the acting loads on the structure the location and general geometry is determined. Based on an initial model different alternatives are considered. Besides the main frame the roof and wall sheeting is calculated. The next step is the bracing system in combination with the gable wall. Lastly the details are determined, for example the column bases and the connections between the beams and the columns.

Used software

- SCIA Engineer 18.0 with student license
- AutoCad 2018 with student license
- LTBeamN 1.0.3
- Microsoft Word 2016
- Microsoft Excel 2016
Location
Alkmaar, Noord-Holland, Netherlands.

Figure 1: Arena location, source: Google Maps

The location is a semi-built area, an unbuilt area is chosen as reference for wind actions.
Geometry

The arena will have a rectangular plan of 66x86 meters. On both sides of a 14.8 meter wide core will be an arched roof with a span of 35.6 meters. The walls on the side will be 8 meters high while the core will be 11 meters. The roof will hang over the walls on both sides. Because of its arch, the roof has its highest point roughly 4.3 meters above the core.
Actions on the roof
CC3

**Permanent actions**

Partial factor $\psi_0$ (according to Dutch national annex)

- Self-weight of ceiling (insulation) $0,2 \text{kN/m}^2$
- Ventilation, lights $0,2 \text{kN/m}^2$
- Total dead load $0,4 \text{kN/m}^2$ NA

**Imposed actions**

- Imposed roof load from maintenance work $1,0 \text{kN/m}^2$ 0

On area of 10 m² **(for trapezoidal sheeting)**

**Wind actions**

$$qp(16) = 1,16 + 1*(1.27-1,16)/5 = 1,2 \text{kN/m}^2$$ 0

**Snow actions**

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

**Temperature actions**

Min 5 degrees
Max 30 degrees

---

**Tabel NB.5 — Partiële factoren voor gevolgklassen 1 en 3 voor belastingen (STR/GEO) (groep B)**

<table>
<thead>
<tr>
<th>CC</th>
<th>Blijvende en tijdelijke ontwerpsituaties</th>
<th>Blijvende belastingen</th>
<th>Overheersende veranderlijke belasting</th>
<th>Veranderlijke belastingen gelijktijdig met de overheersende</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Ongunstig</td>
<td>Gunstig</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>(Vgl. 6.10a)</td>
<td>1,2 $G_k,\text{long}$</td>
<td>0,9 $G_k,\text{int}$</td>
<td>1,35 $\psi_0, Q_k,1$</td>
</tr>
<tr>
<td></td>
<td>(Vgl. 6.10b)</td>
<td>1,1 $G_k,\text{long}$</td>
<td>0,9 $G_k,\text{int}$</td>
<td>1,35 $Q_k,1$</td>
</tr>
<tr>
<td>3</td>
<td>(Vgl. 6.10a)</td>
<td>1,5 $G_k,\text{long}$</td>
<td>0,9 $G_k,\text{int}$</td>
<td>1,65 $\psi_0, Q_k,1$</td>
</tr>
<tr>
<td></td>
<td>(Vgl. 6.10b)</td>
<td>1,3 $G_k,\text{long}$</td>
<td>0,9 $G_k,\text{int}$</td>
<td>1,65 $Q_k,1$</td>
</tr>
</tbody>
</table>
Wind load
Reference height $z_e = 16$ m

Peak velocity pressure $q_p(z)$

Alkmaar is located in area I as defined in the Dutch National Annex.

$$q_p(16) = 1.16 + 1 \times (1.27-1.16)/5 = 1.18 \text{ (roundup)} = 1.2 \text{ kN/m}^2$$
External wind action on walls

Since \( h = 16 \text{m} \) and the arena has a plan of 66x86 meters the scheme above applies, where the wind pressure is equal over the complete height of the arena.

Since \( h = 16 \) and \( d \) is either 62 or 88, \( h/d \leq 1 \)

\[ e = \min(b, 2h) = 32 \text{m} \]
Schemes of external wind pressure coefficients

Wind direction 0°

Wind direction 90°

<table>
<thead>
<tr>
<th>Zone</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>External pressure coefficient</td>
<td>-1,2</td>
<td>-0,8</td>
<td>-0,5</td>
<td>+0,8</td>
<td>-0,5</td>
</tr>
</tbody>
</table>
External wind action on roof
In the case of wind coming from the short side of the arena, the arched roof parts will be schematized as vaulted roofs and the center part of the roof will be schematized as a flat roof with parapets.

In case of the wind coming from the long side of the arena, the arched roof parts will be schematized by a duopitch roof and the center part of the roof as flat roof without parapet.

Flat roofs
The center part of the roof will be schematized as a flat roof with parapets, see scheme below.

With $h = 11\text{m}$, $h_p = 4.3\text{m}$ (difference between the flat part and the highest part of the arches).

However when the wind direction changes these ‘parapets’ are not on the edges of the flat roof and higher wind pressure coefficients are applied.

Vaulted roofs
Duopitch roofs

External pressure coefficients for wind direction 90°

**Tabel NB.11 – 7.4b — Uitwendige drukcoëfficiënten voor zadeldaken**

<table>
<thead>
<tr>
<th>Hellingshoek $\alpha$</th>
<th>Zone voor windrichting $\theta = 90^\circ$</th>
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<tr>
<td></td>
<td>F</td>
</tr>
<tr>
<td></td>
<td>$c_{p_{f},10}$</td>
</tr>
<tr>
<td>$-45^\circ$</td>
<td>-1.4</td>
</tr>
<tr>
<td>$-30^\circ$</td>
<td>-1.5</td>
</tr>
<tr>
<td>$-15^\circ$</td>
<td>-1.9</td>
</tr>
<tr>
<td>$-5^\circ$</td>
<td>-1.8</td>
</tr>
<tr>
<td>$5^\circ$</td>
<td>-1.6</td>
</tr>
<tr>
<td>$+15^\circ$</td>
<td>-1.3</td>
</tr>
<tr>
<td>$+30^\circ$</td>
<td>-1.1</td>
</tr>
<tr>
<td>$+45^\circ$</td>
<td>-1.1</td>
</tr>
<tr>
<td>$+60^\circ$</td>
<td>-1.1</td>
</tr>
<tr>
<td>$+75^\circ$</td>
<td>-1.1</td>
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Schemes of external pressure coefficients

Wind direction 0°

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<th>Zone</th>
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<th>B</th>
<th>C</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-0.37</td>
<td>-0.9</td>
<td>-0.4</td>
<td>-1.2</td>
<td>-0.8</td>
<td>-0.7</td>
<td>+0.2</td>
</tr>
</tbody>
</table>

Wind direction 90°

<table>
<thead>
<tr>
<th>Zone</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-1.8</td>
<td>-1.2</td>
<td>-0.7</td>
<td>+0.2</td>
<td>-1.2</td>
<td>-1.3</td>
<td>-1.35</td>
<td>-0.7</td>
</tr>
</tbody>
</table>
Internal wind action

Since the amount of openings in the facades is unknown Note 2 of formula 7.3 is taken as norm.

\[
\mu = \frac{\sum \text{area of openings where } c_{pe} \text{ is negative or } -0.0}{\sum \text{area of all openings}}
\]  

(7.3)

**NOTE 1** This applies to façades and roof of buildings with and without internal partitions.

**NOTE 2** Where it is not possible, or not considered justified, to estimate \( \mu \) for a particular case then \( c_{pe} \) should be taken as the more onerous of +0.2 and -0.3.

Combining this with the rules as written in chapter 5:

1. The net pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative. Examples are given in Figure 5.1.

\[ c_{fr} = 0.04 \]

\[ 4*h = 4*15.3 = 61.2 \text{ m} \]

Hence the friction on the short side of the building can be neglected. On the long side there is

\[ 88 - 61.2 = 26.8 \text{ m} \]

on which friction has to be taken into account.

The total amount of friction force caused by wind is equal to:

\[ F_{fr} = c_{fr} * l * b * c_{p}(16) = 0.04 * 26.8 * 66 * 1.2 = 84.8 \text{ kN} \]
Snow load
General
\( \alpha_1 = 22^\circ \)
\( \alpha_2 = 15^\circ \)
\( \alpha_{\text{average}} = 18,5^\circ \)
\( \mu_1 = 0,8 \)
\( \mu_{2,18.5} = 1,2 \)
\( s_k = 0,7 \text{kN/m}^2 \)

\( C_e \) and \( C_t \) are equal to 1,0 (Dutch National Annex chapter 5).

The snow load on the arena will be modelled in two ways. The most onerous one will be the decisive one for verification.

Model 1

The arena roof will be estimated by a duopitch roof with values of alpha as described above. Since all angles are below 30 degrees case (i) is an equally divided load with \( \mu \) equal to 0,8. In case (ii), on the flat part of the roof,
Model 2

\[ h = 4,3m \]
\[ b = 36,6m \]
\[ l_s = b \]
\[ h/b = 0,117 \]
\[ \mu_3 = 1,2 \]

Note that Case (i) is equal is to case (i) of Model 1.

Applying the model on the right does not take into account the accumulation of snow on the flat part of the roof. That accumulation will be estimated by the model of the church where in the case of the arena the arches parts are the ‘towers.’

\[ \mu_1 = 0,8 \]
\[ \mu_2 = \mu_w + \mu_w \]
\[ \mu_v = 0 \text{ because } \alpha_2 \leq 15^\circ \]
\[ \mu_w = (b_1 + b_2)/2h \leq \gamma h/s_k \]
\[ \gamma = 2,0 \text{ kN/m}^3 \]
\[ b_1 = 36,6m \]
\[ b_2 = 14,8m \]

\[ h \text{ will be estimated as } 0,5h \text{ of the vaulted roof model above, hence} \]
\[ h = 0 m \]
\[ \mu_w = \text{infinity} \leq 0 \text{ does not hold and therefore} \]
\[ \mu_w = 0,8 \]
\[ l_s = 2h = 4,3m, \text{ however } 5 \leq l_s \leq 15 m, \text{ hence} \]
\[ l_s = 5m \]
The position of the load triangles can be arranged in four different ways. All four cases will be considered. Later on, the different cases will be referred to as Case A – D.
<table>
<thead>
<tr>
<th>Name</th>
<th>Description</th>
<th>Action type</th>
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<tr>
<td>LC1</td>
<td>Self weight</td>
<td>Permanent</td>
</tr>
<tr>
<td>LC2</td>
<td>Dead load</td>
<td>Permanent</td>
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<tr>
<td>LC3</td>
<td>Wind wall 0</td>
<td>Variable</td>
</tr>
<tr>
<td>LC4.1</td>
<td>Wind wall 90.1</td>
<td>Variable</td>
</tr>
<tr>
<td>LC5.1A</td>
<td>Wind roof 0,1A</td>
<td>Variable</td>
</tr>
<tr>
<td>LC5.2A</td>
<td>Wind roof 0,2A</td>
<td>Variable</td>
</tr>
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<td>LC6.1</td>
<td>Wind roof 90.1</td>
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</tr>
<tr>
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<td>Wind roof 0,1B</td>
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<tr>
<td>LC5.2B</td>
<td>Wind roof 0,2B</td>
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<tr>
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</tr>
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<td>Wind roof 90.2B</td>
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</tr>
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<td>LC6.3A</td>
<td>Wind roof 90.3A</td>
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<tr>
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<td>Wind roof 90.3B</td>
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</tr>
<tr>
<td>LC7.1</td>
<td>Wind Internal pressure</td>
<td>Variable</td>
</tr>
<tr>
<td>LC7.2</td>
<td>Wind Internal suction</td>
<td>Variable</td>
</tr>
<tr>
<td>LC8.1</td>
<td>Snow 1</td>
<td>Variable</td>
</tr>
<tr>
<td>LC8.2</td>
<td>Snow 2</td>
<td>Variable</td>
</tr>
<tr>
<td>LC8.3A</td>
<td>Snow 3A</td>
<td>Variable</td>
</tr>
<tr>
<td>LC8.3B</td>
<td>Snow 3B</td>
<td>Variable</td>
</tr>
<tr>
<td>LC8.3C</td>
<td>Snow 3C</td>
<td>Variable</td>
</tr>
<tr>
<td>LC8.3D</td>
<td>Snow 3D</td>
<td>Variable</td>
</tr>
</tbody>
</table>

Load cases as created in Scia Engineer. For more details see Annex 2.

**Combinations**

All relevant combinations, both ULS and SLS are stated in Annex 3.
Design of roof and wall systems
The design of roof and wall profiles is done based on the governing load cases. The profiles are chosen by use of design tables. The used tables are shown in the annex.

Roof
Considering the flat part of the roof (2,16 kN/m²) and the arched parts (1,62 kN/m²) have a significant difference in maximum wind load, they will be designed separately. These values are both higher than the maximum pressure of the imposed roof load (1,0 kN/m²).

Flat roof
The maximum load in ULS is: 1,65*Qₖ = 1,65 * 2,16 = 3,56 kN/m² (suction)
For the deflection the SLS value 2,16 kN/m² is used.
The maximum deflection is equal to L/250 with L equal to the span.

*Chosen profile: 200.375/2 with thickness 1,13mm, see annex.*

Arched roof
The maximum load in ULS is: 1,65*Qₖ = 1,65 * 1,62 = 2,67 kN/m² (suction)
For the deflection the SLS value 1,62 kN/m² is used.

*Chosen profile: 200.375/2 with thickness 1,00mm, see annex.*
Wall Panels
Maximum load on the wall is 1,44 kN/m² in SLS. (suction)
In ULS this is: 1,65 * 1,44 = 2,38 kN/m².
Maximum deflection is h/300.
Span 3m.

*Chosen profile: 80mm thick panel of Kingspan, see annex.*

Profiles between columns to attach panels
Desired span between profiles = 2m.
Maximum load in SLS on beam: 1,44 * 2 = 2,88 kN/m. (suction)
IN ULS 2,88 * 1,65 = 4,75 kN/m.

\[ M_y = \frac{1}{8} \times 21,4 \times 6^2 = 21,4 \text{ kNm. } f_y = 355 \text{ MPa.} \]

\[ W_y = \frac{M_y}{f_y} = \frac{21,4 \times 6^2}{355} = 60,28 \text{ cm}^3. \]

\[ I = 366 \text{ cm}^4. \ E = 210000 \text{ N/mm}^2. \]

\[ d = \frac{5ql^4}{384E} = \frac{5 \times 2,88 \times 6^4}{384 \times 210 \times 366} = 0,063 \text{ m} = 63 \text{ mm.} \]

\[ d_{max} = \frac{L}{250} = \frac{6000}{250} = 24 \text{ mm.} \]

\[ I_{min} = \frac{5ql^4}{384E} = 965 \text{ cm}^4. \]

*Chosen profile: rectangular hollow section 200x100x5,0*

\[ I = 1509 \text{ cm}^4. \]

\[ d \Rightarrow 15 \text{ mm.} \]

\[ g = 22,69 \text{ kg/m} \Rightarrow 0,23 \text{ kN/m} \]

\[ M = \frac{1}{8}ql^2 = \frac{0,23}{8} \times 6^2 = 1,04 \text{ kNm} \Rightarrow \text{neglectable} \]
Design of main load bearing elements

Firstly the mechanical model of the structure is shown.

The structure exists of seven members, B1-B7. However in reality B4 and B5, as well as B6 and B7, are part of the same beam. Furthermore the structure will be designed symmetrically and hence B1 and B2 will be the same profiles, just as B4 and B7. Initially the members are designed based on the model above. The profiles of each member will be chosen in a way that each member suffices the deflection limit for the governing serviceability limit state.

Deflection

The vertical limit is l/250 and the horizontal limit is h/300, where l is equal to the length of the member. In the table below the maximum values per member is shown.

<table>
<thead>
<tr>
<th>Member</th>
<th>length (m)</th>
<th>wmax (mm)</th>
<th>height (m)</th>
<th>umax (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1/B2</td>
<td>NA</td>
<td>NA</td>
<td>8</td>
<td>27</td>
</tr>
<tr>
<td>B3</td>
<td>14,8</td>
<td>59</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>B4-B7</td>
<td>36,6</td>
<td>146</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

In below pictures the maximum displacements for the governing SLS combination (21) is given. This is the second combination with snow action. The most governing combination with wind is SLS11. Firstly the vertical displacements are shown and secondly the horizontal ones.
Sway imperfections
Sway imperfections of the columns are taken into account.

\[ \phi_0 = \frac{1}{200} \]

\( m \ (\#\text{columns}) = 2 \)

\[ \alpha_h = \frac{2}{\sqrt{h}} = \frac{2}{\sqrt{8}} = 0.707 \]

\[ \alpha_m = \sqrt{\frac{0.5(1 + \frac{1}{m})}{\frac{0.5(1 + \frac{1}{2})}} = 0.866} \]

\[ \phi = \phi_0\alpha_h\alpha_m = \frac{1}{200} \times 0.707 \times 0.866 = 0.00306 \]

\[ \phi N_{Ed} = 0.00306 \times 207.67 = 0.6 \ kN \]

This results in the following extra load case:

Immediately there can be noticed that the horizontal deflection is a lot closer to its maximum than the vertical one. Hence the influence of tension bars underneath the main beams (B4 & B7) will be investigated. This will be done however for ULS combinations.

For ULS the maximum stress levels are investigated. Based on different variants and their results a final variant will be chosen. Below the stress levels of ULS21.

First there can be seen that the stress level in the IPE300 beam is too high for the type of steel used (S355). Investigated will be how other variants will have influence on the stress levels in all beams but assumed can be that the IPE beam need to be a larger profile.
Variants
The different variants will be compared based on ULS combination 21, the governing combination. As a start hinged column supports are compared with fixed column support.

Variant 1 fixed column supports

Variant 2 hinged column supports

While the maximum moment in the columns stays roughly equal, there is a difference of almost 300 kNm in the arch. Therefore fixed column bases will be used.

Variant 3

Variant 3 is based on variant 1, where the difference is that the middle supports are both sliding.

Note that the internal forces are almost identical to those of variant 1.

Next pictures are comparisons between the two variants. Note that the profiles are equal in both variants. The upper picture is the result of variant 1 while the lower is variant 3.
**Variant 4**

In variant 4 tension bars are added. Even though differences are small between variant 1 and 3, variant 1 is chosen to reduce the deformation in the core of the frame.

Resulting in a large reduction of internal forces and stresses.

Therefore smaller profiles can be used. Variant 5 will be chosen such that every member is within the limits of deflection.
Variant 5

Profiles

Maximum deflections in SLS21

Maximum stresses in ULS
Internal forces
Column check

ULS

Profile HEA280

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>h (profile)</td>
<td>270 mm</td>
</tr>
<tr>
<td>b</td>
<td>280 mm</td>
</tr>
<tr>
<td>A</td>
<td>97.3 cm²</td>
</tr>
<tr>
<td>t_w</td>
<td>8 mm</td>
</tr>
<tr>
<td>t_f</td>
<td>13 mm</td>
</tr>
<tr>
<td>r</td>
<td>24 mm</td>
</tr>
<tr>
<td>E</td>
<td>210000 N/mm²</td>
</tr>
<tr>
<td>G</td>
<td>79300 N/mm²</td>
</tr>
<tr>
<td>h (column)</td>
<td>8000 mm</td>
</tr>
<tr>
<td>L_cr,y (0.7h)</td>
<td>5600 mm</td>
</tr>
<tr>
<td>I_y (hor. Axis)</td>
<td>13670 cm⁴</td>
</tr>
<tr>
<td>I_z (ver. axis)</td>
<td>4760 cm⁴</td>
</tr>
<tr>
<td>I_y</td>
<td>11.9 cm</td>
</tr>
<tr>
<td>I_z</td>
<td>7 cm</td>
</tr>
<tr>
<td>I_w</td>
<td>7.85E-07 m⁶</td>
</tr>
<tr>
<td>I_t</td>
<td>6.21E-07 m⁴</td>
</tr>
<tr>
<td>W_y,pl</td>
<td>1010 cm³</td>
</tr>
<tr>
<td>f_y</td>
<td>355 MPa</td>
</tr>
</tbody>
</table>

Profile classification

\[
\epsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81
\]

\[
c = h - 2r - 2t_f = 270 - 2 \times 24 - 2 \times 13 = 196 \text{ mm}
\]

\[
\frac{c}{t_w} = \frac{196}{8} = 24.5 \leq 72\epsilon = 58,6 \Rightarrow \text{Class 1}
\]

Critical Moment and lateral torsional buckling reduction
\( k_w = 0.7 \) gives

\[
\kappa_{wt} = \frac{\pi}{k_w L} \sqrt{\frac{E I_w}{G I_t}} = \frac{\pi}{0.7 \times 8000} \sqrt{\frac{210000 \times 7.85 \times 11}{79300 \times 6.21 \times 10^5}} = 1.467
\]

\( M_1 = -121.1 \text{ kNm}, M_2 = 143.9 \text{ kNm} \)

\[
\psi = \frac{M_1}{M_2} = \frac{145.8}{-121.7} = -0.83
\]

\[
C_1 = \sqrt{0.31 + 0.428 \psi + 0.262 \psi^2} = \sqrt{0.31 + 0.428 \times -0.83 + 0.262 (-0.83)^2} = 2.7
\]

\( k_s = 1.0 \)

\( C_3 = 0 \)

\[
\mu_{cr} = \frac{C_1}{k_s} \frac{1 + k_{wt}^2 + (C_2 \xi - C_3 \xi_j)^2 - (C_2 \xi - C_3 \xi_j)}{1 + 1.467^2} = 4.83
\]

\[
M_{cr} = \mu_{cr} \frac{\pi \sqrt{E I G I_t}}{L} = 4.83 \frac{\pi \sqrt{210000 \times 4.76 \times 7 \times 79300 \times 6.21 \times 7}}{8000} E - 6 = 1900 \text{ kNm}
\]

\[
\tilde{\lambda}_{LT} = \frac{M_R}{M_{cr}} = \frac{W_f f_y}{\frac{1}{1900}} = 0.435
\]

\[
h \frac{b}{280} = 0.96 \leq 2 \Rightarrow \text{Buckling curve } b
\]

\( \alpha_{LT} = 0.34 \)

\( \beta = 0.75 \)

\[
\tilde{\lambda}_{LT,0} = 0.4
\]

\[
\Phi_{LT} = 0.5 [1 + \alpha_{LT} (\tilde{\lambda}_{LT} - \tilde{\lambda}_{LT,0}) + \beta \tilde{\lambda}^2_{LT}] = 0.5 [1 + 0.34 (0.435 - 0.4) + 0.75 \times 0.435^2] = 0.577
\]

\[
\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\phi_{LT}^2 - \beta \tilde{\lambda}^2_{LT}}} = \frac{1}{0.577 + \sqrt{0.577^2 - 0.75 \times 0.435^2}} = 0.986
\]

**Moment resistant check**

\( \gamma_{M1} = 1.0 \)

\[
M_{b,Rd} = \chi_{LT} \frac{W_f f_y}{\gamma_{M1}} = 0.986 \times \frac{101 \times 3 - 355000}{1.0} = 354.6 \text{ kNm}
\]

\( M_{Ed} = 145.8 \text{ kNm} \)

\[
\frac{M_{Ed}}{M_{b,Rd}} = \frac{145.8}{354.6} = 0.41
\]
Buckling

\[ \lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{355}} = 76.41 \]

\[ \bar{\lambda}_y = \frac{L_{cr}}{i_y \cdot \lambda_1} = \frac{5600}{119 \cdot \lambda_1} = 0.616 \]

\[ \bar{\lambda}_z = \frac{L_{cr}}{i_z \cdot \lambda_1} = \frac{4000}{70 \cdot \lambda_1} = 0.748 \]

\[ \alpha_y = 0.34 \]

\[ \alpha_z = 0.49 \]

\[ \Phi_y = 0.5 \left[ 1 + \alpha_y (\bar{\lambda}_y - 0.2) + \bar{\lambda}^2_y \right] = 0.5 \left[ 1 + 0.34 (0.616 - 0.2) + 0.616^2 \right] = 0.76 \]

\[ \Phi_z = 0.5 \left[ 1 + \alpha_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}^2_z \right] = 0.5 \left[ 1 + 0.49 (0.748 - 0.2) + 0.748^2 \right] = 0.91 \]

\[ \chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \bar{\lambda}^2_y}} = \frac{1}{0.76 + \sqrt{0.76^2 - 0.616^2}} = 0.829 \leq 1.0 \]

\[ \chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \bar{\lambda}^2_z}} = \frac{1}{0.91 + \sqrt{0.91^2 - 0.748^2}} = 0.695 \leq 1.0 \]

\[ N_{b,Ed} = \frac{X_z \cdot A \cdot f_y}{Y_{M1}} = \frac{0.695 \cdot 9.73E - 3 \cdot 355000}{1.0} = 2400 \text{ kN} \]

\[ N_{Ed} = 208 \text{ kN} \]

\[ N_{Ed} = \frac{208}{2400} = 0.09 \leq 1.0 \]

Combination of bending and normal force pressure

\[ \psi = \frac{M_1}{M_2} = \frac{145.8}{121.7} = -0.83 \]

\[ C_{my}, C_{mz}, C_{mLT} = 0.4 \]

\[ k_{yy} = C_{my} (1 + (\bar{\lambda}_y - 0.2) \frac{N_{Ed}}{N_{Rk}} X_y Y_{M1}) = 0.4 (1 + (0.616 - 0.2) \frac{208}{0.829} \frac{3454}{10}) = 0.414 \]

\[ k_{zy} = C_{mLT} (0.25) \frac{N_{Ed}}{N_{Rk}} X_y Y_{M1} = 1 - \frac{0.0748}{0.4 - 0.25} \frac{208}{0.695} \frac{3454}{10} = 0.957 \]

\[ \frac{N_{Ed}}{N_{Rk}} \frac{M_{yy,Ed}}{X_y Y_{M1}} = \frac{208}{0.829} \frac{3454}{10} + 0.414 \frac{145.8}{0.986} \frac{359.5}{10} = 0.24 \leq 1.0 \]
\[
\frac{N_{Ed}}{N_{Rk}} + k_{xy} \frac{M_{y,Ed}}{M_{y,Rk}} = \frac{208}{0,695} \frac{3454}{1,0} + 0,957 \frac{145,8}{0,980} \frac{359,5}{1,0} = 0,48 \leq 1,0
\]

Shear force

\[
A_v = A = 2bt_f + (t_w + 2r)t_f = 9730 - 2 \times 280 \times 13 + (8 + 2 \times 24)13 = 3178 \text{ mm}^2
\]

\[
V_{pl,Ed} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}} = \frac{3178(355/\sqrt{3})}{1,0} E - 3 = 651 kN
\]

\[
\frac{V_{Ed}}{V_{pl,Rd}} = \frac{34}{651} = 0,05 \leq 1,0
\]

SLS

Maximum deformation check

<table>
<thead>
<tr>
<th>Member</th>
<th>length (m)</th>
<th>wmax (mm)</th>
<th>height (m)</th>
<th>umax (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1/B2</td>
<td>NA</td>
<td>NA</td>
<td>8</td>
<td>27</td>
</tr>
<tr>
<td>B3</td>
<td>14,8</td>
<td>59</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>B4-B7</td>
<td>36,6</td>
<td>146</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

Maximum deformation is 26,1 mm in SLS combination 21 and therefore the SLS condition is satisfied.
Middle beam

ULS

Profile IPE450

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>h (profile)</td>
<td>450 mm</td>
</tr>
<tr>
<td>b</td>
<td>190 mm</td>
</tr>
<tr>
<td>A</td>
<td>98,8 cm²</td>
</tr>
<tr>
<td>tw</td>
<td>9.4 mm</td>
</tr>
<tr>
<td>tf</td>
<td>14.6 mm</td>
</tr>
<tr>
<td>r</td>
<td>21 mm</td>
</tr>
<tr>
<td>E</td>
<td>210000 N/mm²</td>
</tr>
<tr>
<td>G</td>
<td>79300 N/mm²</td>
</tr>
<tr>
<td>L (beam)</td>
<td>14800 mm</td>
</tr>
<tr>
<td>Lcr,y (L)</td>
<td>14800 mm</td>
</tr>
<tr>
<td>Iy (hor. Axis)</td>
<td>33740 cm⁴</td>
</tr>
<tr>
<td>Iz (ver. axis)</td>
<td>1680 cm⁴</td>
</tr>
<tr>
<td>iy</td>
<td>185 cm</td>
</tr>
<tr>
<td>iz</td>
<td>41 cm</td>
</tr>
<tr>
<td>Iw</td>
<td>7.91E-07 m⁶</td>
</tr>
<tr>
<td>It</td>
<td>6.69E-07 m⁴</td>
</tr>
<tr>
<td>Wy,pl</td>
<td>1499,6 cm³</td>
</tr>
<tr>
<td>fy</td>
<td>355 MPa</td>
</tr>
</tbody>
</table>

Profile classification

\[ \varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81 \]

\[ c = h - 2r - 2t_f = 450 - 2 * 21 - 2 * 14.6 = 378.8 \text{mm} \]

\[ \frac{c}{t_w} = \frac{378.8}{9.4} = 40.3 \leq 72\varepsilon = 58.6 \Rightarrow \text{Class 1} \]

Critical Moment and lateral torsional buckling reduction
The critical Moment has been calculated by using software LTBeamN. See the appendix for the parameters and complete results of this calculation.

\[ \mu_{cr} = 17.03 \]
\[ M_{cr} = 3677 \text{ kNm} \]

\[ \lambda_{LT} = \sqrt{\frac{M_R}{M_{cr}}} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{1.01E - 3 \times 355000}{1678}} = 0.38 \]

\[ h = \frac{450}{190} = 2.37 \geq 2 \Rightarrow \text{Buckling curve c} \]
\[ \alpha = 0.49 \]
\[ \beta = 0.75 \]
\[ \lambda_{LT,0} = 0.4 \]

\[ \Phi_{LT} = 0.5[1 + \alpha_{LT} (\lambda_{LT} - \lambda_{LT,0}) + \beta \lambda_{LT}^2] = 0.5[1 + 0.49(0.38 - 0.4) + 0.75 \times 0.38^2] = 0.555 \]

\[ \chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta \lambda_{LT}^2}} = \frac{1}{0.55 + \sqrt{0.55^2 - 0.75 \times 0.38^2}} = 1.01 \Rightarrow \chi_{LT} = 1.0 \]

**Moment resistant check**

\[ y_{M1} = 1.0 \]

\[ M_{b, Rd} = \chi_{LT} \times \frac{W_y f_y}{y_{M1}} = 1.0 \times \frac{1.5E - 3 \times 355000}{1.0} = 523.3 \text{ kNm} \]

\[ M_{Ed} = 215.9 \text{ kNm} \]

\[ \frac{M_{Ed}}{M_{b, Rd}} = \frac{215.9}{523.3} = 0.41 \]

**Buckling**

\[ \lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{355}} = 76.41 \]

\[ \lambda_y = \frac{L_{cr}}{i_y \lambda_1} = \frac{14800}{1850 \times 76.41} = 0.105 \]

\[ \lambda_z = \frac{L_{cr}}{i_z \lambda_1} = \frac{14800}{410 \times 76.41} = 0.472 \]

\[ \alpha_y = 0.21 \]

\[ \alpha_z = 0.34 \]

\[ \Phi_y = 0.5[1 + \alpha_y (\lambda_y - 0.2) + \lambda_y^2] = 0.5[1 + 0.21(0.105 - 0.2) + 0.105^2] = 0.50 \]

\[ \Phi_z = 0.5[1 + \alpha_z (\lambda_z - 0.2) + \lambda_z^2] = 0.5[1 + 0.34(0.472 - 0.2) + 0.472^2] = 0.66 \]
\[
\chi_y = \frac{1}{\Phi_y + \sqrt{\phi_y^2 - \chi_y^2}} = \frac{1}{0,50 + \sqrt{0,50^2 - 0,105^2}} = 1,02 \Rightarrow \chi_y = 1,0
\]

\[
\chi_z = \frac{1}{\Phi_z + \sqrt{\phi_z^2 - \chi_z^2}} = \frac{1}{0,91 + \sqrt{0,91^2 - 0,748^2}} = 0,90 \leq 1,0
\]

\[
N_{b,Rd} = \frac{\chi_z * A * f_y}{y_{M1}} = \frac{0,90 * 9,88E - 3 * 355000}{1,0} = 3143 \text{ kN}
\]

\[
N_{Ed} = 206 \text{ kN}
\]

\[
\frac{N_{Ed}}{N_{b,Rd}} = \frac{206}{3143} = 0,07 \leq 1,0
\]

**Combination of bending and normal force pressure**

\[
\psi = \frac{M_1}{M_2} = \frac{124,3}{124,4} = 1
\]

\[
M_s = 215,9 \text{ kNm}
\]

\[
\alpha_z = -1,0
\]

\[
C_{my}, C_{mz}, C_{mLT} = 0,9
\]

\[
k_{yy} = C_{my} (1 + (\lambda_y - 0,2)) \frac{N_{Ed}}{\chi_y \gamma_{M1}} = 0,9(1 + (0,105 - 0,2)) \frac{206}{1,0} \frac{3143}{1,0} = 0,894
\]

\[
k_{zy} = 1 - \frac{0,1\lambda_z}{C_{mLT} - 0,25} \frac{N_{Ed}}{\chi_z \gamma_{M1}} = 1 - \frac{0,0157}{0,9 - 0,25} \frac{206}{1,0} \frac{3143}{1,0} = 0,990
\]

\[
\frac{N_{Ed}}{N_{Rk}} + k_{yy} \frac{M_{y,Ed}}{M_{y,Rk}} = \frac{206}{1,0} \frac{3143}{1,0} + 0,894 \frac{215,9}{0,976} \frac{532,3}{1,0} = 0,42 \leq 1,0
\]

\[
\frac{N_{Ed}}{N_{Rk}} + k_{zy} \frac{M_{y,Ed}}{M_{y,Rk}} = \frac{206}{1,0} \frac{3143}{1,0} + 0,990 \frac{215,9}{0,976} \frac{532,3}{1,0} = 0,46 \leq 1,0
\]

**Shear force**

\[
A_v = A - 2bt_f + (t_w + 2r)t_f = 9880 - 2 * 190 * 14,6 + (9,4 + 2 * 21)14,6 = 5082 \text{ mm}^2
\]

\[
V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}} = \frac{5082(355/\sqrt{3})}{1,0} E - 3 = 1042 \text{ kN}
\]

\[
V_{Ed} = 92 \text{ kN}
\]

\[
\frac{V_{Ed}}{V_{pl,Rd}} = \frac{92}{1042} = 0,09 \leq 1,0
\]
SLS

Maximum deformation check

<table>
<thead>
<tr>
<th>Member</th>
<th>length (m)</th>
<th>wmax (mm)</th>
<th>height (m)</th>
<th>umax (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1/B2</td>
<td>NA</td>
<td>NA</td>
<td>8</td>
<td>27</td>
</tr>
<tr>
<td>B3</td>
<td>14.8</td>
<td>59</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>B4-B7</td>
<td>36.6</td>
<td>146</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

Maximum deformation is 41.6mm in SLS combination 21 and therefore the SLS condition is satisfied.
Arch beam

ULS

Profile HEA280

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>h (profile)</td>
<td>270 mm</td>
</tr>
<tr>
<td>b</td>
<td>280 mm</td>
</tr>
<tr>
<td>A</td>
<td>97.3 cm²</td>
</tr>
<tr>
<td>tw</td>
<td>8 mm</td>
</tr>
<tr>
<td>tf</td>
<td>13 mm</td>
</tr>
<tr>
<td>r</td>
<td>24 mm</td>
</tr>
<tr>
<td>E</td>
<td>210000 N/mm²</td>
</tr>
<tr>
<td>G</td>
<td>79300 N/mm²</td>
</tr>
<tr>
<td>h (column)</td>
<td>8000 mm</td>
</tr>
<tr>
<td>L_{cr,y}(0,7h)</td>
<td>5600 mm</td>
</tr>
<tr>
<td>I_y (hor. Axis)</td>
<td>13670 cm⁴</td>
</tr>
<tr>
<td>I_z (ver. axis)</td>
<td>4760 cm⁴</td>
</tr>
<tr>
<td>i_y</td>
<td>11.9 cm</td>
</tr>
<tr>
<td>i_z</td>
<td>7 cm</td>
</tr>
<tr>
<td>I_w</td>
<td>7.85E-07 m⁶</td>
</tr>
<tr>
<td>I_t</td>
<td>6.21E-07 m⁴</td>
</tr>
<tr>
<td>W_{y,pl}</td>
<td>1010 cm³</td>
</tr>
<tr>
<td>f_y</td>
<td>355 MPa</td>
</tr>
</tbody>
</table>

Profile classification

\[ \varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81 \]

\[ c = h - 2r - 2t_f = 270 - 2 \times 24 - 2 \times 13 = 196 \text{mm} \]

\[ \frac{c}{t_w} = \frac{196}{8} = 24.5 \leq 72 \varepsilon = 58.6 \Rightarrow Class \ 1 \]

Critical Moment and lateral torsional buckling reduction

The critical Moment has been calculated by using software LTBeamN. See the appendix for the parameters and complete results of this calculation.

\[ \mu_{cr} = 3.76 \]
\( M_{cr} = 576 \, kNm \)

\[
\tilde{\lambda}_{LT} = \sqrt{\frac{M_R}{M_{cr}}} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{1,01E - 3 \times 355000}{576}} = 0,79
\]

\[
h = \frac{270}{280} = 0,96 \leq 2 \Rightarrow \text{Buckling curve b}
\]

\( \alpha_{LT} = 0,34 \)

\( \beta = 0,75 \)

\( \tilde{\lambda}_{LT,0} = 0,4 \)

\[
\Phi_{LT} = 0,5[1 + \alpha_{LT}(\tilde{\lambda}_{LT} - \tilde{\lambda}_{LT,0}) + \beta\tilde{\lambda}_{LT}^2] = 0,5[1 + 0,34(0,79 - 0,4) + 0,75 \times 0,79^2] = 0,80
\]

\[
\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\phi_{LT}^2 - \beta \tilde{\lambda}_{LT}^2}} = \frac{1}{0,80 + \sqrt{0,80^2 - 0,75 \times 0,79^2}} = 0,823
\]

**Moment resistant check**

\( \gamma_{M1} = 1,0 \)

\[
M_{b, Rd} = \chi_{LT} \times \frac{W_y f_y}{Y_M1} = 0,823 \times \frac{1,01E - 3 \times 355000}{1,0} = 295,7 \, kNm
\]

\( M_{Ed} = 151,3 \, kNm \)

\[
\frac{M_{Ed}}{M_{b, Rd}} = \frac{151,3}{295,7} = 0,51
\]

**Buckling**

Stability combinations calculated by Scia
\( \alpha_{\text{critical}} = 10,03 \)

\( N_{Ed} = 333,5 \, kN \)

\( N_{cr} = \alpha_{cr} \times N_{Ed} = 10,03 \times 332,5 = 3334,8 \, kN \)

\( L_{cr,y} = \sqrt{\pi^2 E I_y / N_{cr}} = \sqrt{\pi^2 \times 210000 \times 136700000 / 3334800} = 9217 \, mm \)

However as can be seen from the stability combination in Scia, the critical buckling length is roughly \( L/2 \) and hence

\( L_{cr,y} = \frac{h}{2} = \frac{38100}{2} = 19050 \, mm \)

\( \lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{355}} = 76,41 \)

\( \bar{\lambda}_y = \frac{L_{cr,y}}{i_y \times \lambda_1} = \frac{19050}{119 \times 76,41} = 2,095 \)

\( L_{cr,z} = 6000 \, mm \) (based on bracing system)

\( \bar{\lambda}_z = \frac{L_{cr,z}}{i_z \times \lambda_1} = \frac{6000}{70 \times 76,41} = 1,122 \)

\( \alpha_y = 0,34 \)

\( \alpha_z = 0,49 \)

\( \Phi_y = 0,5[1 + \alpha_y(\bar{\lambda}_y - 0,2) + \bar{\lambda}_y^2] = 0,5[1 + 0,34(2,095 - 0,2) + 2,095^2] = 3,02 \)

\( \Phi_z = 0,5[1 + \alpha_z(\bar{\lambda}_z - 0,2) + \bar{\lambda}_z^2] = 0,5[1 + 0,49(1,122 - 0,2) + 1,112^2] = 1,36 \)

\( \chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \bar{\lambda}_y^2}} = \frac{1}{3,02 + \sqrt{3,02^2 - 2,095^2}} = 0,193 \leq 1,0 \)

\( \chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \bar{\lambda}_z^2}} = \frac{1}{1,36 + \sqrt{1,36^2 - 1,112^2}} = 0,473 \leq 1,0 \)
\[ N_{b, Rd} = \frac{X_y * A * f_y}{Y_{M1}} = \frac{0.193 * 9.73E - 3 * 355000}{1.0} = 666 \text{ kN} \]

\[ N_{Ed} = 332.5 \text{ kN} \]

\[ N_{Ed} \quad N_{b, Rd} = \frac{332.5}{666} = 0.50 \leq 1.0 \]

**Combination of bending and normal force pressure**

\[ \psi = \frac{M_1}{M_2} = -124.4 \quad 151.3 = 0.82 \]

\[ M_s = 32.4 \text{ kNm} \]

\[ \alpha_s = \frac{M_s}{M_2} = \frac{32.4}{-152.8} = -0.21 \]

\[ C_{my}, C_{mz}, C_{mLT} = 0.4 \]

\[ k_{yy} = C_{my}(1 + (\bar{\lambda}_y - 0.2) \frac{N_{Ed}}{X_y N_{Rk}}) = 0.4(1 + (2.095 - 0.2) \frac{332.5}{0.193 \frac{3454}{1.0}}) = 0.56 \]

\[ k_{xy} = 1 - \frac{0.1 \bar{\lambda}_x}{C_{mLT} - 0.25} \frac{N_{Ed}}{X_z N_{Rk}} = 1 - \frac{0.1122}{0.4 - 0.25} \frac{332.5}{0.473 \frac{3454}{1.0}} = 0.627 \]

\[ \frac{N_{Ed}}{N_{Rk}} + k_{xy} \frac{M_{y, Ed}}{M_{y, Rk}} = \frac{332.5}{0.829 \frac{3454}{1.0}} + 0.56 \frac{151.3}{0.823 \frac{358.6}{1.0}} = 0.775 \leq 1.0 \]

\[ \frac{N_{Ed}}{N_{Rk}} + k_{xy} \frac{M_{y, Ed}}{M_{y, Rk}} = \frac{332.5}{0.473 \frac{3454}{1.0}} + 0.627 \frac{151.3}{0.823 \frac{358.6}{1.0}} = 0.52 \leq 1.0 \]

**Shear force**

\[ A_v = A = 2b t_f + (t_w + 2r) t_f = 9730 - 2 * 280 * 13 + (8 + 2 * 24)13 = 3178 \text{ mm}^2 \]

\[ V_{pl, Rd} = \frac{A_v(f_y / \sqrt{3})}{Y_{M0}} = \frac{3178(355 / \sqrt{3})}{1.0} E - 3 = 651 \text{ kN} \]

\[ V_{Ed} = 122 \text{ kN} \]

\[ \frac{V_{Ed}}{V_{pl, Rd}} = \frac{122}{651} = 0.19 \leq 1.0 \]

**SLS**

**Maximum deformation check**

<table>
<thead>
<tr>
<th>Member</th>
<th>length (m)</th>
<th>Wmax (mm)</th>
<th>height (m)</th>
<th>umax (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1/B2</td>
<td>NA</td>
<td>NA</td>
<td>8</td>
<td>27</td>
</tr>
<tr>
<td>B3</td>
<td>14.8</td>
<td>59</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>B4-B7</td>
<td>36.6</td>
<td>146</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>
Maximum deformation is 47.6 mm in SLS combination 21 and therefore the SLS condition is satisfied.
**Tension bars**

The maximum tension in the tension bars is 243,2 kN in ULS21.

Maximum compression is 137,4 kN in ULS6.

To avoid compression the bars need to be pretensioned by 140 kN and hence the capacity of the bars should be 243,2 + 140 = 383,2 kN.

Using tension bars from Maccalloy with strength S460, the minimum diameter of the tension bar should be:

\[
d_{\text{min}} = \sqrt{\frac{4 \times 383,2 \times 1000}{460\pi}} = 32,6 \, mm
\]

Hence tension bars with a diameter of 36 mm will be used.
Bracing system

Bracing will be placed on both ends of the arena between the gable wall and the first regular frame. In the middle one extra bracing will be placed to account for effect of imperfections and friction of the wind. The gable wall will consist of 6 column underneath each side of the arch. The roof bracing is calculated based on the reactions of the wind force on the columns of the gable wall. The reactions of the corner columns will be added in the model of the wall bracing.

Roof bracing

The span between each column is equal to:

\[ s = \frac{35,6}{6} = 5,93 \, \text{m} \]

Therefore, the force on each column is equal to:

\[ q_k = c_{pe} \cdot q_p(z) \cdot s = 0,8 \cdot 1,2 \cdot 5,93 = 5,69 \, \frac{kN}{m} \]

Note that the amount of force on the corner column is only half.

For the bracing, wind load case 4 is governing, meaning the same wind coefficient applied on the whole wall. Due to the different heights of the columns the reactions differ in value. The reaction of each column is equal to: \[ R = \frac{1}{2} q_k L \] with \( L \) equal to the height of the column. The table below provides the reactions of each column, with exception of the corner one.

<table>
<thead>
<tr>
<th>column</th>
<th>height (m)</th>
<th>Reaction (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>13,70</td>
<td>39,00</td>
</tr>
<tr>
<td>2</td>
<td>15,05</td>
<td>42,83</td>
</tr>
<tr>
<td>3</td>
<td>15,21</td>
<td>43,30</td>
</tr>
<tr>
<td>4</td>
<td>14,22</td>
<td>40,48</td>
</tr>
<tr>
<td>5</td>
<td>11,95</td>
<td>34,00</td>
</tr>
</tbody>
</table>

This results in the following model including load case for the roof bracing:

Because both the load and construction are symmetric, the results are shown for half of the construction.
**ULS results - safety factor 1,65**

The maximum normal forces, both tensile and pressure, can be found in the diagonals on either side of the concrete core.

For the horizontal elements we need to distinguish 4 different cases, since the regular frame and gable wall frame are not equal and also the arch beam and the middle beam are different in both frames.

Before the design of the bracing elements themselves, the effects on the main frame and gable wall are analyzed.

The tensile resistance of the arch beam of the main frame is equal to 3454 kN and hence the 219 kN won’t cause a problem. The pressure force is smaller than the governing combination.

The middle frame has a resistance of 3143 kN of which 6% is used with a pressure force caused by a snow combination of 206 kN. The lowest tensile force in the middle beam is the same combination as the bracing is 48 kN tensile force. Hence the resulting normal force is lower than the governing one.

The forces on the gable wall will be taken into account during the design of the gable wall, next chapter.

<table>
<thead>
<tr>
<th></th>
<th>Middle beam</th>
<th>Arch beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main frame</td>
<td>-232 kN</td>
<td>-137 kN</td>
</tr>
<tr>
<td>Gable wall</td>
<td>-3,7 kN</td>
<td>-207 kN</td>
</tr>
<tr>
<td></td>
<td>7,4 kN</td>
<td>120 kN</td>
</tr>
</tbody>
</table>
Wall bracing

The reaction from the corner column needs to be added to the reaction of the roof bracing.

The force on the corner column is equal to:

\[ q_k = c_{pe} \times q_p(z) \times s = 0.8 \times 1.2 \times \frac{5.93}{2} = 2.85 \text{ kN/m} \]

The reaction to the top and bottom of the column is equal to:

\[ R_a = R_c = \frac{3}{8} \times q_k \times \frac{L}{2} = \frac{3}{8} \times 2.85 \times \frac{8}{2} = 4.27 \text{ kN} \]

Adding this reaction to the reaction from the roof gives:

\[ R_{top} = 76.93 + 4.27 = 81.2 \text{ kN} \]

The reaction to the middle of the column is equal to:

\[ R_b = \frac{5}{4} \times q_k \times \frac{L}{2} = \frac{5}{4} \times 2.85 \times \frac{8}{2} = 14.23 \text{ kN} \]

Hence resulting in the following model of the wall bracing.

Where the reaction on the bottom is neglected since the force will be directly absorbed by the support.
Diagonals
The diagonals will be uniform meaning the same diagonal will be used in both the roof and the wall. The diagonals of the wall are 7.21 meters long, while the longest diagonals of the roof are 9.32 and 8.87 meters. The normal forces in the roof diagonals are larger than in the wall and hence the diagonals of the roof are governing. Design will be done based on the governing combination: the diagonal of 8.87 meters has the highest internal force while the 9.32m one is the longest and has a respective amount of normal force as well.

Profile CHS168.3/5.0

<table>
<thead>
<tr>
<th>A</th>
<th>25.7 cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>210000 N/mm²</td>
</tr>
<tr>
<td>G</td>
<td>79300 N/mm²</td>
</tr>
<tr>
<td>L</td>
<td>8866 mm</td>
</tr>
<tr>
<td>Lcr,max</td>
<td>9320 mm</td>
</tr>
<tr>
<td>t</td>
<td>856 cm⁴</td>
</tr>
<tr>
<td>i</td>
<td>58 mm</td>
</tr>
<tr>
<td>W</td>
<td>102 cm³</td>
</tr>
</tbody>
</table>

Slenderness

\[
\frac{L_{cr,\text{max}}}{i} = \frac{9320}{58} = 161 \leq 250
\]

Buckling

\[
\lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{355}} = 76.41
\]

\[
\bar{\lambda} = \frac{L_{cr}}{i \cdot \lambda_1} = \frac{8866}{58 \cdot 76.41} = 2.00
\]
\( \alpha = 0,21 \)

\[
\Phi_y = 0,5 \left[ 1 + \alpha (\bar{\lambda} - 0,2) + \bar{\lambda}^2 \right] = 0,5 \left[ 1 + 0,21(2 - 0,2) + 2^2 \right] = 2,69
\]

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{2,69 + \sqrt{2,69^2 - 2^2}} = 0,223
\]

\[
N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0,223 \cdot 2,57E - 3 \cdot 355000}{1,0} = 203 \text{ kN}
\]

\[
N_{Ed} = 138 \text{ kN (compression)}
\]

\[
\frac{N_{Ed}}{N_{b,Rd}} = \frac{138}{203} = 0,68 \leq 1,0
\]

\[
N_{Ed} = 162 \text{ kN (tension)}
\]

\[
N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M1}} = \frac{2,57E - 3 \cdot 355000}{1,0} = 912 \text{ kN}
\]

\[
\frac{N_{Ed}}{N_{c,Rd}} = \frac{162}{912} = 0,18 \leq 1,0
\]

With the same calculation it can be shown that for the longest diagonal (L = 9,32m):

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{2,91 + \sqrt{2,91^2 - 2,1^2}} = 0,203
\]

\[
N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0,203 \cdot 2,57E - 3 \cdot 355000}{1,0} = 185 \text{ kN}
\]

\[
N_{Ed} = 94 \text{ kN (pressure)}
\]

\[
\frac{N_{Ed}}{N_{b,Rd}} = \frac{94}{185} = 0,51 \leq 1,0
\]

\[
N_{Ed} = 103 \text{ kN (tension)}
\]

\[
\frac{N_{Ed}}{N_{c,Rd}} = \frac{103}{912} = 0,11 \leq 1,0
\]
Struts
The same procedure is followed as for the diagonals.

**Profile CHS88.9/3.2**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>8.62  cm²</td>
</tr>
<tr>
<td>E</td>
<td>210000 N/mm²</td>
</tr>
<tr>
<td>G</td>
<td>79300 N/mm²</td>
</tr>
<tr>
<td>L</td>
<td>6000 mm</td>
</tr>
<tr>
<td>L_cr,max</td>
<td>6000 mm</td>
</tr>
<tr>
<td>i</td>
<td>79.2 cm</td>
</tr>
<tr>
<td>i</td>
<td>30 mm</td>
</tr>
<tr>
<td>W</td>
<td>17.8 cm³</td>
</tr>
</tbody>
</table>

**Slenderness**

\[
\frac{L_{cr,max}}{i} = \frac{6000}{30} = 200 \leq 250
\]

**Buckling**

\[
\lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \frac{\pi}{\sqrt{210000 \over 355}} = 76.41
\]

\[
\lambda = \frac{L_{cr}}{i * \lambda_1} = \frac{6000}{30 \times 76.41} = 2.62
\]

\[
\alpha = 0.21
\]

\[
\Phi_y = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 0.5[1 + 0.21(2.62 - 0.2) + 2.62^2] = 4.18
\]

\[
\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = \frac{1}{4.18 + \sqrt{4.18^2 - 2.62^2}} = 0.134
\]

\[
N_{b,Ed} = \frac{\chi * A * f_y}{\gamma_M} = \frac{0.134 \times 8.62E - 4 \times 355000}{1} = 41 kN
\]

\[
N_{Ed} = 30 kN \text{ (pressure)}
\]

\[
\frac{N_{Ed}}{N_{b,Ed}} = \frac{30}{41} = 0.73 \leq 1.0
\]

**N_{Ed} = 101 kN \text{ (tension)}**

\[
N_{c,Ed} = \frac{A * f_y}{\gamma_M} = \frac{8.62E - 4 \times 355000}{1} = 306 kN
\]

\[
\frac{N_{Ed}}{N_{c,Ed}} = \frac{101}{306} = 0.33 \leq 1.0
\]
Gable wall
Wall panels have a weight of 14,2 kg/m\(^2\). With a span of 2 meters each profile takes 14,2*2 = 28,4 kg/m. A load on the profiles of 0,3 kN/m will be calculated.

Middle beam
ULS
Profile IPE300

<table>
<thead>
<tr>
<th>h (profile)</th>
<th>300</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>150</td>
<td>mm</td>
</tr>
<tr>
<td>A</td>
<td>53,8</td>
<td>cm(^2)</td>
</tr>
<tr>
<td>tw</td>
<td>7,1</td>
<td>mm</td>
</tr>
<tr>
<td>tf</td>
<td>10,7</td>
<td>mm</td>
</tr>
<tr>
<td>r</td>
<td>15</td>
<td>mm</td>
</tr>
<tr>
<td>E</td>
<td>210000</td>
<td>N/mm(^2)</td>
</tr>
<tr>
<td>G</td>
<td>79300</td>
<td>N/mm(^2)</td>
</tr>
<tr>
<td>L</td>
<td>14800</td>
<td>mm</td>
</tr>
<tr>
<td>L(_{cr,y})</td>
<td>14800</td>
<td>mm</td>
</tr>
<tr>
<td>I(_y) (hor. axis)</td>
<td>8360</td>
<td>cm(^4)</td>
</tr>
<tr>
<td>I(_z) (ver. axis)</td>
<td>604</td>
<td>cm(^4)</td>
</tr>
<tr>
<td>i(_y)</td>
<td>125</td>
<td>mm</td>
</tr>
<tr>
<td>i(_z)</td>
<td>34</td>
<td>mm</td>
</tr>
<tr>
<td>I(_w)</td>
<td>1,26E-07</td>
<td>m(^6)</td>
</tr>
<tr>
<td>I(_t)</td>
<td>2,01E-07</td>
<td>m(^4)</td>
</tr>
<tr>
<td>W(_y,pl)</td>
<td>557,3</td>
<td>cm(^3)</td>
</tr>
<tr>
<td>f(_y)</td>
<td>355</td>
<td>MPa</td>
</tr>
</tbody>
</table>

Profile classification
\[
\varepsilon = \sqrt{235/f_y} = \sqrt{235/355} = 0,81
\]
\[
c = h - 2r - 2t_f = 300 - 2 \times 15 - 2 \times 10,7 = 248,6 \text{ mm}
\]
\[
\frac{c}{l_w} = \frac{248,6}{7,1} = 35,0 \leq 72\varepsilon = 58,6 \Rightarrow \text{Class 1}
\]
Critical Moment and lateral torsional buckling reduction

The critical Moment has been calculated by using software LTBeamN. See the appendix for the parameters and complete results of this calculation.

\[ \mu_{cr} = 3.26 \]
\[ M_{cr} = 320 \text{ kNm} \]

\[ \bar{\lambda}_{LT} = \frac{M_R}{M_{cr}} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{557E - 4 \times 355000}{320}} = 0.79 \]

\[ h = \frac{300}{150} = 2.0 \leq 2 \Rightarrow \text{Buckling curve } b \]

\[ \alpha_{LT} = 0.34 \]
\[ \beta = 0.75 \]

\[ \bar{\lambda}_{LT,0} = 0.4 \]

\[ \Phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^2] = 0.5[1 + 0.34(0.79 - 0.4) + 0.75 \times 0.79^2] = 0.80 \]

\[ \chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta \bar{\lambda}_{LT}^2}} = \frac{1}{0.80 + \sqrt{0.80^2 - 0.75 \times 0.79^2}} = 0.825 \leq 1.0 \]

Moment resistant check

\[ \gamma_{M1} = 1.0 \]

\[ M_{b,rd} = \chi_{LT} \frac{W_y \cdot f_y}{\gamma_{M1}} = 0.825 \times \frac{557E - 4 \times 355000}{1.0} = 163.1 \text{ kNm} \]

\[ M_{Ed} = 100.1 \text{ kNm} \]

\[ \frac{M_{Ed}}{M_{b,rd}} = \frac{100.1}{163.1} = 0.51 \]

Buckling

\[ \lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{355}} = 76.41 \]

\[ \bar{\lambda}_y = \frac{L_{cr}}{i_y \cdot \lambda_1} = \frac{14800}{125 \times 76.41} = 1.55 \]

\[ \bar{\lambda}_z = \frac{L_{cr}}{i_z \cdot \lambda_1} = \frac{4933}{34 \times 76.41} = 1.90 \]

\[ \alpha_y = 0.21 \]

\[ \alpha_z = 0.34 \]

\[ \Phi_y = 0.5[1 + \alpha_y(\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = 0.5[1 + 0.21(1.55 - 0.2) + 1.55^2] = 1.84 \]

\[ \Phi_z = 0.5[1 + \alpha_z(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = 0.5[1 + 0.34(1.90 - 0.2) + 1.90^2] = 2.59 \]
\[ 1 = \Phi_y + \frac{1}{\sqrt{1,84^2 - 0,35^2}} = 1,84 + \sqrt{1,84^2 - 0,35^2} = 0,35 \]

\[ 1 = \Phi_z + \frac{1}{\sqrt{2,59^2 - 0,23^2}} = 2,59 + \sqrt{2,59^2 - 0,23^2} = 0,23 \]

\[ N_{b,Rd} = \frac{x_z \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0,23 \cdot 5,38E - 3 \cdot 355000}{1,0} = 438 \, kN \]

\[ N_{Ed} = 22,1 \, kN \]

\[ \frac{N_{Ed}}{N_{b,Rd}} = \frac{22,1}{438} = 0,05 \leq 1,0 \]

However due to bracing:

\[ N_{Ed} = 120 \, kN \]

\[ \frac{N_{Ed}}{N_{b,Rd}} = \frac{120}{438} = 0,27 \leq 1,0 \]

**Combination of bending and normal force pressure**

\[ \psi = \frac{M_1}{M_2} = \frac{-100,1}{-99,8} = 1,0 \]

\[ M_s = 71,4 \, kNm, \alpha_s = -0,72 \]

\[ C_{my}, C_{mz}, C_{mLT} = 0,1 - 0,8 \cdot -0,72 = 0,67 \]

\[ k_{yy} = C_{my}(1 + (\lambda_y - 0,2) \frac{N_{Ed}}{N_{RK}}) = 0,67(1 + (1,55 - 0,2) \frac{22,1}{0,35 \frac{1910}{1,0}}) = 0,67 \]

\[ k_{zz} = 1 - \frac{0,1 \lambda_z}{C_{mLT} - 0,25 \frac{N_{Ed}}{N_{RK}} \gamma_{M1}} = 1 - \frac{0,19}{0,67 - 0,25 \frac{22,1}{0,23 \frac{1910}{1,0}}} = 0,98 \]

\[ \frac{N_{Ed}}{N_{RK}} + k_{yy} \frac{M_{y,Ed}}{\gamma_{M1}} = \frac{22,1}{0,35 \frac{1910}{1,0}} + 0,67 \frac{100,1}{0,825 \frac{197,9}{1,0}} = 0,46 \leq 1,0 \]

\[ \frac{N_{Ed}}{N_{RK}} + k_{yy} \frac{M_{y,Ed}}{\gamma_{M1}} = \frac{22,1}{0,23 \frac{1910}{1,0}} + 1,0 \frac{100,1}{0,825 \frac{197,9}{1,0}} = 0,65 \leq 1,0 \]

**Shear force**

\[ A_v = A - 2bt_f + (t_w + 2r)t_f = 5380 - 2 \cdot 150 \cdot 10,7 + (7,1 + 2 \cdot 15)10,7 = 2567 \, mm^2 \]

\[ V_{pl,Rd} = \frac{A_v f_y \sqrt{3}}{\gamma_{M0}} = \frac{2567(355/\sqrt{3})}{1,0} E - 3 = 526 \, kN \]

\[ V_{Ed} = 46 \, kN \Rightarrow \frac{V_{Ed}}{V_{pl,Rd}} = \frac{46}{526} = 0,09 \leq 1,0 \]
SLS
Maximum deformation check

<table>
<thead>
<tr>
<th>Member</th>
<th>length (m)</th>
<th>wmax (mm)</th>
<th>height (m)</th>
<th>umax (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1/B2</td>
<td>NA</td>
<td>NA</td>
<td>8</td>
<td>27</td>
</tr>
<tr>
<td>B3</td>
<td>14,8</td>
<td>59</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>B4-B7</td>
<td>36,6</td>
<td>146</td>
<td>NA</td>
<td>NA</td>
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</tbody>
</table>

Maximum deformation is 44,7mm in SLS combination 21 and therefore the SLS condition is satisfied.
Arch beam

ULS

Profile IPE300

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>h (profile)</td>
<td>300 mm</td>
</tr>
<tr>
<td>b</td>
<td>150 mm</td>
</tr>
<tr>
<td>A</td>
<td>53,8 cm²</td>
</tr>
<tr>
<td>(t_w)</td>
<td>7,1 mm</td>
</tr>
<tr>
<td>(t_f)</td>
<td>10,7 mm</td>
</tr>
<tr>
<td>r</td>
<td>15 mm</td>
</tr>
<tr>
<td>E</td>
<td>210000 N/mm²</td>
</tr>
<tr>
<td>G</td>
<td>79300 N/mm²</td>
</tr>
<tr>
<td>L (arch)</td>
<td>38100 mm</td>
</tr>
<tr>
<td>(L_{cr,y})</td>
<td>6528 mm</td>
</tr>
<tr>
<td>(I_y) (hor. Axis)</td>
<td>8360 cm⁴</td>
</tr>
<tr>
<td>(I_z) (ver. axis)</td>
<td>604 cm⁴</td>
</tr>
<tr>
<td>(i_y)</td>
<td>125 cm</td>
</tr>
<tr>
<td>(i_z)</td>
<td>34 cm</td>
</tr>
<tr>
<td>(I_w)</td>
<td>1,26E-07 m⁶</td>
</tr>
<tr>
<td>(I_t)</td>
<td>2,01E-07 m⁴</td>
</tr>
<tr>
<td>(W_{y,pl})</td>
<td>557,3 cm³</td>
</tr>
<tr>
<td>(f_y)</td>
<td>355 MPa</td>
</tr>
</tbody>
</table>

Profile classification

\[
\epsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0,81
\]

\[
c = h - 2r - 2t_f = 300 - 2 \times 15 - 2 \times 10,7 = 248,6 \text{ mm}
\]

\[
\frac{c}{t_w} = \frac{248,6}{7,1} = 35,0 \leq 72\epsilon = 58,6 \Rightarrow Class 1
\]
Critical Moment and lateral torsional buckling reduction

The critical length as written in the table above is the length of the arch between the support and the first column, because there is the largest moment. Above picture shows the governing pressure force.

\( k_w = 1,0 \) gives

\[
\kappa_{wt} = \frac{\pi}{k_w L} \sqrt{\frac{E I_w}{G I_t}} = \frac{\pi}{1,0 \times 6528} \sqrt{\frac{210000 \times 1,26E11}{79300 \times 2,01E5}} = 0,620
\]

\( M_1 = -100,1 \, kNm, M_2 = -5,0 \, kNm \)

\[
\psi = \frac{M_1}{M_2} = \frac{-5,0}{-100,1} = 0,050
\]

\[
C_1 = \sqrt{0,31 + 0,428 \psi + 0,262 \psi^2} = \sqrt{0,31 + 0,428 \times 0,050 + 0,262(0,050)^2} = 1,735
\]

\( k_z = 1,0 \)

\( C_2, C_3 = 0 \)

\[
\mu_{cr} = \frac{C_1}{k_z} \left[ 1 + k_{wt}^2 + (C_2 \xi_g - C_3 \xi_j)^2 \right] - (C_2 \xi_g - C_3 \xi_j)] = \frac{1,735}{1,0} \sqrt{1 + 0,620^2} = 2,04
\]

\[
M_{cr} = \mu_{cr} \frac{\pi E I_G}{L} = 2,04 \times \frac{\pi \times 210000 \times 6,04E6 \times 79300 \times 2,01E5}{6528} E - 6 = 140 \, kNm
\]

\[
\bar{\lambda}_{LT} = \frac{M_R}{M_{cr}} = \frac{W_N f_y}{M_{cr}} = \sqrt{\frac{5,57E - 4 \times 355000}{140}} = 1,19
\]

\[
h = \frac{300}{150} = 2 \leq 2 = \text{Buckling curve } b
\]

\( a_{LT} = 0,34 \)

\( \beta = 0,75 \)

\( \lambda_{LT,0} = 0,4 \)

\[
\Phi_{LT} = 0,5(1 + a_{LT}(\bar{\lambda}_{LT} - \lambda_{LT,0}) + \beta\lambda_{LT}^2) = 0,5[1 + 0,34(1,19 - 0,4) + 0,75 \times 1,19^2] = 1,165
\]

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Limits</th>
<th>Buckling curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rolled I-sections</td>
<td>( h/b \leq 2 )</td>
<td>b</td>
</tr>
<tr>
<td>Rolled I-sections</td>
<td>( h/b &gt; 2 )</td>
<td>c</td>
</tr>
<tr>
<td>Welded I-sections</td>
<td>( h/b \leq 2 )</td>
<td>c</td>
</tr>
<tr>
<td>Welded I-sections</td>
<td>( h/b &gt; 2 )</td>
<td>d</td>
</tr>
</tbody>
</table>

Buckling curve: Imperfection factor \( a_{LT} \), \( a = 0,21 \), \( b = 0,34 \), \( c = 0,49 \), \( d = 0,76 \)
\[ \chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta \lambda_{LT}^2}} = \frac{1}{1,165 + \sqrt{1,165^2 - 0.75 \times 1,19^2}} = 0,585 \]

Moment resistant check

\[ \gamma_{M1} = 1,0 \]

\[ M_{b, Rd} = \chi_{LT} \times \frac{W_y \times f_y}{\gamma_{M1}} = 0,585 \times \frac{5,57E - 4 \times 355000}{1,0} = 115,7 kNm \]

\[ \frac{M_{Ed}}{M_{b, Rd}} = \frac{100,1}{197,9} = 0,51 \]

Buckling

\[ \lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{355}} = 76,41 \]

\[ \lambda_2 = \frac{L_{cr, y}}{i_y \times \lambda_1} = \frac{6528}{125 \times 76,41} = 0,683 \]

\[ L_{cr, x} = 6528 \text{ mm (based on bracing system)} \]

\[ \lambda_2 = \frac{L_{cr, z}}{i_z \times \lambda_1} = \frac{6528}{34 \times 76,41} = 2,513 \]

\[ \alpha_y = 0,21 \]

\[ \alpha_z = 0,34 \]

\[ \Phi_y = 0,5[1 + \alpha_y(\lambda_y - 0,2) + \lambda_y^2] = 0,5[1 + 0,21(0,683 - 0,2) + 0,683^2] = 0,78 \]

\[ \Phi_z = 0,5[1 + \alpha_z(\lambda_z - 0,2) + \lambda_z^2] = 0,5[1 + 0,34(2,513 - 0,2) + 2,513^2] = 4,05 \]

\[ \chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \lambda_y^2}} = \frac{1}{0,78 + \sqrt{0,78^2 - 0,683^2}} = 0,855 \leq 1,0 \]

\[ \chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \lambda_z^2}} = \frac{1}{4,05 + \sqrt{4,05^2 - 2,513^2}} = 0,138 \leq 1,0 \]
\[ N_{b, Rd} = \frac{X_y \ast A \ast f_y}{Y_{M1}} = \frac{0,138 \ast 5,38E - 3 \ast 355000}{1,0} = 264 \text{ kN} \]

\[ N_{Ed} = 45,4 \text{ kN} \]

\[ \frac{N_{Ed}}{N_{b, Rd}} = \frac{45,4}{264} = 0,17 \leq 1,0 \]

**Combination of bending and normal force pressure**

\[ \psi = \frac{M_1}{M_2} = -124,4 \]

\[ M_s = 32,4 \text{ kNm} \]

\[ \frac{M_s}{M_2} = 32,4 \leq 152,8 = -0,21 \]

\[ C_{my}, C_{mz}, C_{mLT} = 0,4 \]

\[ k_{yy} = C_{my}(1 + (\ddot{\lambda}_y - 0,2) \frac{N_{Ed}}{X_y N_{Rk}}) = 0,4(1 + (2,095 - 0,2) \frac{332,5}{0,193 \frac{3454}{1,0}}) = 0,56 \]

\[ k_{xy} = 1 - 0,1\ddot{\lambda}_x \frac{N_{Ed}}{C_{mLT} \frac{X_x N_{Rk}}{Y_{M1}}} = 1 - 0,1122 \frac{332,5}{0,4 - 0,25 \frac{3454}{0,473 \frac{358}{1,0}}} = 0,627 \]

\[ \frac{N_{Ed}}{N_{Rk}} + k_{yy} \frac{M_{y, Ed}}{X_y Y_{M1}} = \frac{332,5}{0,829 \frac{3454}{1,0}} + 0,56 \frac{151,3}{0,823 \frac{358,6}{1,0}} = 0,775 \leq 1,0 \]

\[ \frac{N_{Ed}}{N_{Rk}} + k_{xy} \frac{M_{y, Ed}}{X_x Y_{M1}} = \frac{332,5}{0,473 \frac{3454}{1,0}} + 0,627 \frac{151,3}{0,823 \frac{358,6}{1,0}} = 0,52 \leq 1,0 \]

**Shear force**

\[ A_v = A = 2t_bf_r + (t_w + 2r)t_f = 5380 - 2 \ast 150 \ast 10,7 + (7,1 + 2 \ast 15)10,7 = 2567 \text{ mm}^2 \]

\[ V_{pl, Rd} = A_v f_y \sqrt{3} \frac{\gamma_{M0}}{Y_{M1}} = \frac{2567(355/\sqrt{3})}{1,0} E - 3 = 526 \text{ kN} \]

\[ V_{Ed} = 48,2 \text{ kN} \]

\[ \frac{V_{Ed}}{V_{pl, Rd}} = \frac{48,2}{526} = 0,09 \leq 1,0 \]
Check of forces caused by bracing and combination ULS11

In combination ULS11 there is a tensile force of 29 kN in the arch. Combining this with the 207 kN pressure from the bracing results in a total of 178 kN pressure. The same procedure for the resistance check is done.

\[ C_1 = 1,13 \]
\[ k_z = 1,0 \]
\[ C_2 = C_3 = 0 \]
\[ \mu_{cr} = 1,36 \]
\[ M_{cr} = 101 kNm \]
\[ \lambda_{LT} = 1,40 \]
\[ \frac{h}{b} = \frac{300}{150} = 2 \leq 2 \Rightarrow Buckling curve b \]
\[ \alpha_{LT} = 0,34 \]
\[ \beta = 0,75 \]
\[ \tilde{\lambda}_{LT,0} = 0,4 \]
\[ \Phi_{LT} = 1,4 \]
\[ \chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\frac{\phi_{LT}^2 - \beta \lambda_{LT}^2}{\Phi_{LT}^2}}} = 0,47 \]

Moment resistant check

\[ \gamma_{M1} = 1,0 \]
\[ M_{b,Rd} = \chi_{LT} \cdot \frac{W_y \cdot f_y}{\gamma_{M1}} = 0,47 \cdot \frac{5,57E - 4 \cdot 355000}{1,0} = 93,6 kNm \]
\[ M_{Ed} = 10,9 kNm \]
\[ \frac{M_{Ed}}{M_{b,Rd}} = \frac{10,9}{93,6} = 0,12 \]
Buckling

\[ \lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{355}} = 76.41 \]

\[ \bar{\lambda}_y = \frac{L_{ct,y}}{i_y \times \lambda_1} = \frac{6020}{125 \times 76.41} = 0.63 \]

\[ L_{ct,y} = 6020 \text{ mm (based on bracing system)} \]

\[ \bar{\lambda}_z = \frac{L_{ct,z}}{i_z \times \lambda_1} = \frac{6020}{34 \times 76.41} = 2.32 \]

\[ \alpha_y = 0.21 \]

\[ \alpha_z = 0.34 \]

\[ \Phi_y = 0.5[1 + \alpha_y(\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = 0.74 \]

\[ \Phi_z = 0.5[1 + \alpha_z(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = 3.54 \]

\[ \chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \bar{\lambda}_y^2}} = \frac{1}{0.74 + \sqrt{0.74^2 - 0.63^2}} = 0.88 \leq 1.0 \]

\[ \chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \bar{\lambda}_z^2}} = \frac{1}{3.54 + \sqrt{3.54^2 - 2.32^2}} = 0.16 \leq 1.0 \]

\[ N_{b,rd} = \frac{X_y \times A \times f_y}{\gamma_{M1}} = \frac{0.16 \times 5.38E - 3 \times 355000}{1.0} = 307 \text{ kN} \]

\[ N_{Ed} = 178 \text{ kN} \]

\[ \frac{N_{Ed}}{N_{b,rd}} = \frac{178}{307} = 0.58 \leq 1.0 \]

**Combination of bending and normal force**

The moment resistance is utilized for 12% and the normal force resistance for 58% and therefore a combination will result in an approximate combination of 70%.
SLS
Maximum deformation check

The maximum allowable deformation is the smallest span divided by 250.

\[ d_{max} = \frac{L}{250} = \frac{5942}{250} = 24 \text{ mm} \]

Maximum deformation is 2.0 mm in SLS combination 21 and therefore the SLS condition is satisfied.
Middle columns

ULS

Profile HEA450

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>h (profile)</td>
<td>440</td>
<td>mm</td>
</tr>
<tr>
<td>b</td>
<td>300</td>
<td>mm</td>
</tr>
<tr>
<td>A</td>
<td>178</td>
<td>cm²</td>
</tr>
<tr>
<td>t_w</td>
<td>11,5</td>
<td>mm</td>
</tr>
<tr>
<td>t_f</td>
<td>21</td>
<td>mm</td>
</tr>
<tr>
<td>r</td>
<td>27</td>
<td>mm</td>
</tr>
<tr>
<td>E</td>
<td>210000</td>
<td>N/mm²</td>
</tr>
<tr>
<td>G</td>
<td>79300</td>
<td>N/mm²</td>
</tr>
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<td>h_max</td>
<td>15120</td>
<td>mm</td>
</tr>
<tr>
<td>L_cr,y</td>
<td>15120</td>
<td>mm</td>
</tr>
<tr>
<td>I_y (hor. Axis)</td>
<td>63720</td>
<td>cm⁴</td>
</tr>
<tr>
<td>I_z (ver. axis)</td>
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<tr>
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</tr>
<tr>
<td>I_t</td>
<td>2,44E-06</td>
<td>m⁴</td>
</tr>
<tr>
<td>W_{y,pl}</td>
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<td>cm³</td>
</tr>
<tr>
<td>f_y</td>
<td>355</td>
<td>MPa</td>
</tr>
</tbody>
</table>

Profile classification

\[ \epsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81 \]

\[ c = h - 2r - 2t_f = 440 - 2 \times 27 - 2 \times 21 = 344 \text{ mm} \]

\[ \frac{c}{t_w} = \frac{344}{11.5} = 29.9 \leq 72\epsilon = 58.6 \Rightarrow \text{Class 1} \]
Critical Moment and lateral torsional buckling reduction

\[ k_w = 1.0 \text{ gives} \]

\[ \kappa_{wt} = \frac{\pi}{k_w L} \sqrt{\frac{EI_w}{GL_t}} = \frac{\pi}{1.0 \times 15120} \sqrt{\frac{210000 \times 4.15E12}{79300 \times 2.44E6}} = 0.56 \]

\[ C_1 = 1.13 \]

\[ k_z = 1.0 \]

\[ C_2, C_3 = 0 \]

\[ \mu_{cr} = \frac{C_1}{k_z} \left[ 1 + \kappa_{wt}^2 + (C_2 \xi_g - C_3 \xi_j)^2 - (C_2 \xi_g - C_3 \xi_j) \right] = \frac{1.13}{1.0} \sqrt{1 + 0.56^2} = 1.235 \]

\[ M_{cr} = \mu_{cr} \frac{\pi \sqrt{EI_G L_t}}{L} = 1.235 \frac{\pi \sqrt{210000 \times 9.47E7 \times 79300 \times 2.44E6}}{15120} E - 6 = 503 \text{ kNm} \]

\[ \bar{\lambda}_{LT} = \left( \frac{M_R}{M_{cr}} \right) \frac{W_{fy}}{M_{cr}} = \sqrt{\frac{2.90E - 3 \times 355000}{503}} = 1.43 \]

\[ h = 440 \]

\[ \bar{b} = 300 = 1.47 \leq 2 \Rightarrow \text{Buckling curve b} \]

\[ \alpha_{LT} = 0.34 \]

\[ \beta = 0.75 \]

\[ \bar{\lambda}_{LT,0} = 0.4 \]

\[ \Phi_{LT} = 0.5 [1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^2] = 0.5[1 + 0.34(1.43 - 0.4) + 0.75 \times 1.43^2] = 1.44 \]

\[ \chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\frac{\phi_{LT}^2 - \beta \bar{\lambda}_{LT}^2}{1.44 + \sqrt{1.44^2 - 0.75 \times 1.43^2}}} = 0.46 \]

Moment resistant check

\[ \gamma_{M1} = 1.0 \]
\[ M_{b,Rd} = \chi_{LT} \cdot \frac{W_f \cdot f_y}{\gamma_{M1}} = 0,46 \cdot \frac{2,90E - 3 \cdot 355000}{1,0} = 471,9 \text{ kNm} \]

\[ q_k = 5,93 \cdot 1,2 \cdot 0,8 = 5,69 \text{ kN/m} \]

\[ q_d = 1,65 \cdot 5,69 = 9,39 \text{ kN/m} \]

\[ M_{Ed} = \frac{1}{8} \cdot q_d \cdot L^2 = \frac{1}{8} \cdot 9,39 \cdot 15,12^2 = 268,4 \text{ kNm} \]

\[ \frac{M_{Ed}}{M_{b,Rd}} = \frac{268,4}{471,9} = 0,57 \]

**Buckling**

\[ \lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{355}} = 76,41 \]

\[ \tilde{\lambda}_y = \frac{L_{cr,y}}{i_y \cdot \lambda_1} = \frac{15120}{189 \cdot 76,41} = 1,05 \]

\[ L_{cr,x} = 2000 \text{ mm (based on bracing system)} \]

\[ \tilde{\lambda}_z = \frac{L_{cr,z}}{i_z \cdot \lambda_1} = \frac{2000}{73 \cdot 76,41} = 0,36 \]

\[ \alpha_y = 0,21 \]

\[ \alpha_z = 0,34 \]

\[ \Phi_y = 0,5 [1 + \alpha_y (\tilde{\lambda}_y - 0,2) + \tilde{\lambda}_y^2] = 0,5 [1 + 0,21(1,05 - 0,2) + 1,05^2] = 1,14 \]

\[ \Phi_z = 0,5 [1 + \alpha_z (\tilde{\lambda}_z - 0,2) + \tilde{\lambda}_z^2] = 0,5 [1 + 0,34(0,36 - 0,2) + 0,36^2] = 0,59 \]

\[ \chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \tilde{\lambda}_y^2}} = \frac{1}{1,14 + \sqrt{1,14^2 - 1,05^2}} = 0,63 \leq 1,0 \]

\[ \chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \tilde{\lambda}_z^2}} = \frac{1}{0,59 + \sqrt{0,59^2 - 0,36^2}} = 0,94 \leq 1,0 \]

\[ N_{b,Rd} = \frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0,63 \cdot 1,78E - 2 \cdot 355000}{1,0} = 3998 \text{ kN} \]
\[ N_{Ed} = 85.9 \text{ kN} \]
\[ \frac{N_{Ed}}{N_{b,Rd}} = \frac{85.9}{3998} = 0.02 \leq 1.0 \]

**Combination of bending and normal force pressure**

\[ M_1 = 0 \text{ kNm} \]
\[ M_2 = 0 \text{ kNm} \]
\[ M_s = 268 \text{ kNm} \]
\[ C_{my}, C_{mz}, C_{mLT} = 0.95 \]
\[ k_{yy} = C_{my}(1 + (\bar{\lambda}_y - 0.2)) \frac{N_{Ed}}{X_y N_{Rk}} = 0.95(1 + (1.05 - 0.2)) \frac{85.9}{0.63 \frac{6319}{1,0}} = 0.966 \]
\[ k_{zy} = 1 - \frac{0.1\bar{\lambda}_z}{C_{mLT} - 0.25} \frac{N_{Ed}}{X_z N_{Rk}} = 1 - \frac{0.036}{0.95 - 0.25} \frac{85.9}{0.94 \frac{6319}{1,0}} = 0.999 \]
\[ \frac{N_{Ed}}{N_{Rk}} + k_{yy} \frac{M_{y,Ed}}{X_y M_{Y1}} = \frac{85.9}{0.63 \frac{3454}{1,0}} + 0.966 \frac{268}{0.46 \frac{1028}{1,0}} = 0.57 \leq 1.0 \]
\[ \frac{N_{Ed}}{N_{Rk}} + k_{zy} \frac{M_{y,Ed}}{X_z M_{Y1}} = \frac{85.9}{0.94 \frac{3454}{1,0}} + 0.999 \frac{268}{0.46 \frac{1028}{1,0}} = 0.58 \leq 1.0 \]

**Shear force**

\[ A_v = A = 2bt_f + (t_w + 2r)t_f = 17800 - 2 * 300 * 21 + (11.5 + 2 * 27)21 = 6576 \text{ mm}^2 \]
\[ V_{pl,Rd} = \frac{A_v f_y / \sqrt{3}}{\gamma_{Mo}} = \frac{6576(355/\sqrt{3})}{1,0} E - 3 = 1348 \text{ kN} \]
\[ V_{Ed} = \frac{1}{2} * qd * L = \frac{1}{2} * 9.39 * 15.12 = 71 \text{ kN} \]
\[ \frac{V_{Ed}}{V_{pl,Rd}} = \frac{71}{1348} = 0.05 \leq 1.0 \]

SLS

The maximum deflection check is done for the columns with length 15.12 m and 11.93 m.

\[ d_{max} = \frac{h}{300} = \frac{15120}{300} = 50 \text{ mm} \]
\[ d = \frac{5qL^4}{384EI} = \frac{5 * 5,7 * 10^3 * 15120^4}{384 * 210000 * 6,37 * 10^8} = 29 \text{ mm} < d_{max} \]
\[ d_{max} = \frac{h}{300} = \frac{11930}{300} = 40 \text{ mm} \]
\[ d = \frac{5qL^4}{384EI} = \frac{5 * 7,34 * 10^3 * 15120^4}{384 * 210000 * 6,37 * 10^8} = 14 \text{ mm} < d_{max} \]
Corner column

ULS

Profile HEA280

<table>
<thead>
<tr>
<th>h (profile)</th>
<th>270 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>280 mm</td>
</tr>
<tr>
<td>A</td>
<td>97.3 cm²</td>
</tr>
<tr>
<td>t_w</td>
<td>8 mm</td>
</tr>
<tr>
<td>t_f</td>
<td>13 mm</td>
</tr>
<tr>
<td>r</td>
<td>24 mm</td>
</tr>
<tr>
<td>E</td>
<td>210000 N/mm²</td>
</tr>
<tr>
<td>G</td>
<td>79300 N/mm²</td>
</tr>
<tr>
<td>h (column)</td>
<td>8000 mm</td>
</tr>
<tr>
<td>L_{cr,y} (0.5h)</td>
<td>4000 mm</td>
</tr>
<tr>
<td>I_y (hor. Axis)</td>
<td>13670 cm⁴</td>
</tr>
<tr>
<td>I_z (ver. axis)</td>
<td>4760 cm⁴</td>
</tr>
<tr>
<td>i_y</td>
<td>11.9 cm</td>
</tr>
<tr>
<td>i_z</td>
<td>7 cm</td>
</tr>
<tr>
<td>I_w</td>
<td>7.85E-07 m⁶</td>
</tr>
<tr>
<td>I_t</td>
<td>6.21E-07 m⁶</td>
</tr>
<tr>
<td>W_{y,pl}</td>
<td>1010 cm³</td>
</tr>
<tr>
<td>f_y</td>
<td>355 MPa</td>
</tr>
</tbody>
</table>

Profile classification

\[ \epsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81 \]

\[ c = h - 2r - 2t_f = 270 - 2 \times 24 - 2 \times 13 = 196 \text{mm} \]

\[ \frac{c}{t_w} = \frac{196}{8} = 24.5 \leq 72\epsilon = 58.6 \Rightarrow \text{Class 1} \]

Critical Moment and lateral torsional buckling reduction

The critical Moment has been calculated by using software LTBeamN. See the appendix for the parameters and complete results of this calculation.

\[ \mu_{cr} = 250.7 \]
\[ M_{cr} = 1537 \text{ kNm} \]

\[
\tilde{\lambda}_{LT} = \sqrt{\frac{M_E}{M_{cr}}} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{1.01E - 3 \times 355000}{1537}} = 0.48
\]

\[ h = \frac{270}{280} = 0.96 \leq 2 \Rightarrow \text{Buckling curve } b \]

\[ \alpha_{LT} = 0.34 \]

\[ \beta = 0.75 \]

\[ \tilde{\lambda}_{LT,0} = 0.4 \]

\[ \Phi_{LT} = 0.5[1 + \alpha_{LT}(\tilde{\lambda}_{LT} - \tilde{\lambda}_{LT,0}) + \beta \tilde{\lambda}_{LT}^2] = 0.5[1 + 0.34(0.48 - 0.4) + 0.75 \times 0.48^2] = 0.60 \]

\[ \chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta \tilde{\lambda}_{LT}^2}} = \frac{1}{0.60 + \sqrt{0.60^2 - 0.75 \times 0.48^2}} = 0.967 \]

**Calculation of Moment in z direction**

\[ q = 4.27 \frac{kN}{m} \]

\[ M_{Ed,z} = \frac{1}{8} \times q \times \left(\frac{L}{2}\right)^2 = \frac{1}{8} \times 4.27 \times 4^2 = 14.04 \text{ kNm} \]

**Moment resistant check**

\[ \gamma_{M1} = 1.0 \]

\[ M_{b,Rd} = \chi_{LT} \times \frac{W_x f_y}{\gamma_{M1}} = 0.967 \times \frac{3.4E - 4 \times 355000}{1.0} = 116.7 \text{ kNm} \]

\[ M_{Ed} = 14.04 \text{ kNm} \]

\[ \frac{M_{Ed}}{M_{b,Rd}} = \frac{14.04}{116.7} = 0.12 \]

**Buckling**

\[ \lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{355}} = 76.41 \]

\[ \tilde{\lambda}_y = \frac{L_{cr,y}}{i_y \lambda_1} = \frac{2000}{119 \times 76.41} = 0.22 \]

\[ L_{cr,x} = 4000 \text{ mm (based on bracing system)} \]

\[ \tilde{\lambda}_z = \frac{L_{cr,z}}{i_z \lambda_1} = \frac{4000}{73 \times 76.41} = 0.748 \]

\[ \alpha_y = 0.34 \]
\[ a_z = 0.49 \]

\[ \Phi_y = 0.5 \left[ 1 + a_y (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2 \right] = 0.5 \left[ 1 + 0.34 (0.22 - 0.2) + 0.22^2 \right] = 0.53 \]

\[ \Phi_z = 0.5 \left[ 1 + a_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2 \right] = 0.5 \left[ 1 + 0.49 (0.748 - 0.2) + 0.748^2 \right] = 0.91 \]

\[ \chi_y = \frac{1}{\Phi_y + \sqrt{\phi_y^2 - \lambda_y^2}} = \frac{1}{0.53 + \sqrt{0.53^2 - 0.22^2}} = 0.99 \leq 1.0 \]

\[ \chi_z = \frac{1}{\Phi_z + \sqrt{\phi_z^2 - \lambda_z^2}} = \frac{1}{0.91 + \sqrt{0.91^2 - 0.748^2}} = 0.70 \leq 1.0 \]

\[ N_{b,Ed} = \frac{X_z \cdot A \cdot f_y}{Y_{M1}} = \frac{0.70 \cdot 9,73E - 3 \cdot 355000}{1000} = 2400 \text{kN} \]

\[ N_{Ed} = 44.2 \text{kN} \]

\[ \frac{N_{Ed}}{N_{b,Ed}} = \frac{44.2}{2400} = 0.02 \leq 1.0 \]

**Combination of bending and compression**

\[ M_1 = 0 \text{kNm} \]

\[ M_2 = 0 \text{kNm} \]

\[ C_{my}, C_{mx}, C_{mLT} = 0.95 \]

\[ k_{yy} = C_{my} \left( 1 + (\bar{\lambda}_y - 0.2) \right) \frac{N_{Ed}}{\chi_y \cdot Y_{M1}} = 0.95 \left( 1 + (0.22 - 0.2) \right) \frac{44.2}{0.99 \cdot 3454 \cdot 1000} = 0.95 \]

\[ k_{xy} = 1 - \frac{0.0748}{C_{mLT} - 0.25} \frac{N_{Ed}}{\chi_y \cdot Y_{M1}} = 1 - \frac{0.0748}{0.95 - 0.25} \frac{44.2}{0.70 \cdot 3454 \cdot 1000} = 0.998 \]

\[ k_{xz} = C_{my} \left( 1 + (2 \bar{\lambda}_x - 0.6) \right) \frac{N_{Ed}}{\chi_z \cdot Y_{M1}} = 0.95 \left( 1 + (2 \cdot 0.22 - 0.6) \right) \frac{44.2}{0.70 \cdot 3454 \cdot 1000} = 0.966 \]

\[ k_{yz} = 0.6k_{zz} = 0.579 \]

\[ \frac{N_{Ed}}{N_{Rk}} \frac{M_{y,Ed}}{Y_{M1}} + k_{xy} \frac{M_{y,Rk}}{Y_{M1}} + k_{yz} \frac{M_{z,Ed}}{Y_{M1}} \leq 1.0 \]

\[ \frac{N_{Ed}}{N_{Rk}} \frac{M_{y,Ed}}{Y_{M1}} + k_{xz} \frac{M_{y,Rk}}{Y_{M1}} + k_{yz} \frac{M_{z,Ed}}{Y_{M1}} \leq 1.0 \]

**Shear force**

\[ A_v = A = 2bt_f + (t_w + 2r)t_f = 9780 - 2 \cdot 280 \cdot 13 + (8 + 2 \cdot 22)13 = 3178 \text{mm}^2 \]
\[ V_{pl,Rd} = \frac{A_v(y_f/\sqrt{3})}{\gamma_{Ma}} = \frac{3178 (355/\sqrt{3})}{1,0} E - 3 = 651 \text{ kN} \]

\[ V_{Ed} = 33.4 \text{ kN} \]

\[ \frac{V_{Ed}}{V_{pl,Rd}} = \frac{33.4}{651} = 0.05 \leq 1.0 \]

**SLS**

The allowable deflection is 27 mm.

In both horizontal direction, the maximal deflection is equal to 0.6 mm and therefore the SLS check is satisfied.
Rectangular hollow section (RHS)

Profile 200/100/5.0

<table>
<thead>
<tr>
<th>h (profile)</th>
<th>200 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>100 mm</td>
</tr>
<tr>
<td>A</td>
<td>28.9 cm²</td>
</tr>
<tr>
<td>E</td>
<td>210000 N/mm²</td>
</tr>
<tr>
<td>G</td>
<td>79300 N/mm²</td>
</tr>
<tr>
<td>L</td>
<td>5930 mm</td>
</tr>
<tr>
<td>Lcr,y</td>
<td>5930 mm</td>
</tr>
<tr>
<td>Iz (hor. Axis)</td>
<td>1509 cm⁴</td>
</tr>
<tr>
<td>Ix (ver. axis)</td>
<td>509 cm⁴</td>
</tr>
<tr>
<td>iy</td>
<td>189 mm</td>
</tr>
<tr>
<td>iz</td>
<td>73 mm</td>
</tr>
<tr>
<td>Wy,el</td>
<td>151 cm³</td>
</tr>
<tr>
<td>fy</td>
<td>355 MPA</td>
</tr>
</tbody>
</table>

Calculation of Moment in z direction

\[ q = 4.27 \frac{kN}{m} \]

\[ M_{Ed,z} = \frac{1}{8} * q * \left( \frac{L}{2} \right)^2 = \frac{1}{8} * 0.30 * 5.93^2 = 2.18 \, kNm \]

Moment resistant check

\[ \gamma_{M1} = 1.0 \]

\[ M_{c,Rd} = \frac{W_z * f_y}{\gamma_{M1}} = 0.967 * \frac{1.02 * 10^{-4} * 355000}{1.0} = 36.1 \, kNm \]

\[ \frac{M_{Ed}}{M_{c,Rd}} = \frac{2.18}{36.1} = 0.06 \]

Buckling

\[ \lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{355}} = 76.41 \]

\[ \bar{\lambda}_y = \frac{L_{cr,y}}{i_y * \lambda_1} = \frac{5930}{119 * 76.41} = 0.41 \]

\[ L_{cr,x} = 5930 \, mm \]

\[ \bar{\lambda}_z = \frac{L_{cr,z}}{i_z * \lambda_1} = \frac{5930}{73 * 76.41} = 1.06 \]

\[ \alpha_y = 0.49 \]

\[ \alpha_z = 0.49 \]
\[ \Phi_y = 0.5 \left[ 1 + \alpha_y (\overline{\lambda}_y - 0.2) + \overline{\lambda}^2_y \right] = 0.5 \left[ 1 + 0.49(0.41 - 0.2) + 0.41^2 \right] = 0.64 \]

\[ \Phi_z = 0.5 \left[ 1 + \alpha_z (\overline{\lambda}_z - 0.2) + \overline{\lambda}^2_z \right] = 0.5 \left[ 1 + 0.49(1.06 - 0.2) + 1.06^2 \right] = 1.28 \]

\[ X_y = \frac{1}{\Phi_y + \sqrt{\phi_y^2 - \lambda_y^2}} = \frac{1}{0.64 + \sqrt{0.64^2 - 0.41^2}} = 0.89 \leq 1.0 \]

\[ X_z = \frac{1}{\Phi_z + \sqrt{\phi_z^2 - \lambda_z^2}} = \frac{1}{1.28 + \sqrt{1.28^2 - 1.06^2}} = 0.50 \leq 1.0 \]

\[ N_{b,Rd} = \frac{X_z \cdot A \cdot f_y}{\gamma_{M1}} = 0.50 \cdot 2.89 \cdot 10^{-3} \cdot 355000 \cdot \frac{1}{1.0} = 517 \text{ kN} \]

\[ N_{Ed} = 31.1 \text{ kN} \]

\[ \frac{N_{Ed}}{N_{b,Rd}} = \frac{31.1}{517} = 0.06 \leq 1.0 \]

**Combination of bending and compression**

\[ M_1 = 0 \text{ kNm} \]

\[ M_2 = 0 \text{ kNm} \]

\[ C_{my}, C_{mz}, C_{mLT} = 0.95 \]

\[ k_{yy} = C_{my}(1 + (\overline{\lambda}_y - 0.2)) \cdot \frac{N_{Ed}}{X_y \cdot \gamma_{M1}} = 0.95(1 + (0.41 - 0.2)) \cdot \frac{31.1}{0.89 \cdot 1034} = 0.96 \]

\[ k_{yy} = 0.6k_{yy} = 0.58 \]

\[ k_{zz} = C_{my}(1 + (2\overline{\lambda}_z - 0.6)) \cdot \frac{N_{Ed}}{X_z \cdot \gamma_{M1}} = 0.95(1 + (2 \cdot 1.06 - 0.6)) \cdot \frac{31.1}{0.50 \cdot 1034} = 1.03 \]

\[ k_{yz} = 0.6k_{zz} = 0.62 \]

\[ \frac{N_{Ed}}{N_{RR}} + k_{yy} \frac{M_{y,Ed}}{M_{y,RR}} = k_{yz} \frac{M_{z,Ed}}{M_{z,RR}} = \frac{31.1}{0.89 \cdot 1034} + 0.21 \frac{49}{1.0} + 0.62 \frac{31}{1.0} = 0.46 \leq 1.0 \]

\[ \frac{N_{Ed}}{N_{RR}} + k_{xz} \frac{M_{y,Ed}}{M_{y,RR}} = k_{xz} \frac{M_{z,Ed}}{M_{z,RR}} = \frac{31.1}{0.50 \cdot 1034} + 0.58 \frac{268}{1.0} + 1.03 \frac{29}{1.0} = 0.34 \leq 1.0 \]
Details

Arch beam (HEA280) connection

End plate

\( t_p = 25 \text{ mm} \)

\( h = 270 \text{ mm} \)

\( b = 280 \text{ mm} \)

\( V_{z,Ed} = 61 \text{ kN} \)

\[
V_{pl,Rd} = \frac{b \cdot h \cdot f_{yd}}{\sqrt{3}} = \frac{280 \times 270 \times 355}{\sqrt{3}} = 15495 \text{ kN}
\]

Bolt resistance

Bolt grade 10.9

Bolt size M20

Shear resistance

\[
F_{v,Rd} = \frac{A_s \cdot F_{ub} \cdot \alpha_v}{\gamma_{M2}} = \frac{245 \times 1000 \times 0,5}{1,25} = 98 \text{ kN}
\]

Bearing resistance

\[
F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot t_p \cdot f_u \cdot d}{\gamma_{M2}} = \frac{2,5 \times 0,789 \times 25 \times 490 \times 20}{1,25} = 386 \text{ kN}
\]

Tension resistance

\[
F_{t,Rd} = \frac{A_s \cdot F_{ub} \cdot k_2}{\gamma_{M2}} = \frac{245 \times 1000 \times 0,9}{1,25} = 176 \text{ kN}
\]

Bolt spacing

\( e_1 = 52 \text{ mm} \)

\( p = 70 \text{ mm} \)

\( e_2 = 70 \text{ mm} \)

\( \alpha_{we} = 4 \text{ mm} \)

\( c_1 = c_2 = 0,8 \alpha_{we} \sqrt{2} = 4,5 \text{ mm} \)

\( m = 140 - 4 - 7 - 4,5 = 61,5 \text{ mm} \)

\( m_2 = 52 - 4,5 = 57,5 \text{ mm} \)

\( \lambda_1 = \frac{m}{m + e} = \frac{61,5}{61,5 + 70} = 0,47 \text{ mm} \)

\( \lambda_2 = \frac{m_2}{m + e} = \frac{57,5}{61,5 + 70} = 0,36 \text{ mm} \)

\( \alpha = 6,3 \)
<table>
<thead>
<tr>
<th>First (top) row under flange</th>
<th>ex</th>
<th>50 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>leff,cp = 2πm</td>
<td>283 mm</td>
<td>mx</td>
</tr>
<tr>
<td>leff,nc = am</td>
<td>140 mm</td>
<td>e</td>
</tr>
<tr>
<td>leff,1</td>
<td>140 mm</td>
<td>bp</td>
</tr>
<tr>
<td>leff,2</td>
<td>140 mm</td>
<td>w</td>
</tr>
</tbody>
</table>

Group

<table>
<thead>
<tr>
<th>First (top) row under flange</th>
<th>First (top) row under flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>leff,cp = 2πm</td>
<td>386 mm</td>
</tr>
<tr>
<td>leff,nc = am</td>
<td>387 mm</td>
</tr>
<tr>
<td>leff,1</td>
<td>386 mm</td>
</tr>
<tr>
<td>leff,2</td>
<td>387 mm</td>
</tr>
</tbody>
</table>

Middle row

<table>
<thead>
<tr>
<th>Middle row</th>
</tr>
</thead>
<tbody>
<tr>
<td>leff,cp = 2πm</td>
</tr>
<tr>
<td>leff,nc = 4m +1,25e</td>
</tr>
<tr>
<td>leff,1</td>
</tr>
<tr>
<td>leff,2</td>
</tr>
</tbody>
</table>

Bottom row

<table>
<thead>
<tr>
<th>Bottom row</th>
</tr>
</thead>
<tbody>
<tr>
<td>leff,cp = 2πm</td>
</tr>
<tr>
<td>leff,nc = 4m +1,25e</td>
</tr>
<tr>
<td>leff,1</td>
</tr>
<tr>
<td>leff,2</td>
</tr>
</tbody>
</table>

| | Σleff,1 | 527 mm |
| | Σleff,2 | 527 mm |

<table>
<thead>
<tr>
<th></th>
<th>Mpl,1,Rd</th>
<th>Mpl,2,Rd</th>
<th>FT,1,Rd</th>
<th>FT,2,Rd</th>
<th>FT,3,Rd</th>
<th>FT,Rd</th>
<th>h</th>
<th>h*FT,Rd</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Nmm</td>
<td>Nmm</td>
<td>kN</td>
<td>kN</td>
<td>kN</td>
<td>kN</td>
<td>mm</td>
<td>kNm</td>
</tr>
<tr>
<td>Row 1</td>
<td>14177281,2</td>
<td>14177281,2</td>
<td>922</td>
<td>412</td>
<td>353</td>
<td>353</td>
<td>198,5</td>
<td>70</td>
</tr>
<tr>
<td>Row 2</td>
<td>3882812,5</td>
<td>3882812,5</td>
<td>253</td>
<td>230</td>
<td>353</td>
<td>228</td>
<td>128,5</td>
<td>29</td>
</tr>
<tr>
<td>Row 3</td>
<td>11187993,2</td>
<td>11187993,2</td>
<td>728</td>
<td>359</td>
<td>353</td>
<td>353</td>
<td>58,5</td>
<td>21</td>
</tr>
</tbody>
</table>

Σ 120

$M_{Ed} = 84 \text{ kNm}$

$M_{Rd} = 120 \text{ kNm} > M_{Ed}$
Bracing connections

Roof bracing

*Plate welded to the strut*

\[ N_{Ed} = 162 \, kN \]
\[ \alpha = 49,9^\circ \]
\[ N_{Ed,\parallel} = 104 \, kN \]
\[ N_{Ed,\perp} = 124 \, kN \]

\[ F_{vw,\parallel} = \frac{f_u}{\sqrt{3} \beta_w \gamma_{M2}} = \frac{360}{\sqrt{3} \cdot 0.8 \cdot 1.25} = 208 \, MPa \]
\[ a_{we,\parallel} = 4 \, \text{mm} \]
\[ l_{we,\parallel} = 220 \, \text{mm} \]
\[ \tau_{\parallel} = \frac{N_{ed}}{2 a_{we} l_{we}} = \frac{104 \cdot 10^3}{2 \cdot 4 \cdot 220} = 59,3 \, MPa \]
\[ W_{we} = \frac{2}{6} \cdot 4 \cdot 220^2 = 64533 \, \text{mm}^2 \]
\[ a_{we,\perp} = 4 \, \text{mm} \]
\[ l_{we,\perp} = 90 \, \text{mm} \]
\[ \tau_{\parallel} = \frac{N_{ed}}{2 a_{we} l_{we}} = \frac{124 \cdot 10^3}{2 \cdot 4 \cdot 90} = 172,2 \, MPa \]
\[ \tau_{\perp} = \sigma_{\perp} = \frac{1}{\sqrt{2}} \frac{N_{ed} \cdot e}{W_{we}} = \frac{1}{\sqrt{2}} \frac{124 \cdot 10^3 \cdot 86}{64533} = 116,8 \, MPa \]
\[ \sqrt{\sigma_{\perp}^2 + 3 \tau_{\parallel}^2 + \tau_{\perp}^2} = \sqrt{116,8^2 + 3(116,8^2 + 59,3^2)} = 170,9 \, MPa \leq \frac{f_u}{\beta_w \gamma_{M2}} = \frac{360}{0.8 \cdot 1.25} = 360 \, MPa \]

*Plate welded to roof bracing diagonal*

\[ a_{we,\parallel} = 4 \, \text{mm} \]
\[ l_{we,\perp} = 60 \, \text{mm} \]
\[ \tau_{\parallel} = \frac{N_{ed}}{2 \cdot 2 a_{we} l_{we}} = \frac{162 \cdot 10^3}{2 \cdot 2 \cdot 4 \cdot 60} = 168,8 \, MPa \]
Bolt grade 8.8
Bolt size M20

Shear resistance
\[ F_{v,Rd} = \frac{A_s \cdot F_{ub} \cdot \alpha_v}{Y_{M2}} = \frac{245 \cdot 800 \cdot 0.6}{125} = 94 \text{kN} \]

Bearing resistance
\[ F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot t_p \cdot f_u \cdot d}{Y_{M2}} = \frac{2.5 \cdot 0.68 \cdot 10 \cdot 490 \cdot 20}{125} = 134 \text{kN} \]

Tension resistance
\[ F_{t,Rd} = \frac{A_s \cdot F_{ub} \cdot k_2}{Y_{M2}} = \frac{245 \cdot 800 \cdot 0.9}{125} = 141 \text{kN} \]

Bolt spacing
\[ e_1 = 52 \text{ mm} \]
\[ p = 70 \text{ mm} \]
\[ e_2 = 70 \text{ mm} \]

Wall bracing

Plate welded to the strut
\[ N_{Ed} = 98.5 \text{ kN} \]
\[ \alpha = 56.3^\circ \]
\[ N_{Ed,\parallel} = 54.6 \text{ kN} \]
\[ N_{Ed,\perp} = 82.0 \text{ kN} \]

\[ F_{vw,d} = \frac{f_u}{\sqrt{3} \beta_w Y_{M2}} = \frac{360}{\sqrt{3} \cdot 0.8 \cdot 1.25} = 208 \text{ MPa} \]
\[ a_{we,\parallel} = 4 \text{ mm} \]
\[ l_{we,\parallel} = 285 \text{ mm} \]
\[ A_{we} = 2 \cdot 4 \cdot 285 = 2280 \text{ mm}^2 \]
\[ \tau_{\parallel} = \frac{N_{Ed}}{2a_{we}l_{we}} = \frac{54.6 \cdot 10^3}{2 \cdot 4 \cdot 285} = 24.0 \text{ MPa} \]
\[ W_{we} = \frac{2}{6} \cdot 4 \cdot 285^2 = 108300 \text{ mm}^2 \]
\[ \tau_{\perp} = \sigma_{\perp} = \frac{1}{\sqrt{2}} \left( \frac{N_{Ed}}{A_{we}} + \frac{N_{Ed} \cdot e}{W_{we}} \right) = \frac{1}{\sqrt{2}} \left( \frac{82 \cdot 10^3}{2280} + \frac{82 \cdot 10^3 \cdot 86}{108300} \right) = 71.4 \text{ MPa} \]
\[ \sqrt{\sigma_1^2 + 3\tau_1^2 + \tau_{\parallel}^2} = \sqrt{71.4^2 + 3(71.4^2 + 24^2)} = 148.8 \text{ MPa} \leq \frac{f_u}{\beta_w \gamma_{M2}} = \frac{360}{0.8 \times 1.25} = 360 \text{ MPa} \]

**Plate welded to wall bracing diagonal**

\[ a_{we,\parallel} = 4 \text{ mm} \]
\[ l_{we,\perp} = 40 \text{ mm} \]
\[ \tau_{\parallel} = \frac{N_{Ed}}{2 \times 2a_{we} l_{we}} = \frac{98.5 \times 10^3}{2 \times 2 \times 4 \times 40} = 153.9 \text{ MPa} \]

**Bolt grade 8.8**

**Bolt size M16**

**Shear resistance**

\[ F_{v,Rd} = \frac{A_s \times F_{ub} \times \alpha_v}{\gamma_{M2}} = \frac{157 \times 800 \times 0.6}{1.25} = 60 \text{ kN} \]

**Bearing resistance**

\[ F_{b,Rd} = \frac{k_1 \times \alpha_b \times t_p \times f_u \times d}{\gamma_{M2}} = \frac{2.5 \times 0.74 \times 10 \times 490 \times 16}{1.25} = 190 \text{ kN} \]

**Tension resistance**

\[ F_{t,Rd} = \frac{A_s \times F_{ub} \times k_2}{\gamma_{M2}} = \frac{157 \times 800 \times 0.9}{1.25} = 90 \text{ kN} \]

**Bolt spacing**

\[ e_1 = 40 \text{ mm} \]
\[ p = 60 \text{ mm} \]
\[ e_2 = 52 \text{ mm} \]

**Strut to column**

\[ N_{Ed,ver} = 22.3 \text{ kN} \]
\[ N_{Ed,hor} = 124 \text{ kN} \]
\[ N_{Ed,res} = \sqrt{22.3^2 + 124^2} = 126 \text{ kN} \]
Weld around strut

\[ N_{Ed} = 101 \text{kN} \]
\[ a_{we} = 4 \text{ mm} \]
\[ l_{we} = 279 \text{ mm} \]

\[ \tau_\perp = \sigma_\perp = \frac{1}{\sqrt{2}} \frac{N_{Ed}}{A_{we}} = \frac{1}{\sqrt{2}} \frac{101 \times 10^3}{4 \times 279} = 63.9 \text{ MPa} \]

\[ \sqrt{\sigma_\perp^2 + 3\tau_\perp^2 + \tau_\parallel^2} = \sqrt{63.9^2 + 3(63.9^2 + 0^2)} = 127.9 \text{ MPa} \leq \frac{f_u}{\beta_w \gamma_{M2}} = \frac{360}{0.8 \times 1.25} = 360 \text{ MPa} \]

Shear plate stiffener

\[ M_{Ed} = 153 \text{kNm} \]
\[ h = 270 \text{ mm} \]

\[ F = \frac{M_{Ed}}{h} = \frac{153}{270 \times 10^{-3}} = 560 \text{ kN} \]
\[ \alpha = 64,6^\circ \]
\[ F_\parallel = \cos(64,6) \times F = 241 \text{ kN} \]
\[ l_{pl} = 280 \text{ mm} \]

\[ t_{p,\text{min}} \geq \frac{2F_\parallel}{\sigma_y l_{pl}} = \frac{482 \times 10^3}{355 \times 280} = 4,8 \text{ mm} \Rightarrow t_p = 10 \text{ mm} \]

\[ \lambda_1 = \pi \frac{E}{\sigma_y} = \pi \frac{210000}{355} = 76,41 \]

\[ \overline{\lambda_y} = \frac{L_{cr,y}}{i_y \lambda_1} = \frac{0,75 \times 280}{t_p \sqrt{12} \times 76,41} = 0,95 \]

\[ \alpha_y = 0,21 \]
\[ \Phi_y = 0,5[1 + \alpha_y(\overline{\lambda_y} - 0,2) + \overline{\lambda_y}^2] = 0,5[1 + 0,21(0,95 - 0,2) + 0,95^2] = 1,03 \]

\[ \chi_y = \frac{1}{\Phi_y + \frac{\phi_y^2 - \overline{\lambda_y}^2}{\overline{\lambda_y}}} = \frac{1}{1,03 + \sqrt{1,03^2 - 0,95^2}} = 0,70 \leq 1,0 \]

\[ N_{b,Rd} = \frac{\chi_y \times A \times f_y}{\gamma_{M1}} = \frac{0,70 \times 10 \times 280 \times 355 \times 10^{-3}}{1,0} = 695 \text{ kN} \]

\[ N_{Ed} = 482 \text{ kN} \]

\[ \frac{N_{Ed}}{N_{b,Rd}} = \frac{482}{695} = 0,69 \leq 1,0 \]
Tension bar connection

\( N_{Ed} = 243 \, kN \)
\( \alpha = 33,9^\circ \)
\( N_{Ed,\parallel} = 202 \, kN \)
\( N_{Ed,\perp} = 136 \, kN \)

\[ F_{vw,\parallel} = \frac{f_u}{\sqrt{3} \beta_w \gamma_{M2}} = \frac{360}{\sqrt{3} \times 0,8 \times 1,25} = 208 \, MPa \]
\( a_{we} = 5 \, mm \)
\( l_{we} = 369 \, mm \)
\( A_{we} = 2 \times 5 \times 369 = 3690 \, mm^2 \)
\( W_{we} = \frac{2}{6} \times 5 \times 369^2 = 226935 \, mm^2 \)

\[ \tau_{\parallel} = \frac{N_{Ed}}{2a_{we}l_{we}} = \frac{202 \times 10^3}{2 \times 5 \times 369} = 54,7 \, MPa \]

\[ \tau_{\perp} = \sigma_{\perp} = \frac{1}{\sqrt{2}} \left( \frac{N_{Ed}}{A_{we}} + \frac{N_{Ed} \times e}{W_{we}} \right) = \frac{1}{\sqrt{2}} \left( \frac{136 \times 10^3}{3690} + \frac{136 \times 10^3 \times 107}{226935} \right) = 71,1 \, MPa \]

\[ \sqrt{\sigma_{\perp}^2 + 3\tau_{\perp}^2 + \tau_{\parallel}^2} = \sqrt{71,1^2 + 3(71,1^2 + 54,7^2)} = 170,9 \, MPa \leq \frac{f_u}{\beta_w \gamma_{M2}} = \frac{360}{0,8 \times 1,25} = 360 \, MPa \]
Fixed column base
The foundation consists of a concrete block with dimensions 600x700x600mm.

\[ a_c = 2000 \text{ mm} \]
\[ b_c = 1200 \text{ mm} \]
\[ h_c = 700 \text{ mm} \]
\[ r_t = 300 \text{ mm} \]

\[ a_1 = \min(3 \times a_0, a_0 + h, a_c) = \min(3 \times 810, 810 + 700, 2000) = 1510 \text{ mm} \]
\[ b_1 = \min(3 \times b_0, b_0 + h, b_c) = \min(3 \times 440, 440 + 700, 1200) = 1140 \text{ mm} \]

\[ k_j = \sqrt{\frac{a_2 \times b_1}{a_0 \times b_0}} = \sqrt{\frac{1510 \times 1140}{810 \times 460}} = 2.20 \]
\[ f_{jd} = \frac{\beta_j \times k_j \times f_{ck}}{\gamma_c} = \frac{2}{3} \times \frac{2.20 \times 16}{15.6} = 15.6 \text{ MPa} \]
\[ c = t_p \sqrt{\frac{f_{jd}}{3 f_{jd}}} = 30 \sqrt{\frac{355}{3 \times 15.6}} = 67.2 \text{ mm} \]
\[ b_{eff} = 2 \times 80 + 2 \times c = 2 \times 80 + 2 \times 67.2 = 294 \text{ mm} \]

<table>
<thead>
<tr>
<th></th>
<th>ULS1</th>
<th>ULS2</th>
<th>ULS3</th>
<th>ULS10</th>
<th>ULS11</th>
<th>ULS14</th>
<th>ULS15</th>
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<td>142</td>
<td>106</td>
<td>108</td>
<td>73</td>
<td>65</td>
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Where
\[ e = \frac{M_{Ed}}{N_{Ed}} \]

and \( x \) is found by combining the two equations below resulting in an quadratic equation with results \( x_1 \) and \( x_2 \).

\[ N_{Ed}(e + \tau) = N_c(\tau + \frac{a}{2} - \frac{x}{2}) \]

and \( N_c = b_{eff} \times x \times f_{jd} \)

resulting in \( T = N_c - N_{Ed} \)
2x U200

\( A = 2 \times 3220 = 6440 \, \text{mm}^2 \)

\( I_y = 2 \times 19,1 \times 10^6 = 38,2 \times 10^6 \, \text{mm}^4 \)

\( A_{vz} = 2 \times 1770 = 3540 \, \text{mm}^2 \)

**Combined profiles**

\( A = 30 \times 440 + 6440 = 16420 \, \text{mm}^2 \)

\[
 z_T = \frac{\sum A_i z_i}{A} = \frac{30 \times 440 \times 15 + 6440 \left( \frac{200}{2} + 30 \right)}{16420} = 63,0 \, \text{mm} 
\]

\[
 I_y = \frac{30^3 \times 440}{12} + 30 \times 440 \left( 3 \times 15 \right)^2 + 2 \times 19,1 \times 10^6 + 6440 \left( \frac{200}{2} + 30 - 63 \right)^2 
\]

\( = 98,53 \times 10^6 \, \text{mm}^4 \)

\( M_p = N_c \left( 0,41 - \frac{x}{2} \right) = 271 \left( 0,41 - \frac{0,059}{2} \right) = 93,5 \, \text{kNm} \)

\( V_p = N_c = 271 \, \text{kN} \)

\( M_l = T \times 0,30 = 315 \times 0,30 = 94,5 \, \text{kNm} \)

\( V_l = N_c = 315 \, \text{kN} \)

\( W_{y,h} = \frac{98,53 \times 10^6}{200 + 30 - 63} = 590161 \, \text{mm}^3 \)

\( W_{y,d} = \frac{98,53 \times 10^6}{63} = 1562854 \, \text{mm}^3 \)

\( \sigma_{h,max} = \frac{M_{\text{max}}}{W_{y,h}} = \frac{94,5 \times 10^6}{590161} = 160,1 \, \text{MPa} < f_{yd} = 235 \, \text{MPa} \)

\( \tau_{\text{max}} = \frac{V_{\text{max}}}{A_{vz}} = \frac{315 \times 10^3}{3540} = 89,0 \, \text{MPa} < \frac{235}{\sqrt{3}} = 135,7 \, \text{MPa} \)

\( \sigma_z = \frac{M_{\text{max}}}{I_y} z_z = \frac{94,5 \times 10^6}{590161} \left( 230 - 63 - 24,5 \right) = 135,2 \, \text{MPa} \)

\( \sqrt{\sigma_z^2 + 3\tau^2} = \sqrt{135,2^2 + 3 \times 135,7^2} = 205,0 \, \text{MPa} < 235 \, \text{MPa} \)
Welds

Combination ULS21

\[ N_{Ed} = 206 \, kN \]
\[ M_{Ed} = 121 \, kNm \]
\[ V_{Ed} = 33,1 \, kN \]
\[ V_p = 271 \, kN \]
\[ a_{we} = 5 \, mm \]
\[ l_{we} = 810 \, mm \]
\[ A_{we} = 4 \times 5 \times 810 = 16200 \, mm^2 \]
\[ I_{we} = \frac{4}{12} \times 5 \times 810^3 = 8857 \times 10^5 \, mm^4 \]

Static moment \( S_{f,y} = 440 \times 30 \times (63 - 15) = 634195 \, mm^3 \)

Section 1

\[ \tau_\parallel = \frac{V_{Ed}}{A_{we}} + \frac{V_p S_{f,d}}{I_y a_{we}} = \frac{33,1 \times 10^3 \times 634195}{16200} + \frac{271 \times 10^3 \times 634195}{98,53 \times 10^6 \times 4 \times 5} = 2 + 87,1 = 89,1 \, MPa \]
\[ \sigma_{we} = \frac{N_{Ed}}{A_{we}} + \frac{M_{Ed} x_i}{l_{we}} = \frac{206 \times 10^3}{16200} + \frac{121 \times 10^6}{8857 \times 10^5} (405 - 59) = 12,7 + 47,3 = 60,0 \, MPa \]
\[ \tau_\perp = \frac{60,0}{\sqrt{2}} = 42,4 \, MPa \]
\[ \sqrt{\sigma_\perp^2 + 3\tau_\perp^2 + \tau_\parallel^2} = \sqrt{42,4^2 + 3(42,4^2 + 89,1^2)} = 176,2 \, MPa \leq \frac{f_u}{\beta_w \gamma_{M2}} = \frac{360}{0,8 \times 1,25} = 360 \, MPa \]

Section 2

\[ \tau_\parallel = \frac{V_{Ed}}{A_{we}} + 0 = 2,0 \, MPa \]
\[ \sigma_{we} = \frac{N_{Ed}}{A_{we}} + \frac{M_{Ed} x_i}{l_{we}} = \frac{206 \times 10^3}{16200} + \frac{121 \times 10^6}{8857 \times 10^5} (400) = 12,7 + 54,6 = 67,4 \, MPa \]
\[ \tau_\perp = \frac{67,4}{\sqrt{2}} = 47,6 \, MPa \]
\[ \sqrt{\sigma_\perp^2 + 3\tau_\perp^2 + \tau_\parallel^2} = \sqrt{47,6^2 + 3(47,6^2 + 2,0^2)} = 95,3 \, MPa \leq \frac{f_u}{\beta_w \gamma_{M2}} = \frac{360}{0,8 \times 1,25} = 360 \, MPa \]
Anchor bolts

\[ T_{\text{max}} = 315 \, kN \]
\[ T_1 = \frac{T_{\text{max}}}{2} = \frac{315}{2} = 157.5 \, kN \]
\[ N_{t,Ed,\text{max}} = \frac{157.5(200 + 480)}{580} = 184.6 \, kN \]
\[ N_{t,Ed,\text{min}} = 315 - 184.6 = 130.4 \, kN \]
\[ F_{L,Rd} = 0.85 \frac{0.9 \, A_s \, f_{u}}{Y_{M2}} = 0.85 \frac{0.9 \times 865 \times 360}{1,25} = 190.9 \, kN > N_{t,Ed,\text{max}} = 184.6 \, kN \]
\[ F_{L,Rd} = \frac{A \, f_y}{Y_{M0}} = \frac{\pi \times 40^2}{4 \times 1,25} = 295.3 \, kN > N_{t,Ed,\text{max}} = 184.6 \, kN \]

Crossbeams 2xU120

Internal forces

\[ M_{a,d} = 130.4 \times 0.2 = 26.1 \, kNm \]
\[ V_{a,d} = 130.4 \, kN \]
\[ M_{b,d} = 184.6 \times 0.1 = 18.5 \, kNm \]
\[ V_{b,d} = 184.6 \, kN \]

\[ A_{vz} = 1428 \, mm^2 \]
\[ W_{pl,y} = 121400 \, mm^3 \]
\[ V_{pl,Rd} = \frac{A_{vz} \times f_y}{\sqrt{3} \times Y_{M0}} = \frac{1428 \times 235}{\sqrt{3} \times 1.0} = 193.7 \, kN > V_{Ed} = 184.6 \, kN \]
\[ 0.5 \times V_{pl,Rd} = 96.9 \, kN < V_{Ed} \Rightarrow \text{check combination } M + V \]
\[ M_{pl,Rd} = W_{pl,y} \times f_{yd} = 121400 \times 235 = 28.5 \, kNm \]
\[ M_{pl,Rd} > M_{Ed} = 26.1 \, kNm \]
\[ \rho = \left( \frac{2 \times V_{d}}{V_{pl,Rd} - 1} \right)^2 \]
\[ \rho_a = \left( \frac{2 \times 130.4}{193.7 - 1} \right)^2 = 0.119 \]
\[ M_{V,Rd} = \left( W_{pl} - \rho \frac{A_s^2}{4 \times t_{w}} \right) f_{yd} = \left( 121400 - \frac{0.119 \times 1428^2}{4 \times 7} \right) 235 = 27.5 \, kNm > M_{a,d} \]
\[ \rho_b = \left( \frac{2 \times 184.6}{193.7} - 1 \right)^2 = 0.82 \]

\[ M_{V,Rd} = \left( W_{pl} - \rho \frac{A_o^2}{4t_w} \right) f_{yd} = \left( 121400 - \frac{0.82 \times 1428^2}{4 \times 7} \right) 235 = 21.5 \text{ kNm} > M_{b,d} \]

**HEB 100 shear stopper**

\[ F_{y,Ed} = 49 \text{ kN (ULS10)} \]

\[ V_{Ed} = \mu N_c = 0.2 \times 3 = 0.6 \text{ kN} < 49 \text{ kN} \]

\[ A_{vz} = 904 \text{ mm}^2 \]

\[ W_{pl,y} = 104200 \text{ mm}^3 \]

\[ h > \frac{F_{v,Ed}}{b f_{ck} \gamma_c} = \frac{49 \times 10^3}{100 \times 16} = 45.9 \Rightarrow h = 50 \text{ mm} \]

\[ V_{Rd} = \frac{A_{vz} \times f_y}{\sqrt{3} \gamma_M} = \frac{904 \times 235}{\sqrt{3} \times 1.0} = 122.7 \text{ kN} > V_{Ed} = 49 \text{ kN} \]

\[ 0.5V_{Rd} = 61.4 \text{ kN} > V_{Ed} = 49 \text{ kN (little skid)} \]

\[ M_{pl,Rd} = W_{pl,y} \times f_{yd} = 104200 \times 235 = 24.5 \text{ kNm} \]

\[ M_{pl,Rd} > M_{Ed} = F_{v,Ed} \times e = 49 \times \frac{50}{2} = 3.7 \text{ kNm} \]
The foundation consists of a concrete block with dimensions 600x700x600mm.

\[
a_1 = \min(3 \times a_0, a_0 + h, a_c) = \min(3 \times 340, 340 + 600, 600) = 600 \text{ mm}
\]

\[
b_1 = \min(3 \times b_0, b_0 + h, b_c) = \min(3 \times 460, 460 + 600, 700) = 700 \text{ mm}
\]

\[
k_j = \frac{a_1 \times b_1}{a_0 \times b_0} = \sqrt{\frac{600 \times 700}{340 \times 440}} = 1.41
\]

\[
f_{jd} = \frac{f_{ck}}{f_{cd}} = \frac{2}{3} \times 1.41 \times 16 = 10.0 \text{ MPa}
\]

\[
c = t_p \sqrt{\frac{f_{yd}}{3f_{jd}}} = 20 \sqrt{\frac{355}{3 \times 10.0}} = 68.7 \text{ mm}
\]

The effective area is marked in grey in the scheme on the right.

\[
A_{eff} = 113399 \text{ mm}^2
\]

\[
N_{Ed} = A_{eff} \times f_{jd} = 113399 \times 10,0 \times 10^{-3} = 1137 \text{ kN} > N_{Ed} = 85,9 \text{ kN}
\]

**Shear resistance**

\[
N_{Ed} = 1,6 \text{ kN}
\]

\[
V_{Ed} = 72,3 \text{ kN}
\]

\[
F_{v,Ed} = V_{Ed} - C_{f,d}N_{Ed} = 72,3 - 0,2 \times 1,6 = 72,0 \text{ kN}
\]

**HEB 140 shear stopper**

\[
A_{vz} = 1308 \text{ mm}^2
\]

\[
W_{pl,y} = 245400 \text{ mm}^3
\]
\[ h > \frac{F_{v,Ed}}{b f_{ck}} = \frac{72 \times 10^3}{140 \times \frac{16}{1.5}} = 48.2 \Rightarrow h = 50 \text{ mm} \]

\[ V_{Rd} = \frac{A_{wz} \cdot f_v}{\sqrt{3} \cdot y_{M0}} = \frac{1308 \times 355}{\sqrt{3} \times 1.0} = 268.1 \text{ kN} > V_{Ed} = 72.3 \text{ kN} \]

\[ 0.5 V_{Rd} = 134 \text{ kN} > V_{Ed} = 72.3 \text{ kN} \text{ (little skid)} \]

**Bending**

\[ M_{pt,Rd} = W_{p,ly} \cdot f_{yd} = 245400 \times 355 = 87.1 \text{ kNm} \]

\[ M_{pt,Rd} > M_{Ed} = F_{v,Ed} \cdot e = 72.0 \times \left(40 + \frac{50}{2}\right)/1000 = 4.7 \text{ kNm} \]

**Welds**

\[ a_w = 5 \text{ mm} \]

\[ l_w = 92 \text{ mm} \]

\[ l_w = \frac{4}{12} \times 5 \times 92^3 = 1.30 \times 10^6 \text{ mm}^4 \]

**Stresses in the web**

\[ \tau_\parallel = \frac{F_{v,Ed}}{2 \cdot a_w \cdot l_w} = \frac{72 \times 10^3}{2 \times 5 \times 92} = 78.2 \text{ MPa} \]

\[ \tau_\perp = \sigma_\perp = \frac{1}{\sqrt{2}} \frac{F_{v,Ed} \cdot e}{l_w / z_1} = \frac{1}{\sqrt{2}} \frac{1}{1.30 \times 10^6} \times 65 = 117.3 \text{ MPa} \leq \frac{0.9 f_u}{\gamma_{M2}} = \frac{0.9 \times 490}{1.25} = 352.8 \text{ MPa} \]

Von Mises stress:

\[ \sqrt{\sigma_\perp^2 + 3 \tau_\perp^2 + \tau_\parallel^2} = \sqrt{117.3^2 + 3(117.3^2 + 78.2^2)} = 270.8 \text{ MPa} \leq \frac{f_u}{\beta_w \gamma_{M2}} = \frac{490}{0.9 \times 1.25} = 425.6 \text{ MPa} \]

**Stresses in the flange**

\[ \tau_\parallel = 0 \text{ MPa} \]

\[ \tau_\perp = \sigma_\perp = \frac{1}{\sqrt{2}} \frac{F_{v,Ed} \cdot e}{l_w / z_1} = \frac{1}{\sqrt{2}} \frac{1}{1.30 \times 10^6} \times 65 = 191.2 \text{ MPa} \leq \frac{0.9 f_u}{\gamma_{M2}} = \frac{0.9 \times 490}{1.25} = 352.8 \text{ MPa} \]

Von Mises stress:

\[ \sqrt{\sigma_\perp^2 + 3 \tau_\perp^2 + \tau_\parallel^2} = \sqrt{117.3^2 + 3(117.3^2 + 0^2)} = 382.4 \text{ MPa} \leq \frac{f_u}{\beta_w \gamma_{M2}} = \frac{490}{0.9 \times 1.25} = 425.6 \text{ MPa} \]
Bolts

Bolts class 8.8

Bolt size M16

\[ N_{Ed} = 20,2 \, kN \, (tension) \]

\[ t_p = 20 \, mm \]

\[ k_2 = 0,9 \]

\[ F_{ub} = 800 \, MPa \]

\[ A_s = 157 \, mm^2 \]

\[ \gamma_{M2} = 1,25 \]

Tension resistance

\[ F_{L,Rd} = \frac{A_s \times F_{ub} \times k_2}{\gamma_{M2}} = \frac{157 \times 800 \times 0,9}{1,25} = 90,0 \, kN \]

Hence one bolt provides sufficient resistance, however two will be used for symmetry.
Appendix

1 References

O-METALL - supplier of steel plates and panels.
https://www.o-metall.com/nl/trapezbleche/profieplaten_en_steeldeck/hoogprofiel_vanaf_50_mm/852804.htm

KOVOVÉ PROFILY, SPOL. SRO
http://kovprof.cz/profil-spolecnosti/

Kingspan – insulated panels
https://www.kingspan.com/nl/nl/producten/geisoleerde-dak-en-gevelsystemen/geisoleerde-panelen/downloads#!hp=14818&rs_np_1=&rs_rt_2=t-1,t-1,s-4&p=1&ps=16

Macalloy – tension bars

2 Panels and profiles

Profiles – roof

![Profiles diagram](image-url)
Profiles – wall panels

### Outer Sheet 0.63mm (Steel), Inner Sheet 0.63mm (Steel)

#### Single Span Condition

<table>
<thead>
<tr>
<th>Panel Thickness (mm)</th>
<th>Load Types</th>
<th>Uniformly distributed loads kN/m² Span L in metre</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td>80</td>
<td>Pressure</td>
<td>3.12</td>
</tr>
<tr>
<td></td>
<td>Suction</td>
<td>2.75</td>
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<td>100</td>
<td>Pressure</td>
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<td></td>
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<td>3.55</td>
</tr>
<tr>
<td>120</td>
<td>Pressure</td>
<td>3.75</td>
</tr>
<tr>
<td></td>
<td>Suction</td>
<td>3.75</td>
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<td>Suction</td>
<td>3.75</td>
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</table>

#### Double Span Condition

<table>
<thead>
<tr>
<th>Panel Thickness (mm)</th>
<th>Load Types</th>
<th>Uniformly distributed loads kN/m² Span L in metre</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>Pressure</td>
<td>3.57</td>
</tr>
<tr>
<td></td>
<td>Suction</td>
<td>3.46</td>
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<tr>
<td>100</td>
<td>Pressure</td>
<td>3.57</td>
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<tr>
<td></td>
<td>Suction</td>
<td>3.57</td>
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<tr>
<td>120</td>
<td>Pressure</td>
<td>3.75</td>
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<tr>
<td></td>
<td>Suction</td>
<td>3.75</td>
</tr>
<tr>
<td>140</td>
<td>Pressure</td>
<td>3.75</td>
</tr>
<tr>
<td></td>
<td>Suction</td>
<td>3.75</td>
</tr>
</tbody>
</table>

Notes:
1. Values have been calculated using the method described in BS EN 14509: 2013, for dark coloured panels.
2. Deflection limit for pressure loading is L/100 and suction loading is L/100.
3. The actual wind suction load resisted by the panel is dependent on the number of fasteners used and the support thickness as well as the fastener material.
4. The fastener calculation should be carried out in accordance with the appropriate standards. For further advice please contact Kingspan envirocare Technical Services.
5. The allowable steelwork tolerance between bearing planes of adjacent supports is +/- 5mm.
6. Load span tables for the panel specifications not shown are available from Kingspan envirocare Technical Services.

### Dimensions, Weight & Thermal Performance

<table>
<thead>
<tr>
<th>Core Thickness (mm)</th>
<th>80</th>
<th>100</th>
<th>120</th>
<th>140</th>
<th>150</th>
</tr>
</thead>
<tbody>
<tr>
<td>U-value (W/m²K)*</td>
<td>0.26</td>
<td>0.21</td>
<td>0.18</td>
<td>0.15</td>
<td>0.14</td>
</tr>
<tr>
<td>Weight kg/m²</td>
<td>14.2</td>
<td>15.0</td>
<td>15.8</td>
<td>16.6</td>
<td>17.0</td>
</tr>
<tr>
<td>KS1000 CW U-value</td>
<td>0.23</td>
<td>0.19</td>
<td>0.16</td>
<td>0.14</td>
<td>-</td>
</tr>
<tr>
<td>KS1000 CW Weight kg/m²</td>
<td>15.1</td>
<td>15.9</td>
<td>16.7</td>
<td>17.5</td>
<td>-</td>
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<tr>
<td>KS1000 LV U-value</td>
<td>0.25</td>
<td>0.20</td>
<td>0.17</td>
<td>0.15</td>
<td>-</td>
</tr>
<tr>
<td>KS1000 LV Weight kg/m²</td>
<td>15.7</td>
<td>16.5</td>
<td>17.3</td>
<td>18.1</td>
<td>-</td>
</tr>
</tbody>
</table>

The Longspan range has a Thermal Transmittance (U value), calculated using the method required by the Building Regulations Part L2 (England & Wales) and Building Standards Section 6 (Scotland).

* Excluding CurveWall and Louvre. These profiles are listed separately above.

Profiles – C-profile wall
### C 300-S

**Únosnost dle ČSN EN 1993-1-3:**

<table>
<thead>
<tr>
<th>Řádek č. 1</th>
<th>Únosnost bez zvětšky osy sily (rádiová hodnota)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Řádek č. 2</td>
<td>Únosnost s vlivem osy sily 15 kN (nároková hodnota, osova síla v tlaku nebo tahu)</td>
</tr>
<tr>
<td>Řádek č. 3</td>
<td>Únosnost pro sání bez zvětšky osy sily (nároková hodnota)</td>
</tr>
<tr>
<td>Řádek č. 4</td>
<td>Únosnost pro sání s vlivem osy sily 15 kN (nároková hodnota, osova síla v tlaku nebo tahu)</td>
</tr>
<tr>
<td>Řádek č. 5</td>
<td>Maximální zatížení pro deformaci L300 (charakteristická hodnota, únosnost dle MSU není zohledněna)</td>
</tr>
</tbody>
</table>

### PROSTY NOSNIK

#### Přípustné rovnoměrné zatížení [kNm] pro pole rozpětí L [m]

<table>
<thead>
<tr>
<th>Profil</th>
<th>G [kg/m]</th>
<th>3,37</th>
<th>4,88</th>
<th>6,40</th>
<th>7,92</th>
<th>9,43</th>
<th>0,92</th>
<th>11,35</th>
<th>12,86</th>
<th>14,38</th>
<th>15,90</th>
<th>17,42</th>
</tr>
</thead>
<tbody>
<tr>
<td>C 300/2,0</td>
<td>G =7,14 kg/m</td>
<td>10,91</td>
<td>9,59</td>
<td>8,30</td>
<td>7,02</td>
<td>5,83</td>
<td>4,64</td>
<td>3,68</td>
<td>2,90</td>
<td>2,22</td>
<td>1,54</td>
<td>0,86</td>
</tr>
<tr>
<td>C 300/2,5</td>
<td>G =8,33 kg/m</td>
<td>3,49</td>
<td>3,06</td>
<td>2,67</td>
<td>2,29</td>
<td>1,92</td>
<td>1,54</td>
<td>1,17</td>
<td>0,80</td>
<td>0,43</td>
<td>0,09</td>
<td>-0,03</td>
</tr>
<tr>
<td>C 300/3,0</td>
<td>G =10,72 kg/m</td>
<td>3,49</td>
<td>3,06</td>
<td>2,67</td>
<td>2,29</td>
<td>1,92</td>
<td>1,54</td>
<td>1,17</td>
<td>0,80</td>
<td>0,43</td>
<td>0,09</td>
<td>-0,03</td>
</tr>
</tbody>
</table>

#### HOLLOW RECTANGULAR SECTION

**Dimensions**

- **b, h, d, t:** Dimensions of the section
- **Weight:** Weight of the section
- **Area:** Cross-sectional area
- **Ixx, Ixy, Iyy:** Inertia moments
- **J:** Polar moment of inertia
- **Surface Area:** Surface area of the section

### Diagram

- **X-Y coordinate system**
- **Rectangular section**
- **Dimensions and properties**

---

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3 Load cases screenshots

Load case 1: Own weight
Calculated by Scia based on choice of steel profile.

Load case 2: Dead load
Dead load is equal to 0,4 kN/m². The span is 6 m and hence one main beam will take 6 * 0,4 = 2,4 kN/m

Load case 3: Wind wall 0
The main frame will have a span of 6 meters. In the below picture the area of the roof which is taken by one beam is indicated by red and green hatch. Due to the different wind pressure coefficients on the different areas of the roof multiple load cases have to be considered. However with the wind coming from this direction the coefficients of D and E are constant along their respective walls.
In the table below the wind pressure coefficients have been multiplied by $c_p(16)$ and by the span, hence giving the wind action on the walls per area.

<table>
<thead>
<tr>
<th>Zone</th>
<th>Wind action [kN/m]</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>-8,64</td>
<td>-5,76</td>
<td>-3,6</td>
<td>5,76</td>
<td>-3,6</td>
</tr>
</tbody>
</table>

Load case 4: Wind wall 90
Load case 4: Wind wall 90.1
The blue rectangle covers partially zone A (3,4m) and B (2,6m). Hence the action on the wall for the blue area is: \((3,4 \times -8,64 + 2,6 \times -5,76)/6 = -7,39\) kN/m.

Load case 4: Wind wall 90.2
Note that load cases Green and Yellow are identical for wind action on the wall (-5,76 kN/m).
Load case 5 Wind roof 0 degrees
The main frame will have a span of 6 meters. In the below picture the area of the roof which is taken by one beam is indicated by red and green hatch. Due to the different wind pressure coefficients on the different areas of the roof multiple load cases have to be considered.

In above table the wind pressure coefficients are multiplied by the peak velocity pressure $q_v(z)$ and the span (6m). In the black rectangle coefficients F and G are both partially present. Taking into account the area of each coefficient the average coefficient is calculated:

$$(0.25*F + 0.35*G) / 0.6 = -6.96$$

Furthermore I has to be considered both positive and negative and hence for the wind coming from a direction with 0 degrees four different (sub)load cased are considered.
Load case 5.1A: Wind roof 0.1A
Black rectangle with I positive (pressure)

Load case 5.1B: Wind roof 0.1B
Black rectangle with I negative (suction)

Load case 5.2A: Wind roof 0.2A
Red rectangle with I positive (pressure)

Load case 5.2B: Wind roof 0.2B
Red rectangle with I negative (suction)
Load case 6: Wind roof 90 degrees

<table>
<thead>
<tr>
<th>Zone</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind action [kN/m]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Zone A</td>
<td>-12.96</td>
<td>-8.64</td>
<td>-5.04</td>
<td>1.44</td>
<td>-8.64</td>
<td>-9.36</td>
<td>-9.72</td>
<td>-5.04</td>
</tr>
</tbody>
</table>

Load case 6.1 Wind roof 90.1
Blue rectangle
Load case 6.2A Wind roof 90.2A
Green rectangle with D positive (pressure)

Load case 6.2B Wind roof 90.2B
Green rectangle with D negative (suction)

Load case 6.3A Wind roof 90.3A
Yellow rectangle with D positive (pressure)

Load case 6.3B Wind roof 90.3B
Yellow rectangle with D negative (suction)
Load case 7: Internal wind

As described before there are 2 cases of internal pressure: pressure and suction. Logically they have been divided into two sub cases.

Load case 7.1: Internal wind pressure

The internal wind pressure will be acting on both roof and walls and is equal to:

\[ W_{i,1} = q_p \cdot c_{pi} = 1.2 \cdot 0.2 = 0.24 \text{ kN/m}^2 \]

Multiplying by the span of 6 meters gives:

\[ Q_1 = 6 \cdot 0.24 = 1.44 \text{ kN/m} \]

Load case 7.2: Internal wind suction

\[ W_{i,1} = q_p \cdot (16) \cdot c_{pi} = 1.2 \cdot (-0.3) = -0.36 \text{ kN/m}^2 \]

Multiplying by the span of 6 meters gives:

\[ Q_2 = 6 \cdot -0.36 = -2.16 \text{ kN/m} \]
Load case 8: Snow
Load case 8.1 Snow 1
Model 1 (i)
Uniform distributed load of $s_k\cdot c_t\cdot c_e\cdot \mu = 0,7 \cdot 1,0 \cdot 1,0 \cdot 0,8 = 0,56 \text{ kN/m}^2$
Taking into account the span of 6 meters gives a line load of $6\cdot0,56 = 3,36 \text{ kN/m}$

Load case 8.2 Snow 2
Model 1 (ii)
Like load case 1 all $\mu$ factors will be multiplied by $s_k$ and the span to obtain the line load.
$0,7 \cdot 1,2 \cdot 6 = 5,04 \text{ kN/m}$

Load case 8.3A Snow 3A
Model 2 (ii) A
$0,7 \cdot 0,6 \cdot 6 = 2,52 \text{ kN/m}$
<table>
<thead>
<tr>
<th>Name</th>
<th>Load cases</th>
<th>Coeff. [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLS1</td>
<td>LC1 - Self weight, LC2 - Dead load</td>
<td>1</td>
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<tr>
<td>SLS2</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.1A - Wind roof 0,1A, LC7.1 - Wind Internal pressure</td>
<td>1 1 1 1 1</td>
</tr>
<tr>
<td>SLS3</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.1A - Wind roof 0,1A, LC7.2 - Wind Internal suction</td>
<td>1 1 1 1 1</td>
</tr>
<tr>
<td>SLS4</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.1B - Wind roof 0,1B, LC7.1 - Wind Internal pressure</td>
<td>1 1 1 1 1</td>
</tr>
<tr>
<td>SLS5</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.1B - Wind roof 0,1B, LC7.2 - Wind Internal suction</td>
<td>1 1 1 1 1</td>
</tr>
<tr>
<td>SLS6</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.2A - Wind roof 0,2A, LC7.1 - Wind Internal pressure</td>
<td>1 1 1 1 1</td>
</tr>
<tr>
<td>SLS7</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.2A - Wind roof 0,2A, LC7.2 - Wind Internal suction</td>
<td>1 1 1 1 1</td>
</tr>
<tr>
<td>SLS8</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.2B - Wind roof 0,2B, LC7.1 - Wind Internal pressure</td>
<td>1 1 1 1 1</td>
</tr>
<tr>
<td>SLS9</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.2B - Wind roof 0,2B, LC7.2 - Wind Internal suction</td>
<td>1 1 1 1 1</td>
</tr>
<tr>
<td>SLS10</td>
<td>LC1 - Self weight, LC2 - Dead load, LC4.1 - Wind wall 90.1, LC6.1 - Wind roof 90.1, LC7.1 - Wind Internal pressure</td>
<td>1 1 1 1 1</td>
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<td>SLS11</td>
<td>LC1 - Self weight, LC2 - Dead load, LC4.1 - Wind wall 90.1, LC6.1 - Wind roof 90.1, LC7.2 - Wind Internal suction</td>
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<td>LC1 - Self weight, LC2 - Dead load, LC4.2 - Wind wall 90.2, LC6.2A - Wind roof 90.2A, LC7.1 - Wind Internal pressure</td>
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<td>LC1 - Self weight, LC2 - Dead load, LC4.2 - Wind wall 90.2, LC6.2A - Wind roof 90.2A, LC7.2 - Wind Internal suction</td>
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<td>SLS14</td>
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<td>1 1 1 1 1</td>
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<td>SLS15</td>
<td>LC1 - Self weight, LC2 - Dead load, LC4.2 - Wind wall 90.2, LC6.2B - Wind roof 90.2B, LC7.2 - Wind Internal suction</td>
<td>1 1 1 1 1</td>
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<tr>
<td>SLS20</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.1 - Snow 1</td>
<td>1 1 1</td>
</tr>
<tr>
<td>SLS21</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.2 - Snow 2</td>
<td>1 1 1</td>
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<td>SLS22</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.3A - Snow 3A</td>
<td>1 1 1</td>
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<tr>
<td>SLS23</td>
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<td>SLS24</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.3C - Snow 3C</td>
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<td>SLS25</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.3D - Snow 3D</td>
<td>1 1 1</td>
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<tr>
<td>Name</td>
<td>Load cases</td>
<td>Coeff. [-]</td>
</tr>
<tr>
<td>-------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------------</td>
</tr>
<tr>
<td>ULS1</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.1A - Wind roof 0,1A, LC7.1 - Wind Internal pressure</td>
<td>1,5 1,5 1,65 1,65</td>
</tr>
<tr>
<td>ULS2</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.1B - Wind roof 0,1B, LC7.1 - Wind Internal pressure</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS3</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.1A - Wind roof 0,1A, LC7.2 - Wind Internal suction</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS4</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.1B - Wind roof 0,1B, LC7.2 - Wind Internal suction</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS5</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.1B - Wind roof 0,1B, LC7.2 - Wind Internal suction</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS6</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.2A - Wind roof 0,2A, LC7.1 - Wind Internal pressure</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS7</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.2A - Wind roof 0,2A, LC7.2 - Wind Internal suction</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS8</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.2B - Wind roof 0,2B, LC7.1 - Wind Internal pressure</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS9</td>
<td>LC1 - Self weight, LC2 - Dead load, LC3 - Wind wall 0, LC5.2B - Wind roof 0,2B, LC7.2 - Wind Internal suction</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS10</td>
<td>LC1 - Self weight, LC2 - Dead load, LC4.1 - Wind wall 90.1, LC6.1 - Wind roof 90.1, LC7.1 - Wind Internal pressure</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS11</td>
<td>LC1 - Self weight, LC2 - Dead load, LC4.1 - Wind wall 90.1, LC6.1 - Wind roof 90.1, LC7.2 - Wind Internal suction</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS12</td>
<td>LC1 - Self weight, LC2 - Dead load, LC4.2 - Wind wall 90.2, LC6.2A - Wind roof 90.2A, LC7.1 - Wind Internal pressure</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS13</td>
<td>LC1 - Self weight, LC2 - Dead load, LC4.2 - Wind wall 90.2, LC6.2A - Wind roof 90.2A, LC7.2 - Wind Internal suction</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS14</td>
<td>LC1 - Self weight, LC2 - Dead load, LC4.2 - Wind wall 90.2, LC6.2B - Wind roof 90.2B, LC7.1 - Wind Internal pressure</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS15</td>
<td>LC1 - Self weight, LC2 - Dead load, LC4.2 - Wind wall 90.2, LC6.2B - Wind roof 90.2B, LC7.2 - Wind Internal suction</td>
<td>0,9 0,9 1,65 1,65 1,65</td>
</tr>
<tr>
<td>ULS20</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.1 - Snow 1</td>
<td>1,3 1,3 1,65</td>
</tr>
<tr>
<td>ULS21</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.2 - Snow 2</td>
<td>1,3 1,3 1,65</td>
</tr>
<tr>
<td>ULS22</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.3A - Snow 3A</td>
<td>1,3 1,3 1,65</td>
</tr>
<tr>
<td>ULS23</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.3B - Snow 3B</td>
<td>1,3 1,3 1,65</td>
</tr>
<tr>
<td>ULS24</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.3C - Snow 3C</td>
<td>1,3 1,3 1,65</td>
</tr>
<tr>
<td>ULS25</td>
<td>LC1 - Self weight, LC2 - Dead load, LC8.3D - Snow 3D</td>
<td>1,3 1,3 1,65</td>
</tr>
</tbody>
</table>
5 Calculations sheets LTBeamN
- Middle beam
- Arch beam
- Middle beam gable wall
- Corner column gable wall
CALCULATION SHEET

I.1 - General parameters

- Projected total length: L = 14.8 m
- Initial discretization of the beam: n_p = 100 elements

I.2 - Material

- Name: Steel
- Young modulus: E = 210000 MPa
- Shear modulus: G = 80769 MPa
- Poisson factor: ν = 0.3
- Density: ρ = 7850 kg/m³

I.3 - Sections

- Alignment of sections: Top

![Figure 1: Profile in long with section numbers.](image1)

- Section No. 1: IPE 450
  - Abscissa from the left end of the beam: s = 0 m
  - Type: In catalogue (CITUA)
  - Main geometrical properties:
    - A_p = 98.82 cm²
    - I_p = 1499.7 cm⁴
    - W_p = 1701.8 cm³
  - Other geometrical properties:
    - A = 98.82 cm²
    - W_p = 1701.8 cm³
  - Stiffness relations:
    - c: Continuous
    - ν: Continuous
    - d: Continuous
    - f: Continuous
    - w: Continuous

- Section No. 2: IPE 450
  - Abscissa from the left end of the beam: s = 14.8 m
  - Type: In catalogue (CITUA)
  - Main geometrical properties:
    - A_p = 98.82 cm²
    - I_p = 1499.7 cm⁴
    - W_p = 1701.8 cm³
  - Other geometrical properties:
    - A = 98.82 cm²
    - W_p = 1701.8 cm³
  - Stiffness relations:
    - c: Continuous
    - ν: Continuous
    - d: Continuous
    - f: Continuous
    - w: Continuous
### 1.4 - Lateral restraints

![Diagram of restraint numbers](image1)

- **Restraint No. 1:**
  - Type: Pinned
  - Abcissa from the left end of the beam: $s = 0$ m
  - Vertical position from the shear centre: $z = 0$ cm
  - Restriction conditions:
    - $v$: Fixed
    - $c$: Fixed
    - $v'$: Free
    - $c'$: Free

- **Restraint No. 2:**
  - Type: Continuous
  - Coordinates of the left end:
    - Abcissa from the left end of the beam: $s = 0$ m
    - Vertical position from the shear centre: $z = 22.5$ cm
  - Coordinates of the right end:
    - Abcissa from the left end of the beam: $s = 14.8$ m
    - Vertical position from the shear centre: $z = 22.5$ cm
  - Restriction conditions:
    - $v$: Fixed

### 1.5 - Supports

![Diagram of support numbers](image2)

- **Support No. 1:**
  - Abcissa from the left end of the beam: $s = 0$ m
  - Support conditions:
    - $w$: Free
    - $w'$: Fixed

- **Support No. 2:**
  - Abcissa from the left end of the beam: $s = 14.8$ m
  - Support conditions:
    - $w$: Fixed
    - $w'$: Free

### 1.6 - Loads

- **Moment diagram**

![Moment diagram](image3)

- **Axial force diagram**

![Axial force diagram](image4)

### Table 1: Moment diagram

<table>
<thead>
<tr>
<th>$s$ (m)</th>
<th>$M_{MN}(N)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-124.29</td>
</tr>
<tr>
<td>1.48</td>
<td>-1.82</td>
</tr>
<tr>
<td>2.96</td>
<td>53.76</td>
</tr>
<tr>
<td>4.44</td>
<td>161.46</td>
</tr>
<tr>
<td>5.92</td>
<td>202.28</td>
</tr>
<tr>
<td>7.4</td>
<td>210.66</td>
</tr>
<tr>
<td>8.88</td>
<td>205.27</td>
</tr>
<tr>
<td>10.36</td>
<td>161.44</td>
</tr>
<tr>
<td>11.84</td>
<td>63.36</td>
</tr>
<tr>
<td>13.32</td>
<td>-1.97</td>
</tr>
<tr>
<td>14.8</td>
<td>-124.25</td>
</tr>
</tbody>
</table>

- Type of loading: Internal

**Active:** Yes
II. LTB Calculation

Requested number of modes: 1
Blocked moment diagram: No
Blocked axial force diagram: No

II.1 - LTB Modes

<table>
<thead>
<tr>
<th>Mode</th>
<th>No</th>
<th>M_{max} [kN.m]</th>
<th>M_{max} [kN.m]</th>
<th>N_{max} [kN]</th>
<th>N_{max} [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>17.25</td>
<td>3277.3</td>
<td>7.4</td>
<td>0</td>
<td>7.4</td>
</tr>
</tbody>
</table>

II.2 - Mode Shapes

- Mode 1

<table>
<thead>
<tr>
<th>Mode</th>
<th>No</th>
<th>M_{max} [kN.m]</th>
<th>M_{max} [kN.m]</th>
<th>N_{max} [kN]</th>
<th>N_{max} [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>17.25</td>
<td>3277.3</td>
<td>7.4</td>
<td>0</td>
<td>7.4</td>
</tr>
</tbody>
</table>
I.1 - General parameters
Projected total length : \( L = 36.1 \, \text{m} \)
Initial discretization of the beam : \( n = 100 \) elements

I.2 - Material
Name : Steel
Young modulus : \( E = 210000 \, \text{MPa} \)
Shear modulus : \( G = 80769 \, \text{MPa} \)
Poisson ratio : \( \nu = 0.3 \)
Density : \( \rho = 7850 \, \text{kg/m}^3 \)

I.3 - Sections
Alignment of sections : Top

Figure 1 : Profile in cog with section numbers.

- Section No. 1 : HEA 280
Abacause from the left end of the beam : \( x = 0 \, \text{m} \)
Type : In catalogue (OTUA)

Main geometrical properties :

- Section No. 2 : HEA 280
Abacause from the left end of the beam : \( x = 36.1 \, \text{m} \)
Type : In catalogue (OTUA)

Main geometrical properties :
1.4 - Lateral restraints

- Restraint No. 1:
  - Type: Pinned
  - Abcissa from the left end of the beam: $x = 0$ m
  - Vertical position from the shear centre: $z = 0$ cm
  - Restraint conditions:
    - $v$: Fixed
    - $w$: Free
    - $w'$: Free

- Restraint No. 2:
  - Type: Continuous
  - Coordinates of the left end:
    - Abcissa from the left end of the beam: $x_l = 0$ m
    - Vertical position from the shear centre: $z_l = 13.5$ cm
  - Coordinates of the right end:
    - Abcissa from the left end of the beam: $x_r = 38.1$ m
    - Vertical position from the shear centre: $z_r = 13.5$ cm
  - Restraint conditions:
    - $v$: Fixed

1.5 - Supports

- Support No. 1:
  - Abcissa from the left end of the beam: $x = 0$ m
  - Support conditions:
    - $u$: Free
    - $w$: Fixed
    - $w'$: Free

- Support No. 2:
  - Abcissa from the left end of the beam: $x = 38.1$ m
  - Support conditions:
    - $u$: Fixed

1.6 - Loads

- Moment diagram:

Table 1 - Moment diagram:

<table>
<thead>
<tr>
<th>$x$ (m)</th>
<th>MIN(MN)</th>
<th>MAX(MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-198.43</td>
<td></td>
</tr>
<tr>
<td>1.85</td>
<td>-27.20</td>
<td></td>
</tr>
<tr>
<td>5.72</td>
<td>-73.06</td>
<td></td>
</tr>
<tr>
<td>11.43</td>
<td>-99.06</td>
<td></td>
</tr>
<tr>
<td>9.24</td>
<td>-1.73</td>
<td></td>
</tr>
<tr>
<td>12.50</td>
<td>24.99</td>
<td></td>
</tr>
<tr>
<td>18.46</td>
<td>30.17</td>
<td></td>
</tr>
<tr>
<td>21.46</td>
<td>32.21</td>
<td></td>
</tr>
<tr>
<td>29.86</td>
<td>52.21</td>
<td></td>
</tr>
<tr>
<td>32.09</td>
<td>-0.46</td>
<td></td>
</tr>
<tr>
<td>36.5</td>
<td>41.4</td>
<td></td>
</tr>
<tr>
<td>37.3</td>
<td>-54.4</td>
<td></td>
</tr>
<tr>
<td>38.1</td>
<td>-151.21</td>
<td></td>
</tr>
</tbody>
</table>
II. LTBEAM CALCULATION

- Axial force diagram:

<table>
<thead>
<tr>
<th>Mode</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Active: No

Table 2: Axial force diagram.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Eccentric concentrated loads:
No load has been defined.

Eccentric distributed loads:
No load has been defined.

II.1 - LTBeam modes

Table 2: LTBeam modes.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

II.2 - Mode shapes

- Mode 1

Table 4: Mode 1.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 7: Axial force diagram.

Figure 8: Mode shape in 3D (Mode 1).

Figure 9: Lateral displacement component of the shear centre (Mode 1).

Figure 10: Rotation in lateral flexure component of the shear centre (Mode 1).

Figure 11: Longitudinal rotation (torsion) component of the shear centre (Mode 1).

Figure 12: Warping component of the shear centre (Mode 1).
I - PARAMETERS

1.1 - General parameters
Projected total length: \( L = 14.8 \text{ m} \)
Initial discretization of the beam: \( n = 100 \) elements

1.2 - Material
Name: Steel
Young modulus: \( E = 210000 \text{ MPa} \)
Shear modulus: \( G = 80769 \text{ MPa} \)
Poisson factor: \( \nu = 0.3 \)
Density: \( \rho = 7850 \text{ kg/m}^3 \)

1.3 - Sections
Alignment of sections: Top

Figure 1: Profile view with section numbers.

- Section No. 1: IPE 300
  - Abacasis from the left end of the beam: \( x = 0 \text{ m} \)
  - Type: In catalogue (OTUA)

Main geometrical properties:
\[
egin{align*}
  A &= 53.81 \text{ cm}^2 \\
  I_x &= 836.1 \text{ cm}^4 \\
  I_y &= 603.78 \text{ cm}^4 \\
  r_x &= 19.87 \text{ cm (Vierendeel)} \\
  r_y &= 126332 \text{ cm}^2 \\
\end{align*}
\]

Other geometrical properties:
\[
egin{align*}
  A_{0x} &= 32.1 \text{ cm}^2 \\
  W_{x,0x} &= 557.07 \text{ cm}^3 \\
  V_{x,0x} &= 80.5 \text{ cm}^2 \\
  W_{y,0x} &= 626.36 \text{ cm}^3 \\
\end{align*}
\]

Stiffness relations:
\[
egin{align*}
  &E: \text{Continuous} \\
  &G: \text{Continuous} \\
  &\nu: \text{Continuous} \\
  &\psi: \text{Continuous} \\
\end{align*}
\]

Figure 2: Section No. 1 (IPE 300).

- Section No. 2: IPE 300
  - Abacasis from the left end of the beam: \( x = 14.8 \text{ m} \)
  - Type: In catalogue (OTUA)

Main geometrical properties:
\[
egin{align*}
  A &= 53.81 \text{ cm}^2 \\
  I_x &= 836.1 \text{ cm}^4 \\
  I_y &= 603.78 \text{ cm}^4 \\
  r_x &= 19.87 \text{ cm (Vierendeel)} \\
  r_y &= 126332 \text{ cm}^2 \\
\end{align*}
\]

Other geometrical properties:
\[
egin{align*}
  A_{0x} &= 32.1 \text{ cm}^2 \\
  W_{x,0x} &= 557.07 \text{ cm}^3 \\
  V_{x,0x} &= 80.5 \text{ cm}^2 \\
  W_{y,0x} &= 626.36 \text{ cm}^3 \\
\end{align*}
\]

Stiffness relations:
\[
egin{align*}
  &E: \text{Continuous} \\
  &G: \text{Continuous} \\
  &\nu: \text{Continuous} \\
  &\psi: \text{Continuous} \\
\end{align*}
\]

Figure 3: Section No. 2 (IPE 300).
I.4 - Lateral restraints

**Figure 4**: Profile in long with restraint numbers.

- Restraint No. 1:
  - Type: Pendual
  - Abacissa from the left end of the beam: \( x = 0 \) m
  - Vertical position from the shear centre: \( z = 0 \) cm
  - Restraint conditions:
    - \( v \): Fixed
    - \( \psi \): Free
    - \( w \): Free

- Restraint No. 2:
  - Type: Continuous
  - Coordinates of the left end: \( x_v = 0 \) m
  - Vertical position from the shear centre: \( z_v = 15 \) cm
  - Coordinates of the right end: \( x_v = 14.8 \) m
  - Vertical position from the shear centre: \( z_v = 15 \) cm
  - Restraint conditions:
    - \( v \): Fixed

I.5 - Supports

**Figure 5**: Profile in long with support numbers.

- Support No. 1:
  - Abacissa from the left end of the beam: \( x = 0 \) m
  - Support conditions:
    - \( u \): Free
    - \( w \): Fixed
    - \( w' \): Free

- Support No. 2:
  - Abacissa from the left end of the beam: \( x = 14.8 \) m
  - Support conditions:
    - \( u \): Fixed
    - \( w \): Fixed
    - \( w' \): Free

I.6 - Loads

**Figure 6**: Moment diagram.

- Moment diagram:

**Table 1**: Moment diagram.

<table>
<thead>
<tr>
<th>( x )</th>
<th>( M()KNm)</th>
<th>( W()MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-88.71</td>
<td>0</td>
</tr>
<tr>
<td>1.48</td>
<td>-13.63</td>
<td>0</td>
</tr>
<tr>
<td>2.96</td>
<td>12.30</td>
<td>0</td>
</tr>
<tr>
<td>4.44</td>
<td>41.1</td>
<td>0</td>
</tr>
<tr>
<td>5.92</td>
<td>57.58</td>
<td>0</td>
</tr>
<tr>
<td>7.4</td>
<td>73.15</td>
<td>0</td>
</tr>
<tr>
<td>8.88</td>
<td>97.96</td>
<td>0</td>
</tr>
<tr>
<td>10.36</td>
<td>47.78</td>
<td>0</td>
</tr>
<tr>
<td>11.84</td>
<td>12.61</td>
<td>0</td>
</tr>
<tr>
<td>13.32</td>
<td>39.22</td>
<td>0</td>
</tr>
<tr>
<td>14.8</td>
<td>18.13</td>
<td>0</td>
</tr>
</tbody>
</table>

- Axial force diagram:
II. LTBEAM CALCULATION

Requested number of modes : 1
Blocked moment diagram : No
Blocked axial force diagram : No

II.1 - LTBEAM modes

Table 2 - LTBEAM modes:

<table>
<thead>
<tr>
<th>Mode</th>
<th>No.</th>
<th>Xmax</th>
<th>Ymax</th>
<th>Zmax</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.259</td>
<td>-319.86</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

II.2 - Mode shapes

- Mode 1

Table 4 - Mode 1:

<table>
<thead>
<tr>
<th>Mode</th>
<th>No.</th>
<th>Xmax</th>
<th>Ymax</th>
<th>Zmax</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.259</td>
<td>-319.86</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
I - PARAMETERS

1.1 - General parameters
Projected total length : $L = 8$ m
Initial discretization of the beam : $n_0 = 100$ elements

1.2 - Material
Name: Steel
Young modulus : $E = 210000$ MPa
Shear modulus : $G = 80769$ MPa
Poisson factor : $\nu = 0.3$
Density : $\rho = 7850$ kg/m$^3$

1.3 - Sections
Alignment of sections : Top

Figure 1: Profile in cross with section numbers.

- Section No. 1: HEA 280
  Abaciss from the left end of the beam : $x = 0$ m
  Type: In catalogue (OTUA)

Main geometrical properties:
- $a = 97.26$ cm$^2$
- $A_{u} = 31.74$ cm$^2$
- $W_{u} = 1012.8$ cm$^3$
- $W_{u} = 518.13$ cm$^3$

Other geometrical properties:
- $A_{u} = 97.26$ cm$^2$
- $W_{u} = 1012.8$ cm$^3$
- $W_{u} = 518.13$ cm$^3$

Stiffness relations:
- Continuous

- Section No. 2: HEA 280
  Abaciss from the left end of the beam : $x = 8$ m
  Type: In catalogue (OTUA)

Main geometrical properties:
- $a = 97.26$ cm$^2$
- $A_{u} = 31.74$ cm$^2$
- $W_{u} = 1012.8$ cm$^3$
- $W_{u} = 518.13$ cm$^3$

Other geometrical properties:
- $A_{u} = 97.26$ cm$^2$
- $W_{u} = 1012.8$ cm$^3$
- $W_{u} = 518.13$ cm$^3$

Stiffness relations:
- Continuous
1.4 - Lateral restraints

- Restraint No. 1:
  Type: Pendulum
  Abacuses from the left end of the beam: x = 0 m
  Vertical position from the shear centre: z = 0 cm
  Restraint conditions:
  \( v \) : Fixed
  \( w \) : Fixed
  \( \psi \) : Fixed

- Restraint No. 2:
  Type: Pendulum
  Abacuses from the left end of the beam: x = 8 m
  Vertical position from the shear centre: z = 0 cm
  Restraint conditions:
  \( v \) : Fixed
  \( w \) : Fixed
  \( \psi \) : Fixed

1.5 - Supports

- Support No. 1:
  Abacuses from the left end of the beam: x = 0 m
  Support conditions:
  \( u \) : Fixed
  \( w \) : Fixed
  \( w' \) : Fixed

- Support No. 2:
  Abacuses from the left end of the beam: x = 2 m
  Support conditions:
  \( u \) : Free
  \( w \) : Fixed
  \( w' \) : Fixed

- Support No. 3:
  Abacuses from the left end of the beam: x = 4 m
  Support conditions:
  \( u \) : Free
  \( w \) : Fixed
  \( w' \) : Fixed

- Support No. 4:
  Abacuses from the left end of the beam: x = 6 m
  Support conditions:
  \( u \) : Free
  \( w \) : Fixed

1.6 - Loads

- Moment diagram:
  Active: Yes
  Table 1: Moment diagram
<table>
<thead>
<tr>
<th>( x(m) )</th>
<th>( M(MN) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>2.22</td>
</tr>
<tr>
<td>2</td>
<td>0.88</td>
</tr>
<tr>
<td>3</td>
<td>2.27</td>
</tr>
<tr>
<td>4</td>
<td>-1.33</td>
</tr>
<tr>
<td>5</td>
<td>1.7</td>
</tr>
<tr>
<td>6</td>
<td>-0.66</td>
</tr>
<tr>
<td>7</td>
<td>-1.03</td>
</tr>
<tr>
<td>8</td>
<td>-0.13</td>
</tr>
</tbody>
</table>

- Axial force diagram:
II - LTB CALCULATION

Requested number of modes : 1
Blocked moment diagram : No
Blocked axial force diagram : No

II.1 - LTB modes

Table 2: LTB modes.

<table>
<thead>
<tr>
<th>Mode</th>
<th>k</th>
<th>M_{max} [MN.m]</th>
<th>M_{max} [%]</th>
<th>N_{max} [kN]</th>
<th>N_{max} [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>-153.6</td>
<td>8</td>
<td>0</td>
<td>8</td>
</tr>
</tbody>
</table>

II.2 - Mode shapes

- Mode 1

Table 4: Mode 1.

<table>
<thead>
<tr>
<th>Mode</th>
<th>k</th>
<th>M_{max} [MN.m]</th>
<th>M_{max} [%]</th>
<th>N_{max} [kN]</th>
<th>N_{max} [%]</th>
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</thead>
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</tr>
</tbody>
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CZECH TECHNICAL UNIVERSITY IN PRAGUE

FACULTY OF CIVIL ENGINEERING

MASTER THESIS

ICE HOCKEY ARENA WITH ARCHED ROOF

PART C: DRAWINGS

2019

B. HALBERSTADT
<table>
<thead>
<tr>
<th>LIST OF DRAWINGS</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRAwing 1: PLAN VIEW AND SECTIONS</td>
</tr>
<tr>
<td>DRAwing 2: DETAILS</td>
</tr>
</tbody>
</table>